THE DEVELOPMENT OF MATERIAL-ADAPTED STRUCTURAL FORM

PART II: APPENDICES

THE BEAM AND ARCH IN GREEK ANTIQUITY A-01

1.1 Introduction

During the seventh and eighth centuries BC, Hellenic Greek civilization began to transform itself from a construction culture dominated by timber to one dominated by stone. Though evidence is fragmentary, the conventions of Greek monumental architecture developed during the seventh and early sixth centuries BC.[1] In the course of this transition, Greek builders had to relearn the skills necessary to build with stone. These skills existed in Bronze Age Greece but were lost for some four centuries before Hellenic Greece emerged. Classical Greek buildings relied on post and lintel construction. Noticeably absent is evidence of the arch.

The re-emergence of stone construction in Greece is associated with the development of the Doric Order, an aesthetic system based on naturally occurring proportions that can be defined by mathematical ratios related to the square. There is little evidence showing the development of the Doric Order or its system of construction. It is often posited that the Doric Order is a direct translation of traditional timber construction to stone.[2] The validity of this hypothesis will be examined below.

Using stone to make beams, thus subjecting the material to bending stress, leads to the question of how the Greeks adapted stone to that purpose. Timber is almost equally strong in tension and compression, making a regular prismatic section practical. Conversely, stone resists compression better than tension. If the Doric Order is a system translated from traditional timber construction, then how was the form of the stone beams adapted, if at all, to the particular structural characteristics of stone? The second part of this chapter traces the development of the stone beam in Greek antiquity – including several instances where iron bars were apparently used to relieve stress in the stone.

The Greeks rarely employed the arch, one of the most structurally efficient forms to make from stone. Did they know of the arch? If yes, why did they not use it? While technological pursuits were not considered worthy intellectual exercises in Hellenic Greece, the Greeks were capable mathematicians and accomplished builders. Was the arch aesthetically unacceptable, or are there other reasons the arch was not developed? The final part of this chapter addresses these questions.

[1] Coulton, p15-16.
[2] Anderson et al., p67-68; Dinsmoor, p55-58; Coulton p37-41; Vitruvius, p111-113.

1.2 Origins of Greek Construction

Between 1100 and 700 BC the Greeks built little or no monumental architecture. During the eighth century BC, buildings began to increase in scale in conjunction Greek society's growing interest in Greece's heroic past. Homer wrote *The Iliad* and *The Odyssey* at this time. In the seventh century BC, the Greeks began to build monumental architecture to reflect their growing power as a political and military entity. The intent was as much to impress and endure as to create a space in which public activity could occur.[3]

When the Greeks became interested in monumental architecture, they could look to their past architectural experience for guidance, develop totally new approaches or combine the two. A problem with looking in the past was that some older structures were not relevant to their needs. For instance, during Greece's Bronze Age, circular *tholos* tombs were constructed with corbelled vaults. The problem was that such tombs had no place in the religious and social conditions of the time, therefore the remains of Bronze Age architecture could not really supply helpful models to the eighth-century architects.[4] Today, we have difficulty remembering the building technology employed in the 19th century despite all of the recorded reference material available to us. How can one expect a people with little or no written records to adequately apply knowledge and technology not used for three centuries?

It is difficult to trace the evolution of Greek monumental architecture between the eighth and sixth centuries BC because most of the earliest temples were constructed of wood, which has long since decayed or been destroyed by fire. What is known comes mainly from the layout and size of surviving masonry foundations. The remains of terracotta panels used to clad the wooden superstructure also provide some clues.[5]

Terracotta was the first new material used by Greek builders in their search for a more monumental mode of expression. Greek artisans worked proficiently with terracotta. They produced large, colorful and crisply shaped objects. The use of terracotta was a natural reaction for builders not yet familiar with monumental techniques, but trying to achieve an impression of solidity and durability. One longstanding aspect of this experimentation with terracotta was its use as roof tiles.[6]

The quest for a 'permanent' architecture, which is itself a product of cultural ideas stemming from civic and spiritual life, ultimately led to the use of stone. The sociological foundation of ancient Greek culture supports this statement, but technologically it is an incomplete analysis. Choosing to use stone because of its durability or permanence – compared to timber – is self-evident. It would be helpful if more were known about how and why the Greeks learned to fashion stone into specific forms.

[3] Dinsmoor, p30.
[4] Coulton, p30-31.
[5] Anderson, Spiers and Dinsmoor, p63-64.
[6] Coulton, p41.

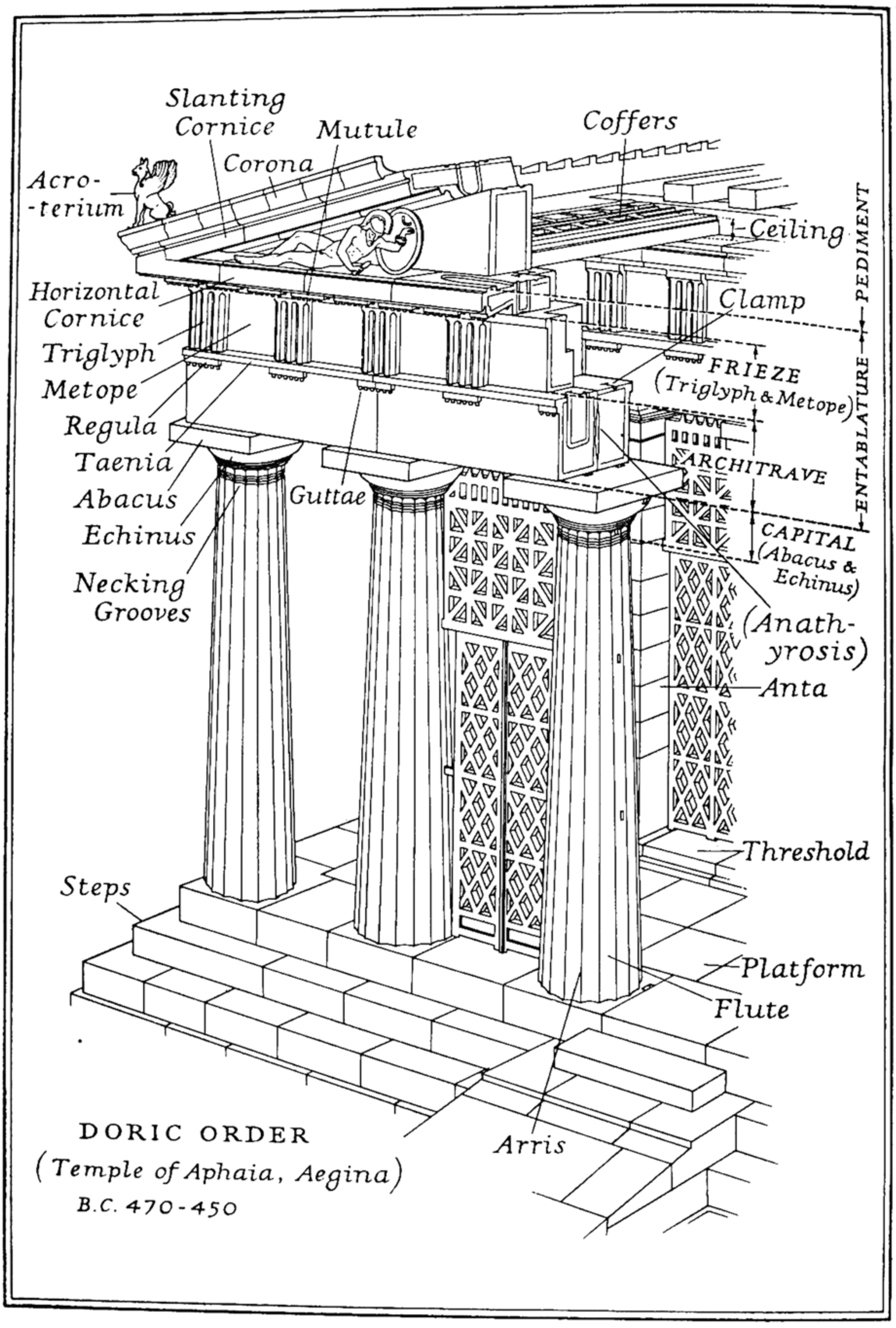

Fig. 1: Architectural vocabulary of the Doric Order. (Lawrence)

1.3 The Doric Order and Material Substitution

1.3.1 The Substitution Theory

The earliest known Greek temples built of stone during the Hellenic period are characterized by the Doric Order. (**Fig. 1**) The origins of the Doric Order have been much argued over.[7] The principal aesthetic language is influenced by Greece's Mycenaen heritage and its relationship with Egypt, an important trading partner. It has been said that the Doric Order is the product of directly substituting stone for timber, though evidence is scant.[8] The earliest stone temple ruins we know of show a mature command of the Doric Order, while the woodwork of earlier Greek buildings has perished.

Accepting for the moment that the Doric Order in stone represents the translation of traditional wooden forms into stone then we can repeat Vitruvius's explanation of what the architectural details in stone would have been in wood.[9] **Figures 2, 3 and 4** show the hypothetical progression from timber construction, to timber-terracotta, and finally to all stone.

The straight substitution theory, as proposed by Vitruvius, concludes that the triglyphs in the frieze reproduce the ends of beams. The pins that secured the beams by passing through the projecting taenia surmounting the architrave and through the regula became guttae. The mutules, or projecting blocks on the soffit of the cornice, are clearly the ends of the rafters of the roof, likewise with guttae. We can likewise interpret all the other details as translations of wood into stone with the exception of the metopes, which are interpreted as being the terracotta facing between the triglyph beam-ends.[10]

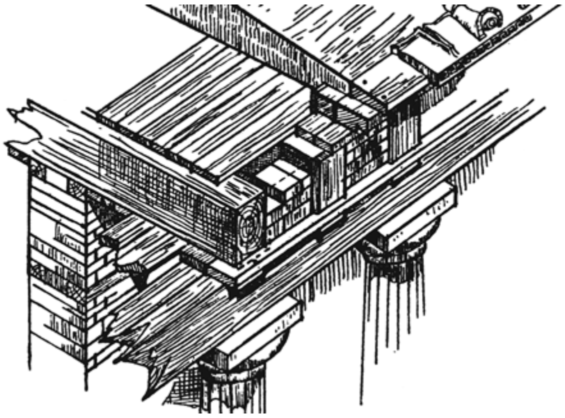

Fig. 2: Conjectured reconstruction of Proto-Doric, wooden entablature. (Anderson, after Durm)

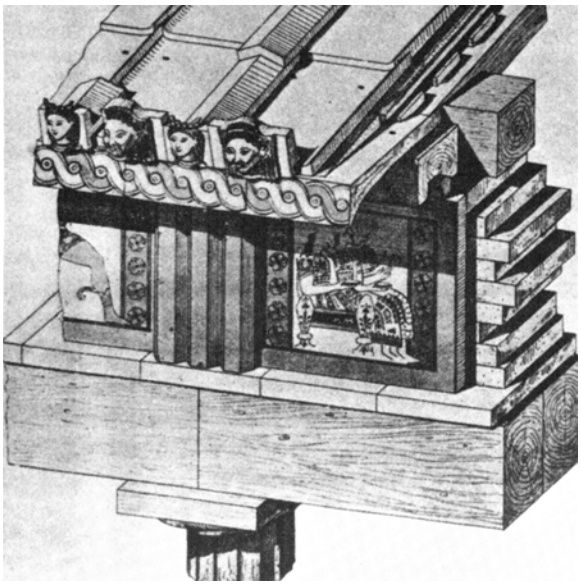

Fig. 3: Entablature of the Temple Apollo at Thermum, showing terracotta frieze. (Anderson, after Kawerau)

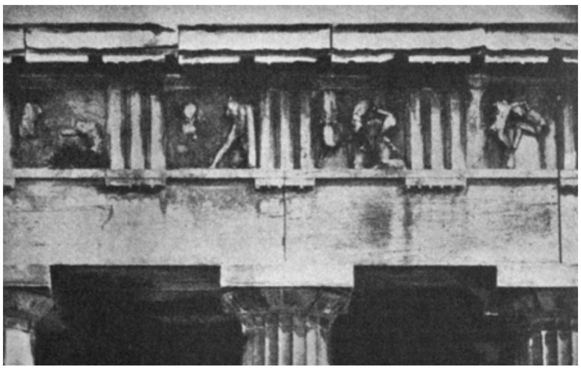

Fig. 4: Doric frieze of the Theseum. (Anderson)

[7] Dinsmoor, p55-58; Coulton, p37-41; Vitruvius, p11-113; Lawrence, p99-107; Viollet-le-Duc, p51-53.
[8] Viollet-le-Duc, Dinsmoor, Coulton.
[9] Vitruvius, p111-113.
[10] Anderson, Spiers and Dinsmoor, p68.

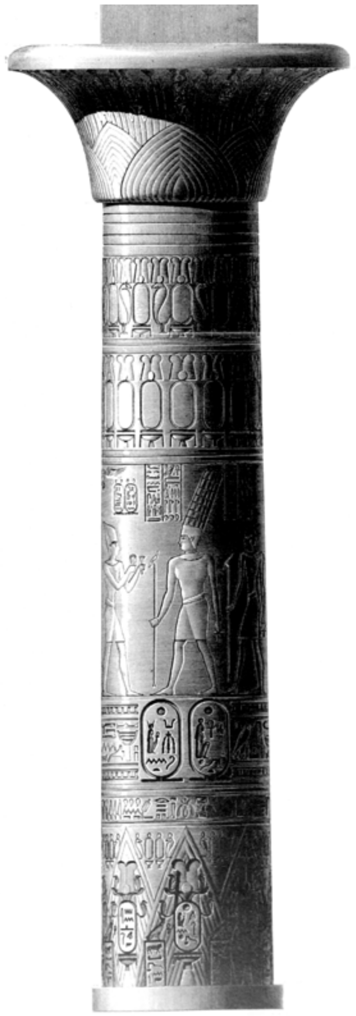

Fig. 5: Upward tapering column from Temple of Karnak in Thebes, Egypt. 1530-323 BC. (Gillispie and Dewachter)

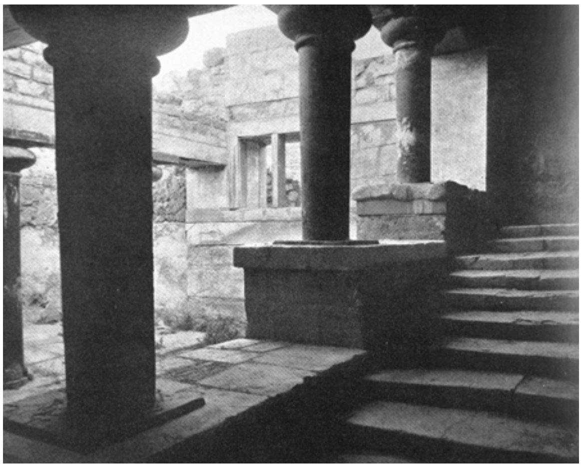

Fig. 6: Downward tapering columns, Cnossus, Mycenae. c.1600 BC. (Lawrence)

Contemporary works on the subject typically accept this interpretation as fact. We can base such conclusions only on what we have learned from the terracotta castings that clearly express the architectural language that would later be carved more permanently into stone. The problem is that we can only assume that these castings did indeed copy the wood architecture they were meant to aesthetically replace and functionally protect from deterioration.

1.3.2 The Possibility of Outside Influence

To state that the stone temple is a reproduction of its timber counterpart ignores the possible influences on Greek building traditions from the various Greek colonies and other kingdoms where stone construction technology was still current. The Greeks were not technologically isolated. By the early eighth century BC there was a Greek trading settlement in Syria and a Greek colony established on the west coast of Italy, the first of a whole series of colonies founded in the later eighth and seventh centuries on the coast of Sicily and south Italy. In about 660 BC, the Egyptian pharaoh Psamtik I, known to the Greeks as Psammetichos, gained control of his country from the Assyrians with the help of Ionian and Carian mercenaries. From then on there was close contact between Greece and Egypt.[11]

The basic forms of Doric architecture had been created by the beginning of the sixth century.[12] William Anderson suggests that the wooden columns of primitive Greek architecture had no relationship to the Doric column, and that the Greeks imitated the latter from certain Egyptian stone columns.[13]

Doric columns taper upwards like Egyptian columns, not downwards like those of the

[11] Coulton, p32.
[12] Coulton, p32.
[13] Anderson, Spiers and Dinsmoor, p62.

Bronze Age, and the number of flutes in most early Doric columns is sixteen, an easy number to produce, but also the number commonly used in Egypt.[14] (**Figs. 5 and 6**) Conversely, there is not sufficient evidence to determine whether or not the wood columns of the seventh and eighth centuries BC were cylindrical or tapered and oriented up or down.[15]

Anderson and Spiers write,

> The heavy proportions of the earliest known Greek Doric columns of stone, little more than four diameters high, and the fact that from the very beginning the echinus formed an essential feature between the shaft and the abacus, while the abacus was of much greater width than the upper diameter of the shaft, militate seriously against the theory that there was any connection between the Greek Doric column and the so-called "Proto-Doric" examples at Beni-Hasan or at Karnak and Der-el-Bahari in Thebes.[16]

However, early stone columns were noticeably thicker then their wooden counterparts and more similar in proportion to Egyptian columns.[17] Anderson proposes that the well-known timidity of the Greeks in stone construction would be enough to account for the sudden thickening of the proportions when translating from a prototype hitherto of wood.[18] A recurring theme that will emerge later in this text is the idea of stability during construction. A slender column requires a higher level of construction management to ensure it does not topple. That Greek columns would later become more slender may just indicate the inferior state of construction technology and management at this early date.

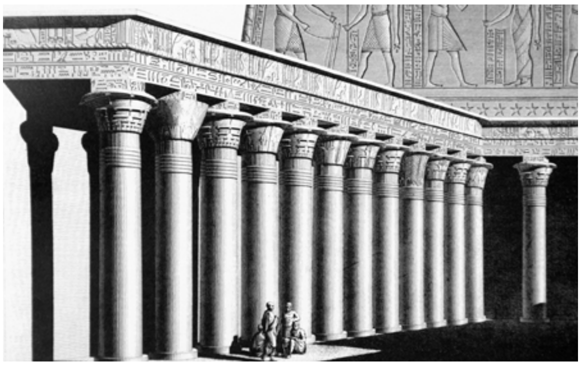

Fig. 7: Compare similarities of the entablature of this Egyptian temple to the Doric Order. Grand Temple of Edfou. 237-57 BC. (Gillispie and Dewachter)

Coulton cites further evidence to support the possibility of Egyptian and Bronze Age Greek influence, though not conclusively. The early Doric capital, for instance, finds close parallels in Greek architecture of the Bronze Age. The Doric architrave, roughly square in section, and crowned by a continuous projecting band (*laenia*), also has parallels in Egypt, although strictly the projecting band there belongs to the cornice. (**Fig. 7**) However, the peg-like projections on the Doric architrave have no precedents in Egypt or Bronze Age Greece, and, Coulton concedes, are probably derived from functional pegs in wooden construction, although that need not mean that such pegs appeared in primitive architecture in those particular positions.[19]

While the possibility exists that the Doric Order was influenced by the architecture of Egypt and Bronze Age Greece, the evidence is not conclusive in light of how little we know of the actual form of the wood architecture that preceded the emergence of Doric construction.

[14] Coulton, p39.
[15] Anderson, Spiers and Dinsmoor, p65.
[16] Anderson, Spiers and Dinsmoor, p67.
[17] Coulton, p39.
[18] Anderdon, Spiers and Dinsmoor, p67-68.
[19] Coulton, pp39,41.

1.3.3 From Wood to Stone? – Columns

The normal principle involved in the construction of the wood temples was probably the post and lintel using simply supported beams. The actual form of these earlier wooden columns is unknown. In Anderson and Spier's second edition[20], they suggest that the columns may have tapered downward in accordance with Mycenaean precedent,[21] but there is no actual evidence to conclude that. If Doric columns are to be considered as copies of the wooden columns, then the wood columns should have tapered upwards.

Coulton writes,

> The slender proportions of early stone Doric columns, with heights of 6.5 to 7 diameters, are often cited as evidence for their derivation from wooden prototypes. Since the Greeks themselves for centuries made Ionic columns in stone with heights of 9-10 diameters, the argument is weak. As recent vernacular architecture shows, wooden posts are likely to have a height of 10-20 diameters and a spacing of 7-15 diameters.[22] If, as is likely, the Greeks made wooden columns with a height of 6.5-7 diameters and a spacing of about 4 diameters, they were probably imitating the effect of stone columns in wood, not vice versa.[23]

On the Greek mainland, one of the earliest peripteral temples of which remains have been found sufficient to determine its approximate form is the temple of Apollo at Thermum in Aetolia. This temple has five columns on the fronts, fifteen columns on the flanks, and a single row down the center of the cella in order to carry the roof. (**Fig. 3**) The cella walls were of unburnt brick.[24] Only the footings of the columns were of stone; the columns themselves and the entablature were of wood. The diameter of the columns in temple of Apollo at Therum have been determined to by 0.80 m (2 ft 7.5 in), based on the size of the stone footings. If this diameter is multiplied by 7 diameters to calculate their height, then they would be 5.60 m (18 ft 4.5 in) height.

The temple of Apollo's wood columns may have been dimensioned as they were for three reasons: 1. Dressing the tree trunk close to its original diameter would require the least effort. 2. Keeping the trunk round made for easier transport as it could be rolled on the ground. 3. Using a stout column would make construction easier since it would probably stand without temporary bracing. Care would have been taken to avoid accidentally bumping already standing columns when raising adjacent ones or hitting the top of the unbraced column while installing the entablature.

Therefore, Coulton's argument that wood may have been a substitute for stone can be refuted for constructive reasons and the case for substitution is supported. It is, however, important to note that the proportions are more likely designed for stability during construction, and perhaps to minimize fabrication effort, rather than in-service stresses, buckling or other structural reasons.

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[20] Anderson and Spiers, *The Architecture of Ancient Greece*, 1907.
[21] Anderson, Spiers and Dinsmoor, p65.
[22] Coulton, after Payne, p37-38.
[23] Coulton, p38.
[24] Being unburnt brick made this material inadequate to be employed to construct columns or structural walls, like the Romans would later do so proficiently.

1.3.4 From Wood to Stone? – Entablatures

Due to the lack of direct evidence, we can only rely on terracotta fragments, especially those of Thermum, to indicate the form of the original wood structure making up the entablature. Thus, we must accept the unverifiable interpretation that the terracotta details are literal models of the original wooden features in the primitive entablature as described by Vitruvius and generally accepted ever since. There have been alternatives suggested, variously drawing from similar details found in Egyptian, Agean, Mycenean and Greek Bronze Age precedents. These alternatives are not any more provable than straight substitution.

1.3.5 Conclusion on Substitution

Coulton argues that none of the archetypical Doric details seems to arise naturally out of the simple wooden structures of a primitive society, although they could have been built out of wood. They involve beams carefully sawn to shape – often specially sawn into two pieces so that they can be joined together again by thin planks necessary to explain other features of the Doric system. He would expect primitive builders to use the saw sparingly.[25]

Elsewhere, Coulton writes,

> The evidence seems to suggest… that the Doric Order was not the result of a slow development, and there is no reason to believe that it represents a coherent structural system in any material. Rather it was the invention of a builder or a number of builders in the north-east Peloponnese who, around the middle of the seventh century, were trying to create a monumental style in architecture.[26]

Coulton defeats his own argument because it is logical that the literal translation of architectural details that evolved over a century or two in wooden temples would perfectly explain why the emergence of Doric architecture is marked by such an apparently advanced and mature state of design. To that end, while the evidence for substitution is not compelling, because there actually is no physical evidence of the timber construction to reference, the typical interpretation as described by Vitruvius seems appropriate in light of the substantive evidence given by the terracotta moldings.

The question that I am concerned with is to what extent did the Greeks adapt their architectural language, particularly with respect to proportion, as a result of the very different properties of wood and stone. Coulton actually addresses this above when proposing that the timber architecture was actually a *substitution for stone.* Perhaps the Greeks did indeed draw upon their own Bronze Age history, as well as the knowledge gained from their relationship with Egypt, to come up with a new temple form appropriate to their needs. The choice to use wood can be logically explained by the fact that large stone works were not produced in Greece for over three centuries, thus the technical competence did not exist. Furthermore, Greek civilization was just emerging, being in the beginning stages of a kind of renaissance. Economics was certainly a factor in building their first constructions in timber rather than stone. As the Greek civilization grew in wealth and technical competence, stone architecture emerged, not because it was not considered before but because the conditions did not favor its use.

[25] Coulton, p37-38.
[26] Coulton, p39.

According to the evidence that is available to me, it would appear that the temples built of wood were built of conservative proportions for that material, particularly with respect to span. It would help if I had more information concerning foundations found of the wood temples that would indicate more about column diameter and spacing. There just is not enough information to say that proportions were initially changed because of material concerns. In fact, it appears spans grew larger with the stone, and the Greeks did invent new forms and construction practices to address those conditions.

1.4 Tools of Construction

1.4.1 Construction Knowledge

The knowledge of how to build the monumental stone architecture of the Bronze Age was lost for over three centuries when the Greeks began to again build with stone. The ruins of the Bronze Age structures were the only models available to the Greeks of the Hellenic period on how to build in stone. However, those ruins could not inform the Greek builders how the stone was quarried, dressed, transported and lifted into place. In the following, I will briefly review some of the tools and construction practices used by the Greeks to build their stone temples. It can be supposed that the Greeks learned a great deal from their colonies and trading partners, such as Egypt.

1.4.2 Construction Management

From the fourth century BC the architect was expected to exercise detailed control over matters of workmanship. They would have had to inspect each course of stone before the next was laid, approve the tightness of the joints and the quality of the metal clamps between blocks, and authorize payments to the various workmen and contractors involved. In the late fourth and early third centuries, there was no distinct concept of separating architect from engineer in Geece. Eupalinos of Megara was called an *architekton* by Herodotos. Eupalinos supervised the construction of a tunnel at Samas c. 530 BC. The tunnel was cut from both ends to meet in the middle.[27]

1.4.3 Labor and Economy

Labor was a precious commodity in Greece. As such, a number of mechanical devices were employed that perhaps differ from the practices of Egypt, whose influence on Greek construction practices deserves a more in-depth review. The first lifting aid to be introduced, after the use of levers, was the compound pulley system. Coulton believes that it would not have been possible to build the monumental works of Greece without them. Such a system would have made it possible to conduct building operations with a comparatively small, professional workforce. The ramp, employed by the Egyptians, requires a large workforce. Although it is probable that such a workforce could have been raised during the early days of Greek architecture, it was not a normal feature of the Greek city-state.[28]

[27] Coulton, p16.
[28] Coulton, p144. Pliny describing the Greek's use of the ramp, said they used sandbags to make the ramp. When a block was put in position, sand was let out of the bags to lower the block into place.

It is not certain when Greek architects knew of cranes, but there is good evidence that they came rapidly into prominence from c.515 BC onwards. From about that time building methods were to some extent adapted to avoid weights beyond the capacity of a simple crane. Figure 8 shows a number of lifting devices probably employed by the Greeks.[29] Note the inventive use of simple mechanisms and the care taken to protect the integrity of those faces of the stone that would be visible when the building was complete by the clever placement and form of lifting points in the stone.

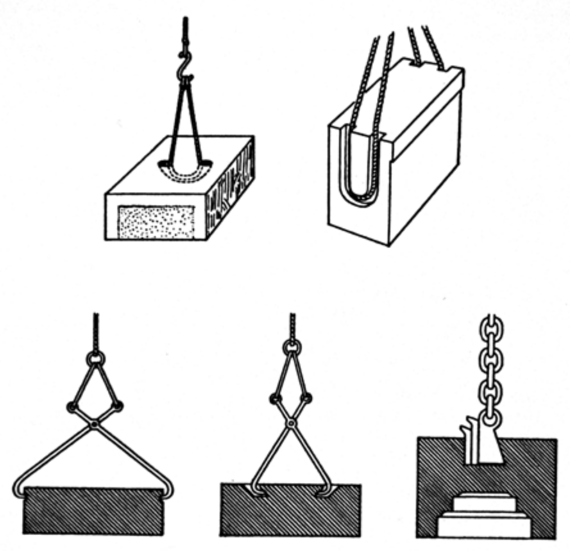


Fig. 8: Various lifting devices and lifting points cut in stone. (Dinsmoor)

1.4.4 *Technical Developments*

The Greeks developed progressively advanced ways of transporting stones. As the stones got bigger, the technological challenges of transporting them got more difficult. Early stones were probably transported on carts or sleds, but as the stone size increased, the weight probably became too great for the strength of wheel axles at the time. As a result, means of transporting the stones as integral parts of the wheel were developed as seen in Figure 9.[30]

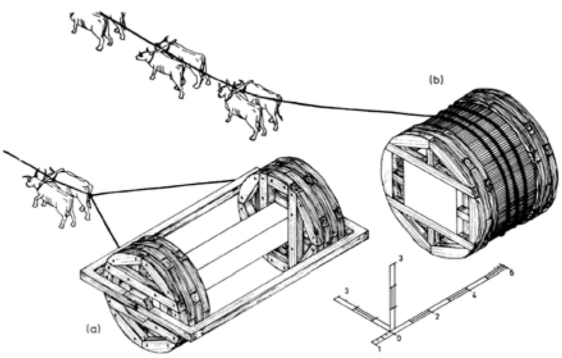


Fig. 9: Two modes for moving colossal stones. (Coulton)
(a) Metagenes' Method (c.550 BC)
(b) Paconius' Method (First Century)

Over time the Greeks developed their techniques and construction methods for working with stone. First, stones got progressively bigger until the Ionic period, when colossal temples were built. Concurrent with this increase in scale, means were developed to reduce the mass of the individual stones that had to be maneuvered. Economies were sought, as evidenced by the return to two-skinned wall-construction in the Hellenistic period.

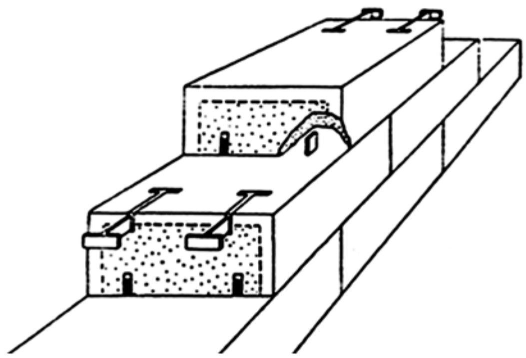


Fig. 10: I-shape clamps in ashlar of the Parthenon, Athens. (Hamilton)

There was surely seismic activity in Greece during the Hellenistic period that severely damaged or destroyed stone temples. In the

[29] Dinsmoor, p174.
[30] Coulton, p.141-142.

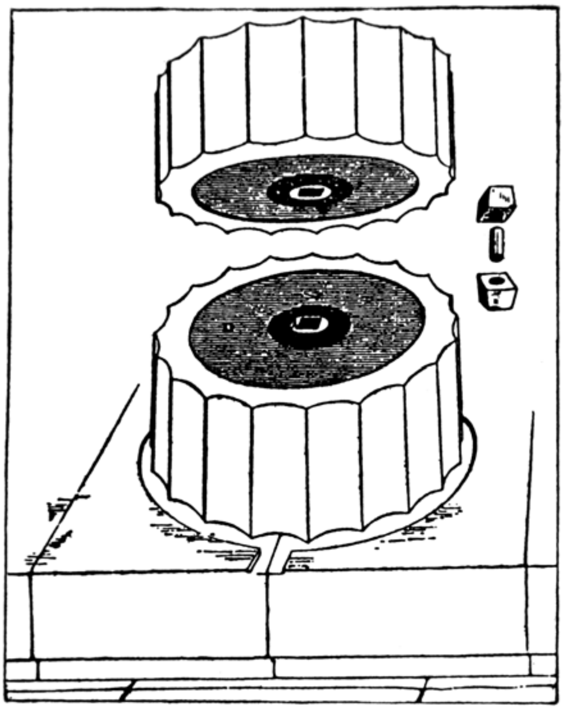


Fig. 11: Dowelled column sections. (Anderson)

Fig. 12: Iron Forge of Hephaistas, from an Attic vase, 6th century. (Hamilton)

late sixth century BC, the Greeks fabricated metal **I**-shaped clamps[31] to key successive courses of stone together and dowels to keep sectional columns together. (**Figs. 10 and 11**) As it turns out, the seismic behavior of a monolithic column is comparable to a segmental column. Research conducted by Greek engineers has shown that the dowels may actually be detrimental to the seismic performance of a stone column.[32] In the next section, I will examine in more detail the use of iron by the Greeks.

1.5 Iron in Greek Antiquity

1.5.1 Iron Production

Iron was widely produced in antiquity to varying degrees of quality and quantity. Iron was first considered a precious metal. It was used principally for making armor and weapons. The applications of iron became more diverse as the production of iron increased. Iron tools were vital improvements because no previous material had the hardness and strength properties of iron. Greek smiths of the classic period made lifting tackle, lewises, tongs, clamps and dowels from iron of low carbon content. Greek smiths also manufactured armor and cutting edges for tools and weapons using steel. **Figure 12** shows an armorer's furnace of the sixth century BC painted on a Greek vase now preserved in the British Museum. Inside the pot, iron as a viscous solid practically free of carbon would sink to the bottom. Above this, the iron richer in carbon, mainly eutectic steel, would float, and above that the slag. The slag would be ladled off and thrown away. The intermediate layer was highly valued for

[31] Coulton, p142. On the subject of these technological advances made by the Greeks, Coulton writes, "Such matters are not nowadays considered the meat of architecture, however, and since no general principles of interest appear to be involved, there is no need to go into them in detail." What a shame that he does not, even in 1977, consider these issues important and relevant. The metal used could have been iron or bronze.
[32] Psycharis et al., p1108.

weapon making. The soft iron at the bottom of the pot was suitable for structural and other common uses.[33]

The evidence available shows that the best prehistoric furnace could, in eight to ten hours and consuming about 90 kg (200 lbs) of charcoal, produce a bloom of partly-fused iron weighing about 23 kg (50 lbs). Much of the iron from the ore was run to waste in the slag. The use of limestone as a flux to separate the iron from the slag was only introduced in the seventeenth century AD. To expel the slag retained by the bloom required much hammering with some re-heating and the consumption of about another 11 kg (25 lbs) of charcoal. By Imperial Roman times, blooms that weighed about 45 kg (100 lbs) could be produced.[34]

Vitruvius, writing in the first century BC, does not mention use of waterpower to work bellows or furnaces, or to operate a hammer. He probably would not have failed to mention the fact, particularly if the forgings made were beams and the like for buildings.[35] The first recorded application of waterpower to the making and forging of iron, according to R.J. Forbes, appears in twelfth-century documents belonging to Cistercian monasteries in France.[36] Therefore, we can only assume that if any Greek or Roman builder did employ heavy structural members of iron, they were made by welding together numerous small pieces by hand-hammering, a process excessively laborious and expensive, even if done by slave labor; and the product would probably have been unreliable.[37]

1.5.2 Connection Technology

S.B. Hamilton suggests that the Greeks may have riveted together several metal plates to fabricate a compound iron beam.[38] The earliest evidence of riveting technology are some bronze bowels with riveted stiffening rims found in an excavation of a cemetery at Ur[39] dated before 2500 BC.[40] Parts of iron weapons and tools were fastened together with bronze rivets in Homeric times c.1300-700 BC.[41] Therefore, it is possible Greeks of the Hellenic era were familiar with and used rivets to fabricate metal artifacts.

Welding by hand hammering would have become a part of the Greek blacksmith's art by the middle of the fifth century BC. The oldest known example of welded iron is a small model headrest found in the tomb of Tut-ankh-amun, beneath the golden mask, where it was placed about 1350 BC. The welds were not good and a broken one had been repaired by soldering. However, Egyptian objects of later date, such as perfectly welded rings, have been found. [42] The iron clamps shown in **Figure 10** would have been hammer welded. The size of hand-hammer welded objects is limited by the enormous time and effort required to weld iron in this way and the fact that the quality of this method is very inconsistent.[43]

[33] Hamilton, p31.
[34] Hamilton, after Singer, p30.
[35] Hamilton, p31.
[36] Hamilton, after Singer, p30-31.
[37] Hamilton, p31.
[38] Hamilton, p36.
[39] Ur is located in present-day Iraq.
[40] Hamilton, after Maryon, p31.
[41] Hamiltion, after Gray, p31.
[42] Hamilton, p31.
[43] Hamilton, p36.

1.5.3 Known Examples of Greek Application of Iron

As stated before, the Greeks fabricated I-shaped clamps to tie successive courses of stone together. (**Figs. 10 and 13**) While the Egyptians used wooden butterfly ties between their stones, it is possible that the Greeks learned of the benefits of connecting the stones together from the Egyptians and then further developed the system using metal.[44]

The clamps of the type shown in **Figure 13** have been recovered from the Parthenon in Athens. These clamps were built up by welding together iron bars about 4 in. wide and 3/8 in. thick, or less. They had been formed in different ways; one by folding a strip of iron on itself and welding it to form the whole section; the other by welding together four separate pieces. The clamps were 13 in. and 12 in. long, 4 in. wide, about 3/8 in. thick, and were set nearly 4 in. deep into the stone. Two clamps and two dowels from stones of the Parthenon were examined at the Strength of Materials Laboratory of the Athens Technical University. The metal was mainly ferrite, but some parts were carbonized and contained as much as 0.85 percent carbon. The tensile strength varied from 262 to 524 MPa (38 to 76 ksi[45]). The clamps would appear to have been heated and lowered into their sinkings, which had been partly filled with molten lead. They were preserved in good condition.[46]

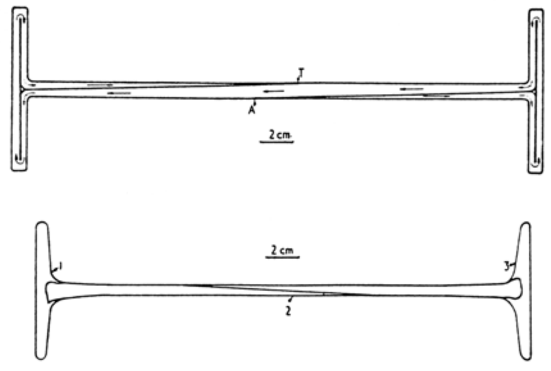


Fig. 13: Detail of ashlar clamp fabrication. (Hamilton)

There is evidence that iron bars were used as transfer beams to reduce stress in architraves. The Greeks also may have tried to reinforce their stone beams with iron, but the evidence is suspect and inconclusive.[47] I will explore these applications of iron in the next section on stone beams in Greek antiquity.

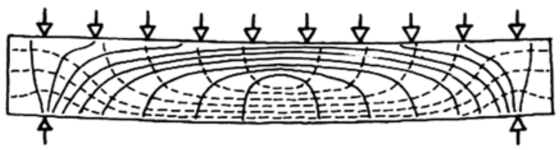


Fig. 14: Internal static equilibrium of a beam. Solid lines = Compression and Dashed lines = Tension. (Mainstone)

1.6 The Beam, a Background

1.6.1 Early Forms

Geometrically, the beam is one of the simplest of all elemental structural forms, but is one of the most structurally complex. A beam can span between supports without exerting any horizontal thrust only because tensile and compressive forces are contained within the beam. The simple form of a beam belies the complexity of what is going on inside. (**Fig. 14**)

There does not seem to be much evidence that the Greeks sought to maximize the structural capacity of the materials they used. It was common for them to set their wooden beams

44 Dinsmoor, p174.
45 ksi = kilo-pounds per square inch. 1 kilo-pound = 1000 pounds.
46 Hamilton, pp31, 33.
47 Dinsmoor, p176 and p103-105; Hamilton, p34-37.

resting on their long faces. This probably had to do with construction practice because this orientation is inherently more stable and provides a larger bearing surface. I cannot believe the Greeks laid the timber in that way out of ignorance since the superior stiffness characteristics of a wood beam bearing on its short side are readily observed.[48]

The structural development of the beam has been spurred by practical needs and a desire to make the most efficient use of new materials and techniques. The major developments are nearly all-recent ones of the last two hundred years in conjunction with the development of the field of strength of materials and the evolution of structural theory. A reason for this late development was the need to correctly visualize the internal actions to see what was needed and potentially feasible.[49]

1.6.2 Scale

As the scale of a colonnade increases, the load on the architrave will increase more rapidly than its strength, and for a given set of proportional rules there is a maximum size beyond which the architrave will collapse. Spans, therefore, had to remain small. Another factor is the necessity to preserver enough space between columns for functional and aesthetic reasons. Therefore, the columns cannot be too close together or too wide.[50]

The scale of columns is never a practical concern. Kamminga and Cotterell show that columns in ancient architecture carried very little stress. The limestone columns in the Temple of Aphaia, built in 490 BC, only carry a stress of 0.4 MPa (58 psi), which is only about a two-hundredth of the compressive strength of limestone.[51] The greater problem in the construction of columns is their stability and resistance to buckling. It is for these reasons that I think columns remained so stout up until the times of the Romans, who introduced construction process and technology.

When a wooden beam is bent, the fibers on the compression face begin to buckle at high loads. As the bending moment is increased the kinking progresses towards the center of the beam until the stress on the opposite side reaches the tensile strength of the wood, and the beam fractures. Stone is much stronger in compression than in tension. Ignoring the effects of buckling, a limestone column of the Temple of Aphaia with a diameter of 0.98 m (38.6 in) could stand more than 3.5 km (2.2 miles) high before crushing under its own weight.[52] In contrast, that same column, if suspended by one end, would have to be only about 3 m (10 ft) long before rupturing due to tensile stress.[53]

For stone beams, two or more beams could be set side by side where greater carrying strength is required. It became common in New Kingdom Egypt and Mycenaean Greece to reduce the load from the structure above by arranging for much of it to be carried down directly to each side of the opening rather then weighing heavily upon the central part of the

[48] A beam resting on its long side is more stable than on its short side because the center of gravity is lower. A frequent problem in light-wood framing construction is the failure of improperly secured beams resting on their short sides (for strength reasons) when a beam falls over and the rest fail progressively in domino-fashion.
[49] Mainstone, p145.
[50] Coulton, p76.
[51] Cotterell and Kamminga, p105.
[52] Cotterell and Kamminga, p105.
[53] Calculated assuming an ultimate tensile stress of 7MPa and a density of 2300 kg/m^3.

Fig. 15: Lion Gate, Mycenae, c.1300. The false arch relieves much of the load above the lintel. (Lawrence)

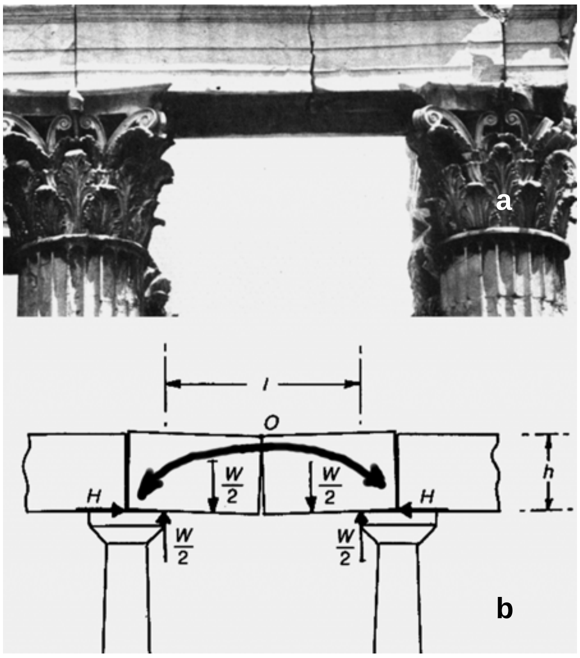


Fig. 16(a): Cracked architrave, temple of Zeus, Athens. (Mainstone)

Fig. 16(b): Force diagram of cracked architrave. (Kamminga and Cotterell)

lintel.[54] Lion Gate, shown in **Figure 15**, is such a structure. The lintel is also tapered so that the greatest depth of material is in the middle where the beam has the highest stress. This reduces the dead load of the beam, thus making it possible to increase the beam's length for the same weight of material or increasing the allowable load.

In general, Greek beams were not highly stressed. The architraves of the Temple of Aphaia spanned a distance of 1.43 m (4.7 ft) between columns; two limestone blocks, 0.85 m (33.5 in) deep by 0.43 m (17 in) wide, composed each architrave.[55] Estimating that each half of the architrave supported a load of 10 tons, the bending moment at their centers would have been 17.5 kNm. The maximum bending stress, calculated from the equation $M_b = \sigma_{max}bh^2/6$, is 0.3 Mpa (43.5 psi), which is only about a fiftieth of the tensile strength of limestone. So it is evident that the architect was well within the limit of the strength of his stone for the architraves. Kamminga and Cotterell conclude the Greeks were "decidedly timid in their approach to stone lintels because they did not understand the mechanics."[56]

1.6.3 Beams to Arches

Provided that sufficient horizontal thrust can be sustained, even a cracked architrave is stable. Earthquakes have cracked many architraves of Greek buildings and yet they have not fallen. In the temple of Zeus in Athens an architrave has fractured and sagged slightly, leaving a wedge-shaped crack. (**Fig. 16(a)**) The thrust line and forces of a cracked architrave are shown in **Figure 16(b)**.[57]

[54] Mainstone, p146.
[55] Cotterell and Kamminga, after Heyman, p114.
[56] Cotterell and Kamminga, p114.
[57] Cotterell and Kamminga, p119-120.

1.7 The Stone Beam in Greek Antiquity

1.7.1 Introduction

In general, the Greeks only used beams of uniform cross-section and the shape of Greek beams was generally governed by aesthetic considerations.[58] However, Greek builders did develop ways of fabricating their beams to achieve certain constructive or structural aims. While the following beam forms are highly interesting, they are not the norm.

During the fifth and early fourth centuries, few Greek buildings were on a colossal scale, and the blocks that they required remained within the lifting capacity of simple cranes. Further development would not have been necessary until the new generation of colossal Ionic temples was begun from the mid-fourth century onwards.[59]

1.7.2 Lightening

Unnecessarily heavy blocks were obviously awkward to quarry, lift and transport. There are several recorded examples of blocks that were lightened by hollowing them out. (**Fig. 17**) The earliest is the lintel of Temple A at Prinias (c. 630 BC), which was hollowed out on top to form a **U**-shaped beam. The architrave of the temple of Apollo at Syracuse (early sixth century) and the lintel of the temple of Dionysos at Naxos (late sixth century) were cut to an **L**-shaped section. There was no structural advantage because the beams must have been filled to their original rectangular section in order to carry the parts above. This means the beams would be weaker relative to a solid section. The purpose must have been to lighten them for lifting or transport.[60]

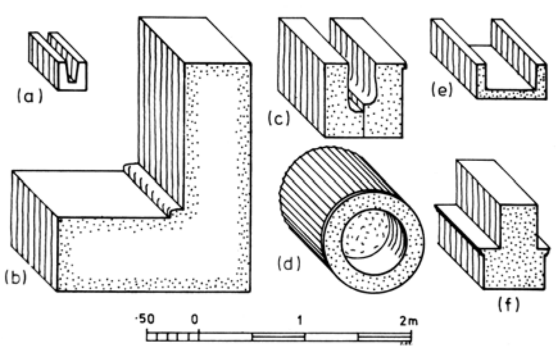


Fig. 17: Various stones lightened by removing material. (Coulton)

The tendency to hollow out blocks in these and similar ways is more common in, though not restricted to, the archaic period. At Delphi, where building stone was often imported by sea from a considerable distance and the site lies at about 550 m above sea level. In the temple of Apollo at Bassai (c. 430-400 BC) there are **U**-shaped ceiling beams. The weight of the blocks before they were hollowed out, about 2.4 tons, would have presented no difficulty in lifting and there is no structural advantage, for the stone removed reduced the weight and the strength of the beams about equally, leaving the maximum stress almost unchanged. While the main structure of the temple is local limestone, the ceiling beams are said to be of Parian marble, which would have had to been transported more than 400 km by sea, then hauled 22 km overland to a height of about 1130 m above sea-level. Hollowing out the ceiling beams at the quarry, and so reducing their weight by about half, could substantially reduce the cost of this expensive operation.[61]

[58] Cotterell and Kamminga, p111.
[59] Coulton, p144.
[60] Coulton, p145-146.
[61] Coulton, p146.

The fact that the beams were lightened in this way suggests that some Greek architects realized that the stone was often substantially under-stressed, and that their beams were dimensioned primarily on aesthetic grounds. It is never the visible front and lower faces that are cut away, and with the **U**-section beams the rear face is also unmodified. Thus these beams are not really comparable with modern steel beams, although they may have similar sections. The section of a steel beam is calculated to give the maximum strength for the minimum amount of material, while with the Greek stone beams no material is saved by giving them an **L**- or **U**-shaped section, for the material cut away is useless chips; nor does there appear to be any significant gain in strength.[62]

1.7.3 Increasing Beam Depth

The depth of the typical Greek stone beam is governed by the aesthetic proportion of the architrave, which often resulted in beams that were wider than they were tall – an orientation found in early timber construction. However, there are instances where the structural depth of the beams was increased, thus increasing their strength.

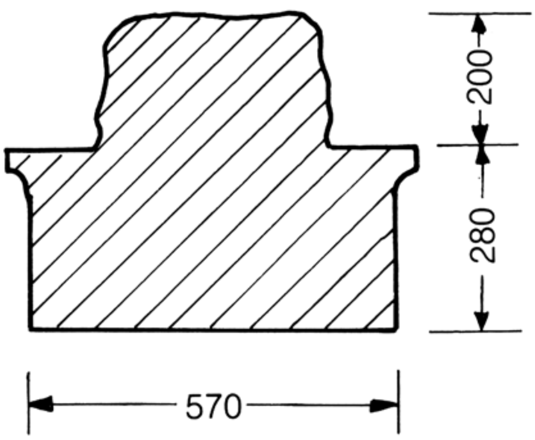


Fig. 18: Roof beam with strengthening rib. Not known if on purpose or simply to minimize work to dress the stone. (Kamminga and Cotterell)

The first positive development came in the construction of the architraves. Some early architraves consisted of two courses. The first course was a roughly square main beam that carried the second course of decorative details (*taeneia, regulae* and *guttae*) laid above it. In later buildings, when the architrave consisted of more than one block, it was always divided in breadth not in height, so that each block was higher than it was broad, and door lintels were built up on the same principle.[63]

There are examples of some stone beams carrying coffered ceilings that had appreciable ribs that would be unseen but would increase the depth of the beam, thus making the beam stronger. (**Fig. 18**). In some cases the rib is only a few centimeters high, and may just be the remains of the mantle of stone normally allowed by the quarrymen all round a block, and here left unworked because it came between the beds for the coffer slabs.[64]

The existence of these ribs suggests that some Greek builders were aware that the strength of a beam lies more in its height than in its breadth. However, this conclusion is belied by the observation that the shape of the visible beams in a Greek temple depended more on visual than structural considerations. According to Coulton,

> The true answer should be given by the shape of wooden roof-beams, which, being hidden, should have been dimensioned without regard to appearance. The cuttings for such beams suggest that they were usually roughly square in section (which would mean cutting away as little wood as possible in forming

[62] Coulton, p147.
[63] Coulton, p148.
[64] Coulton, p147.

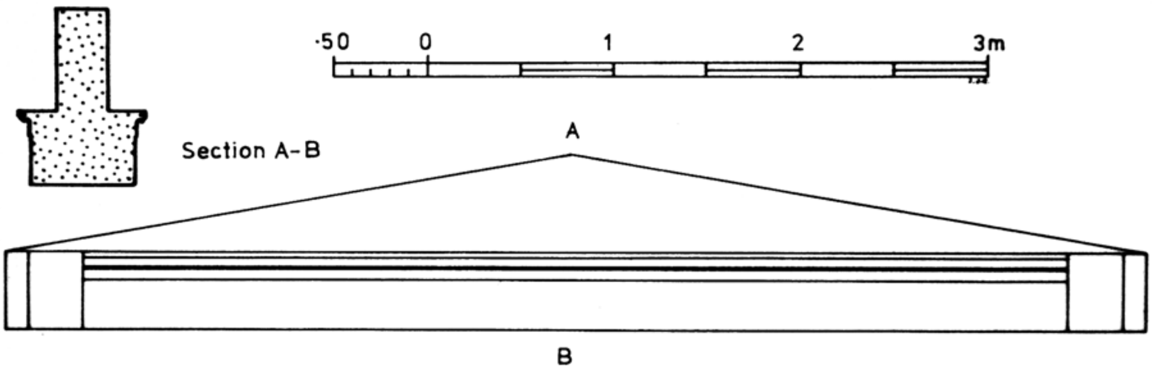


Fig. 19: Roof beam of variable section, Hieron at Samothrace (late 4th Century). Section and elevation. (Coulton)

> them from a circular log); sometimes they were laid with their height greater than their breadth, but more often with their breadth greater then their height, perhaps because that was judged a more stable position. Certainly the arrangement of roof woodwork provides no strong evidence for a systematic appreciation of this elementary principle, that the height of a beam is more important than its breadth.[65]

A notable exception to the above analysis is the Hieron at Samothrace, built in the fourth century BC by Mnesikles. The beams of the Hieron's vestibule, which carried a marble ceiling across a span of 6 m, all had hidden ribs that tapered from 0.5 m high at the center, where the bending moment was greatest, to nothing at either end. **(Fig. 19)** It is difficult not to conclude that the architect of the Hieron had some knowledge of the mechanics of beams and deliberately tapered the depth of the rib in order to minimize the weight of the beam while still preserving its strength.[66] I think that the architect probably observed that when beams broke they usually broke in the middle, therefore he may have reasonably concluded that if he put more material where the beam tended to break it would not break there anymore. Alternatively, the beam was simply cut to the slope of the roof, as is evidenced by the similarity between it and the pediment shown in Figure 14. Nonetheless, Mnesikles's beam seems to have been an anomaly whose radical form was not reproduced elsewhere.

1.7.4 The Cantilever

Another interesting development was that of the use of the cantilever to effectively reduce the stress in the architrave in the Parthenon (447-432 BC) The cantilever was variously used by the Greeks to support balconies, stairways and statues[67], but the idea of how to employ the cantilever in the construction of the Parthenon was much more complex.

Coulton writes,

> In the Parthenon, iron bars were built into the wall of the pediment to carry the main pediment statues, so relieving the cornice of their weight. However, the effectiveness of the slabs acting as counterweights is reduced by hollowing them out behind, so that for most of their height they are only about 0.25 m

[65] Coulton, p148.
[66] Cotterell and Kamminga, p111-112.
[67] Coulton, p149.

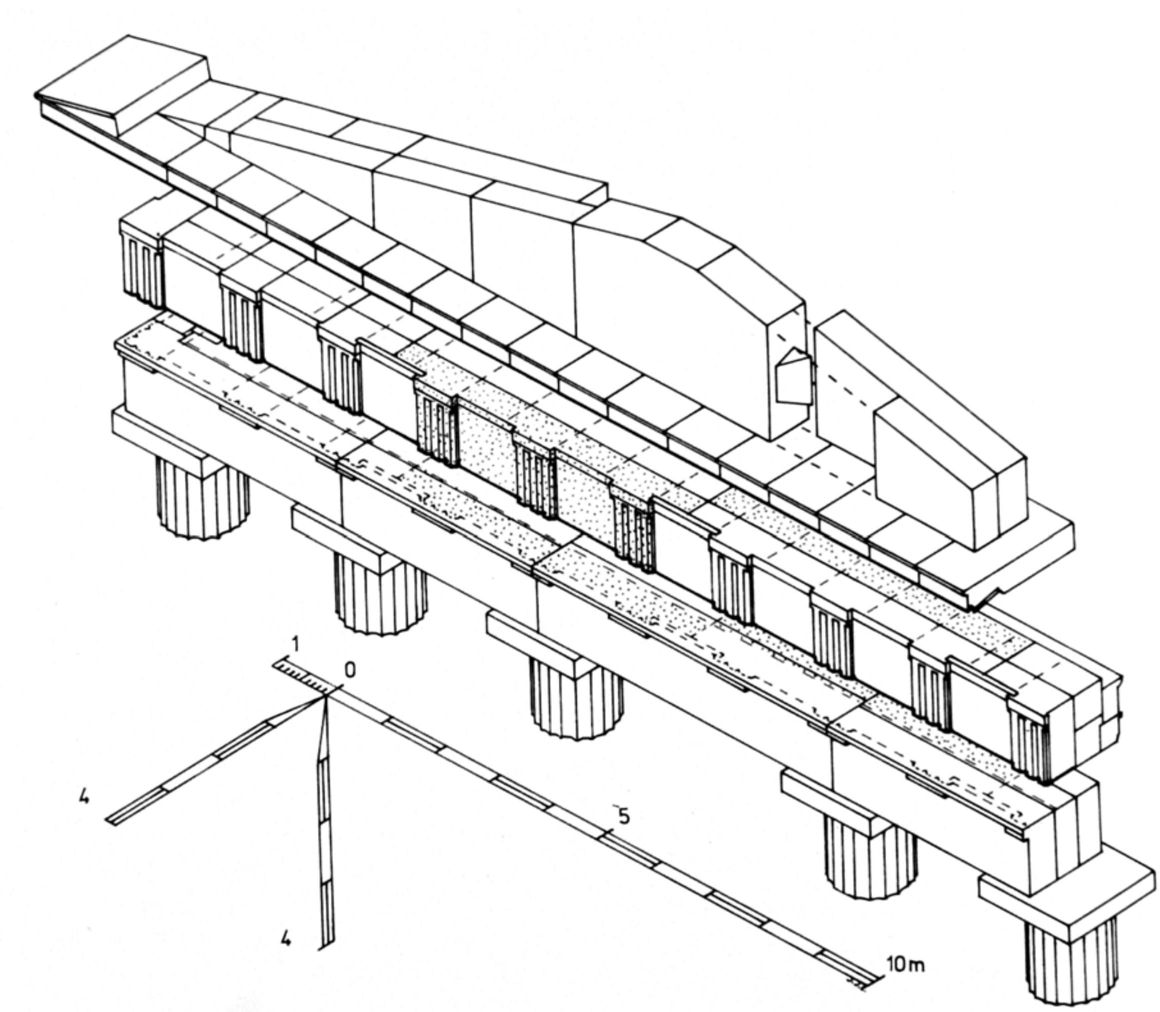


Fig. 20: Cantilevered frieze beams, the Propylaia, Athens, c.437-432 BC. (Coulton)

> thick. Iron bars were bent up behind the slabs, so that in order to let the statues down they would have to force the slabs forward as well as upward, and that was prevented by clamping the slabs firmly to the rear part of the pediment wall. It would have been simpler, however, not to have hollowed out the pediment slabs at all, and it seems likely that the operation of the cantilevers was not fully worked out beforehand.[68]

In other instances, Greek architects used the double cantilever beam, arranged so that the loads on the two arms balance. They used this device as a means of relieving the strain on an architrave in monumental architecture. In order to relieve the central architrave of the main east and west façades in the Propylaia at Athens, Mnesikles arranged the frieze above them in long blocks of triglyph-metope-triglyph-metope-triglyph, with the middle triglyph of each block coming directly over the column on one side of the wide span. (**Fig. 20**) The central architrave thus carried only the thin marble slab forming the metope required to mask the joints between the two long frieze blocks. This suggests that Mnesikles had a clear understanding of the operation of his cantilever.[69]

[68] Coulton, p149-151.
[69] Coulton, p151.

1.7.5 Structural Iron in Greek Temples

Besides the use of iron clamps and pins, there are various claims of larger iron structural work being employed by the Greeks. Most of these claims are based on the book by Dinsmoor, which have been somewhat unquestioningly accepted by many authors as fact. Nevertheless, there is enough evidence to consider that his claims may be true to some extent. One claim of Dinsmoor's is that large iron tie beams were incorporated into the foundations of the Thebian Treasury at Delphi. Two claims that are of great interest to the subject of the stone beam, are about an iron transfer beam used at Propylia and the use of iron as a type of under-spanning or reinforcement in the Temple of Zeus at Acragas.

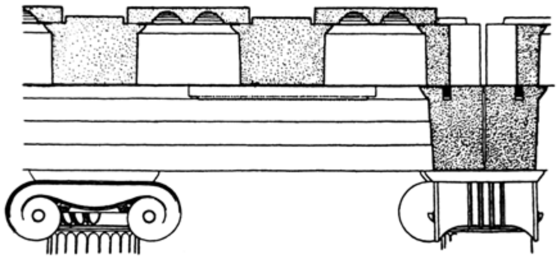


Fig. 21: Iron relief beam shown by dashed line and seen in section of archtirave. Propylaia, Athens. (c.437-432 BC) (Dinsmoor)

1.7.6 Iron Transfer Beam at Propylia

In the west porch of the central building of Propylia (437-432 BC), built by Mnesikles, two rows of Ionic columns support the marble ceiling. The ceiling beams are arranged so that one comes over each column and one at the mid-span of each architrave. Since each beam, with its share of ceiling, weighs over 10 tons, Mnesikles decided to reinforce the architraves with iron bars let into their top face; but the cuttings for the iron bars stop short about 0.90 m (35 in) from each end of the architrave. (**Fig. 21**) If the iron bar worked as intended, the load would not fall on the center of the architrave, but would be transferred to points just in from the supporting columns; thus only the vulnerable central part of the architrave was protected. By choosing to use the shorter bar, the section of the bar could be smaller, making it easier to forge.[70]

The bar was meant to perform as a simple beam, reflecting a recognition of that material's superior strength, though there was no attempt to exploit the high tensile strength of iron,

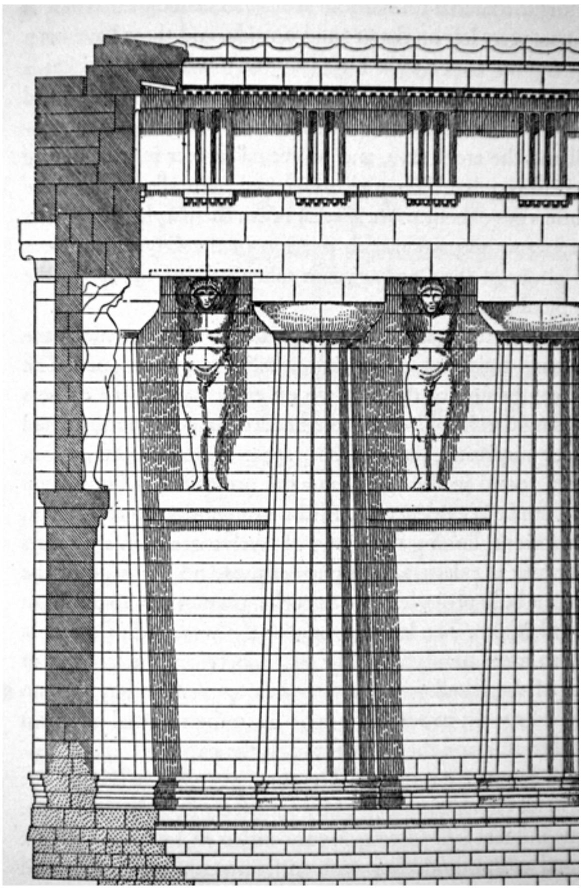


Fig. 22: Iron relief beam on underside of architrave, noted by dashed line. Note support provided by arms of statues. The Olympieum at Akragas, c.500-460 BC. (Lawrence)

[70] Coulton, p148-149.

otherwise the bar would have been placed on the bottom face of the architrave, anchored at the ends.[71] Treating the load applied by the ceiling beam as a central load, the bending moment at the center of each beam would be 46 kN·m , producing a bending stress of 0.7 MPa (101.5 psi), which is quite acceptable. Evidently, Mnesikles did not trust the strength of his marble beams. By adding the iron bars Mnesikles halved the maximum bending moment, thereby reducing the bending stress in the marble to 0.3 MPa (43.5 psi). In the course of time the iron bars rusted, allowing the ceiling beams to rest directly on the architraves, some of which are now fractured, most likely by earthquake and not overload[72]

1.7.7 Iron Relief Beam

In the gigantic temple of Zeus at Akragas (500-460 BC) there are cuttings in the under-face of some architrave blocks that may have held iron bars running from capital to capital. (**Fig. 22**) The cuttings show that the ends of each bar were not anchored, so that the bars were again regarded as simple beams with their tensile strength unexploited. At Akragas the bars were probably envisaged as supporting the architrave itself, which was here built up like a wall out of three courses of comparatively small blocks, while in the Propylaia they were in the top face because they were envisaged as supporting the ceiling beams.[73]

The architraves in the temple of Zeus are made up of 27 blocks, the largest weighing only about 14 tons. Since the architraves between the half-columns projected forward from the screen-wall, and there was a joint between blocks in the center of each span, some direct support seemed necessary between the columns; giant male figures were therefore placed on a ledge of the screen wall, carrying the architrave on their upraised forearms.[74]

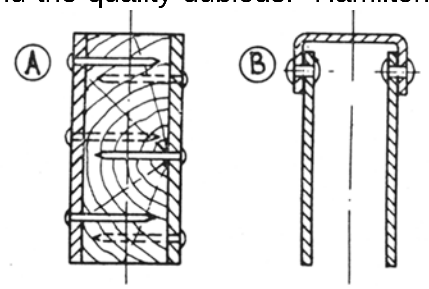
This method of construction solved many of the technical problems associated with colossal scale. The smaller blocks would make quarrying and transport simpler, thus reducing cost. The problem of raising massive blocks to considerable height would also be much reduced. The huge architraves of the temple of Apollo at Selinous would have been too heavy for any but an elaborate type of crane, so they were probably dragged up temporary earth ramps. The largest blocks of the temple of Zeus could be lifted by two fairly simple cranes. Almost all of the blocks have **U**-shaped grooves at each end, like those seen in **Figure 8** (top-right), to take loops of rope which could be attached to cranes.[75]

The supposition that the grooves were meant to reinforce the architrave was first posited by Dinsmoor and stated as fact in the 1950 edition of his book.[76] He records the dimensions of these iron 'beams' as having been 4.6 to 4.9 m (15 or 16 feet) long and as much as 12.7 cm (5 in) wide and 30.5 cm(12 in) high.[77] The interpretation of the cuttings at Akragas has been doubted, because the huge figures between the half columns would appear to have supported the architrave blocks effectively on their upraised elbows, without the iron bars.

[71] Coulton, p149.
[72] Cotterell and Kamminga, p114.
[73] Coulton, p149.
[74] Coulton, p82-84.
[75] Coulton, p84.
[76] There is no mention of these beams in Dinsmoor's 1927 edition of *The Architecture of Ancient Greece*, but are stated as fact in the 1950 edition. Dinsmoor first presented these ideas in two articles: W.B. Dinsmoor in *Bulletin de Correspondence Héllenique*; Ecole Français d'Athènes 1912, pp. 453-5; W.B. Dinsmoor, "Structural iron in Greek architecture," *Amer. Jour. of Architecture*, 1922, 2nd Ser. XXVI (2), pp. 148-158.
[77] Dinsmoor, p104-105.

However, the unprecedented system of building employed in this temple may have led the architect of this temple to take all possible precautions to ensure its stability. Coulton also cites another instance in which an iron bar has been found intact in the bottom face of a lintel of a minor doorway in the Erechtheion at Athens (c. 420-405 BC), though its ends are invisible so it is not possible to say how it was meant to work.[78]

Hamilton refutes Dinsmoor's claims, but only in part. Hamilton calculated that each beam would have weighed 660 kg (1455 lbs), and there were at least thirty-eight of them. But all that remains of the 25 tons of iron, and of the industrial undertaking of forging it into beams, is a few rust stained grooves in stones scattered over the ground.[79] The labor required to fabricate such beams would have been enormous and the quality dubious. Hamilton posits that if these grooves did indeed incorporate relieving beams of some sort, then they could more plausibly have been of the Flitch or compound iron beam types illustrated in **Figure 23**.[80] The function of whatever filled these grooves is not clear. Nor is it clear what were the form and materials used. Based on the examples at Propylia and in the Erechtheion, we could assume that housing a relief beam of some sort was the intended purpose of these grooves and that, like at Propylia, they were probably unnecessary.



A. FLITCH BEAM; IRON PLATES NAILED TO WOOD
B. COMPOUND IRON BEAM

Fig. 23: Alternative beam forms that could have been used in the Olympieum at Akragras. (Hamilton)

1.8 The Arch

1.8.1 Origins of the Arch

The longest natural stone arch in the world is in Utah (USA), and has a span of nearly 90 m (295 ft),[81] but there are no naturally occurring monolithic arches to be found in any of the areas where the man-made arch first appeared. It is possible that some type of accidentally occurring arches could have been observed, such as a wedged boulder, or two pieces of stone or timber inclined inwards to meet as an inverted V.[82]

A prototype to the arch is an arch made by bending two saplings or bundles of reeds until their free ends meet. In the marshes of southern Iraq, for instance, roughly parabolic 'arches' up to 6 m in span have, at least until recently, been made by setting bundles of giant reeds in the ground in two rows spaced that far apart, then bending their heads to meet one another and tying them together. Their action is not, of course, the purely compressive one characteristic of pure arch action.[83] The structural principle here is not as important as the physical form and its effect on the thinking of builders about constructed form.

78 Coulton, p149.
79 Hamilton, p35.
80 Hamilton, p36.
81 Cotterell and Kamminga, p119.
82 Mainstone, p98.
83 Mainstone, p98.

The arch dates to the third millennium BC in Mesopotamia and Egypt where it was built of brick.[84] The earliest known true arches of mud-brick have survived in Egypt from early in the third millennium BC. They were constructed of only a few bricks, very likely by the process observed by Somers Clarke in Upper Egypt in 1896 for constructing small window arches without using any centering:

> The walls being raised to the necessary height from which the window arches should spring, a little pile of four or five bricks was set up on either jamb, with plenty of tough mortar between like dough. One master craftsman manipulated one little pile, one the other. Each put a hand on the inner side of the pile and tilted it sedately and slowly, giving it a curved form as it moved until the two touched at the top. A cry, 'One more brick!' – in it went, and the little arch was complete.[85]

1.8.2 The Corbelled Arch

Most primitive arch form is the corbelled arch, which does not actually behave as a true arch, but rather a series of cantilevers. The corbelled arch is constructed form horizontal courses of brick or stone. Each course slightly overlaps the one below it until the two sides meet. (**Fig. 24**) Corbelled arches were usually built without any centering and therefore had to have steep sides to stay stable during erection.[86] It must have been recognized – or soon discovered in the course of building – that no block could project more than half its length beyond the one below if it were not to tip over and fall, and that a considerably smaller projection was safer. It must similarly have been recognized that the total weight of masonry bearing down behind the edge of the opening must always exceed the total weight that projected in front of it.[87]

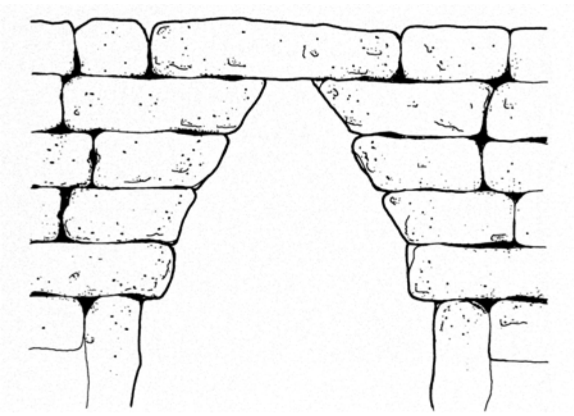


Fig. 24: Corbelled arch. (Mark)

The chief merit of these early forms was that they could be constructed with little or no centering or other temporary support beyond what might be desirable to facilitate the handling of the larger blocks. Their chief shortcomings were the limited spans that could be built and the massive abutment necessary for corbelled arches to balance the weight of the cantilevered stones.[88]

[84] Cotterell and Kamminga, p119.
[85] Mainstone, p98-99.
[86] Cotterell and Kamminga, p119.
[87] Mainstone, p99.
[88] Mainstone, p100.

1.8.3 The True Arch

The optimum form of a true arch can be determined by the catenary curve, which is very close to being parabolic. The semi-circular form of the true arch, characteristic of antiquity, is an inefficient form that must accommodate the true force curve in an arch that is thicker than it needs to be for strength requirements. The development of the true arch, the first type probably being that described by Somers Clarke above, seems to have been closely linked with that of its longitudinal extension, the barrel vault. In the type of vault illustrated in **Figures 25(a) and 25(b)**,an end wall served in lieu of centering while the first few incomplete rings of bricks were set on edge leaning back against it. As the vault advanced away from this end wall, each fresh ring leant back against the previous one and temporarily gained such support as it needed from it. Where a greater depth than that of a single brick was required, further rings were built above the first one.[89]

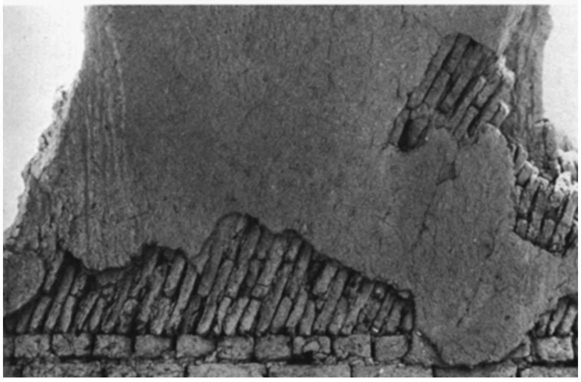


Fig. 25(a): Vault of storeroom. Ramesseum, Thebes. (Mainstone)

Fig. 25(b): Detail of brick constructed vault. (Mainstone)

Despite numerous prototypes, the stone voussoir-arch was not adopted by the Egyptians very quickly. They preferred massive lintels, the inverted-V form and the corbelled arch. The reason cannot have been arguments of aesthetics. Curved forms were often given to corbelled arches and inverted-V structures, at least from the Middle Kingdom onwards. The reason was probably that nothing would have been gained in the overall economy of construction. Handling much smaller blocks of stone would have been easier, but the skill of the builders would have had to been higher, foundation issues would have become more complex, and there were limited timber stocks locally available.[90]

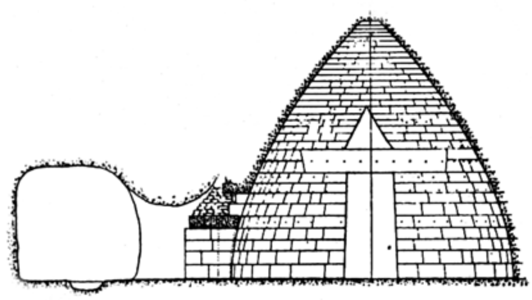


Fig. 26: Section and plan of 'Treasury of Atreus, Mycenae, c.1300 BC. Corbelled dome has a span of 6 meters. (Lawrence)

[89] Mainstone, p100.
[90] Mainstone, p101.

1.8.4 Applications of Corbelled and True Arches in Ancient Greece

Corbelled tombs in a masonry technique characteristic of Greek masons are found in areas such as Thessaly, Macedonia, Thrace, Asia Minor and Etruria dating from the Bronze Age.[91] The Greeks used corbelling more commonly from the late fifth century onwards, largely because the kind of public works to which it was suited began to be constructed on a more elaborate scale. Many spans were between three and five meters, with the corbelled dome of the Treasury of Atreas (1330-1300 BC) reaching a diameter of 6 m (20 ft). (**Fig. 26**) No other corbelled or arch structure approach the spans of the corbelled domes of Mycenaean *tholos* tombs which are up to 14.40 m (47.2 ft) in diameter.[92]

Concerning the true arch, we know that the Egyptians constructed true type arches with brick, though they did not use such forms prominently in the construction of their great tombs and monuments. Given the relations Greece had with Egypt from 660 BC onwards, it is likely the Greeks would have learned of the arch from them. In fact, the voussoir arch did begin to appear in Greece around the fourth century BC.[93] The emergence of the true arch in Greece was probably facilitated by the availability of timber for centering. In those rare instances when the Greeks did employ the true arch, it was usually used to replace the corbelled arch, not the massive lintel.

The form of the true arch used by the Greeks was semi-circular, like that of the Egyptians before and the Romans later. They probably chose the semicircular section because it was easy to set out. The widest known Greek archway, part of a tomb at Leukadia, spans only 7.35 m (24.1 ft). The full superiority of the true arch over corbelling was certainly never exploited. There is no evidence that the Greeks built any arch spans that approach the spans achieved by Roman builders, such as the 24.5 m (80.4 ft) span of the Bridge of Fagricius at Rome (62 BC). An example of Greek arch construction is the vault in the Temple of Apollo at Didyma (c.300 BC), shown in **Figure 27.** Corbelling may have been preferred over the arch for two practical reasons: centering or falsework is not a necessity; it is easier to execute in accurate, dry-laid masonry, for all the blocks are cut with their two main faces parallel. The faces of a voussoir arch are angled.[94]

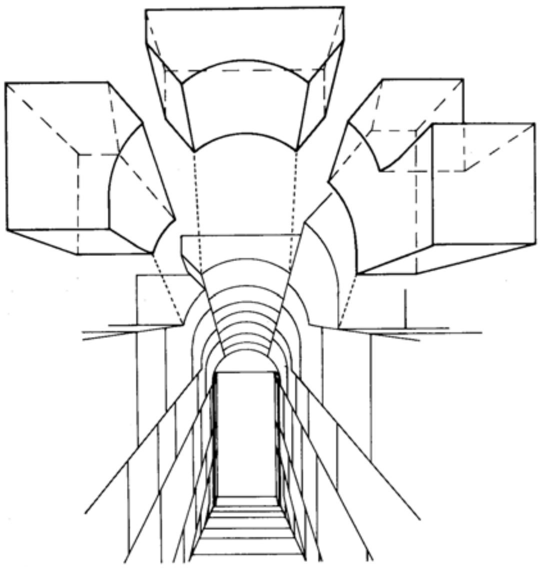


Fig. 27: Sloping barrel vault above ramp to altar court. Temple of Apollo at Didyma, c.300 BC. (Coulton)

[91] Coulton, p152.
[92] Coulton, p152-153.
[93] Mainstone, p101.
[94] Coulton, p153-154; note 49, p180.

1.8.5 Why the Arch was not Used – Cultural Tradition, Established Form

The conventional forms of the Greek temple were established at an early date, and its function – it was primarily a symbol of the sanctuary and a shelter for the god's statue – remained unchanged from start to finish. It seems to me that there could be no claim that the Greeks were timid about their abilities to cut stone accurately in the later period of their development, but the formal architectural language of their temples had been firmly set, and this did not include arches. A vault across the cella, where the simple post-and-lintel system was restrictive, would generate lateral thrusts that would have required visible buttressing. The reason why arches are not found in other building types, lies in the fact that the prestige of temple building was so great, even in the Hellenistic period, that its forms were as far as possible transferred to all other building types. The structural system associated with those forms was naturally transferred as well.[95]

1.9 Conclusions

While the evidence is not conclusive, the proposition that the Greeks translated the architectural details of traditional wood construction into stone seems valid. The preceding review of this development also reveals that it is difficult to argue that the form of the structural members in either wood or stone was dictated by structural reasons. It does not appear that the Greeks exploited the potential structural capability of either material. In the transition from wood to stone, there is no appreciable recognition of the distinctly different structural characteristics of stone. A better analysis would be that the forms were governed by constructabilty issues, particularly with respect to stability during construction and limiting the amount of work necessary to fashion the building components of the traditional wood architecture. Once these proportions were established, they became the formalistic model for the aesthetic system, from which the Doric and Ionic Orders were derived. The fundamental form of Greek monumental construction did not change significantly because it became culturally important to maintain a consistent image of what a temple is.

The Greeks were conservative in their use of stone for beams. Save for a few isolated cases of inventiveness, such as seen in the work of Mnesikles, there does not seem to have been much advancement in the knowledge of the strength of materials and what makes a beam break. The cases described above about the beam of variable depth and the use of iron relieving beams appear not to have had any appreciable effect on the Greek building culture as a whole.

The Greeks probably did not use the arch more widely because of the conservative nature of their building culture. In addition, the formal language of Greek temple design was established by the time the arch was introduced. This language precluded the use of curved forms such as the arch or vault.

Coulton explains that the limited development of building technology in Greek society can be attributed to a culture that valued intellectual pursuits of a theoretical nature over what were deemed 'practical matters'. Because the fundamental building form did not evolve, there was clearly no impetus toward developing more efficient systems of construction. However,

[95] Coulton, p140.

Greek builders did admirably address questions of scale within the same framework by significantly changing the details of their construction. Their primary achievement was to use smaller blocks to make larger temples, which simplified the logistics of handling the stones and thereby kept the amount of work and resources necessary to build such structures within acceptable limits. One may presume that if they had applied their minds to the technological problems of construction and loosed themselves from the tight constraints of their system of design, they would have advanced more rapidly.

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TELFORD'S MENAI SUSPENSION BRIDGE

A-02

2.1 Introduction

The Menaï Suspension Bridge, designed and built by the Scottish engineer Thomas Telford, was the longest bridge in the world when completed in 1826. It had a center span of 170.7 meters (560 ft). The process to complete this bridge laid the foundation for the beginning of a new bridge building era based on principles of suspension.

In 1814, Telford first proposed a suspension bridge for another project, the Runcorn Bridge. He proposed a bridge with an ambitious total span of 609.6 m (2000 ft) and a center span of 305.1 m (1000 ft). Notably, Telford proposed the first parallel wire cable, but this is not the most enduring legacy of the Runcorn Bridge. Even thought the Runcorn Bridge was never built, its design sparked widespread interest in suspension bridges in Great Britain. As a result, a number of small-scale bridges were built as a result. Additionally, Telford's experiments with the characteristics of wrought-iron wires, bars and cables contributed significantly to strength of materials knowledge.

Although Telford was aware that suspension bridges had already been built in China, South America, and, most recently, in the United States, he had little practical knowledge of their construction. Therefore, Telford's development of the chain-cable was independent of any precedent. In the course of designing the cables for the Runcorn Bridge, Telford interacted with Samuel Brown, a Scottish engineer, who had developed a linked bar-cable. Brown's work subsequently influenced Telford's development, but Telford made some key improvements and his design endured as the basis of all subsequent bar-type suspension cables. The French studied the works of Brown and Telford, and used this knowledge in the development of the wire-cable suspension bridge. Thus, Telford's work had a far-reaching influence on the overall development of suspension bridges.

This chapter reviews what developments had been made in the construction of suspension bridges before Telford began to develop his own. It will briefly review Telford's life, focusing on those aspects that could have influenced his thinking about the design and development of suspension bridges. Thomas Telford was a mason by trade, so it is interesting to consider how he acquired the knowledge to evolve from building purely compressive structures in stone to a tensile structure in which he applied wrought iron, a very different material that had been little used at the time. Finally, this chapter will examine how Telford developed an idea for a suspended center for constructing a cast-iron arch into one for a variety of suspension cable designs. This culminated in the Menaï Suspension Bridge's eye-bar chain-cable design.

2.2 Early Non-Western Suspension Bridge Development

2.2.1 The Earliest Suspension Bridges

It is presumed that the earliest known suspension structures stood in the Himalayas because of the number and diversity of suspension type bridges in that region. Suspension bridges also became common in western South America.[1] The earliest suspension bridge cables were certainly made of readily available, natural materials such as cane, bamboo, hemp, animal hide and vines. Vines, as such found in South America, could probably have been initially employed as found, with spans gradually increasing as the skill of weaving various vines together to form stronger and longer cables developed. **Figure 1** shows a recent bridge constructed of vines and reinforced with a rope.

Fig. 1: Three-cable suspension bridge manufactured using vines. (Brown)

It cannot be said with any certainty when the earliest suspension bridges were constructed due to the perishable nature of the materials first employed. Based on what is known about the age of certain bamboo and iron-chain cable bridges in China, the earliest suspension bridges of the most primitive kind could date before the ninth century BC.[2]

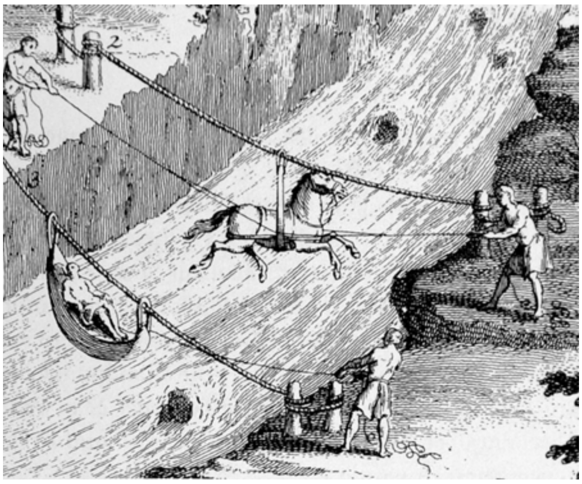

Fig. 2: Single rope suspension bridge showing system of moving harness from one side to another. (Brown)

These earliest suspension bridges probably consisted of one, two, or three cables, each requiring one cable to support the principal load. The hand-over-fist method of pulling oneself across a rope bridge while the person's legs are crossed around the rope might have been used initially. But more advanced systems evolved. In an account given by Don Antonio de Ulloa on bridges found in South America in the 18th century, he described the cables as being made of several bujucos, a vine plant native to South America, twisted together to form the length required. A wheel was used on one side to properly tension the cable. To cross the bridge, a

Fig. 3: Basic three-rope suspension bridge. (Brown)

[1] Peters[2], p13 and 14.
[2] Peters[2], p16.

Fig. 4: Quan-Xian Bridge, with bamboo suspension cables. (Peters[2], after Tang)

person or cargo was put in a leather sling suspended from the main cable and attached to two ropes. The ropes were used to pull the sling to the other side.[3] (**Fig. 2**) In the two-cable configuration, the second cable would be above the other. It provided something to hold on to while traversing. Two cables in the three-cable configuration acted as a type of handrail, such as seen in **Figure 3**.[4]

The three-cable variant can be augmented with more cables, creating a V- or U-shape. (**Fig. 1**) Suspension bridges of this form are typically found in Tibet. In all Chinese examples of this type, the cables were made of cane. This bridge form evolved to eventually be composed of cables laid side by side with planking fastened across them and two or more lateral cables hung as guide rails. The first recorded bridge of this type was built as early as 285 BC in the Province of Sichuan.[5]

2.2.2 Cane and Bamboo Cables

Cane and bamboo cable are stronger than hemp ropes of the same diameter. Marco Polo, the Venetian explorer, noted their strength saying,

> They have canes of the length of fifteen paces, and these, by twisting them together, they form into ropes three hundred paces long. So skillfully are they manufactured, that they are equal in strength to ropes made of hemp.[6]

Peters writes,

> In the later bamboo ropes, the tensile strength could be augmented by a careful choice of material for the various layers. Splints from the inner part of the culm formed the core of the rope around which a cover of tightly braided strips from the outer, harder silicon-rich layer, was woven. These strips were highly resistant to abrasion. The covering was made of approximately 5 meter-long [16.4 ft] splints which were spliced simply by overlapping the ends. No two splices were allowed to occur at the same position in the rope. The braided cover was flexible and could yield slightly under sudden tugs on the rope. This was a great advantage for tensile resistance as, the tighter the rope was pulled, the tighter the cover gripped the core. The force was thus transferred to all fibers equally, an important consideration as uneven distribution of stress would have caused overloading of some of the fibers and consequently more failures.[7]

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[3] Telford[2], p480.
[4] Peters[2], p14.
[5] Peters[2], p14.
[6] Peters[2], p14.
[7] Peters[2], p14-15.

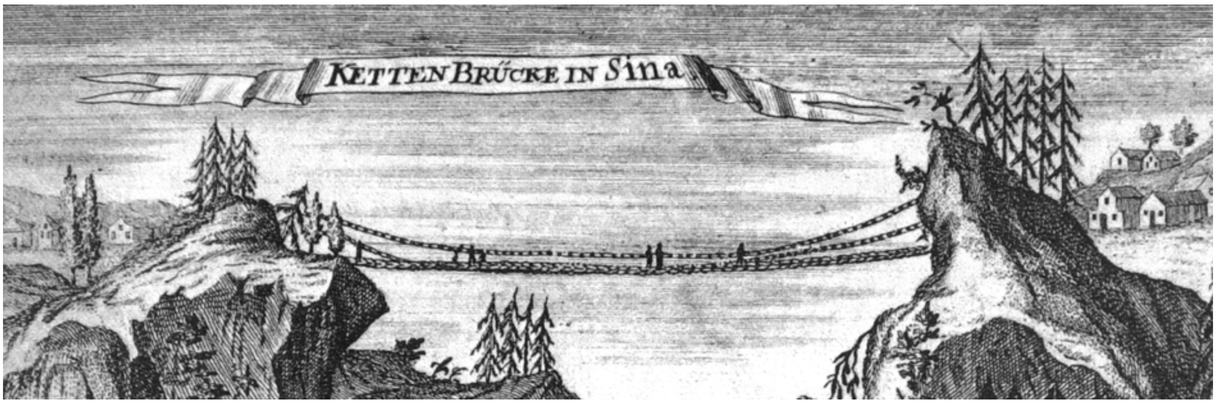

Fig. 5: Lan Jing Bridge with iron-link chain-cable in Yunnan, China. First illustrated in Kircher, 1667. (Peters[2], after Schramm)

Figure 4 shows the Quan-Xian Bridge in China. This bridge is comprised of multiple bamboo cables hung side-by-side to construct the traversing surface, and another ten cables comprise the handrails. This bridge was first mentioned in the third century BC, and is shown in its original condition before the cables were replaced with wire ones in 1976. In the tenth century AD, the bridge had five spans for a total length of 380 m (1247 ft). Today it measures 330 m (1083 ft) and has ten spans. The largest span is 61 m (200 ft). Over the centuries, the traversing cables were continually replaced. The old cables were used to replace the cables of the handrails.[8]

2.2.3 The Wrought Iron Chain

The origin of the iron chain in construction is just as obscure as that of the bamboo cable. In China, iron chains were first mentioned in documents of the 'Spring and Autumn' Period (770-476 BC), where records mention that 3609 iron mines were then in use.[9]

Tom F. Peters, a historian of building technology, writes,

> The high standard of Chinese metallurgy, particularly in the mountainous regions of Sichuan and Yunnan, with their abundance of high-grade iron ore, favored the emergence of the iron chain bridge. It soon spread throughout the Himalayan Range. Travelogues dating back to 519 and 646 AD bespeak the existence of many such structures on the pilgrim road from China to India. As with the bamboo cable bridges, the chain ones were also single spans with few exceptions.[10]

The earliest chain bridge, called Lan-Jin, is traditionally supposed to have been erected at Jing Dong in Yunnan in the year 65 AD[11]. **Figure 5** shows an image of this bridge first published by Kircher in 1667. This image was widely reproduced, including in an English travelogue by Turner in 1800.[12]

[8] Peters[2], p15-16.
[9] Peters[2], p16.
[10] Peters[2], p17. The Luding Bridge over the Tatu River, had a record span of 100 m when it was built in 1703, which would have been the largest span of its time.
[11] Traces of a chain bridge discovered at Liuba may predate the Lan-Jin Bridge. Peters[2], p21.
[12] Peters[2], p21.

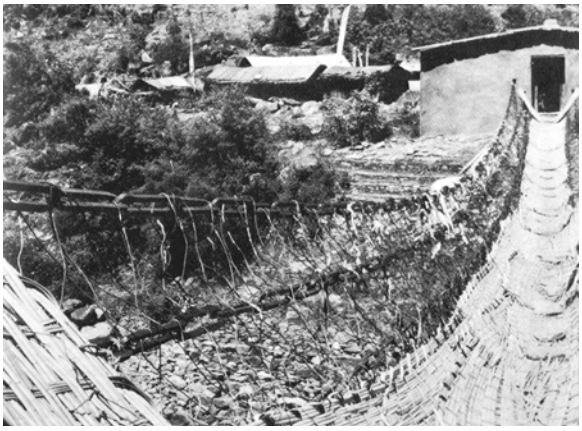

Fig. 6: Docsum Sempa Bridge, with iron-link chain-cables. (Peters[2], after von Schulthess)

Fig. 7: Iron chain at Wangdiphograng, Bhutan. (Peters[2] after von Schulthess)

2.2.4 Thang-stong rGyal-po

A noted builder of wrought-iron chain bridges was the Tibetan monk, Thang-stong rGyal-po (1385-1464). Most chain bridges in Bhutan and Tibet are traditionally attributed to him. Many chain bridges of the type he erected have been in daily use in that region for 550 years. The best documented is the Tamchugang Bridge over the Paro River. The deck rested on parallel chains. Additional chains formed the railings.[13] **Figure 6** shows a the Docsum Sempa Bridge over the confluence of the Gongri and Yangtse Rivers, Bhutan. This bridge was originally erected by Thang-stong in the 15th century AD. **Figure 7** shows a detail of a chain left abandoned at Wangdiphodrang, Bhutan. Thang-stong gave up trying to build a bridge there after the chains had 'been thrown down' several times. Necking of the links indicates they were probably over-stressed.[14]

2.2.5 Eye-Bar Chain Bridges in China

The eye-bar differs from the normal chain link in that it is made of a single bar, not closed in a loop. Its ends are either bent to form 'eyes' at either end, or flattened and punched with holes. There were many eye-bar chain bridges in China. The Kiai Tsu Chang bridge built in the eighteenth century, still stands on the former Sichuan-Xi Kang border. Its chains are of round bars bent to form eyes, and welded at the ends.[15]

2.2.6 Influence of Non-Western Development in the West

There is no evidence that any practical ideas were transposed from Asia or South America to the West. All references by Telford, and other authors in England at the beginning of the 19th century, treat the existence of such bridges only as curiosities. They assure themselves that English engineering, ingenuity and iron-making skills can do a superior job regardless of what has been done in the past.[16]

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[13] Peters[2], p18-19. The Tamchugang Bridge was last photographed in 1967 before it collapsed during a storm the following year.
[14] Peters[2], p19-20.
[15] Peters[2], p21.
[16] Cumming; Telford[2].

2.3 Early Western Suspension Bridge Development

2.3.1 The Earliest Iron Chains in Europe

A small bridge on the Gotthard Pass route in Switzerland is the first western bridge recorded to have been suspended by an iron chain. This bridge was built around 1218. An iron votive chain that was constructed over the southern French town of Moustier Sainte-Marie in the Alpes de Haute Provence is traditionally dated to that same century. That chain still spans 200 m (656 ft) between two crags. It is made of eye-bar links, 65 cm (25.6 in) long and 20 mm (0.8 in) in diameter.[17]

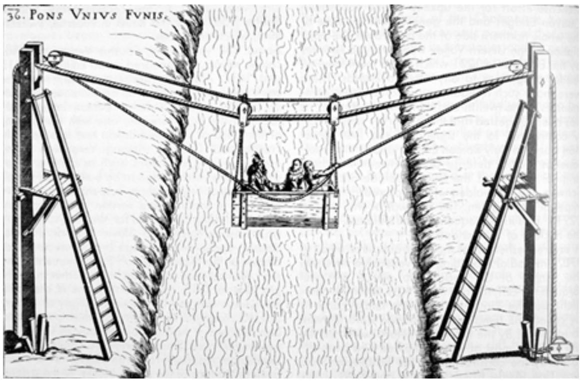

Fig. 8: Ropeway design, Verantius. (Peters[2])

2.3.2 Verantius

Faustus Verantius published three suspension bridge proposals in his book *Machinae novae,* which appeared in two editions in 1615 and 1617. This book was published in Latin, Italian, Spanish, French and German. It was a popular reference. The three bridges Verantius proposed are illustrated in **Figures 8, 9** and **10**. The first is a ropeway; similar to the one-rope span previously described that employed a leather sling with two ropes to pull a charge from one side to the other. An improvement of Verantius's system is to use a continuous cable to pull the basket across rather than two separate ropes. The second bridge is one for a more typical suspension bridge, albeit designed to be temporary for military purposes, where the deck is level, suspended from above by what are probably hemp-rope cables. The last shows a cable-stayed system using eye-bar chains with an auxiliary catenary chain supporting mid-span. The use of segmental construction in the deck would be far too flexible. We can presume that Verantius did not adequately think through the associated problems with his design.[18]

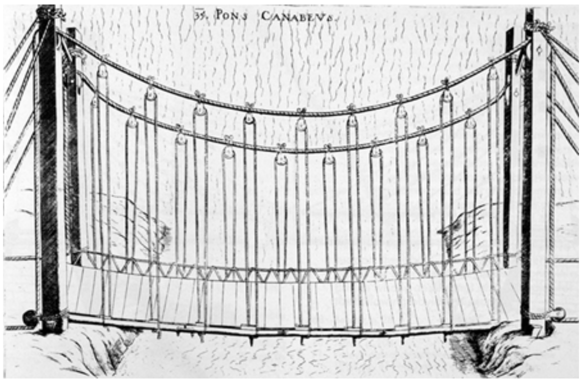

Fig. 9: Temporary military rope bridge, Verantius. (Peters[2])

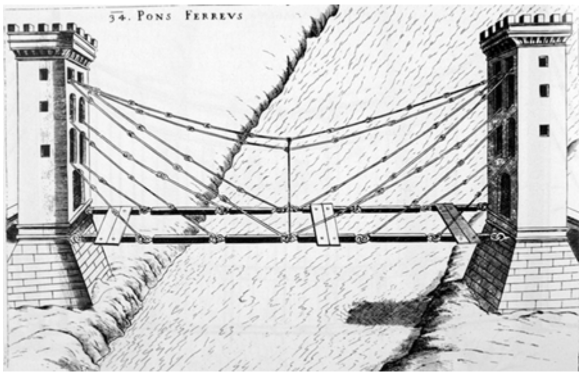

Fig. 10: Chain bridge proposal, Verantius. (Peters[2])

[17] Peters[2], p24-25.
[18] Peters[2], p25-26.

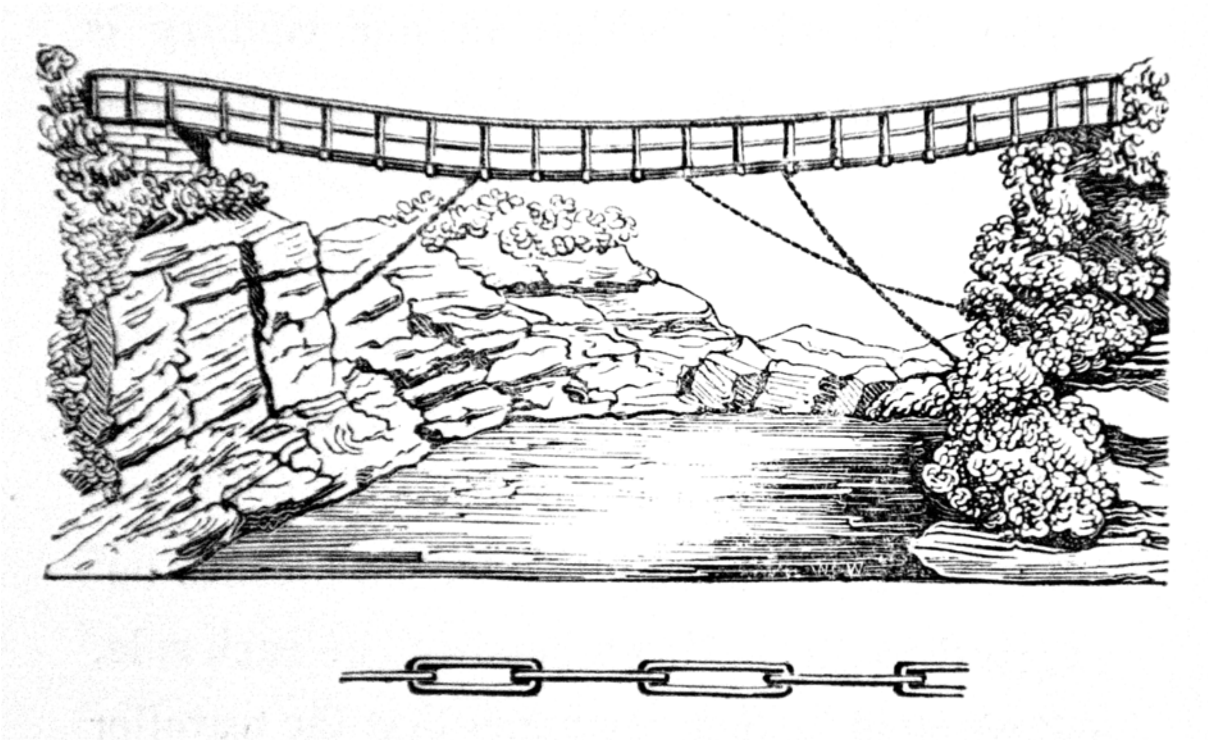

Fig. 11: Winch Catwalk over the River Tees, County Durham, England, 1802. First built 1741. (Cumming)

2.3.3 The Winch Bridge

The Winch Bridge, across the river Tees in England, was built in 1741. **Figure 11** shows the Winch Bridge in 1824, after it had been reconstructed because the original bridge collapsed in 1802. This bridge was still standing in 1908. Lacking evidence to the contrary, the Winch Bridge appears to be the first permanent suspension bridge in Europe to use chains.[19] The bridge is a 'catenary' type where the deck is laid directly on two or more chains strung side by side. The image shows that secondary stay chains were employed in an effort to stabilize the bridge from movement and undulation. The chains themselves were composed of chains composed of rectangular shaped links. The Winch Bridge and the Lan Jin Bridge,[20] were widely published on the European Continent and in England, and can thus be considered as ancestors of later developments.[21]

2.3.4 Niklaus von Fuss, Russia

In 1794, Niklaus von Fuss proposed a design for a long-span chain bridge over the Neva in St. Petersburg, though no record of its design is now known to exist. Fuss was a Swiss mathematician and Leonhard Euler's assistant and successor as Secretary of the Russian Imperial Academy of Sciences at St. Petersburg. It would have been illuminating to examine Fuss's plan because it predated the first Western stiffened suspension bridge actually built, that of James Finley in Pennsylvania, by just two years.[22]

[19] Peters[2], p27.
[20] First illustrated in Kircher, 1667.
[21] Peters[2], p27.
[22] Peters[2], p28.

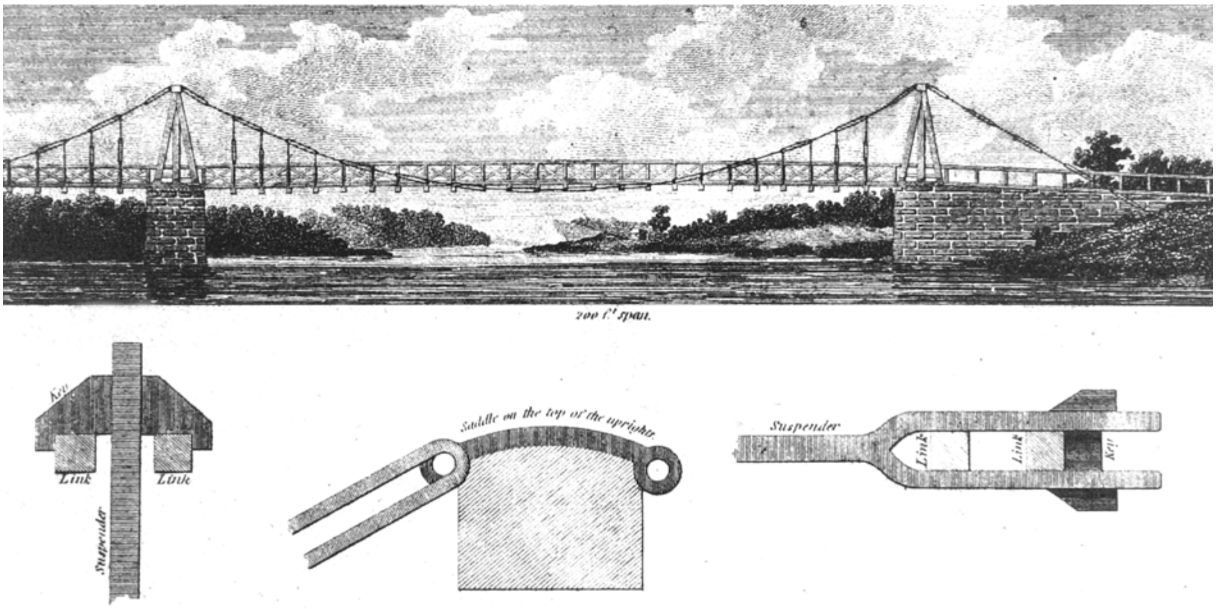

Fig. 12: Finley Patent Bridge, 1808. (Peters[2])

2.3.5 *James Finley, United States*

In 1801, James Finley, an American justice of the peace, erected a 21.3 m (70 ft) span suspension bridge for carriage traffic at Jacob's Creek. He patented his design in 1808 and by 1820, about 40 bridges on his plan are said to have been built in the United States. (**Fig. 12**) Finley's chains were comprised of rectangular links. Finley built Merrimack Bridge in 1810 with a main span of 74.4 m (244 ft), marginally greater than the cast-iron arch bridge at Sunderland, which attracted attention to this type of bridge.[23] The largest Finely type bridge was 93.3 m (306 ft) in length with two spans over the Schuylkill Falls at Fairmount, Pennsylvania.[24]

2.3.6 *England*

Around the turn of the 19th century, the British developed a new form of chain derived from experiments made with iron chains for standing rigging and anchor chains for warships. William Hawks was the first to patent a British eye-bar link with punched holes in 1805, and Thomas Telford subsequently followed suit. The British development was clearly autonomous from that of the Chinese and the ideas of Verantius.[25]

Samuel Brown was, according to Peter Barlow, the original inventor of iron cables. This is clearly false in light of our knowledge of developments in China, Switzerland, France and the United States. Brown used bars with welded eyes, not punched like Telford. Brown notably built the Union Bridge and the Brighton Chain-Pier. (**Figs. 13 and 14**) Brown's Union Bridge was the first suspension bridge in England completed with the bar-type chain-link cable. The Union Bridge crossed the Tweed at Norham Ford with a span of 110 m (361 ft).[26]

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[23] Paxton and Mun, p87.
[24] Peters[2], p31. Fairmount is now part of Philadelphia.
[25] Peters[2], p33.
[26] Gibb, p142.

Fig. 13: Union Bridge, Samuel Brown, 1818. (Brown)

Fig. 14: Brighton Chain Pier, Brown. (Sutherland)

After Telford published his plans for the Runcorn Bridge, a number of bridges were erected in Great Britain, including two small chain bridges built in southern Scotland. One was the first Dryburgh Abbey Bridge, built in 1817 by John and William Smith for the Earl of Bucan. (**Fig. 15**) This bridge had a span of 79.2 m (260 ft) between anchorages and consisted of a catenary cable with two stay cables at either end of the bridge.[27] Redpath and Brown built Kings Meadows Bridge, also in 1817, over the Tweed River in Scotland. (**Fig. 16**) They initially designed a stayed bridge, one of the first such bridges, but later added a catenary cable.[28]

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[27] Peters[2], p32-33; Cumming, p45.
[28] Peters[2], p36.

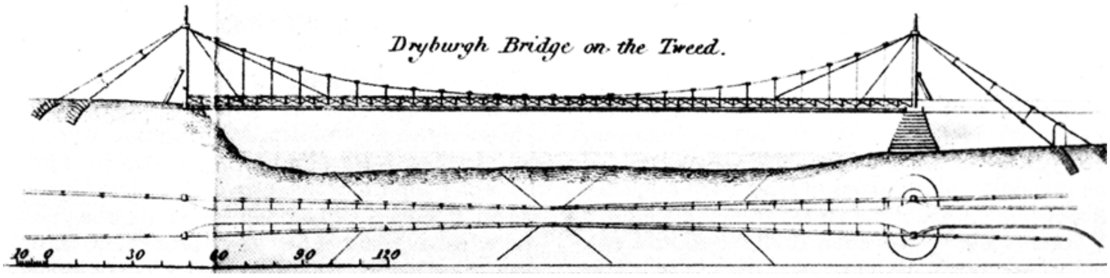

Fig. 15: First Dryburgh Abbey Bridge over the Tweed, John and William Smith, 1817. (Peters[2])

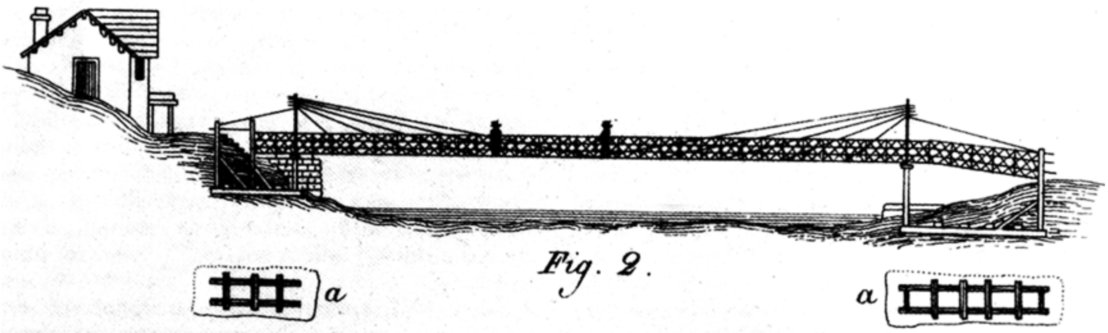

Fig. 16: Kings Meadows Bridge over the Tweed, Redpath and Brown, 1817. (Peters[2])

2.3.7 Other Developments beyond the Menaï Bridge

At this point in the history of suspension bridges, it is proper to introduce the developments made by Thomas Telford while designing the Menaï Bridge. I will examine this case in detail below. First, to complete the brief summary of suspension bridge development laid out in this and the last section, I will briefly review other developments that occurred contemporaneously with the construction of the Menaï Bridge.

Capt. Napier erected a wire bridge with a 38.1 m (125 ft) span at Thirlstane, over the Etterick before 1821.[29] Several cable-stayed bridges were also constructed during this early period of English suspension bridge construction. Cumming notes the beneficial constructive properties that permit a cable-stayed bridge to be constructed from each side independent of the other. He also proposes that a chain pier can be constructed with one half of such a bridge cantilevering out over the sea.[30]

2.3.8 The French

In 1820 and 1821, the French government sent Claude Louis Marie Henri Navier, a prominent engineer and mathematician, to Britain to report on the state-of-the-art of suspension bridge construction.[31] From that time until just after mid-century, the French applied their scientific and theoretical knowledge to developing wire-cable suspension bridges. The Seguin Brothers, French bridge builders, and Guillaume Henri Dufour, a Swiss engineer were at the forefront of this development.

[29] Stevenson.
[30] Cumming, p xii-xiii and 44-45.
[31] Cumming, p50.

2.3.9 Wire Cable Suspension Bridges in US

When Lewis Wernweg's innovative timber arch bridge near Philadelphia, the 'Colossus'[32], was destroyed by fire in 1838, Charles Ellet Jr. replaced it in 1842 by the first permanent wire cable suspension bridge to be built in the United States.[33] From that time onwards, the principal development of the suspension bridge moved back across the Atlantic, where, through the efforts of Ellet, John Augustus Roebling, Othmar Ammann, David Steinman, and others, it remained at the forefront of suspension bridge technology until the latter part of the 20th century. All of these great efforts can principally trace their origin to the work of Thomas Telford. His ideas and work hugely influenced the first great period of suspension bridge construction from 1814 to the mid-nineteenth century.

2.4 Thomas Telford, His Life and Experience

2.4.1 Early Life

Thomas Telford was born in Eskdale, Scotland in 1757 to working-class parents. The school system in Scotland at the time was considered very good and was accessible to Telford, therefore he received a good educational foundation. At the age of 14 Telford left school to start an apprenticeship as a mason. Telford worked throughout Scotland as a mason, including Edinburgh, until the age of 24, when upon his own initiative he set out for London, where he felt he could learn more about his trade than in Scotland.[34]

Upon his arrival in London, Telford's first employer was William Pulteney, a gentleman from Telford's hometown of Eskdale. Within a few months, Telford went to work for an architect named Sir William Chambers. In 1784, Telford was at Portsmouth Dockyard, where for the next two years he was engaged on the building of an official residence designed by Samuel Wyatt for the Commissioner. As a matter of circumstance[35] and certainly Telford's own industriousness and initiative, he gained total control over the project.[36]

The job at Portsmouth was Telford's first position of independence and responsibility and marked his emergence from the rank of manual laborer. While at Portsmouth, Telford was introduced to all the activities of a big harbor and naval base. He saw and could study engineering works in tidal areas, and made his first contact with what the English engineer John Smeaton would have called civil engineering, as opposed to architecture. All the time Telford was acquiring knowledge. Anything that he thought might be useful to him he collected and recorded in a notebook.[37]

Writing a friend, Telford noted,

> Knowledge is my most ardent pursuit, a thousand things occur that would pass unnoticed by good easy people who are contented with trudging on in the beaten Path but I am not contented unless I can reason on every particular. I am now very deep in Chemistry – the manner of making Mortar, led me to

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[32] The Colossus, built in 1803, had a world-record tying span of 103.6 m. Ref. **Appendix A-03, p.A.87**.
[33] Peters[2], p31.
[34] Gibb, p1-5.
[35] The person who had been in charge was considered by the client to be unreliable and neglectful.
[36] Gibb, p5, 6 and 10.
[37] Gibb, p10.

> enquire into the Nature of Lime. In pursuit of this, having look'd in some books on Chemistry I perceived that the field was boundless – and that to assign reasons for many Mechanical processes it required a general knowledge of that Science. I wish … that you saw me at the present instant surrounded by Books, Drawings – Compasses Pencils and Pens etc. etc. great is the confusion but pleases my taste and *that's enough*.[38]

During his time at Portsmouth, Telford would have been exposed to iron and woodworking, different tools, and a completely different way of thinking about construction. It would be interesting to research what activities were going on at the dockyard during that time. Had the Royal Navy already begun research on the use of iron chains in lieu of hemp rope? Telford would have been exposed to a variety of tensile structures, such as the ships rigging and anchor cables. These structures would have provided valuable insight in his thinking about materials and structure. Until then, his experience was largely limited to masonry structures subject to compression, and timber structures used for the centering of masonry arches.

2.4.2 From Mason to Civil Engineer

The Commissioner's house was finished in the autumn of 1786, and soon after Telford moved to Shropshire, England, where he was appointed Surveyor of Public Works for the County of Salop, probably due to the influence of his former employer, William Pulteney.[39] This position led to all other commissions Telford received to oversee large civil engineering works on roads and canals.

Telford designed and built one of his first bridges, the Montford Bridge in 1790. It is a three arch stone bridge over the Severn River. It was begun in 1790, and for it Telford imported Matthew Davidson, the master mason of Langhom, with whom he had perhaps worked his apprenticeship. Davidson remained with Telford until his death while engaged on the Caledonian Canal at Invernss nearly thirty years later. 'His skill and workmanship were unsurpassed, and his caution provided a valuable and sometimes necessary foil to Telford's enthusiasm for experiment and new ideas.'[40]

The years 1793 and 1795 saw the emergence of Telford as a civil engineer. He was appointed General Agent to the Ellesmere Canal in late October 1793 and Engineer to the Shrewsbury Canal in February 1795. The period from 1793 marked him as an innovating civil engineer capable of daring projects. By 1793 he had the confidence of the canal commissions, landed gentry, and had already made many of his friends among practical men and scientists that he would work with throughout the rest of his life.[41]

Among the men Telford befriended were a number of iron founders. One of these men was William Hazledine, a millwright and iron founder, who had supplied a pump to one of Telford's canal projects. Hazledine was highly respected by Telford, particularly for his inventiveness. Hazledine would eventually supply all of the ironwork for the Menaï Suspension Bridge and it would be interesting to trace what influence Hazledine had on the

[38] Gibb, p10.
[39] Gibb, p11.
[40] Gibb, p21.
[41] Penfold, p12 and 17.

Fig. 17: Ironbridge, Coalbrookdale, Abraham Darby, 1779. (Walker)

actual design of the chain cable. An interesting connection to make is that Hazledine cast the structural elements of the first iron frame building, a flax mill at Ditherington in Shrewsbury, designed by Charles Bage in 1796-97. It is probable that Telford knew the details of this building. Bage's experiments on the strength of iron bars are recorded in Telford's memorandum book and it seems likely that Bage borrowed the results of the experiments on cast iron performed at Ketley by Reynolods for the Longdon Aqueduct in 1795, which was designed with Telford. Bage apparently worked with Telford on survey works in Shrewsbury[42] around 1788, though it is unclear what personal relationship they might have had.[43]

While working on the Ellesmere Canal, Telford made the acquaintance of John Wilkinson. Telford called Wilkinson the 'King of the Iron Masters' and considered himself 'fortunate in being on good terms with most of the leading men of property and abilities.'[44]

2.4.3 Ironbridge and the Suitability of Iron in Bridges

While Surveyor of the Public in Shrewsbury, he was not far from the Coalbrookdale ironworks and Ironbridge, which was the first all cast-iron bridge when built in 1779 over the Severn River. (**Fig. 17**) It is not perfectly certain from whom the idea for its construction originated. Abraham Darby, proprietor of the Coalbrookdale ironworks that cast and erected the bridge, is generally credited with its design; however, it has been argued that John Wilkinson had some share in its conception.[45]

Telford's proximity to Coalbrookdale obviously afforded him the opportunity not only to study the construction of the first iron bridge, but also to become acquainted with the whole process of iron production and fabrication at the Coalbrookdale Ironworks.

About Ironbridge, Telford observed,

> The banks of the river adjacent to the bridge are exceedingly high and steep, and composed of alluvial matter which slips over the points of the coal strata. The effect of this operation not having been sufficiently provided against, some years ago, the top part of one of the stone abutments was pressed in a few inches, and of course raised up the iron work about the middle of the arch. Steps have been since taken to secure the western abutment; but the other, by having valuable houses built close up to it, is more entangled, and it may in time suffer from that cause; but the iron work has not been the least affected by the weather, or the intercourse over or under the bridge during 34 years.[46]

[42] …at the prison and in the town of Madeley.
[43] Penfold, p17.
[44] Penfold, p12.
[45] Telford[2], p539.
[46] Telford[2], p488.

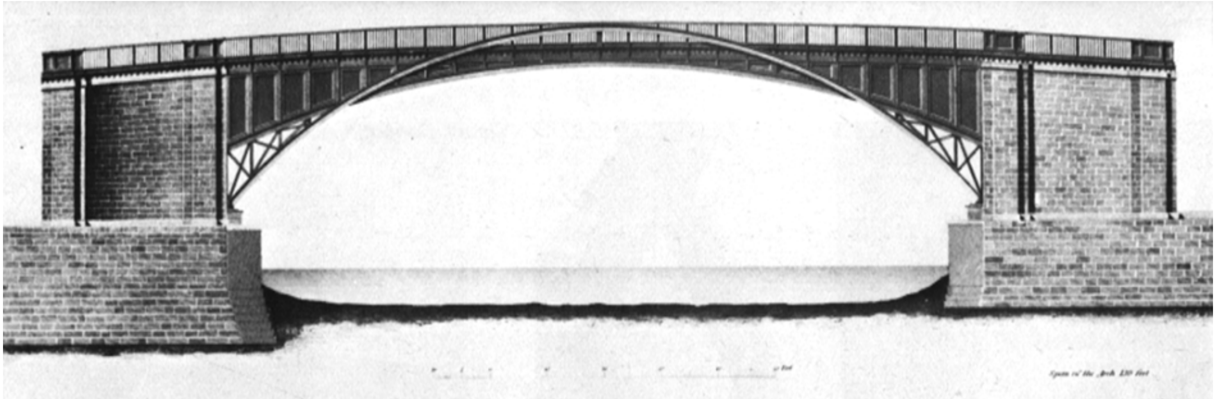

Fig. 18: Buildwas Bridge, cast-iron arch, Telford, 1795. (Telford[2])

To address the problem of the abutments Telford explained,

> Had the abutments been at first sunk down into the natural undisturbed measures, and constructed of dimensions and form capable of resisting the ground behind, and had the iron work, instead of being formed in ribs nearly semicircular, been made flat segments, pressing against the upper parts of the abutments, the whole edifice would have been much more perfect, and a great proportion of the weight of metal saved. … [Furthermore,] the circular rings of the spandrels are less than perfect than if the pressure had been upon straight lines; for a circle is not well calculated for resistance, unless equally pressed all around.[47]

Evidently, engineers must have considered Ironbridge with some skepticism even though it had attracted much attention because a second iron bridge was not built until 1895. In addressing the faults he observed in the construction of Ironbridge, Telford wrote,

> We consider it our duty to introduce these observations, in order to shew the necessity for great precaution in similar works, and how liable first attempts are to be defective; but they derogate nothing from the merit of projecting a great arch of cast iron, introducing a material almost incompressible, which is readily moulded into any shape, and which is peculiarly applicable in the British Isles, where the mines or iron are inexhaustible and the means of manufacturing cast iron unrivalled.[48]

These preceding observations demonstrate Telford’s appreciation of the compressive strength of cast iron and its applicability to arch structures. This structural type was very familiar to the highly trained mason. Telford recognizes the possibilities that the new material offered in terms of weight savings that would make possible the construction of flatter arches. He warns against irrational caution by his peers to adopt the new material, showing the same initiative and comfort with risk that raised him to the professional level he attained. Finally, Telford also notes how the iron industry in England was ready and particularly suited to the production of cast iron structures.

[47] Telford[2], p539.
[48] Telford[2], p539.

2.4.4 Buildwas Bridge

Thomas Telford built the second iron bridge in England. This bridge traversed the Severn River just three miles north of the first iron bridge at Coalbrookdale. The Buildwas Bridge replaced an old stone bridge of several arches that had been carried away in a very high flood in early 1795.[49] The floods damaged or washed out other bridges too, but Ironbridge, just downstream of the first Buildwas Bridge, had survived the floods undamaged.[50] There appears to be very little evidence of interest in iron bridges or aqueducts among engineers and ironmasters until a letter from Coalbrookdale that recorded the performance of Ironbridge, dated 15 February 1795, was published in several newspapers. Interest in iron bridges and aqueducts increased markedly after this event.[51]

Telford knew Ironbridge well since it was not far away from Shrewsbury, where he was Surveyor General. Telford had related that this bridge had greatly interested him from the moment he first saw it and he was looking for the opportunity to put into practice his own ideas for avoiding the chief mistakes he believed Ironbridge's designers had made. One of these mistakes was to underestimate the stability of the earthen abutments with an arch that was much lighter in comparison to the typical masonry bridge. At Ironbridge, the abutments caused problems because they pushed out.[52]

To replace the washed out Buildwas Bridge, Telford recommended a cast-iron arch of 39.6 m (130 ft) span. (**Fig. 18**) For the design, Telford asked to William Reynolds and John Wilkinson for their opinions. The Coalbrookdale Company performed both the masonry and ironwork for Buildwas Bridge, and it was finished in 1796.[53]

Telford evidently had some trouble convincing the Coalbrookdale Company to depart from their former mode of construction, but he at last prevailed in building the bridge as shown in **Figure 18**. The bridge actually incorporated two arches, each with a different rise and springing. The bearing ribs have a curve of 17 in 130, or nearly one-eighth of their span. The suspending ribs rise 10.4 m (34 ft), or about one-fourth of the span. There are cast-iron braces and horizontal ties. There are 46 covering plates, each 5.5 m (18 ft) in length, and 2.5 cm (1 in) in thickness. They have haunches 10.2 cm (4 in) in depth, and are screwed together at each joint. The arches were designed to act compositely as one arch.[54]

The banks are low at the site of Buildwas Bridge.[55] In 1800 Telford said that he used this double-arch configuration because 'the roadway could not with propriety be raised to a great height' and had to be kept as level as possible with the banks. Moreover the uncommonly high flood had led him to consider a design 'which would avoid piers and allow for rapid changes in river level' and, more cogently, because of the instability of the banks, he used 'a very flat arch (the segment of a very large circle) calculated to resist the abutments if disposed to slide inwards, as at Coalbrookdale.' The back of each abutment was 'wedge shaped, so as to throw off laterally much of the pressure of the earth'.[56]

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49 Telford[2], p539-540.
50 Gibb, p21.
51 Cohen, p140. Comment given by J.G. James.
52 Gibb, p21.
53 Telford[2], p488-489.
54 Telford[2], p540.
55 Telford[2], p488-489.
56 Penfold, p13 and 15.

When Buildwas Bridge was completed, Telford's design was criticized because the two arches, which have different length and curvature, were connected, thus exposing them to different degrees of expansion and contraction. Telford dismissed the criticism, saying 'this appears just in theory; and that no discernible effect has hitherto been produced, is probably from the difference being small.'[57]

Telford was a pioneer in the use of iron in bridge building. For twenty years the Coalbrookdale bridge, inspired by Wilkinson, remained unchallenged and uncopied. His design for the Buildwas Bridge was, for the times bold and original. He first recognized that the real advantage iron offered was that it allowed the introduction of a much flatter arch; and that the weight on the foundations could be greatly reduced. The success of the Buildwas Bridge not only added to Telford's growing reputation, but also drew attention to an important market for surplus iron, for which the ironmasters were then anxiously looking.[58] It was only one year after the construction of the Buildwas Bridge, that Charles Bage. And English industrialist, introduced the first cast-iron beam into mill-building construction.

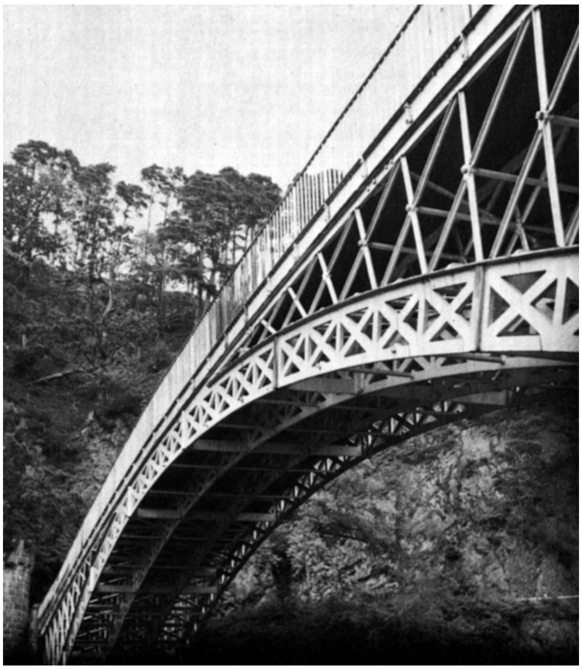

Fig. 19: Craigellachie Bride, cast-iron arch. Telford, 1810. (Ruddock)

Fig. 20: Longdon Viaduct, c.1796. (Ruddock)

Fig. 21: Pont Cysyllte Viaduct, 1805. (Sealey)

2.4.5 What did Telford Know?

Concerning all of his knowledge of engineering and construction, Telford is largely self-taught. From the time he was an apprenticed mason, Telford sought to increase his skills and knowledge. It is for this reason that Telford first moved to Edinburgh and then to London. His knowledge came from all sources – personal experience and observation, reading books, and from those people with whom he surrounded himself.

At Portsmouth Dock, he studied all the works of the port and wrote to a friend that he was studying chemistry because of its application

[57] Telford^2, p540.
[58] Gibb, p22-23.

to the production of mortar. By the time he built Buildwas Bridge, he already had an enormous resource of knowledge to draw from, not only his own, but some of the preeminent iron founders, engineers and building contractors of the time.

Telford learned project management and was an effective leader. None of the great engineering works he was involved in could have been successfully executed had Telford not been astute at managing the economic, political and public influences on projects. All of this knowledge was necessary to realize the construction of the Menaï Suspension Bridge, then the longest bridge in the world.

When Telford was asked to submit proposals for two bridges to cross the Menaï he had already established himself as an accomplished engineer of large works such as roads and canals as well as having built numerous bridges in stone and cast-iron. His article, *The Theory and Practice of Bridge Building*, published around 1811, attests to what Telford knew at that time. The following are just a few pieces of information with which he had to approach the design for a bridge to cross the Menaï.

2.4.6 Strength of Materials

The first scientific treatises on the strength of materials, containing descriptions of duplicable experimentation, were just beginning to be written at the time. Abstract, mathematical theories had yet to be translated into practical tools for design.[59] As Telford wrote,

> Though we highly value the sublime geometry, we are inclined to think that the unnecessary parade of calculus in the application of science to the arts, has been one of the chief causes of the dislike, which many able practical men of our country have shewn to analytical investigation.[60]

From all of his early experiences Telford gained an astounding amount of knowledge about structural materials – stone, wood[61] and cast-iron. As a highly trained mason, Telford's knowledge of stone, its properties, workability and application, certainly must have lacked deficiency. Associated with his masonry work, Telford had to have been well familiar with carpentry and many aspects of building with timber because of its importance in the construction of centering for masonry arches.

By 1810, cast iron was not a new material to Telford. He had built the second iron bridge in the world in 1895, the Buildwas Bridge and completed another cast-iron arch bridge in 1810, Craigellachie Bridge. (**Fig. 19**) Additionally, Telford was intimately involved in the construction of two aqueducts, the Longdon and Pont Cysyllte, in which cast iron was used. (**Figs. 20 and 21**) In the Longdon Viaduct, completed c.1796, the piers were made of cast-iron. The canal troughs of the Longdon and Pont Cysyllte Aqueducts were constructed from cast-iron plates. During this period, Telford conducted experiments to determine the strength properties of cast iron. He was also privy to the experiments being done by Charles Bage for his mill-building beams.

[59] Peters^2, p31.
[60] Telford^2, p490.
[61] Telford used timber for falsework in masonry construction, was exposed to that material's use in ship and dock works, and used it for a foot bridge in Glasgow.

Iron quality of the time was undependable. Manufacturing methods were poor and uneven and hidden flaws were common. Not much was known about metallurgy.[62] In his article on bridge building, Telford recommended cast iron for 'receiving thrusts, forming supports, or dowelling stones.' The only applications Telford recommended for wrought iron, 'made from wood charcoal,' were in ties, bolts, and nails.[63] The historical record does not show that Telford knew as much about wrought iron as he did cast iron prior to his involvement with the design of the Menaï Suspension Bridge. Nor was there much information available in general.

2.4.7 Telford on Learning

While Telford valued formal education, he strongly believed experience was the best teacher. In his biography[64], Telford wrote,

> I ever congratulate myself upon the circumstances which compelled me to begin by working with my own hands, and thus to acquire early experience of the habits and feelings of workmen; it being equally important to the Civil Engineer, as to Naval or Military Commanders, to have passed through all grades of their profession.[65]

2.4.8 Structural Theory and Empiricism

Telford's article on bridge building makes it clear that Telford was keenly aware of the various theories on structure of the time. He seems to have largely ignored such theory on the basis that it did not often match his observations in practice. In his article on bridge building, Telford wrote,

> It was only about a century ago, when Newton had opened the path of true mechanical science, that the construction of arches attracted the attention of mathematicians. Since that time, volumes have been written respecting the equilibrium of arches. It has been found one of the most delicate, as it is one of the most important applications of mathematical science. Yet, with all due deference to the eminent men, who have prosecuted this subject, we are much inclined to doubt whether the greater part of their speculations have been of any value to the practical bridge builder. He is still left to be guided by a set of maxims derived from long experience, and as yet little improved theory. In truth, his works seldom fail even where they differ farthest from the deductions of the theorist; and at all events, he finds that a much greater latitude is allowable than theory seems to warrant. He is therefore surely excusable in doubting of the justice of such theories, at least until they are more consonant to the approved practice.[66]

T.G. Cumming sums up the attitude of the British engineer in the second edition of his book on suspension bridges, published in 1828. Cumming related the story of when Navier, the French mathematician and structural theoretician, visited England in 1820 and 1821 to examine and report on the state of English suspension bridge technology. With this knowledge, he attempted to apply the French scientific approach to the design of a

[62] Peters[2], p31.
[63] Telford[2], p521.
[64] Telford[1].
[65] Gibb, p209.
[66] Telford[2], p489-490.

suspension bridge that was to traverse the Seine in Paris. The intended span was to be 149.4 m (490 ft), with 170.7 m (560 ft) from anchorage to anchorage. Towards the close of 1827, the chains were in place, and installation of the deck was proceeding when, according to Cumming, the balance weights proved too light and the bridge fell into the river.[67] Tom F. Peters attributes the failure to a movement in the abutment compounded by a burst water main near by.[68]

Cumming wrote that the ruins of Navier's bridge "proudly shew that English modes of calculation, combined with practical skills, are infinitely superior to being 'initiées aux connoissances mathématiques les plus élevées.' The reason is obvious; no man can make progress in the highest departments of mathematical learning who does not consume by far the greater part of his time in them; while with a certain degree of power in comparing quantities, and knowing the exact nature of a thing to be done, it is easy to render stability certain without having recourse to refined calculations."[69]

2.4.9 Knowledge of the State-of-the-Art in Engineering

Telford was an avid reader and observer. His article on bridge building reveals a broad breadth of knowledge about bridge building not only in Great Britain but also from around the world, including the suspension bridges of China and South America. Telford habitually asked friends to record all the engineering works they observed in the course of traveling. In this way, Telford was probably quite familiar with what was being done on the European continent.[70]

Telford was known for his seemingly tireless commitment to work. Telford never married and kept no home. In London, Telford lived in a coffeehouse. When asked if he had any regrets about not having married or settled down, Telford simply responded that he kept 'sentiment in its true perspective.'[71]

2.5 Menaï Bridge, 1810-1811

2.5.1 Background on the Menaï Suspension Bridge

At the turn of the 19th century, pressure was growing for a better transportation link between London and Holyhead, England's gateway to Ireland. Economic growth due to the industrial revolution generated an increase of trade across the Irish Sea. Politicians were seeking to shorten the trip between London and Dublin. To meet this objective, it was determined that the road to Holyhead had to bridge the Menaï Strait, which separated the island of Anglesea from Carnarvonshire.[72]

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[67] Cumming, p51.
[68] Peters[2], p149.
[69] Cumming, p51-52.
[70] Gibb, p208.
[71] Gibb, p2; correspondence with Tom F. Peters.
[72] Telford[2], p542.

In 1801, Charles Abbott, then Secretary of State for Ireland, directed the engineer John Rennie to survey the Menaï Strait, and propose a plan for a bridge.[73] Rennie identified two possible locations for a bridge: one at the narrowest part of the strait with a projecting rock named Ynys-y-Moch; and the second upon the several projecting rocks known collectively as “The Swellies.” Rennie proposed a single cast-iron arch with 137.2 m (450 ft) span for Ynys-y-Moch and a three cast-iron arches over the Swellies, each spanning 106.7 m (350 ft). Of the two, Rennie preferred to build the bridge at the Swellies, even though he estimated it to be the more expensive of the two proposed projects. The reason he cited was the difficulty of constructing a proper centering for the single span at Ynes-y-Moch because of the depth of water and rapidity of the tide there.[74]

While Rennie’s designs were under consideration, a strong opposition to the erection of a bridge at the Swellies arose by some of the commercial and trading interests of Carnarvonshire and its neighborhood, who contended that the bridge would cause additional eddies, wind and water, thereby increasing the difficulty and danger of passing the Swellies. William Provis writes that ‘they were probably influenced quite as much by the consideration that it would open a more perfect communication with the Bangor markets [on the other side of the strait], and consequently operate to the prejudice of their own.’ These objections did not have a chance to be fully addressed because there was no money available to proceed with the project. At the time, it was estimated that the expected returns on any investment would not be enough for private individuals to undertake it, and the government was engaged in an expensive war that left little in the treasury for a bridge.[75]

In 1810, Rennie was asked to submit revised plans and estimates. Other engineers were consulted. Various sailing authorities testified that previous objections to the bridge because of the effects it would have on shipping in the strait were unfounded, but ferry boat interests and the trading interests of Carnarvonshire continued their objections. Additionally, it was found that the roads leading from the Swellies were less satisfactory than those near Ynys-y-Moch. The condition of the road was delaying delivery of the Royal Mail. With these concerns in mind, the Lords of the Treasury instructed Thomas Telford to make an accurate survey of the two roads to Holyhead, to propose the best lines that could be adopted, and to consider the best mode of traversing the Menaï Strait.[76]

2.5.2 Telford’s First Proposal for a Bridge across the Menaï Strait, 1810-1811

When Telford surveyed the two sites identified by Rennie, he defined the following criteria to respect for the design of any bridge:

- a considerable number of small coasting-vessels, from 16 to 100 tons, navigate the Menaï, and that there have been a few from 100 to 150 tons.

- the largest ships to consider, from 150 to 300 tons, are only 26.8 m (88 ft) above the water line with their top-gallant-masts struck. These are extreme cases in the Menaï, and if provided for should satisfy the navigation requirements of the strait.

[73] Provis, p3.
[74] Telford[2], p542.
[75] Provis, p5.
[76] Provis, p7-8.

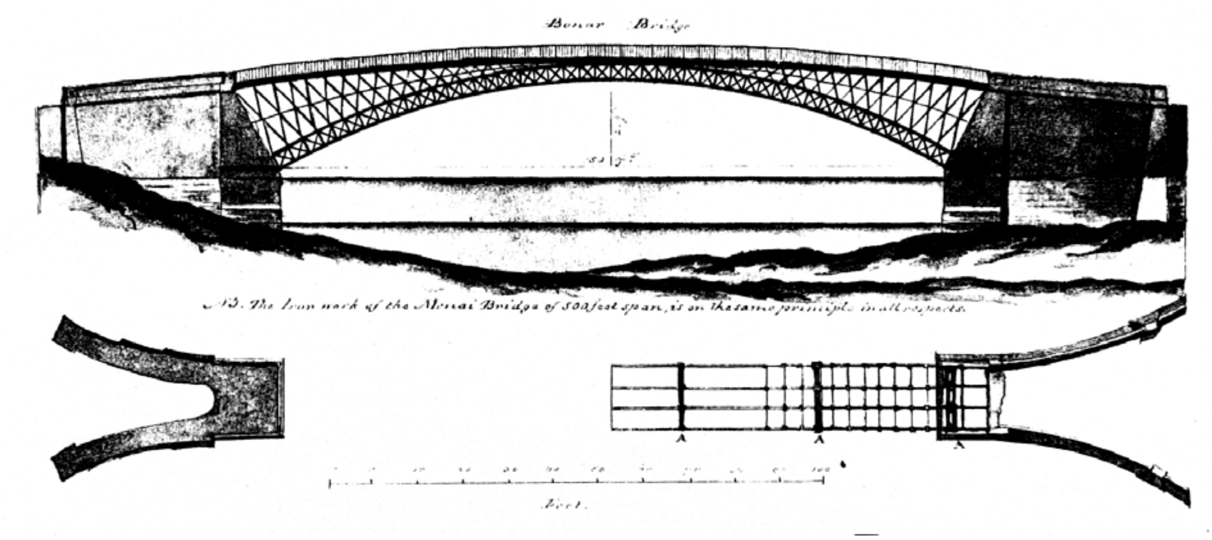

Fig. 22: Cast-iron arch type bridge as proposed by Telford to cross the Menaï Strait, 1811. (Telford[2])

- Telford posed the question if the height should not be limited to vessels less than 100 tons, by which the expense of a bridge would be considerably diminished. The design ship ultimately was settled to be a 150-ton design ship "with all on end.[77]

Telford's first design was for the Swellies and consisted of three cast-iron arches of 79.2 m (260 ft) span each, with a stone arch of 30.5 m (100 ft) span between each two of them. The spandrels of the cast-iron arches were left open 'so as to oppose as little resistance as possible to the wind, and to preserve the general lightness of character.' The arches had a rise of 27.4 m (90 ft) on a spring tide, sufficient to allow vessels not exceeding 150 tons to pass under the Bridge at high water with all on end, and vessels of 300 tons might pass under by merely striking their topgallant masts.[78]

Telford's design for Ynys-y-Moch comprised of a single cast-iron arch of 152.4 m (500 ft) span. (**Fig. 22**) The height to the under side of the arch was proposed to be 30.5 m (100 feet) from the high water of spring tides, and the breadth of the roadway 40 feet. The estimated expense was £127,331, £31,367 less than the design for the Swellies. The principle objection as expressed by a Dr. Hutton, was the difficulty of erecting a proper centering for supporting the arch. Additionally, a traditional structure for supporting the centering would interfere ships navigating the strait until the arch was complete. The Royal Navy made clear that at no time could the strait be closed to shipping or could a bridge interfere with that shipping.[79]

2.5.3 A Novel Approach to the Construction of Centering

Telford appreciated the importance of good centering. He wrote,

> To construct and erect [the centers] is one of the most masterly operations in bridge building…. In a centre, the principal objects to be kept in view, are to

[77] Telford[2], p543.
[78] Provis, p9.
[79] Provis, p9.

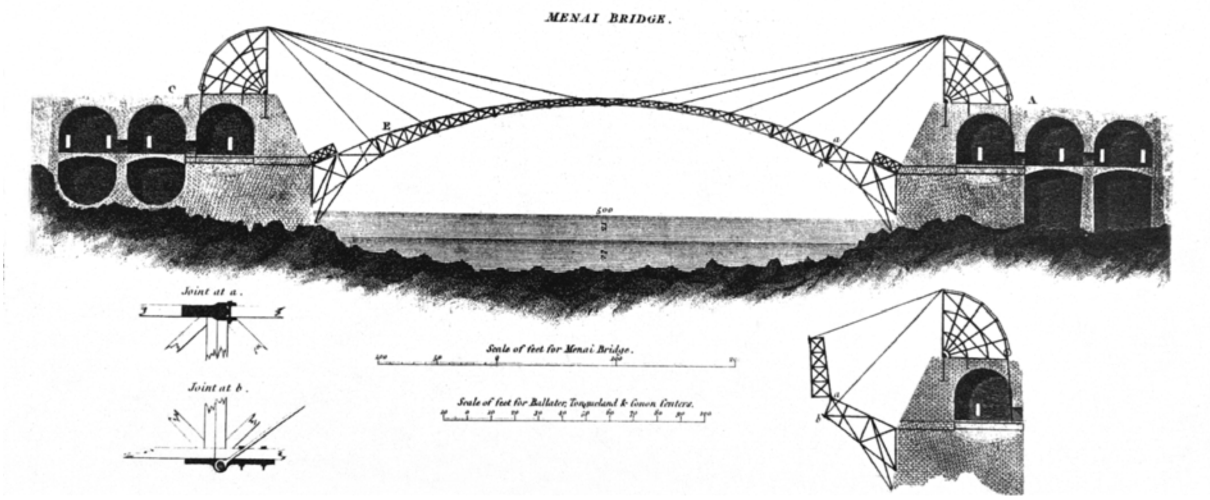

Fig. 23: Telford's proposal for a suspended centering upon which to built the cast-iron arch. (Telford[2])

> construct and fix such a frame as shall support the weight of the arch-stones, through all the progress of the work, from the springing of the arch, to the fixing of the key-stone, without changing its shape, and to admit of its being removed with safety and ease.[80]

In April 1811, Telford proposed suspending the centering for the 152.4 m span cast-iron arch at Ynys-y-Moch. (**Fig. 23**) The centering was to have consisted of four parallel timber rib-frames spanning the waterway in sections supported by a series of 968 mm^2 (1.5 in^2) iron stays. These stays radiated in eights, two to a frame, at angles of approximately 12°, 15°, 20°, 30° and 47° from the horizontal from timber tower-frames at each side of the bridge. Each stay was to have been continuous, presumably welded, from the rib-frame to about 15.2 m (50 ft) from the tower-frame where it was attached via a flexible chain to a winch.[81] Telford seems to have taken for granted the fact that his proposed plan was feasible. Telford stated that the weight of the bridge would be nearly half that of a similar stone arch.[82]

2.5.4 Design of the Stays for Suspending the Centering

In his calculations Telford assumed the breaking stress of a bar to be 552 MPa (80 ksi[83]) and multiplied this figure by the cross-sectional area of the bar to give a "suspending power" of 81650 kg (180 kips[84]). He claimed that since the weight each bar would have to support, including self-weight, would be about 13,600 kg (30 kips), or one-sixth of the weight that might be safely suspended.[85] Although this was probably considered a preliminary calculation, it is clear that Telford is not accounting for the increased tensile stress incurred by inclining the stay from the vertical. Taking the extreme case of a stay that is angled 12° from the horizontal, Telford's stay would have to support upwards of 65,450 kg (144.3 kips), which does not provide such margin of safety with the ultimate stress Telford used for his

[80] Telford[2], p531.
[81] Paxton and Mun, p88.
[82] Telford[2], p544.
[83] ksi = kilo-pounds per square inch, e.g. 80 ksi = 80,000 lbs/in^2
[84] 1 kip (kilo-pound) = 1000 pounds
[85] Provis, p9.

calculation. Paxton and Mun calculate that Telford's value of 552 MPa probably exceeded the true value by about 30%.[86] Hence, Telford's initial calculations were fundamentally flawed. His oversight of the increased stress imposed by changing the angle of a tension member to anything but vertical illuminates the crude state of structural theory and knowledge of strength of materials. Conversely, it is interesting that Telford, believing he had made his calculation correct, tried to convince others to use what was at that time an untested material, wrought iron, by designing for a factor of safety equal to 6.

2.5.5 Cast-Iron Crushing Strength: Gun Metal

There appears to be no record of Telford conducting tests on wrought iron to obtain the value of its ultimate stress used in his calculation for dimensioning the stay bars, he did use data from experiments on cast iron to confirm his preliminary sizing of the actual arch ribs. William Reynolds of Coalbrookdale performed the tests. Reynolds found that it requires 203,210 kg (448 kips) to crush a cube of 6.35 mm (¼") of cast-iron, of the quality named 'gun metal' – i.e. 49,422 MPa (7,168 ksi). Based on this value, Telford determined that the ribs would be 'kept in their true position, that the strength provided is more than ample.'[87] With the ample cross bracing in Telford's design the problem of buckling probably would not have been a problem but the phenomenon does not seem to have been recognized as of that time. Barlow and Hodgkinson conducted the first recorded investigation into buckling. Hodgkinson only published the results in 1840.

2.5.6 Project Delayed

Though some critics considered Telford's single arch at Ynys-y-Moch less objectionable, there were still vested interests opposed to any bridge being built over the Menaï. To assuage some concerns, it was offered to cut away some of the most dangerous rocks in the strait at that point if the bridge was injurious to navigation. After a number of hearings, the Committee recommended to the Parliament that Telford's arch proposal be accepted. Nevertheless, no action to pursue the plan further was taken until 1818.[88]

2.6 Runcorn Bridge, 1814-1818

2.6.1 The Runcorn Bridge Proposal, 1814-1818

While the fate of the Menaï Bridge lay muddled in indecision, Telford was approached to propose a plan for a bridge to cross the Mersey River to Liverpool at a narrow part in the waterway called Runcorn Gap. After approaching all parties most interested, Telford was convinced that it would be impossible to propose any plan that might at all interfere with the free and uninterrupted navigation of the river. Additionally, Telford thought it would be very difficult to erect piers within the low water lines because of the great depth of loose, shifting sand in those areas. Telford therefore concluded that because of the above concerns, a bridge of typical construction was not suitable, and determined to propose a suspension

[86] Paxton and Mun, p88.
[87] Telford^2, p544.
[88] Provis, p12-13.

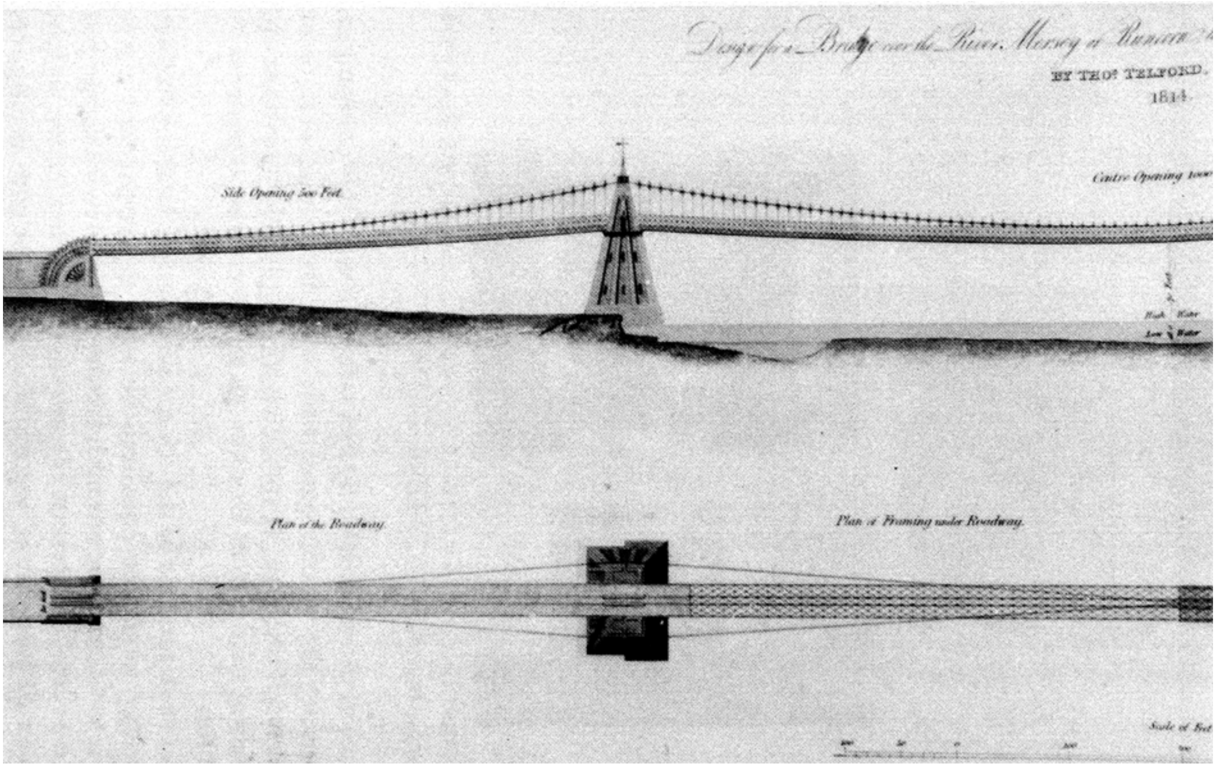

Fig. 24: Telford's 1814 proposal for a 609.6 m (2000 ft) long suspension bridge at the Runcorn Gap with a 1000 ft. center span. Elevation and plan from anchorage to centerline. (Penfold, after Rickman)

bridge.[89] Telford proposed a bridge 609.6 m (2000 ft) long, with a center span of 304.8 m (1000 ft) and two side spans of 152.4 m (500 ft). (**Fig. 24**) The committee responsible asked Telford to study of the best way to construct the suspending cables to meet this objective.[90]

2.6.2 What Telford Knew of Suspension Bridges in 1814

In his article on the theory and practice of bridge building, Telford records that he is aware that 'bridges of ropes and chains had long been suspended both in America and India, though generally of perishable materials, rudely put together.'[91] It is likely Telford, who was well read, had seen the illustration of the Lan Jin Bridge, mentioned earlier.[92]

Telford almost certainly knew of the Winch Bridge, built in England in 1741 and collapsed in 1802, but he does not mention it in his article. Provis, Telford's assistant, does mention Finley in his monograph on the Menaï Bridge, but dismissed Finley's work, making it evident that no useful knowledge was to be had from Finley's work – probably due to the restricted flow of information as a result of the War of 1812. Therefore, Telford's own efforts were for the most part independent of any precedent.

Telford was aware of the catenary curve, as defined by Hooke being 'the figure into which a heavy chain or rope arranges itself, when suspended at the two extremities.' Telford was aware of its application to the design of arches in that, when inverted, the catenary defines

[89] Provis, p11-13.
[90] Paxton and Mun, p93.
[91] Provis, p13.
[92] Peters^2, p22-24.

the proper form of an arch when all of the stones are of equal size and weight. Telford also understood that in practice such a curve is not accurate and that a non-uniform load has to be considered due to the mass of the haunches. Telford also understood that if the arch itself were not so heavy, traffic traversing it would also cause deformation.[93]

Overall, Telford knew about mathematics and the state of science and theory at the time. He was skeptical of mathematical calculation and theoretical design because such theory and calculation did not often correspond with what he observed in practice. It was Telford's inclination to use any knowledge that would help him build something, but he would not let anything like 'theory' stop him from building what his judgment told him he could do.

Presented with the design challenge before him, Telford considered that his proposed centering of the arch at Ynys-y-moch was an adaptation of the suspension principle. There seemed to him in the case of the Runcorn Bridge 'no reason to doubt the practicability or permanency of a Suspension Bridge, providing its parts were properly combined, and durable materials used in its construction.'[94]

2.6.3 Material Testing of Wrought-Iron Wire and Bars

In developing the Runcorn Bridge plan, Telford researched the properties of wrought iron and cable forms. Telford's experimental approach to suspension bridge design was similar to Finley's in some respects but this was probably coincidence. Nevertheless, Telford believed 'British dexterity upon superior material' would improve on the North American bridges.

In the spring and summer of 1814, Telford made over 200 experiments to determine the tensile strength of wrought iron. These tests were mostly performed at Brunton's London Manufactory.[95] Telford tested wrought iron with diameters of 1.27 mm to 48.10 mm (0.05 to 1.50 in), and on lengths varying from 9.4 m to 274.3 m (31 to 900 ft). The experiments were made perpendicularly, horizontally and with different degrees of curvature. It was found that the mean strength of a bar of good malleable charcoal iron 645 mm^2 (one inch square) was 407 Mpa (59 ksi) with a low of 372 MPa (54 ksi), and that an iron wire one tenth of an inch diameter will suspend 317.5 kg (700 lbs), or 603 MPa (87.5 ksi). That is, the wire was found to support 1.6 times the load than the bar. It was also found that when the wire of one tenth of an inch diameter is suspended with a versed sine of one fiftieth of the chord line it will support one tenth of the weight suspended perpendicularly in addition to its self-weight, which is only 1.45 kg per 30.5 m (3 lbs 3 oz per 100 feet). When the versed sine was one twentieth of the chord the wire supported one third of the load it supported perpendicularly.[96] Results of the bar tests were published in 1817 by Barlow in his *Treatise on the Strength of Timber and Iron*, though the results of the wire tests were not published until 1826 in the third edition of the book.[97] Maybe the wire tests were not initially published because they could not explain the discrepancy between the ultimate strengths of the wire and bar.

From these experiments, Telford observed stretching commenced at about 70% of the breaking load or about 278 MPa (40.3 ksi). Paxton and Mun believe that figure is high,

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[93] Telford[2], p490.
[94] Provis, p13.
[95] Paxton and Mun, p90.
[96] Paxton and Mun, p90.
[97] Peters[2], p34.

probably due to inaccuracies in Brunton's testing equipment. A value of 185 MPa to 232 MPa (26.9 to 33.6 ksi) would have seemed more appropriate.[98] Telford considered it safest to adopt the least strength exhibited by any bar tested instead of taking the average. He concluded that a bar of 645 mm^2 (1 in^2) would suspend 27,442 kg (60.5 kips) under direct vertical strain.'[99] At the time, the word *strain* was used for what we call *stress* today. Therefore, Telford calculated the ultimate stress of wrought-iron to be 417 MPa (60.5 ksi).

For design, Telford adopted 232 MPa and 417 MPa (33.6 to 60.5 ksi) for the stretching and breaking limits of wrought iron. John Rennie, in 1809, and Capt. Samuel Brown, in 1816-17, had also conducted similar experiments but they do not seem to have noted when the value of yielding commenced.[100]

2.6.4 Telford Eliminates Small Link Chain Forms

Telford's investigations into chain strength confirmed that the small link chain was not the best for application to the suspension principle. For this purpose he required that the 'metal should be kept as far as practicable in straight lines and also have few joinings.' In 1814, Telford developed two principle cable designs, one with wire cables and the other with composite bar cables.[101] William Alexander Provis, Telford's assistant since 1805, and who was later to be resident engineer on the construction of the Menaï Suspension Bridge, developed the details for both proposals and published them in 1828.[102]

2.6.5 Wire Cable Design, 1814

The first proposals for using wire in bridge cables were advanced by Telford in 1814 for the design of the Runcorn Bridge.[103] Telford's design consisted of 42 cables. They were continuous, parallel strands of nominal 2.55 mm (0.01 in) diameter iron wire totaling nearly 24,140 km (15,000 miles).[104] In this design a higher ultimate load value per wire was taken for the cables with a curvature depth of 1/20 span – i.e. 310.3 kg or 600 MPa (684 lbs, or 87.1 ksi), than for the 1/50 span curvature depth roadway cables – i.e. 272.2 kg (600 lbs). This indicates that Telford considered a larger safety factor necessary for the latter curvature. From subsequent experiments at Ellesmere on a 274.3 m (900 ft) length of 2.55 mm diameter wire Telford obtained an average value of 285.8 kg, or 553 MPa (630 lbs, or 80.2 ksi). He constructed a wire model of the bridge 15.25 m (50 ft) long and considered to be 1/1200 of its strength. Each suspension cable was probably represented by a 2.55 mm diameter wire. The model, which was without proper joints or bracing, withstood a load of 1360.8 kg (3000 lbs), equivalent to about 1½ times the dead load on the central span. According to Provis the model could have carried considerably more weight with proper detailing.[105]

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[98] Paxton and Mun, p90.
[99] Provis, p15.
[100] Paxton and Mun, p90.
[101] Paxton and Mun, p90.
[102] Peters[2], p34.
[103] Peters[2], p34.
[104] Paxton and Mun, p90.
[105] Paxton and Mun, p90-91.

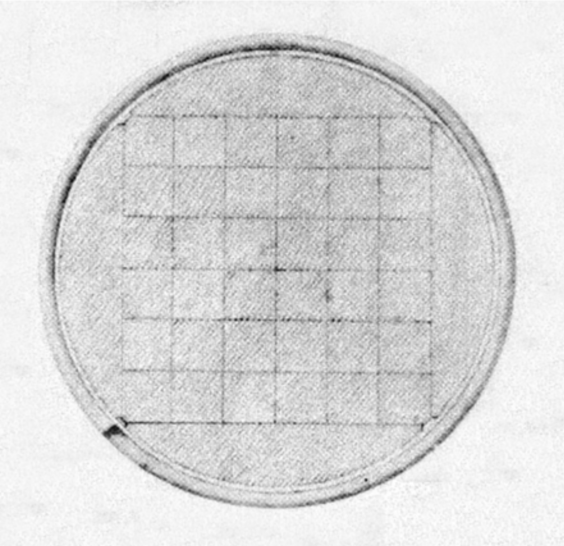

Fig. 25: Section of bar-cable design developed and tested by Telford. (Penfold, after Telford)

Even allowing for the greater strength of the wire cables, the estimates indicate that they would have cost nearly £149,000. That is about 75% more than the figure published in 1817 for an alternative bar cable design. The respective cost estimates for the wire and bar cable were £56 and £25 per ton for a design strength advantage in the ratio of about 37:27.[106] During the same period Telford was working on his proposal for the Runcorn Bridge, he was also working on a bridge with a smaller span of 61 m (200 ft) over the Mersey River in Latchford, near Warrington. Telford proposed wire and chain variants for both bridges. In both cases, the wire versions were abandoned for financial reasons. Telford never built a wire cable bridge. The first wire suspension bridge appears to have been the Schuylkill River Bridge built in the spring of 1816 at Fairmount, Pennsylvania.[107]

2.6.6 Bar Cable Design, 1814-1817

In the alternative bar cable design, shown in **Figure 25**, the cables were to have consisted of thirty-six 322 mm^2 (0.5 in^2) bars butt-welded to form continuous elements and making a 1935 mm^2 (3 in^2), with an iron segment on each face to enable the bars to be pressed firmly together. Water-proofing was to have been achieved by filling interstices with a mixture of beeswax and resin, covering the surface of the cable with flannel saturated with this composition, and the whole firmly wrapped round with wire. Buckles were to have been provided at 1.52 m (5 ft) intervals. A specimen length of cable was fabricated in association with Bryan Donkin, a civil engineer who supported its practicability.[108]

Telford envisaged, as he did for the wire design, that stresses in the bridge would be equalized between the upper and lower cables. He used the same strength factors, but applied an ultimate stress of 417 MPa (60.5 ksi). He designed for a maximum stress under dead and live load of about 204 MPa (29.6 ksi).[109]

Work on the Runcorn Bridge project continued for two years, during which time much discussion, consideration of other designs and further experiments were made.[110]

2.6.7 Experiments to Determine Strength of the Cable with Various Curvatures

Other experiments added to show what relative force would make a cable fail when stretched between two fixed points with various degrees of curvature. These tests were performed

[106] Paxton and Mun, p90.
[107] Peters^2, p34.
[108] Paxton and Mun, p92.
[109] Paxton and Mun, p92.
[110] Paxton and Mun, p92.

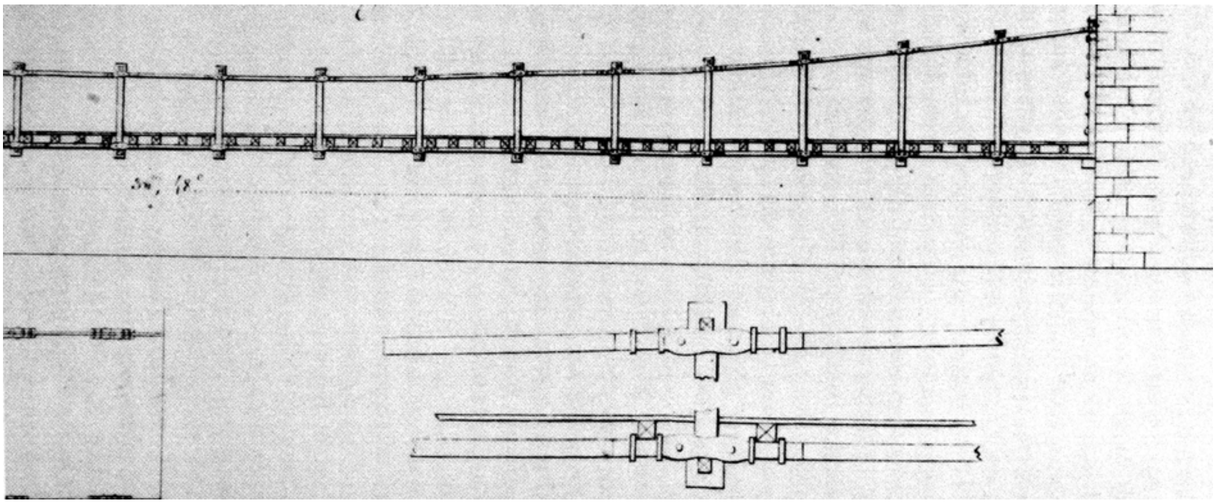

Fig. 26: Brown's alternative bar-chain proposal for the Runcorn Bridge (1817) was based on the above design that Brown used to construct a large scale model at his chain cable factory on the Isle of Dogs. Brown claims this bridge was erected in 1813 or 1814. (Sutherland)

using wires varying from 1.2 mm to 2.55 mm (0.048 to 0.100 in) in diameter. Chord lengths of 9.6 m, 30.5 m, and 274.3 m (31.5 ft, 100 ft, and 900 ft) were tried with the deflections of 1/30th and 1/20th of the chord length. It was found that a wire of 2.55 mm diameter broke under a load 1/10th of that it could support by direct vertical pull when the load was distributed at three points equidistant from each other and the supports and the curvature was 1/30th of the chord line. When the curvature was 1/20th of the chord line, the 2.55 mm diameter wire could support one-third of what it could under direct pull with the same loading points.[111]

It was concluded that as long as the weight of the bar itself was less than that required to tear it asunder, there would be a surplus of power to employ in suspending a roadway. Based on the tests, Telford concluded that, in principle, a span of 304.8 m (1000 ft) was within the maximum extent to which iron bars could be stretched. Therefore, there would be enough capacity to support a roadway.[112] Paxton and Mun contend that these conclusions are not realistic. They consider Telford's design value of 278 MPa (40.3 ksi) obtained from Brunton's equipment too high, leaving too little a margin of safety for the design.[113]

2.6.8 Arrangement of the Bar Cables and Their Rejection Due to Cost

The chains, and every other part of the Bridge to which tension could be applied, were specified be of the best malleable[114] iron. There were to be 16 main chains, each fabricated as described above.[115]

The difference in price between the wire and the bar cables was not great, and both would cost far more than an eye-bar chain.[116] Using a welded bar-cable in lieu of the wire clearly

[111] Provis, p15.
[112] Provis, p15.
[113] Paxton and Mun, p92.
[114] malleable iron = wrought iron, which has less than 0.06% carbon content
[115] Provis, p17-18.
[116] Peters^2, p34.

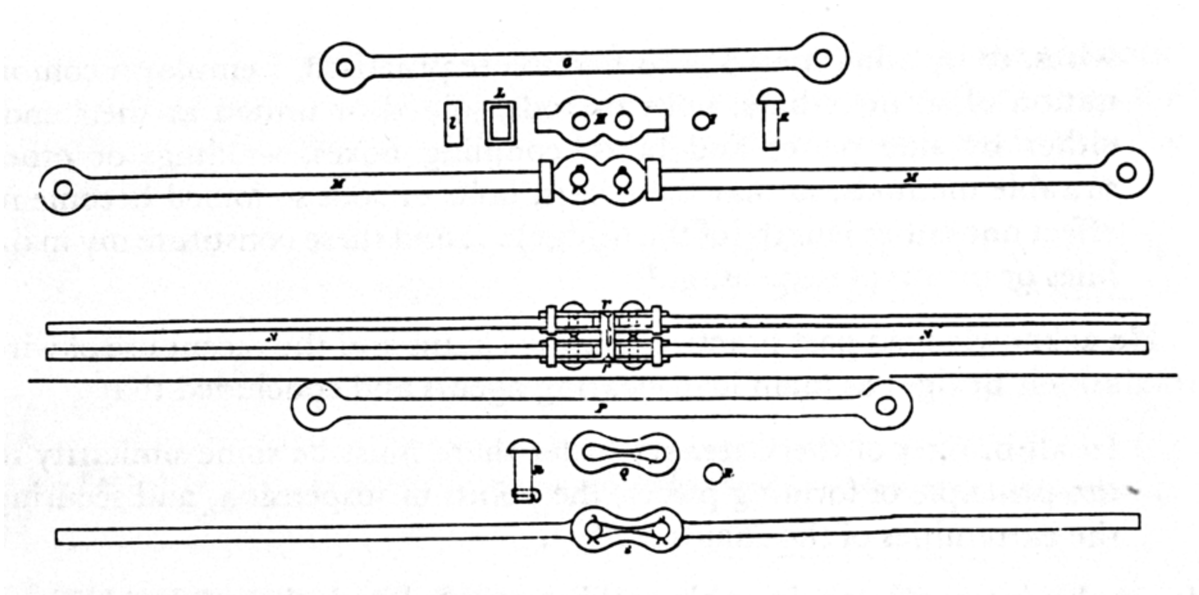

Fig. 27: Drawing of chain bars and connectors from Samuel Brown's 1818 patent. (Sutherland)

offered no advantage, particularly when the wire was 1.6 times stronger than that of the bar. With the costs still too high in either case, Telford modified his design one more time.

2.6.9 Modified Bar Cable Design, 1817

A more economical version of the cable for the Runcorn Bridge emerged in July 1817. As part of a review of various suspending chain forms, Telford visited the chain cable manufactory of Samuel Brown on the Isle of Dogs in February 1817. There, he examined and traveled across a large scale model of an iron suspension bridge, presumably the one about 30.5 m (100 ft) span which Brown later stated that he had erected in 1813 or 1814.[117]

With suggestions from Telford, Brown submitted a proposal for the Runcorn Bridge. According to Telford, the modified Brown design consisted of 8 main chains of flat 127 mm x 38 mm (5 in x 1.5 in) links. The spans and curvature depths were the same as Telford's 1814 Runcorn proposal. (**Fig. 26**) Brown's design had a suspending power of only 700 tons, less than 1/3 of that provided by Telford's alternative design, described below.[118]

Before they met on the Isle of Dogs, Telford and Brown developed their suspension bridge systems independently. Telford and Brown were unable to agree on the form, cross sectional area and curvature to be adopted for the main suspending members of the Runcorn project and, in the circumstances, there was little basis for a joint effort. From April 1817 onwards they almost certainly developed independent proposals.[119]

Within about four months Brown had prepared and submitted a patent for iron suspension bridges. It included a 304.8 m (1000 ft) span example in which the number of main chains was increased to sixteen, four rows of four flat bars. (**Fig. 27**). The patent did not cover individual chains consisting of a series of parallel eye-bars, possibly so as not to infringe on

[117] Paxton and Mun, p92-93.
[118] Paxton and Mun, p93.
[119] Paxton and Mun, p93.

an 1805 patent William Hawks, although Brown's links were much shorter.[120] Telford specifically rejected smaller links from a constructabilty standpoint, as well as what his early tests told him about he importance of keeping the links as straight as possible to have the most efficient transfer of stress. His work on the Menaï Bridge would support this view.

I do not have an image of Telford's modified chain design, so I do not know how it differs from Brown's form. At about this time the Runcorn Project was going to be canceled and Telford would return to the design of the Menaï Bridge. The chain design of the Runcorn can be considered the direct forerunner of the chain employed in the Menaï Bridge, therefore I have to assume that the Runcorn design included pinned, parallel links as seen below.

Telford introduced a number of improvements in his modified design in trying to simplify the construction and reduce costs. The cables under and adjoining the roadway were abandoned, suspension now being solely from the main cables, a change that eliminated the longitudinal sag in the deck. This refinement resulted in considerable saving in ironwork. A further reduction in suspended weight was achieved by adopting a much lighter deck, the result of which, with the retention of the previous cable arrangements, had the effect of reducing the design stress to about 179 Mpa (26 ksi). Other improvements were the lowering of the cables from 4.6 m to 2.1 m (15 to 7 ft) above the roadway at mid-span while maintaining the same cable curvature, and in achieving a more direct line of anchorage. In the 1814 designs the suspension lines were carried over cast-iron frames of quadrant elevation at each side of the bridge, a development of the 1811 Menaï centering proposal.[121]

2.6.10 More Experiments – Force to Pull a Cable into Position

With Donkin, Telford conducted tests to determine the force required to pull each cable into position. A bar chain with a cross-sectional area of 564 mm^2 (0.875 in^2) was suspended between points 38.1 m (125 ft) apart and the forces required to bring the chain to curvature depths of 1/15.6 to 1/20 span were ascertained. Telford concluded that a curve with a central deflection of 1/20 span force of 2½ times the weight of the chain was required. This finding was later applied in the design of the Menaï Bridge.[122]

2.6.11 Socio-Economics and the Assurance of Physical Tests

With financial commitments to the project not meeting expectations, the Bridge Commission, decided to verify the strength of iron at full span independent of Telford. An experiment was made using an iron rod 304.8 m (1000 ft) long over a valley near Liverpool. The strength of the iron was found to exceed the values calculated by Telford. This outcome and the public exhibition of the findings resulted in greater support for the scheme. By early 1818, subscriptions had reached about £25,000, but this was still insufficient. A 240 : 1 scale model of the bridge was made, probably in 1817, and existed at Ellesmere Port in 1905.[123]

[120] Paxton and Mun, p93; Peters[2], p33.
[121] Paxton and Mun, p92.
[122] Paxton and Mun, p94.
[123] Paxton and Mun, p95.

2.6.12 Legacy of the Runcorn Bridge Design

The Runcorn Bridge was not built due to lack of funding, but it had an important and lasting influence on the general development of suspension bridges. As Provis commented, the project established a new era in the art of bridge building and 'the publication of Mr. Telford's design led to the construction of bridges and piers on the suspension principle in almost every part of the kingdom.'[124] Telford's work, and to a lesser extent that of Samuel Brown, gave technical credibility to suspension structures.[125] Additionally, these first suspension bridges provided important smaller scale tests to affirm basic principles Telford was to incorporate in his Menaï Bridge.

Telford's energies had not been wasted. The results he had obtained from his experiments and the information he had collected were all to come into use in the design of the Menaï Bridge. Telford's experimental work on the strength of iron in tension was unprecedented in range and detail and contributed to 'strength of materials' knowledge for many years through the publications of Peter Barlow, Navier, Charles Stewart Drewry and others. Telford's proposed use of parallel wire cables in 1814, supported by the making and testing of a 50 ft. long wire model, is of considerable historical significance in the evolution of the suspension bridge, His proposal was of unprecedented scale and conceptually close to modern practice.[126]

2.7 Menaï Suspension Bridge, 1818-1826

2.7.1 The Menaï Bridge, 1818-1820

The publication of Telford's Runcorn Bridge design led to an enquiry in the latter part of 1817, by the Chancellor of the Exchequer, as to the practicability of a suspension bridge over the Menaï Strait. On 16 February 1818, Telford was on site and by May had submitted an outline plan and report, for a 16 cable bridge at Ynys-y-moch with a 170.7 m (560 ft) center span, supported from cast iron tower frames with back-stays tied into masonry approach arches.[127] (**Fig. 28**)

The shores at Ynys-y-Moch afforded easy access and excellent foundations. The design would span the whole channel between the low water lines, and the roadway would be kept uniformly 30.5 m (100 ft) in height above the top of a spring tide, leaving the whole of the navigable water-way uninterrupted. The height of the towers was proposed to be 15.2 m (50 ft) above the level of the roadway. The main chains were to have a deflection of 11.3 m (37 ft).[128]

The 1818 proposal for Menaï Bridge included wire cables, which were later changed to bar-cables like those invented for the Runcorn Bridge.[129] Telford's proposals to use wire and continuous iron bar suspension members in lieu of chains demonstrates an efficient approach to suspension design since chains are necessarily heavier because of their highly

[124] Paxton and Mun, p95.
[125] Provis, p13.
[126] Paxton and Mun, p102.
[127] Paxtons and Mun, p95.
[128] Provis, p17-18.
[129] Paxton and Mun, p96.

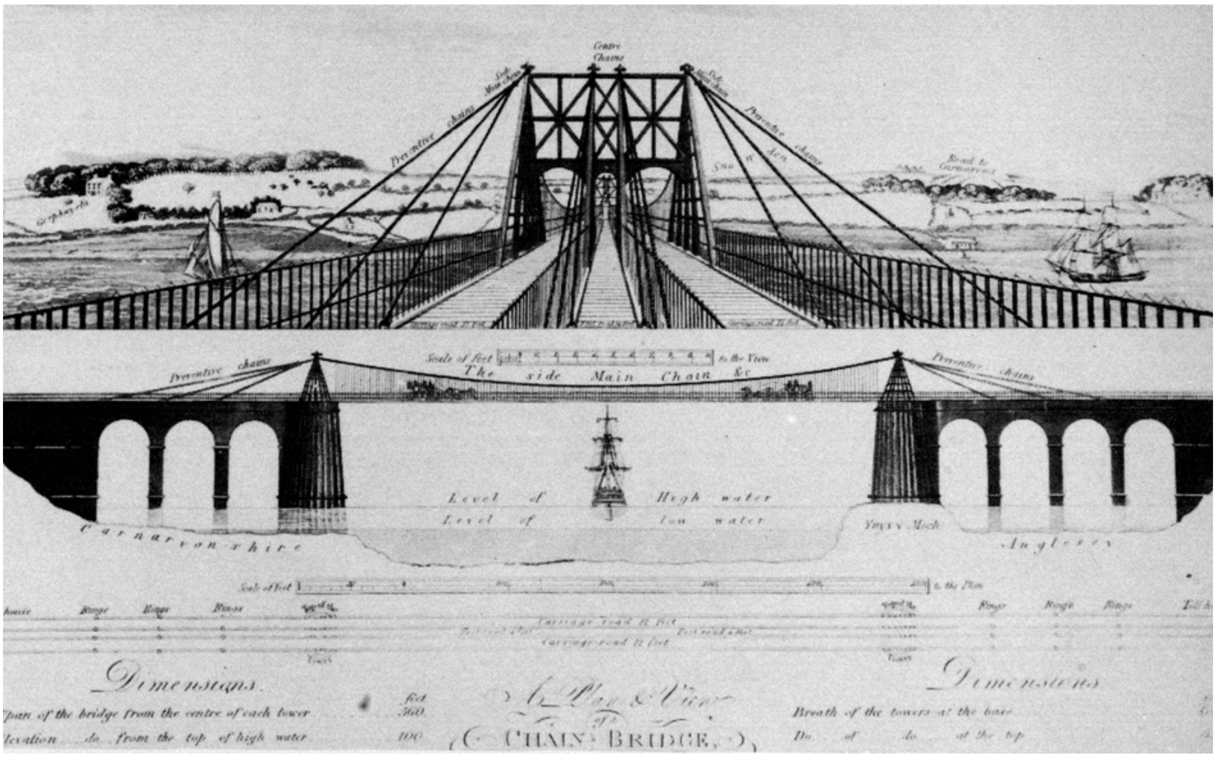

Fig. 28: Telford's 1818 Menaï Suspension Bridge proposal using wire cables anchored like stays into masonry of the approach viaducts. (Penfold)

stressed link connections. This is one of the earliest examples of what is now modern practice with respect to the use of steel wire cables.[130]

2.7.2 Strength of Iron

Telford's calculations assumed the breaking stress of a bar to be 552 MPa (80 ksi) and multiplied this figure by the cross-sectional area of the bar to give a 'suspending power' of 81,650 kg (180 kips). While the term *stress* was not then in use – it was often referred to as *strain*, it is clear that Telford understands what it is and how it differs from the actual capacity of a structural member of a given cross-sectional area.[131]

2.7.3 Wire to Bar-Chain

The decision to abandon the bar-cable in favor of a bar-chain was taken some time between April 1819 and July 1821. Telford was undoubtedly influenced by Brown's eye-bar links, which were successfully to construct the Union Bridge over the Tweed in Berwickshire, completed in 1820.[132] (**Fig. 13**) The Union Bridge has a single 133 m span. Brown used round eye-bars with welded eyes. (**Fig. 29**) The Union Bridge is one of the few that Brown built that still stands. It served as a model for the first wire bridges in Switzerland and France.[133]

[130] Penfold, p54.
[131] Penfold, p54.
[132] Penfold, p100-102.
[133] Peters^2, p33.

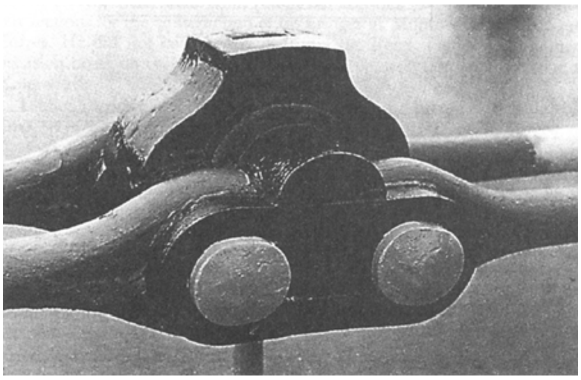

Fig. 29: Chain link and hanger connection of Union Bridge. (Sutherland)

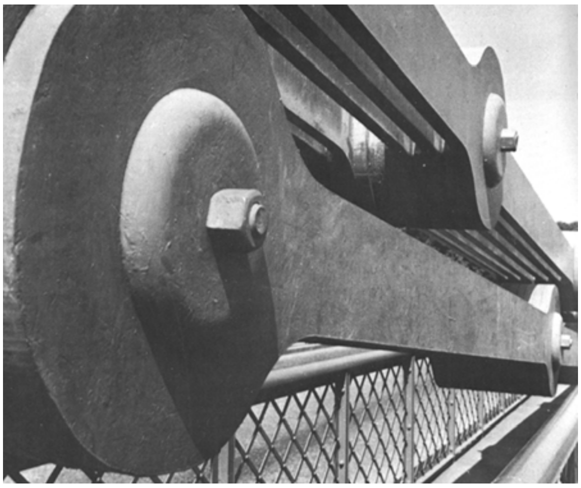

Fig. 30: Menaï chain link (replacement, similar to original). (Ellis)

Employing Brown's design as a baseline was the conservative thing for Telford to do. Brown's Union Bridge 'proved' that Telford's idea could work and using the eye-bar was preferable to using an untried form on a project of unprecedented scale. Telford may also have wished to accommodate Rennie's preference for chains. Whatever the reason, the bar link was the most practicable solution at the time.

Telford improved upon Brown's basic design by cross-bolting the bars in parallel instead of resting T-bar hangers on top of the individual lines of chain. In this way, Telford's chains acted uniformly as one unit while each line of links acted independently in Brown's system. The chain link forms of Telford and Brown can be compared in **Figures 29 and 30**.[134]

2.7.4 Bar-Chain Dimensions

The Menaï Bridge chain consisted of five parallel eye-bars with rectangular cross-sections. They were screw-pinned together near their ends to short connection plates, to which the suspenders were attached.[135] Each bar and a plate assembly was intended to be exactly 3048 mm (10 ft), but the boring of the eyes of the main chain bars and plates lengthened them.[136] The chains were arranged in four rows of four with suspension from alternate pairs in each line.[137]

2.7.5 The Commencement of Work and Power Politics

Telford appointed William Provis, his longtime assistant, Resident Engineer in June 1818. Work commenced with the employment of the first man, a carpenter, in July 1818. The first stone was laid on August 10, 1819. In the interim, preparations were made to secure good stone for the masonry piers, approaches and abutments. Special quarries had to be opened for this purpose to ensure a supply of good quality of stone as close to the construction site as possible. During this year, opposition to the project reorganized and more hearings were held to hear arguments for and against the project. It was not until May 1819 that Parliament signed a law authorizing the construction of the Menaï Bridge, saying it is 'of the greatest public importance' that the bill should pass into law.[138]

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[134] Penfold, p100-102.
[135] Paxton and Mun, p96.
[136] Provis, p56.
[137] Paxton and Mun, p96.
[138] Provis, p22-23.

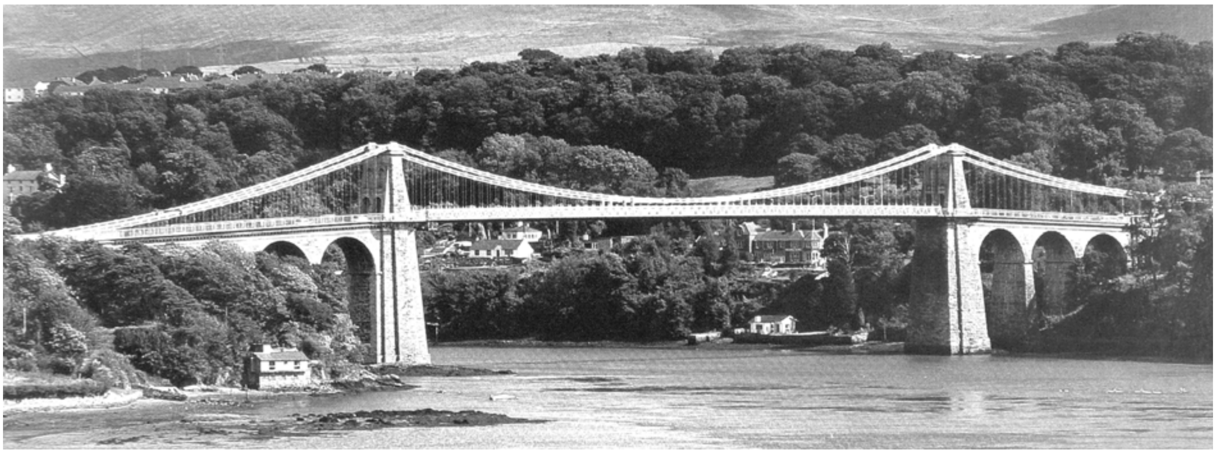

Fig. 31: The Menaï Suspension Bridge. Completed 1826, Telford. (Brown)

2.7.6 The Menaï Bridge, 1821-1826

After the adoption of bar-chains, two principal modifications were made to Telford's original design. The height of the towers was increased from 37 ft. to 50 ft. above the level of the roadway. As a result, the chains had to be lengthened and it was decided to anchor them in rock at the abutments rather than splayed and anchored in the masonry of the approach piers.[139] (**Figures 28 and 31**) The suspenders of the side-spans seen in **Figure 31** do not support the roadway but do stiffen the cable against undulations caused by moving loads across the bridge deck.

The recommendation to increase the tower height was made by Davies Gilbert, a mathematician and Holyhead Road Commissioner, based on the results he obtained while investigating the properties of the catenary curve as applicable to suspension bridges in 1820. Gilbert calculated the maximum tension in the main chains of the 1818-19 Menaï Bridge design to be equivalent to nearly three times the weight of the chains but assumed a central deflection of only 7.6 m (25 ft) instead of 9.1 m (30 ft). He was able to demonstrate that if the depth of curvature were doubled to 15.2 m (50 ft), the maximum stress would be reduced by nearly half.[140]

The second major design modification was to increase the cross-sectional area of the main chains from 123,870 mm^2 to 167,741 mm^2 (192 to 260 in^2). Telford did not consider this or the increase in tower height to be necessary. But he deferred to the opinion of Rennie in respect to the cross-sectional area, and to Gilbert and Barlow for the depth of curvature.[141]

As the Menaï design developed, the originally proposed cast-iron towers were replaced with traditional masonry. This probably had to do with concerns about the stress introduced by thermal movement of the chains; and the unequal stress produced by the arrangement of the back stays on one side of the bridge not being of an equal angle to the chain of the center span. Telford must have been concerned that such stresses might introduce too much tension and fracture the cast iron. Another factor in the decision to not use cast iron may

[139] Paxton and Mun, p96.
[140] Paxton and Mun, p97.
[141] Paxton and Mun, p96.

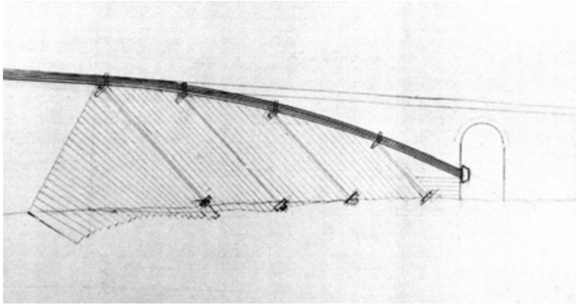

Fig. 32: Anchorage as designed for the modified bar-cable design of the Runcon Bridge project. (Penfold, after Telford)

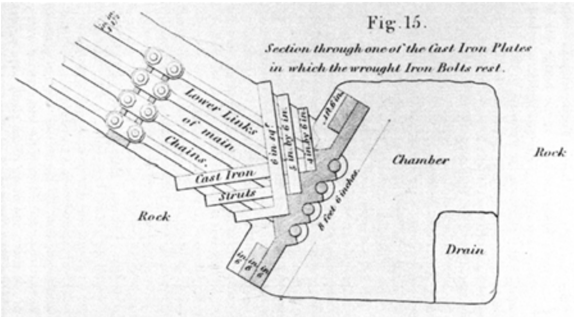

Fig. 33: Anchorage as built for the Menaï Suspension Bridge. (Penfold, after Provis)

have been an emerging understanding of impact loading and its application to cast–iron bridge construction.

To secure the masonry towers, over which the chains would bear, against the effects of any motion and unequal stress from the chains, Telford had all the masonry doweled. To do this effectually, two holes, about 32 mm (1.25 in) diameter, were drilled through every stone of every course, and 152 mm (6 in) into the course below. These holes were then filled with Parker's or British cement and wrought-iron dowels. The dowels were 305 mm (12 in) long and 25.4 mm (1 in) in diameter, were driven through the cement to the bottom of the drilled holes, so that each had a hold of 152 mm (6 in) in each of the two courses. The cement completely filled the spaces round the dowels and, with the upper parts of the holes filled with grout, protected them from corrosion.[142]

In the latter part of 1822, progress on pier building made necessary the finalization of the saddle and anchorage designs. A comparison between the revised Runcorn Bridge and the improved anchorage in rock at Menaï is shown in **Figures 32 and 33**. The piers had reached roadway level, and before proceeding further it was decided to increase the tower heights by a further 0.6 m (2 ft) in order that the roadway at mid-span could be lifted by a similar amount, thus obviating the deck from sagging below a horizontal line with temperature changes.[143] Later, a winter-summer differential of 280 mm (11 in) at mid-span was observed associated with a movement of about 38 mm (1.5 in) at each saddle.[144]

2.7.7 Ironwork Manufacture

William Hazledine was contracted for the manufacture and delivery of the ironwork soon after the drawings had been completed in July 1821. The ironwork was to be of 'best Shropshire hammered iron'. It was manufactured at Upton Forge and finished and tested in Hazledine's Coleham workshops in Shrewsbury. Most of it was transported via the Ellesmere and Chester Canals and then by sea from Chester to Menaï. Every operation in connection with the manufacture, finishing and testing of the ironwork was performed under the control of John Provis, brother of William Provis, the resident engineer for the bridges at Menaï and Conway, which was under construction not far from the Menaï site. The scale of the work was unprecedented. The sixteen main chains were each 521 m (1710 ft) long and consisted

[142] Provis, p36-37.
[143] Penfold, p102-103.
[144] Paxton and Mun, p97.

of 14,960 eye-bars about 2896 mm (9.5 ft) long, some 16,000 connection plates about (1.5 ft) long, and 6,000 76 mm (3 in) diameter screw-pins 406 mm (16 in) long.[145]

Hazledine's facilities were originally inadequate to meet the technological challenge of the work and the first cargo of main chain bars was not delivered to Menaï until 31 October 1822. In the winters of 1822-1823 and 1823-1824 the forge at Upton was flooded several times.[146]

2.7.8 Quality Control of the Chain Links

Notwithstanding all the care that had been taken at Shrewsbury to make the main chain bars of equal length, it was found impracticable to get them out of the smith's hands without some little differences. Before a bar was removed from the shop, a gauge, consisting of a plain rod of iron with a circular plug at each end, was tried into the eyes of the bar, and if the plugs dropped in correctly, the bar was considered to be of the exact length. It often happened that they would not fit exactly, and the bar was then either lengthened or shortened until the gauge fitted. However, the gauge could not always be applied when the bars were at the same temperature, which resulted in bars of incorrect length when even after adjustments were made. Another problem was the impossibility of producing the iron with uniform quality, resulting in bars that did not contract equally when the temperature was equally reduced.[147]

It was determined to bore each eye of every bar when cold. This was done on site using a specially constructed machine made of castings that were then on site fitted up with the necessary new work. The boring operation secured two benefits: it brought all the bars to an equal length at their bearing points and their eyes were made perfectly at right angles with the length of the bar where the pins would touch, thereby ensuring the equal bearing of every bar, in every part of its thickness.[148]

Even so, it did not prove an easy task to achieve the parallel five-bar chain. These and other setbacks resulted in insufficient ironwork being available at the bridge site in the summer of 1824. In retrospect, Thomas Rhodes, who had previously worked with Telford on the ironwork of the Caledonian Canal and who supervised the ironwork fixing at Menaï Bridge, thought that link manufacture could be improved in the future by turning the pins true, boring the links correctly to length and passing their ends through a rolling mill.[149]

2.7.9 Strength Testing of the Chain

The whole of the suspended ironwork was to be proved before it was accepted for use, by applying an actual stress to it. Telford calculated that the actual strain that would be experienced was equivalent to 85 MPa (12.3 ksi). Accordingly, he instructed that each bar be subjected to a tension of 170 MPa (24.6 ksi), equal to a load of 35 tons.[150] Each chain link was actually subjected to a tensile load of 32.5 tons. The remaining load was calculated to have been applied by the friction of the testing machine. While the bar was under the maximum testing stress, it was hit sharply with a hammer on its side and examined for signs of fracture. If no fracture appeared, the strain was relieved and the link was put in a gauge

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[145] Penfold, p103.
[146] Paxton and Mun, p98.
[147] Provis, p43
[148] Provis, p43-44.
[149] Penfold, p103 and 106.
[150] Gibb, p171.

with two fixed pins. If the link fit, i.e. had not been elongated, it was accepted, stamped with John Provis' proof mark (an indented cross), and set aside for use.[151]

Of 35,649 bars and plates tested about 6.7% were discarded, most of these bars were either too long or too short. Many of the plates were imperfectly welded under the forge hammer.[110] A number of the bars failed near their ends, probably from repeated heating and cooling while forming the eyes.[152]

2.7.10 Corrosion Protection

The question of how best to preserve the iron from corrosive effects of the atmosphere was addressed, but none of the experiments that were tried was completely successful. It was not possible to get any coating substance to take a firm hold of the iron, and consequently chipped off when bruised or rubbed. This was especially the case if considerable oxidation had taken place before the coating was applied, which was found quite impossible to prevent it completely. A bar began to oxidize as soon as it was taken from under the forge hammer. If a coating was applied as expediently as possible, it was found that the iron could be adequately protected from corrosion. However, the ironwork for the bridge had to be conveyed five miles from the forges at Upton to where it was to be finished at Shrewsbury, then to be carted 16 km (ten miles), boated along the Ellesmere and Chester Canals 87 km (fifty-four miles) to Chester, and conveyed from there by sea 96 km (sixty miles) to the site. It was uniformly found that when it arrived there that much of the artificial surface had been rubbed or knocked off, and in these parts rust had taken place.[153]

Provis writes,

> The plan finally adopted was to take each piece of iron after it had been finished and proved, clean it as perfectly as possible from the oxide and dirt, then heat it in a stove till the hand could only just be borne upon it, and afterwards immerse it in linseed oil. After remaining in the oil a few minutes, it was taken out, returned to the stove, and the oil dried on the iron by means of a moderate heat applied for three or four hours. When taken the second time from the stove, the oil was found to have dried to a thin hard varnish, which completely preserved the iron from the atmosphere, until rubbed off by friction.[154]
>
> The difficulty of preserving iron nearly altogether from the effects of the atmosphere does not appear insurmountable, if the iron could be secured from friction, and particularly if the coating that is applied possesses elasticity; but if iron work is to be subjected to rubbing and knocking about, I am not aware of any thing that can be applied to prevent its partial corrosion. These observations might induce some to suppose, that the Menaï Bridge must necessarily be a structure of a very perishable nature, but this need not be the case; it can be preserved to a very distant period by merely keeping the iron work covered with paint, or any other substance that will exclude the atmosphere. Or should it at any time be weakened, either from continued

[151] Provis, p34-35.
[152] Paxton and Mun, p98.
[153] Provis, p35.
[154] Provis, p35-36.

> friction at the joints, or from long exposure, [the bridge] is so constructed that nay part of it may at any time be taken out and replaced by new work.[155]

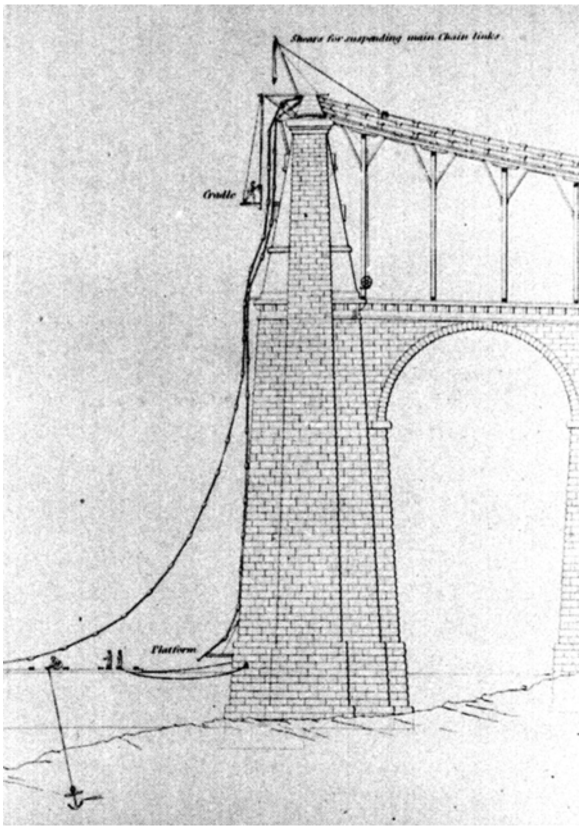

Fig. 34: Erection of the main chains of the Menaï Bridge. (Penfold, after Provis)

2.7.11 Main Chain Tunnel Links, Corrosion and Securtiy

Chain links to be anchored in the tunnels were made of larger stock to ensure adequate strength should corrosion compromise even half of the links in the tunnel. This precaution was taken because of the difficulty in maintaining the links in the confined and damp environs of the tunnels.[156]

2.7.12 Corrosion Protection and Link Assembly

Between each bar, plate, and washer, there was inserted two thicknesses of strong flannel, saturated with white lead and oil. This was intended to protect the linseed coating from rubbing off from friction between the links. Later, Borradaile's patent felt was substituted, having been found to better answer the purpose of jointing than the flannel.[157]

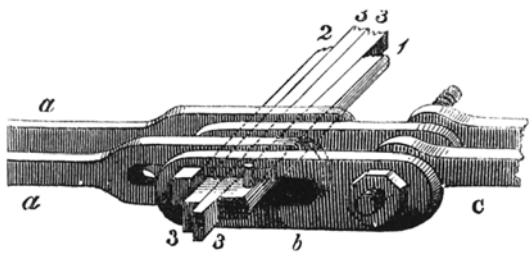

Fig. 35: Erection of the main chains of the Menaï Bridge. (Penfold, after Provis)

2.7.13 Fixing and Raising the Chains

In March 1823, erection of ironwork was started and Thomas Rhodes, who had been Matthew Davidson's assistant, came to take charge. The main chains were swung across a narrow valley to test their tensile strength. A 1:4 scale chain was fabricated to work out the lengths of the vertical suspending rods. Provis explained that the information might have been obtained by calculation, but 'with a practical man, an experiment is always more simple and satisfactory than theoretical deductions.'[158]

The most intensive period of ironwork erection began in the spring of 1824. The 25.4 mm x 82.6 mm (1 in. x 3.25 in) bars for the side spans were assembled on scaffolding in close proximity to their final positions. In the tunnels leading to the anchorages the chains were

[155] Provis, p36.
[156] Provis, p44.
[157] Provis, p45.
[158] Gibb, p171.

fixed from the castings towards the piers to meet the chains fixed from the saddles downwards. On completion of the side spans the chains for the central span were floated out, attached to a tail end of a chain hanging down the face of the Carnarvonshire tower, and then hoisted up to the saddles on the Anglesea tower by means of specially designed capstans. (**Fig. 34**) All 16 chains were erected from 26 April to 9 July 1825, 9 of them taking less than 2 hours to put in place.[159]

2.7.14 Adjusting for Length Differences

From the difficulty experienced in getting the chain links perfectly equal, it was predicted there would still be length differences when the chains were put into place and tensioned. Such differences might prevent achieving perfect parallelism, or cause unequal stress on the several chains. Therefore, this it was necessary to allow the capability to shorten or lengthen each chain, after it was fixed, thereby bringing them to proper adjustment.[160]

The design shown in **Figure 35** was adopted. Describing the device, Provis writes,

> The principle it will be seen was to make a set of bars and connecting plates, with circular eyes at one end the same as the common bars and plates, and the other ends with eyes elongated to 13 ½ inches [343 mm]. The bars and plates being put together, and screwed at the circular eyes by three inch [76 mm] pins as usual, the lengthened eyes of one end of a set of bars and of one end of a set of plates, were thereby brought opposite each other. At each end of the long eye (supposing those of a set of bars and a set of plates to form now but one) a pin, two inches thick [51 mm], with a head at one end and a cotter-hole at the other, was run through, and the cotter being dropped in, the whole were thus connected. One side of each of the pins was semicircular, so as to fit close to the end of the eye, and the other flat, so that if the two pins were brought close together they would touch throughout their lengths and thick nesses. If the long eyes were exactly opposite each other, the cotter-pins, being each two inches thick [51 mm], would leave a space between them of 9 ½ inches [241 mm], which being filled with iron wedges driven in tolerably hard, secured the chain at its shortest length. Now in order to lengthen the chain it was only necessary to slacken these wedges or to draw out a pair of them altogether, and the long eyes sliding past each other would bring the cotter-pins so much nearer, and to that extent elongate the chain: and if the whole of the wedges were taken out and the pins allowed to come close together, then the chain would be lengthened to its greatest extent, or 9 ½ inches [241 mm].[161]

In putting up the chains it was intended to keep them at nearly their mean length, so as to allow either shortening or lengthening. Allowance was made for some lengthening of the chain when the tension from its own weight brought all the joints hard up.[162] The mechanism Thomas Telford used to adjust the chains of the Menaï Suspension Bridge was comprised of keys, wedges, links, and mortised connections, typical of the earliest iron structures.[163]

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[159] Paxton and Mun, p98-99.
[160] Provis, p53.
[161] Provis, p53.
[162] Provis, p54.
[163] Peters[1], p40.

2.7.15 Maintenance – Replacing the Links while in Service

When putting up the second chain, several links were bent, making it necessary to take the damaged bars out and replace them with others. A screw-frame, designed for the purpose, was fixed to the ends of the damaged links. Screws were worked round by levers, until all the tension on the damaged link was relieved. Screw-pins at the ends of the link were then drawn out, the bars of the link taken away one at a time, new ones substituted, and the pins again put through and secured. This procedure proved to be efficient and safe.[164] Even a whole chain could be replaced without great risk and without interruption of traffic accrss the bridge.[165] The chains were, in fact, replaced in the 20th century.

2.7.16 Maintenance Designed into the Structural Detailing

Suspending rods pass between the chains and are installed in pairs so that the general strength of the bridge is not materially affected by taking one away.[166] The chains and the flooring, as well as the suspending rods, are constructed and united so that each of the parts may be taken out and replaced separately. The design allows the bridge to remain in service for most repairs.

2.7.17 Finalizing the Construction

The final phase of the Menaï Bridge's construction included raising the chains and installing the deck. The latter task proved troublesome as the effects of wind induced vibration began to be understood. Various construction details had to be changed as knowledge was gained by each instance of damage caused by these vibrations.[167]

The Menaï Suspension Bridge was opened January 30, 1826, with the driving of the first mail car over the bridge at 1:30 AM.[168]

2.7.18 Longevity of the Ironwork

In 1922, H.T Tudsbury and A.R. Gibbs made a thorough examination of the bridge. The chain bars in the tunnel sections leading to the anchorages were not corroded significantly. Of the main chain bars in the open air, the majority were found to be in good condition. 110 bars were considered to be badly corroded and seven excessively corroded. Of the 110 bars, in no case did any chain of five bars contain less than the intended design cross-sectional area. Strain gauges were applied to the chains at several locations. The maximum-recorded dead load stresses near the supports were found to be in the range of 89 MPa to 126 MPa (12.9 to 18.3 ksi). Later investigations using the Maihek extensometer indicated values between 100 MPa and 108 MPa (14.6 and 15.7 ksi). The breaking stress of bars tested by the National Physical Laboratory ranged from 284 MPa to 361 MPa (41.2 to 52.4 ksi) and the yield stress from 188 MPa to 233 MPa (27.3 to 33.8 ksi). After being continually stressed for about a century the iron still possessed what was probably its original order of strength.[169]

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[164] Provis, p67.
[165] Provis, p67.
[166] Cumming, p8-9.
[167] Provis, p38-70.
[168] Provis, p72.
[169] Paxton and Mun, p101.

Although there were some doubts about the strength of the structure, it was not until 1938-41 that Baker's deck and the original main chains were replaced. This reconditioning of the bridge does not seem to have been dictated so much by any particular structural deficiency. However, modern traffic demands required a greater live load capacity.[170]

2.8 The Influence of Telford's Suspension Bridge Work

2.8.1 On the General Development of Suspension Bridges

The Menaï Bridge design exercised a fundamental influence on the construction and development of suspension bridges from 1818 onward for several decades. It established the suspension bridge as the most economic means of providing the largest bridge spans for carriage traffic in the western world. This claim can also be made for Brown's Union Bridge. It was started about a year after the Menaï, but it was finished first,. However, Union Bridge was less than one-third the length, height and weight of the Menaï Bridge, and at a sheltered location.[171]

The project also provided the basis for improvements in suspension bridge design practice by example and through the publications of Davies Gilbert, Navier, Provis, Drewry and others, including Edward Cresy. The use of underground solid rock anchorages was a development of particular significance. Other improvements relating to the curvature and stresses, undulation and theoretical developments are reviewed below.[172]

2.8.2 Parallel Bar Chains

This innovation represented an improvement on Brown's arrangement, as it was more adaptable to large cross-sectional areas and the catenary of uniform strength. Leading designers, including William Tierny Clark and I.K. Brunel, adopted and improved on the basic Menaï Bridge chain using the latest structural theory and technology.[173]

2.8.3 Theoretical Developments

The Menaï Bridge project fostered the development of theoretical methods in suspension bridge design. By 1818, knowledge that forces are proportional to the sides of any triangle which are parallel to their direction, was used to determine the cross-sectional areas of wrought iron in tension by Samuel Ware, Peter Barlow, John Loudon, and William Chapman. Ware's theoretical investigations and catenary tables facilitated suspension bridge design from 1822. Olinthus Gregory promoted Ware's work in textbooks published in 1825 and 1833.[174]

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[170] Paxton and Mun, p101.
[171] Paxton and Mun, p102.
[172] Paxton and Mun, p102.
[173] Paxton and Mun, p102.
[174] Paxton and Mun, p105.

The most significant development was Davies Gilbert's work in encouraging the adoption of greater, more efficient depths of curvature in suspension bridge chains; and also safer chain stresses. His approximations were referenced well into the twentieth century.[175]

Eaton Hodgkinson's 1828 theoretical investigations and calculations vis-à-vis the Menaï Bridge encouraged a more scientific approach to design. Developments up to 1832 were summarized and evaluated by Drewry in the first British textbook entirely devoted to suspension bridges. From about 1825 onwards there was an increasing awareness amongst leading engineers of the value of a more theoretical approach to suspension bridge design that began to be reflected in the training and practice of the new generation of civil engineers.[176]

2.8.4 Undulation

The effects of undulation detailed in the authoritative accounts of Provis impelled work towards a solution of the problem by Clark, James Meadows Rendell, Barlow, Brunel, and Provis himself. For Hammersmith Bridge, completed in 1837, Clark made and wind-tested a model which he used to devise an arrangement of longitudinal trussed railings to counter undulation.[177]

2.8.5 Influence on Cable-Stayed Bridges

Although the bridge was not built, the first Menaï proposal for an arch constructed with a suspended centering was publicized in a parliamentary report and Nicholson's Journal. Suspension stay bridge designs were subsequently developed by Loudon, James Anderson, J. Seward and J.S. Brown of Redpath & Brown, Edinburgh, who designed and erected the Kings Meadows wire stay footbridge of 110 ft. span at Peebles in 1817. (**Fig. 16**) The basic concept envisaged by Telford eventually became an accepted engineering technique for large spans. His idea may have originated from the use of ropes to lower the iron arch ribs of Ironbridge into position.[178]

2.8.6 Project Management / Project Delivery

When reading of the life of Thomas Telford it is confounding to imagine how he managed such a large amount of work at the same time. Provis describes Telford's ability to manage projects and practices he introduced to bring better value to constructed projects. Provis writes,

> Telford had the art of devolving responsibility without losing control, of wielding a personal administration without becoming enmeshed in trivial detail… His influence was not confined to his immediate associates or his own staff; it touched most who came into occasional contact with him; and in directly the whole profession of civil engineers. It is moreover little realised how far Telford was responsible for the system of carrying out public works by contract, that is now accepted as a matter of course. Under him there grew up a body of contractors who brought their methods of business to a new standard, whether

[175] Paxton and Mun, p102.
[176] Paxton and Mun, p105.
[177] Paxton and Mun, p105.
[178] Paxton and Mun, p88.

> on the side of skill or on that of honesty. Such firms as Cargill and Simpson, though they have long ceased to exist, have had their imperishable effect on contracting as a business. Telford introduced or elaborated the system of monthly payments; of the retention of definite sums as guarantee of satisfactory workmanship and punctual completion; of the retention of definite sums as guarantee of satisfactory workmanship and punctual completion; of a period of maintenance during which the contractor is responsible for the state of new work. He broke down the Government rule that the lowest tender must be accepted irrespective of experience or capacity, or the adequacy of his financial standing. He devised procedure for dealing with disputes and disagreements ; although so far as he was himself concerned his own prestige, honesty and independence were so recognised that he was accepted as sole and final arbitrator in all his contracts, without challenge or complaint. In fact so great was the confidence in his sense of justice, that on occasion, being appealed to settle the dispute between the county authorities and the contractor over Tewkesbury Bridge, both parties agreed to proceed with the work on a completely redesigned bridge, without either contract or specification, on the basis of a single drawing signed by Telford, the prices to be left to him to settle.[179]

It is frustrating to realize that the battle in the construction industry between ‘least cost’ and ‘best value’ procurement is over two hundred years old. In the United States, best value is becoming the norm after decades of least cost procurement. Interestingly, in Switzerland the trend seems to be the opposite, and will no doubt lead to the detriment of overall construction and engineering quality.

2.8.7 Quality Standards in Construction

Poor construction practice by contractors is a timeless problem. Telford used the weight of his influence, which came from the sheer volume of work he generated, to improve the quality the contractors’ work quality. Provis relates,

> In the Highlands the strictness of his specification and the rigid inspection of John Mitchell had at the outset involved some of the contractors, unused to such a standard, in loss. That was not Telford’s wish. He felt strongly that a contented contractor was essential to the successful carrying out of a big programme; but he must also be competent. Telford, therefore, always endeavoured to use one or other of the small group of contractors that he had trained to his ways, selecting them individually, each for the class of work for which he was best suited. Many of the great public works contractors of later times derive directly or indirectly from those who worked for Telford: and have maintained the same ideals and spirit. *Success in engineering still depends on the co-operation of contactor and engineer.*[180] (emphasis added)

Today, public agencies and corporations can use their power to encourage “best practice” from the construction industry. But it must be a conscious and deliberate campaign.

[179] Provis, p186.
[180] Provis, p186.

2.9 Conclusions

Telford developed the basis of suspension bridge technology used for a little over a century. His initial ideas of parallel wire cable bridges were prescient of modern suspension bridge design. While the bar-chain concept was not solely his, some credit goes to Samuel Brown and his bar-chain, it seems unlikely that Telford would not have arrived at the same conclusions independently. It took Telford until 1818 to produce a design for a bar-chain only because he spent the previous four years developing more efficient designs using wires and continuous bars.

Telford's early explorations show an ambitious will to design the most efficient system. It is interesting to note how engineers like Telford push unknown limits, proceeding on a project knowing only by their engineering judgment that their idea is sound and can be built, yet the details of how to accomplish these feats are often unclear. The problems that arose were overcome by engineering ingenuity, perhaps some luck, and, yes, some science.

Telford, the consummate practical man who valued experience and physical models, is mainly responsible for the growth of the field of strength of materials. Telford's experiments on the properties of wrought iron and suspension cables form the foundation of much of the research done after, and the knowledge he created could be put directly to use.

The form of Telford's Menaï Bridge bar-chain was the product of a long development. It largely represents the compromise made between structural efficiency and economics. Even though Telford's wire cable had 1.6 times the strength, it was still far more expensive than the bar-chain design. The bar chain itself ideally represents the effects connections have on tension members. The eyes of the bars, necessary to transfer stress from one link to another, clearly make the chain much heavier than if constructed of a continuous wire or bar. Here too, a compromise is made between the dead load of the chain and maximum permissible live load of the bridge.

Maintenance issues are also considered in the design of the chain. The problem of corrosion was adequately addressed. The debate in the early 20th century concerning whether wire or bar-chain cables were preferable largely revolved around the issue of corrosion protection. It was argued that the chain cable could be more easily examined for corrosion and that corrective actions were easier to execute by designing a system for maintenance as Telford had done for Menaï. Telford designed the bridge to be replaced piecemeal without service interruption.

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BRITANNIA BRIDGE, WHY A TUBE?

A-03

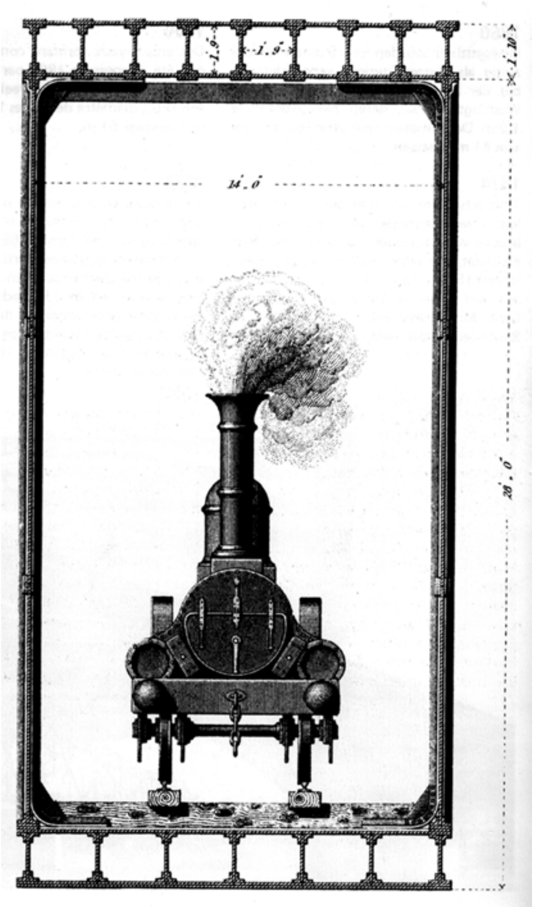

Fig. 1: Cross-section of the Britannia Bridge, Robert Stephenson, 1850. (Peters[2])

3.1 Introduction

The design and construction of the Britannia and Conway tubular bridges is a canonical achievement in the history of structural engineering. The research and development process of the tubular bridge is perhaps one of the best documented in engineering history. There are two contemporary accounts of the project written by persons intimately involved in the bridge's design, development and construction.[1] Robert Stephenson was the chief engineer for the bridges acting in his capacity as chief engineer for the entire Chester and Holyhead Railroad. His project manager, Edwin Clark, assisted with design and construction of the bridge. Clark's book represents Stephenson's account as well as his own. Stephenson engaged William Fairbairn, a respected English industrialist, shipbuilder, and all-round iron fabricator, as a consulting engineer to develop the tubular concept. Fairbairn's book, largely written to defend his role in the conceptual design of the tubular beam, records in detail the experiments that led him to the tubular beam with cellular flanges characteristic of the Britannia Bridge. (**Fig. 1**) Fairbairn, in turn, engaged Eaton Hodgkinson, a respected experimental engineer, help develop a theoretical explanation for the structural behavior of a tube. Also, he defined a formula with which to calculate the strength of a tubular beam. Hodgkin son's work is recorded in various reports and Clark's book.

[1] Ref. Clark and Fairbain[1] in bibliography.

The railway from Chester to Holyhead formed an important part of the shortest line of communication between London and Dublin. The journey between Chester and Holyhead was 3 hours and 5 minutes. It was determined that about 25 minutes could be saved by building a railroad bridge across the Menaï Strait and the Conway River as Thomas Telford had done in 1826 with two suspension bridges to carry road traffic.[2]

When it was first suggested to cross the Menaï with the mail train, a study was performed by the Admiralty of the British Navy to determine if alternative harbors existed other than at Holyhead. In an 1836 report, the Admiralty determined that Holyhead was the best point of departure but if it was intended to construct a railroad it was not likely that a steam locomotive with a loaded train could traverse Telford's suspension bridge at Bangor. Further, they objected to any arch being built that might obstruct the strait navigation. Captains Black and Fair independently verified this conclusion.[3]

Parliament passed a bill authorizing construction of a bridge by the Chester and Holyhead Railway over the Menaï Strait in July 1844. Robert Stephenson[4], the son of pioneer railroad builder George Stephenson, was chief engineer of the Chester and Holyhead Railroad at the time and it fell to him to engineer a bridge to cross the strait.

In the construction of the Chester and Holyhead Railway, Stephenson had to overcome two formidable obstacles – the Menaï Strait and the Conway River. The Menaï has a deep and rapid tidal flow and both crossings required bridges of uncommon span, for the time, especially considering the need for sufficient strength and rigidity to support railroad loads. Moreover, the Admiralty specified that centering or other substructures, as was commonly used at the time to erect such large structures, could not be used.[5]

For the Conway Bridge, Stephenson chose to build his bridge parallel to the existing suspension bridge completed by Telford in 1826. Stephenson chose not to consider building a new bridge at either of the sites originally considered for Telford's bridge across the Menaï Strait.[6] Instead, he chose a site where a rock, called Britannia Rock, sat in the center of the strait. Stephenson determined that this rock was large enough to support an intermediary pier in the middle of the strait.[7]

The following chapter will examine why Stephenson chose to build a giant tube, an unprecedented structural type at the time, instead of more traditional structures like a suspension bridge or an unconventional system like a truss, which was then being developed in North America. In addition, the state of beam theory, developments in beam construction, and the knowledge that then existed about wrought iron (which had never been used on such a scale in any bridge or building structure to that time) will be examined. This chapter closes with a review of the influences this bridge had on structural engineering.

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[2] Dempsey, p46-47.
[3] Dempsey, p54-55.
[4] For the biography of Robert Stephenson, reference: Smiles, Samuel. *The Life of George Stephenson and His Son Robert Stephenson*, rev. ed. (New York, 1868), and Rolt, L.T.C. *The Railway Revolution: George and Robert Stephenson* (New York, 1962).
[5] Fairbairn[1], p1.
[6] **Ref. Appendix A-02**.
[7] Dempsey, p58-59.

3.2 Stephenson Considers a Suspension Bridge, 1844

3.2.1 State of Suspension Bridge Construction in 1844

In 1844, there were only two principle structural types for spanning long distances, the arch and the suspension bridge. Given the design constraints defined by the Admiralty and requirements specific to the railway (such as controlling gradients), Stephenson's most obvious solution for spanning the Menaï Strait was with a suspension bridge. Telford had done just that for the road bridge examined in **Appendix A-01**, which was located just one mile up the strait from the site chosen by Stephenson at Britannia Rock.

Since the completion of Telford's Menaï Suspension Bridge, numerous other suspension bridges were built, including William Thierny Clark's Hammersmith Bridge (1827) and I.K. Brunel's Hungerford Footbridge (1845), both in London. For the Hammersmith Bridge, Clark made and wind-tested a model from which he devised an arrangement of longitudinal trussed railings to counter wind-induced oscillations.[8]

While there were notably successful suspension structures built before 1845, there had been a rash of disasters in the 1830s. Bridges at Montrose (1830), Morpeth (1830), Broughton (1831), Yore (1831), Durham (c.1832), and, shown in **Figure 2**, Samuel Brown's Brighton Chain Pier (1833) seriously undermined confidence in this structural type in England until about mid-century.[9] An unusually severe gale seriously damaged Telford's Menaï Bridge on 7 January 1839. Emergency repairs to the deck were made and the bridge was put back into service by 21 January. Subsequently, William Provis, Telford's resident engineer during the original construction, replaced the whole of the deck with a stronger and stiffer design. The replacement deck was completed in the summer of 1840.[10] These bridges were principally damaged due to wind-induced oscillation with the notable exception of the Durham Bridge, which is referenced below.

Fig. 2: Brighton Chain Pier, Samuel Brown, 1823. Damage caused by wind-induced oscillations in 1833. (Peters[3])

3.2.2 George Stephenson's Recommendation to use Telford's Menaï Bridge, 1839

In 1839, George Stephenson proposed to use Telford's suspension bridge, then being repaired and strengthened, to transport railway cars across. He suggested that Telford's bridge could be used if the locomotives were disengaged and horses were used to haul the railway cars across, where another locomotive would then be engaged.[11]

Thomas Page studied George Stephenson's proposal to determine whether the Menaï Suspension Bridge had, or could be modified to achieve, the load capacity required to carry rail traffic. Page calculated that the chains, with a sectional area of 167,741 mm^2 (260 in^2),

[8] Paxton and Mun, p101 and 105.
[9] Paxton and Mun, p101.
[10] Paxton and Mun, p100.
[11] Rosenberg and Vincenti, p83, note 11.

were under a stress of 77.2 MPa (11.2 ksi[12]) due to a dead load of 786,420 kg (1734 kips), which includes the addition of 132,086 kg (291 kips) due to the repairs of 1839 and 1840. This calculation assumed the weight to be equally distributed to all the chains. It did not account for eccentric loading caused by undulation. This stress is 27.6 MPa (4 ksi) more than Telford and George Rennie *testified* the chain would be subject to in a report to a committee of the House of Commons 29 April 1819. Page assumed the weight of the railway cars was to be applied on one side of the longitudinal centerline and, incorrectly, that the load was borne by only one half of the chains. It was calculated that ten railway cars at 5080 kg (11.2 kips) each, without an engine, would impose an additional 10.3 MPa (1.5 ksi) on the chains, totaling 87.6 MPa (12.7 ksi), or 2,494 kg (5.5 kips). This exceeded more than the chain was designed for. Consequently, Page concluded, "The passage of connected railway trains would be injurious to the general stability of the bridge."[13] Considering the cost of a new bridge, it is amazing that a more thorough analysis of the *actual* capacity of the chains was not made instead of using a limit stress determined when the properties of wrought iron were less well understood and was used in a novel structural type of unprecedented scale.

3.2.3 Robert Stephenson Considers the Suspension Bridge Principle, 1844

Stephenson considered, and rejected, a suspension bridge on the Telford model. He based his opinion on his own experience with a small suspension bridge designed by Samuel Brown[14] that Stephenson and his father built for the Stockton and Darlington railway near Durham in 1825. He observed "that the train sank down and raised before and behind it a wave in the deck, which racked the bridge to pieces in a few years."[15]

Stephenson described his experience with the Durham Bridge in his *Report*, presented to the Directors of the Chester and Holyhead Railway in February 1846. Stephenson writes,

> The injurious consequences attending the ordinary mode of employing chains in suspension bridges were brought under my observation in a very striking manner, on the Stockton and Darlington Railway, where I was called upon to erect a new bridge for carrying the railway across the river Tees, in lieu of an ordinary suspension bridge, which had proved an entire failure. Immediately on opening the suspension bridge for railway traffic, the undulations into which the roadway was thrown, by the inevitable unequal distribution of weights of the train upon it, were such as threaten the instant downfall of the whole structure. These dangerous undulations were most materially aggravated by the chain itself, for this obvious reason, - that the platform or roadway, which was constructed with ordinary trussing, for the purpose of rendering it comparatively rigid, was suspended to the chain, which was perfectly flexible, all the parts of the latter being in equilibrium. The structure was, therefore, composed of two parts, the stability of the one being totally incompatible with the that of the other; for example, the moment an unequal distribution of weights upon the roadway took place, by the passage of a train, the cure of the chain altered, one portion descending at the point immediately above the greatest weights, and consequently causing some other portion to ascend in a corresponding degree,

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[12] ksi = kilo-pounds per square inch. 1 ksi = 6.9 MPa. 1 kip, or kilo-pound = 1,000 pounds = 453.6 kg
[13] Dempsey, p56-57.
[14] **Ref. Appendix A-02, p.A.36**.
[15] Hamilton, p460. Quote from Parliamentary Papers, *Report of the Commission Appointed to Inquire into the Application of Iron to Railway Structures*, 1849. The report mistakes the year it was destroyed (1830) for the year it was built (1825).

> which necessarily raised the platform with it, and augmented the undulation. So seriously was this defect found to operate, that immediate steps were taken to support the platform underneath by ordinary trussing; in short, by the erection of a complete wooden bridge, which took off a large portion of the strain upon the chains. If the chains had been wholly removed, the substructure world have been more effective; but as they were allowed to remain, with the view of assisting, they still partake of those changes in the form of the cure consequent upon the unequal distribution of the weight, and eventually destroyed all the connections of the wooden frame-work underneath the platform, and even loosened and suspended many of the piles upon which the frame-work rested, and to which it was attached. The study of these and other circumstances connected with the Stockton bridge leads me to reject all idea of deriving aid from chains employed in the ordinary manner.[16]

Stephenson was convinced that “the suspension principle is utterly inapplicable for sustaining railway traffic.”[17] Thus, he ruled out using the suspension bridge, at least as then constructed in common practice.

3.3 Stephenson’s Arch Proposal, 1844

Having eliminated, at least for the time being, the applicability of a suspension bridge, and having considered other alternatives such as a timber arch, Stephenson proposed a cast-iron arch with two spans of 137.2 m (450 ft) each.[18] At the time, Stephenson was convinced that the arch was the most suitable structure to span such a great distance while providing the requisite stability and stiffness the trains needed. The arches were designed to leave a clear height of 30.5 m (100 ft) from high water and spring at 15.2 m (50 feet) above high water.[19]

Since it was not possible to erect a centering in the strait, Stephenson proposed to build the half-arches on either side of the pier simultaneously using an early example of the balanced-cantilever method. Stephenson planned on connecting each half-arch with tie-rods.[20] Presumably, the other two half-arches would be constructed using a type of suspended centering as first proposed by Telford for his Menaï bridge,[21] or cantilevered and tied back to the abutments. The principle adopted for the half-arches in the center could also have been derived by the research of Sir Marc Brunel on brick cantilevers reinforced with iron strapping. (**Fig. 45**)

Stephenson’s arch proposal was rejected by the Admiralty because it would not permit any bridge to be built that obstructed navigation. It required that there be a clear opening between piers of 137 m by 31.4 m (450 ft x 103 ft). They also criticized the thickness of the proposed intermediate pier on Britannia Island for disturbing the wind flow.[22] The Admiralty had final authority in this matter since the navy was the principle means by which England projected its power.

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[16] Dempsey, p59-60.
[17] Dempsey, p59.
[18] Fairbairn[1], p2.
[19] Dempsey, p60.
[20] Dempsey, p60.
[21] **Ref. Appendix A-02, p.A.50**.
[22] Peters[1], p165; Clark, p8.

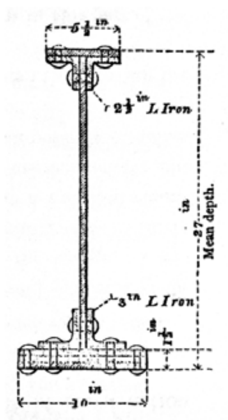

Fig. 3: Ware Bridge wrought-iron girder section with proportions appropriate to cast iron, 1841. Robert Stephenson. (Fairbairn[1])

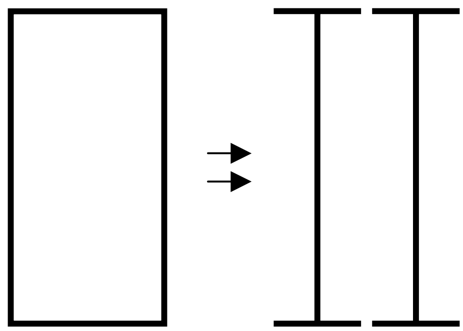

Fig. 4: Illustration of the double I-beam model Robert Stephenson thought could be applied to the analysis of a tubular beam, 1841. (Dooley)

The Admiralty's requirement eliminated the possibility of building an arch unless its springing was 15.2 m (50 ft) higher above the proposed position, which would substantially increase the cost of the piers and abutments as well as create a problem of grade for the adjoining levels of the railway.[23] There does not appear to be any evidence that Stephenson considered using a tied-arch, which was then being introduced in Great Britain.

After Stephenson's proposal to employ an arch was rejected, he was left with no choice but to engineer an entirely new kind of long-span bridge.

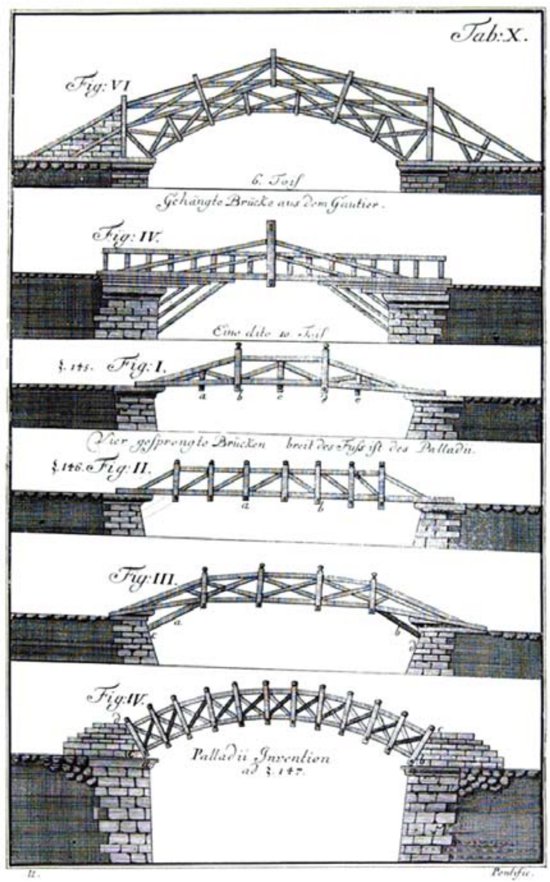

Fig. 5: Several wooden trusses illustrated by Leupold, 1726. (a) and (b): two bridges proposed by Gautier. (c), (d), (e) and (f) show for designs by Palladio, the first three of which are considered Palladio's interpretation of a friend's description of bridges he had seen in northern Europe. (Peters[3])

3.4 Stephenson Reconsiders a Suspension Bridge, and the Emergence of the Tubular Idea, 1845

After his arch proposal had been rejected, Stephenson again considered a suspension bridge. He was aware that a contemporary of his, James Meadows Rendel, had already used wooden trusses for the purpose of preventing oscillation in the platform of suspension bridges, however he thought a

[23] Dempsey, p60-61.

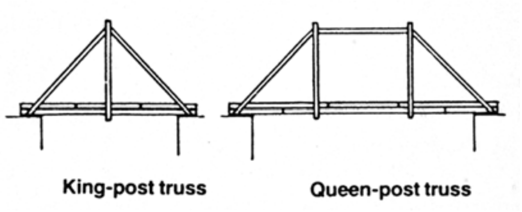

Fig. 6: (a) King-post truss; (b) Queen-post truss. (James[2])

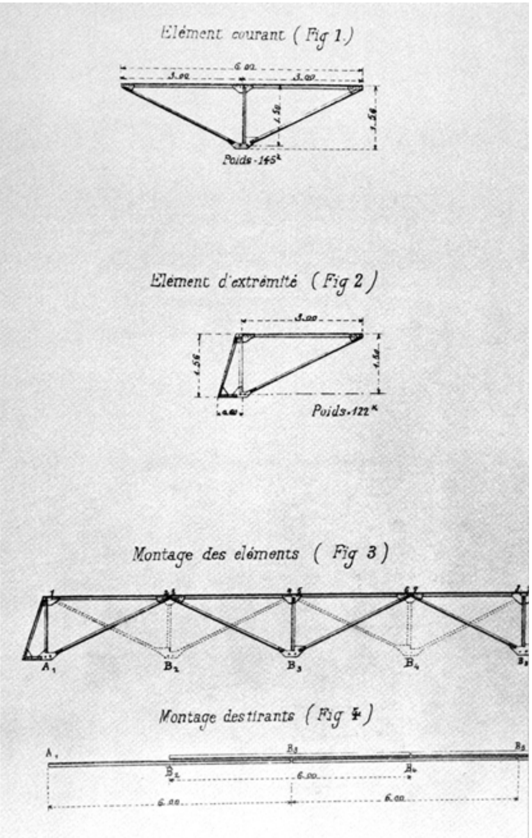

Fig. 7: Economic bridge system based on modular design, Eiffel, 1884. (Eiffel)

stronger trussing system would be needed. After considering a number of alternatives, Stephenson thought he would need a truss, with cross braces in the top and bottom, supported by suspension chains. In lieu of wooden trussing, Stephenson considered using wrought-iron plates similar to the wrought-iron **I**-section girders he used to build the Ware Bridge in 1841. (**Fig. 3**) The resulting form was a rectangular tube supported by suspension chains. Stephenson realized the tube could be viewed as a beam composed of two **I**-sections side-by-side. (**Fig. 4**) Simplifying the form in this way, typical of engineering, allowed Stephenson to consider it feasible to analyze such a structure based on methods already established for calculating the strength of cast-iron beams.[24]

3.5 Why did Stephenson not use a Truss?

3.5.1 The Question

The preceding account of how Stephenson got to the idea of a giant tube presents an obvious question, why did Stephenson not simply replace the members of a wooden truss with members of wrought iron of similar form? There is no apparent record that the truss was seriously considered by Stephenson on its own merit. To answer this question the general history of truss development and the state it had reached in England by 1845 will be reviewed.

3.5.2 Early Truss Development in Europe

The earliest recorded examples of trusses come from the Italian publications by Andrea Palladio, Vincenzo Scamozzi and Faustus Verantius in the late 16th and early 17th centuries. The plate shown in **Figure 5** is from Jacob Leupold's *Theatrum Pontificale…*

[24] Clark, p13-27. From Robert Stephenson's chapter "Introductory Observations on the History of the Design."

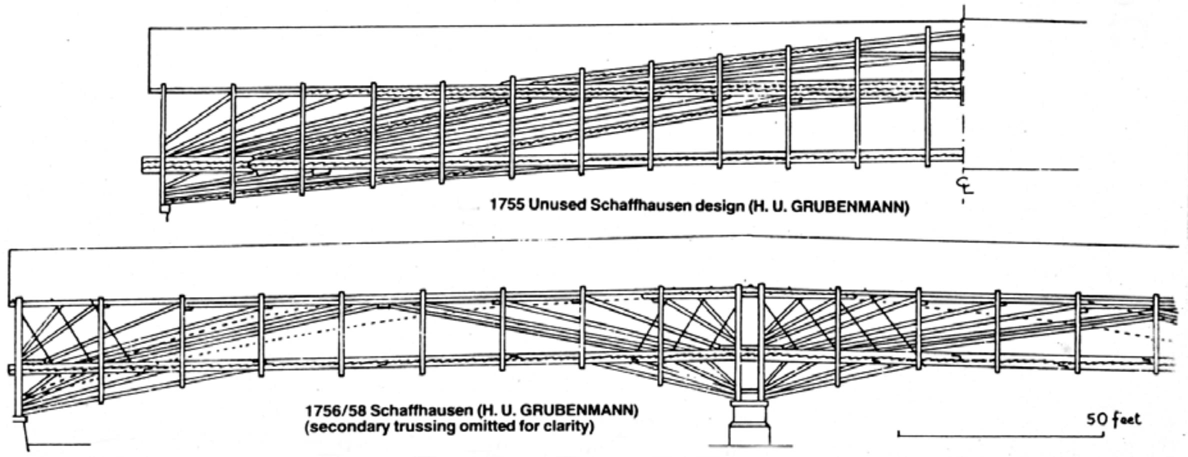

Fig. 8: Schaffhausen Bridge, H.U. Grubenmann. (a) First proposal for single span through-truss of 119 m (390 ft), 1755. (b) As built, two span through truss with spans of 59 m (193 ft) and 52 m (171 ft), 1756-58. (James[2])

published in 1726.[25] Palladio's designs are recorded in the last four illustrations (**Figs. 5 (c), (d), (e) and (f)**). Except for the last design, these illustrations are apparently his interpretations of descriptions given to him by a friend who had seen truss bridges in what is now Germany.[26] Tom F. Peters, a historian of building technology, explains that these bridges were designed by a system of overlays in which basic structural modules, principally the king and queen post, are multiplied to create ever-larger spans.[27] (**Fig. 6**) When viewed this way, the structure shown in **Figure 5(a)** becomes more rational and Palladio's bridges can be appreciated without the bias of our knowledge of trusses today. Gustave Alexandre Eiffel employed such a modular system for his *ponts economique*. (**Fig. 7**)

Until the end of the 18th century development of the long-span wooden through-truss bridge took place primarily in Switzerland, parts of Germany and the Tyrol region of Austria. In these areas, the basic raw material for such structures, long lengths of stout pine, was plentiful and cheap. Carpenters, rather than architects and engineers, were almost entirely responsible for this development. New ideas were tested in model form. By the second half of the 18th century, through-trusses were being constructed with spans up to 61 m (200 ft) using rectangular frames in conjunction with struts, polygons and arches. (**Figs. 8 and 9**) These bridges were typically covered to protect the wood from water damage.[28]

The greatest wood bridges built in Europe before 1800 are those built by the several generations of the Grubenmann family of Switzerland. The most celebrated bridge is the Schaffhausen Bridge built by Hans Ulrich Grubenmann in 1758. (**Fig. 8**) An early design for a single 118.9 m (390 ft) span was rejected in favor of a more conservative two span bridge with spans of approximately 58.8 m and 52.1 m (193 ft and 171 ft). The actual dimensions are somewhat in dispute according to J.G. James.[29]

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[26] James[2], p119.
[27] Peters[3], p10.
[28] James[2], p116.
[29] James[2], p124.

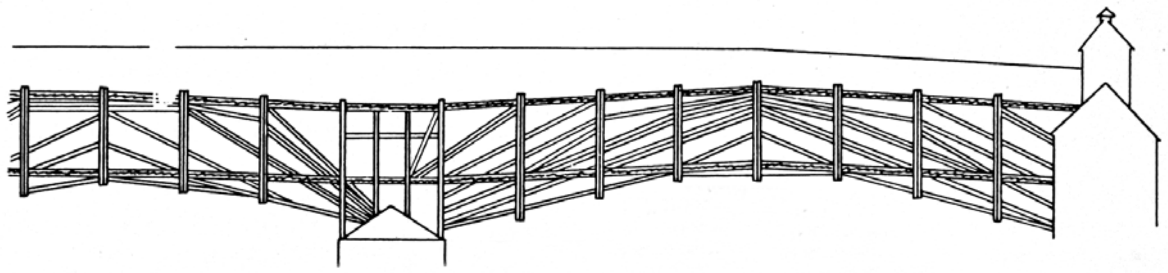

Fig. 9: Through-truss with polygonal reinforcement at Meissen, Reuss. The right side was built in 1764 and the left in 1784. (James[2])

Many bridges with spans up to about 30.5 m (100 ft) were built until the 1850s with traditional polygonal reinforcement like the bridge shown in **Figure 9**. Reuss built this bridge near Meissen, Germany. The right side was built in 1764 and the left in 1784. Trusses with diagonal bracing in all panels only began to appear in Switzerland in the 1840s.[30]

Knowledge of building these bridges was largely kept confined to the craftsmen who built the structures. Literature concerning the construction of through-truss bridges only emerged at the end of the 18th century and was predominantly written in German. Between 1765 and 1790, bridge books were published by Walter, Voch, Löscher, Wahl, and Reuss. At the turn of the 19th century many wooden bridges in and around Switzerland were burnt by the armies of Napoleon and his allies.[31] The destruction of Switzerland's bridges by Napoleon at the turn of the 19th century seems to have suppressed development there.

J.G. James notes that it is important to consider the qualities of wood when reviewing these early structures, particularly those that employ polygonal strengthening that was typical of the time. James writes,

> The basic problem in designing a timber bridge was not to plan some elegant minimal framework which could be calculated to bear the required load on the first day. It was to devise one which would retain its ability to do so despite the illnesses that wood is heir to – progressive sagging, warping and rotting, particularly at the joints. Some designers used members which on rigid frame theory are 'redundant'; some sought to make vulnerable members readily replaceable; others sought to incorporate methods for periodic adjustment. The final and best solution was the use of iron tensioning rods but this required the intermarriage of two technologies and a rapprochement between woodworker and ironworker.[32]

In France and Great Britain, there was a remarkable absence of covered through-truss bridges. Generally, France and England did not have large stocks of cheap timber and the rationale of through-trusses and the value of covering them was not appreciated. Apart from

[30] James[2], p127.
[31] James[2], p126.
[32] James[2], p116.

relatively short king-post and queen-post trusses, very few wooden truss bridges of any sort are known. A few designs were published, but French and British carpentry literature was generally more concerned with joinery than large structural framing.[33]

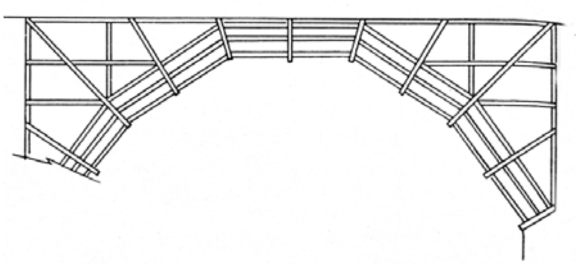

Fig. 10: Design for a deck bridge supported by polygonal arch, Gautier, 1714. (James[2])

Masonry arch traditions inherited from Rome dominated bridge design in France. France's academic-oriented approach to structural design was centered on arch theory. This influenced the development of the deck-bridge supported by polygonal or circular arches in most of Europe. An exception was the area around Switzerland described above, in the 18th and 19th centuries. (**Fig. 10**) One positive benefit of this misguided development was the accelerated introduction of iron into structures.[34] J.G. James writes,

> Although the boldness of the Swiss wooden bridges were admired, their form of construction was considered by the *Ponts et Chaussées* pundits to be unscientific. The purpose of the rectangular frame, except to carry the roof (which they regarded, in any case, as unnecessary), seems to have been unappreciated and the Swiss examples were not imitated. Outside Switzerland the French lead was followed all over Europe – even in parts of Germany, thanks to copious writings of Bavarian engineer Wiebeking. However deck-arches were inherently more flexible than through-trusses, particularly after water had seeped through the uncovered roadway and wrought its inevitable effect.[35]

From 1790 to 1850, truss development continued in America, and advanced impressively. The development there was in large part due to a rapidly expanding, newly formed country that had enormous spaces to traverse. In addition, fresh pine forests and a new generation of carpenter-builders, unbiased by building traditions in Europe, combined to abet the exploitation of new building concepts and techniques. The Americans took up where the Swiss and Germans left off and empirically developed truss forms suited to mass-production and capable of carrying railway loads.[36]

3.5.3 American Bridges, Before the Railroads

In the late 18th and early 19th centuries, Americas were moving west and south in large numbers. The wide rivers on the American continent posed severe challenges to travelers because there was no funding to construct massive piers and erect masonry arches. For an important crossing at Fairmount (now Philadelphia) across the Schuylkill River, the Scottish immigrant Robert Smith proposed timber arches in 1769. Thomas Paine, best known for his political writings and less so for his interest in bridge construction, proposed to build a prefabricated iron bridge in the 1780s. This was an exceptional idea. James writes,

[33] James[2], p127.
[34] James[2], p127.
[35] James[2], p116.
[36] James[2], p116.

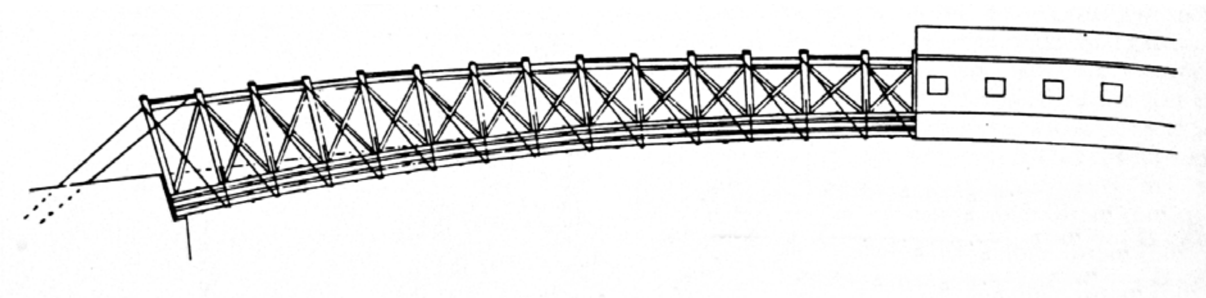

Fig. 11: The “Colossus,” an arched through-truss with a single span of 103.6 m (340 ft) built at Philadelphia, Wernwag, 1813. (James[2])

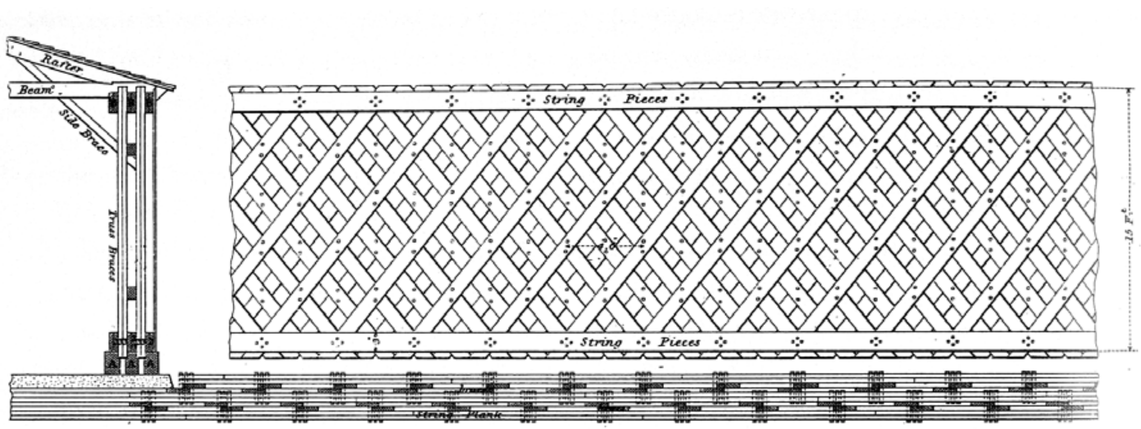

Fig. 12: Town lattice bridge, Ithial Town, patented 1820. (Peters[1])

> In the end, two general solutions well-suited to the rough and ready American conditions were evolved: the chain bridge and the timber truss. Early chain bridges were of limited stability and regarded with mistrust for major sites but good timber and carpenters were to be had almost everywhere.[37]

The greatest of the early timber truss builders was Lewis Wernwag, who immigrated to the United States from Württemberg, Germany, in the 1780s. He first worked as a mechanic before rising to manager of an iron nail factory in Phoenixville, Pennsylvania. Wernwag pioneered the use of auxiliary iron bracing in wooden bridges. His best-known bridge, the “Colossus”, built at Philadelphia in 1813, had a span of 103.6 m (340 ft). (**Fig. 11**) The panels, braced and counter-braced, increased in depth towards the abutments: the lower chord, 1067 mm (42 in) deep, had three courses separated by iron spacers to minimize rot. Iron counter-braces were added to the wooden ones, and iron back-stays were provided. They were anchored into the abutments. The bridge was destroyed by a fire in 1838.[38]

The most influential bridge type to emerge from this early period in American truss development was the lattice bridge, patented by Ithial Town in January 1820.[39] (**Fig. 12**) One lattice bridge, erected over the Susquehanna River at Columbia, was comprised of twenty-nine spans, each 61 m (200 ft) wide, the total length being about a 2 km (1.25 miles). This

[37] James[2], p169.
[38] James[2], p172.
[39] James[2], 172.

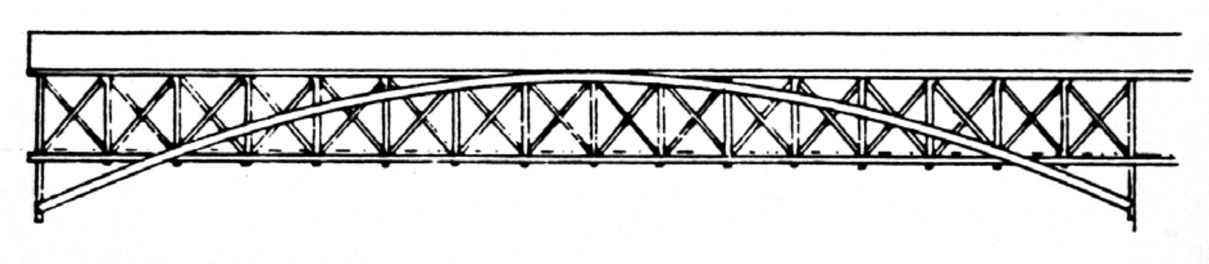

Fig. 13: Burr truss at Waterford, New York, Theodor Burr, 1804. (James[2])

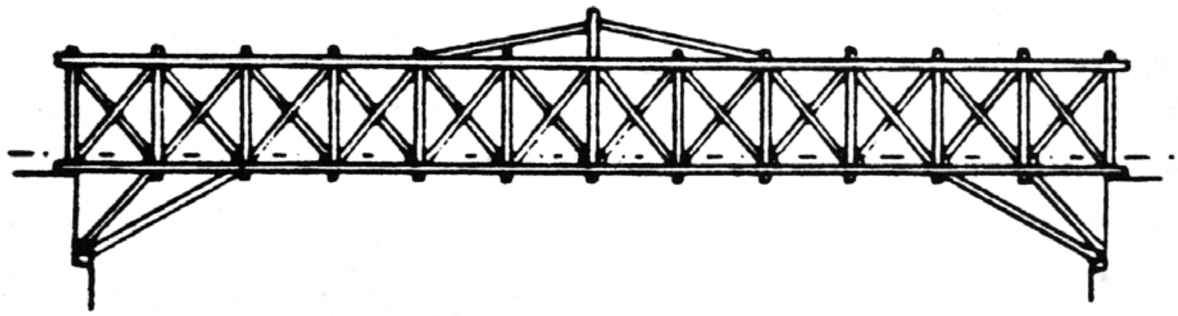

Fig. 14: Long truss, Stephen Harriman Long, patent designs 1836. (James[2])

bridge incorporated wrought-iron ties in the lower chord, fitted with screwed nuts to introduce prestressing. This assured that made sure all joints were stressed and countered the effects of shrinkage. Tensioning the ties introduced a precamber into the bridge that offset deflections induced by live load and creep. Dempsey noted that "one distinguishing advantage of this mode of construction is its simplicity; the braces and counter-braces being all cut exactly to the same length, and square on the ends, which simply rest in blocks attached to the top and bottom chords, and are without mortising or jointing in the members; the tie-rods pass through these blocks, and the whole structure is so simple, that it may be readily taken down, removed to another site, and re-erected with the utmost facility and precision."[40]

3.5.4 American Railroad Bridges

In the 1830s and 1840s, American railways grew at a phenomenal rate. This growth required a corresponding number of long bridges. Wood continued to be the primary bridge building material into the 1850s, but improved truss forms were developed to address the rapid increase in the weight of locomotives. These new bridges also had to have level decks as opposed to a longitudinal camber characteristic of most of the early truss systems.[41]

The Burr truss, a successful design pre-dating the railway boom, was used on many early railroads. (**Fig. 13**) Carl Culmann illustrated several Burr truss bridges, some seen on his

[40] Dempsey, p35-36.
[41] James[2], p175.

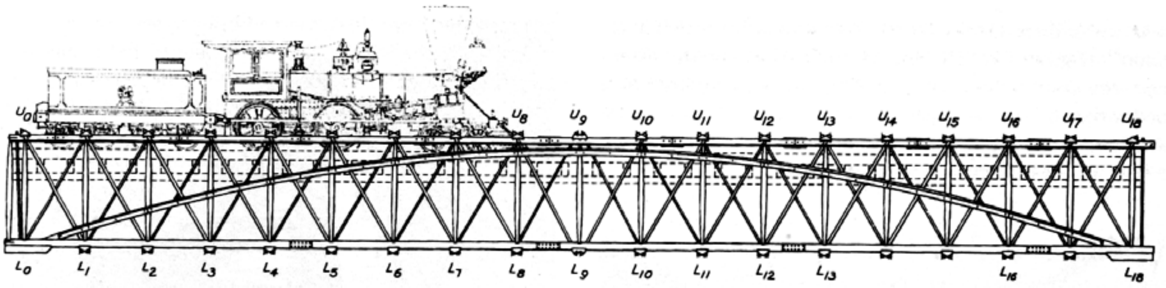

Fig. 15: Haupt truss, Herman Haupt, patent design 1839. (James[2])

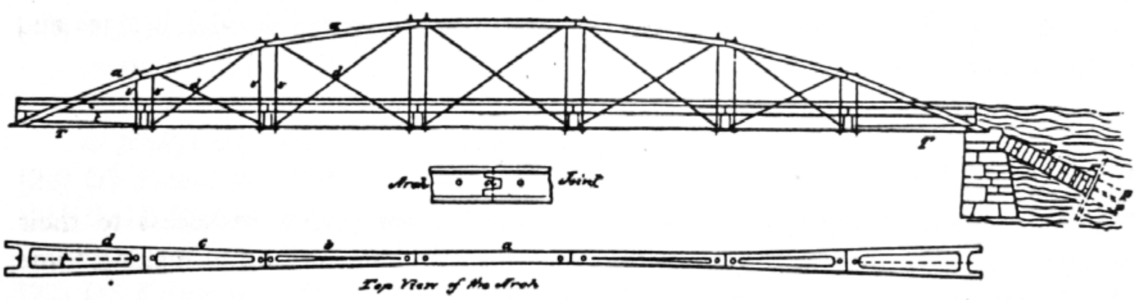

Fig. 16: Whipple all-iron bowstring truss, Squire Whipple, patent design 1841. (Sutherland[2])

American tour of 1849-50, as well as some drawn from a compilation of drawings made in 1850 by G. Duggan.[42]

America's massive bridge-building program resulted in increasingly higher prices for large timbers. Therefore, many engineers specified Town's lattice bridge as a cheap solution, if only a temporary one. Town had toured Europe in 1829-30 to market his system. In Europe the lattice bridge was first constructed of iron. Town was aided by the French-trained engineer Moncure Robinson to gain the confidence of railway engineers. Robinson provided valuable publicity for the system in Europe as well as America. In 1834, Robinson became chief engineer on the Philadelphia and Reading Railroad. In 1838, he showed the British engineer David Stevenson some of the Town lattices he had installed including one bridge with eleven 30.5 m (100 ft) spans.[43]

Town's 'invention' had proved so lastingly lucrative that several other designers sought their fortune through bridge patents. One example is Stephen Harriman Long, who produced a stream of patents and pamphlets. (**Fig. 14**) A more important figure was Herman Haupt, who graduated as a military engineer in 1835. He went into railway engineering and was exposed early in his career to the Burr truss, which he modified. (**Fig. 15**) Like Long, Haupt wrote several articles and pamphlets. After anonymously publishing *Hints on Bridge Construction* around 1841, he began writing a full-length book in 1844, published in 1851. This became a standard textbook and was reprinted for the rest of the century despite outdated contents. While the book provides an invaluable view of 19th century American bridge practice, it omits

[42] James[2], p175.
[43] James[2], p175.

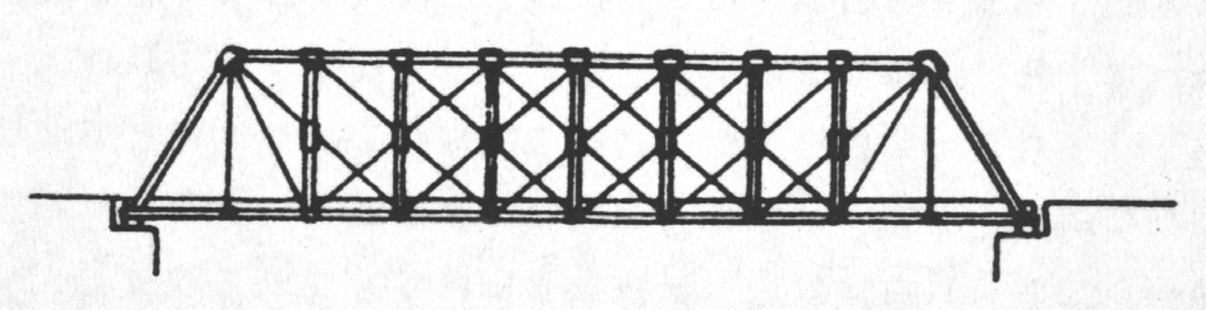

Fig. 17: Whipple all-iron parallel chord truss, Squire Whipple, c.1847. (James[1])

reference to the work of Squire Whipple, whose own analytical treatise was by then available. Whipple's influence on Haupt's book is still debated.[44]

The well-known all-iron bowstring bridge of Squire Whipple, patented in April 1841, had a cast-iron top chord and cast-iron posts with wrought iron lower chord and diagonals. (**Fig. 16**) In his famous book[45], written around 1846-47, Whipple advocated the parallel chord form, with double- and triple-intersection bracing for long spans.[46] (**Fig. 17**)

Iron trusses in America displaced wooden trusses in the 1850s and 1860s. Typical spans of iron trusses exceeded those of wooden trusses with spans well over 100 m. In contrast, early iron trusses rarely reached 32.5 m (100 ft) in the 1840s.[47] The evolution of the truss in America between 1790 and 1850 led to the rationalization and increased material efficiency of this structural type. On this foundation, truss development returned to the Europe where Carl Culmann, Wilhelm Ritter and others developed graphic statics.

3.5.5 Truss Development in England to 1845

By the 1840s, the truss was being developed primarily in the United States and Russia.[48] The insular British and French engineers almost totally ignored American truss systems when wooden bridges were used on railways.[49] English engineers spent a great deal of effort in the 1830s and 1840s developing the trussed girder, a cast-iron girder strengthened with wrought iron, but these structures are not particularly comparable to the trusses being described here. The trussed girder will be discussed in more detail in the next section. The following is a review of some developments that did occur in England before 1845, knowledge of which Robert Stephenson could have been aware.

In 1824, George Smart patented a proposal for using wrought-iron bars arranged diagonally that Dempsey describes as being of the lattice bridge type. It has parallel lower and upper chords with bars crossing diagonally to each other at 18° to the horizontal. This seems too low for a lattice bridge. In any case, Dempsey wrote in 1850 that he thought an inquiry into the principle and practical value of such structures was warranted.[50]

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[44] James[2], p176-177.
[45] Whipple, Squire. *A Work on Bridge Building, etc…* Utica, New York. 1847.
[46] James[2], p180.
[47] James[2], p180.
[48] Rosenberg and Vincenti, p83, note 13.
[49] James[2], p184-185.
[50] Dempsey, p33-34.

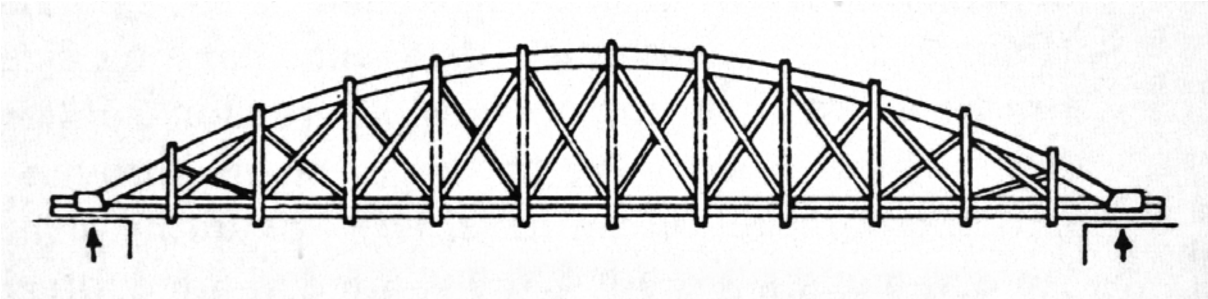

Fig. 18: Tied-arch with cross-bracing, or bowstring truss. (James^2)

In the 1830s, Robinson, Stevenson and John Weale all published favorable accounts of Town's "improved" lattice bridges. In France, similar publicity was obtained via Robinson's friend Michel Chevalier.[51] As John Weale was a popular publisher, Robert Stephenson surely must have seen this volume.

William Scarth Moorsom, one of the few English engineers to favor American railway methods tried the lattice bridge. But, like French engineers, he only trusted if for road traffic. A number of lattice bridges were built in Switzerland with spans ranging from 21.3 m to 39.6 m (70 ft to 130 ft), but they were intended only for road use.[52]

By the mid-1840s, the importance of German texts on theoretical analysis surpassed those in French. Lentz analyzed pioneering Irish lattice bridges constructed of iron in 1845 and Carl Ghega analyzed the American truss systems. Ghega published the most prestigious civil-engineering journal of the mid-19th century, the *Allgemeine Bauzeitung*, from Vienna. That journal published the analytical study of American trusses by Bavarian engineer Carl Culmann, which led to him becoming an engineering professor at Zurich, where he led the development of graphic statics.[53]

The only real contribution to the development of trusses by the British would appear to be the iron tied-arch.[54] (**Fig. 18**) This is not surprising, given their dependence upon arches for long span bridges; and the attendant problems of constructing proper foundations to contain the lateral thrust of those structures.

With the rapid development of iron railway bridges in the 1840s, American and Russian engineers appear to have thought primarily in terms of trusses and British engineers in terms of riveted-plate structures. Rosenberg and Vincenti attribute the British emphasis on plate structures to the growing availability of wrought iron in Britain and to their prior experience with riveted-plate iron-ship construction.[55] Timoshenko attributes the American and Russian concentration on iron trusses to their prior experience with wood trusses, which flowed logically from an abundance of wood as well as other economic factors peculiar to the United States and Russia.[56]

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[51] James^2, p180.
[52] James^2, p180.
[53] James^2, p184.
[54] James^2, p185.
[55] Rosenberg and Vincenti, p92, note 112.
[56] Timoshenko, p184-185.

3.5.6 Why Stephenson did not Use a Truss

As of 1845, English engineers knew about truss structures, but made scant use of the concept. Cast iron was still the preeminent engineering material that had led to the peculiar development of the trussed girder.[57] There was little information available for dimensioning a truss. Texts by Whipple and Haupt were only published *after* Stephenson had begun development of the tubular girder. Even if early publications on trusses had been available to Stephenson in 1845, he might understandably have pursued the design of the tubular beam anyway because he and Fairbairn had more experience with riveted-plate structures. To consider the truss would have meant starting the project with even less knowledge than was available to apply to the tubular beam. Rosenberg and Vincenti elaborate that "experience and technical information weigh heavily with engineers in making major design decisions, especially where technical failure my lead to considerable loss of life."[58] One can argue that the truss would have saved money, but that is beside the point when the critical goal of the Directors was to open that line as soon as possible. If Stephenson had considered changing to the truss around 1847, when Whipple's book would have been available, he would have had to start a new series of experiments because: the scale of his bridge was far greater than any iron trusses built to that time; and the span would be nearly twice the length of the longest existing wooden truss bridges.

Ultimately, I think Stephenson did not consider the truss for the following reasons:

1. From Stephenson's comments at the beginning of this section it is clear that he did not see the beam as an independent entity, but supported by what would come to be known as an auxiliary chain. This indicates he did not trust the beam to be secure on its own for such a span.
2. In 1845, the longest bridges in England built on the beam principal, i.e. structures that do not exert a horizontal thrust, were trussed cast-iron girders.[59] These bridges were typically limited to spans of 18.3 m (60 ft) or less and only exceptionally approached 32.5 m (100 ft). Stephenson's proposed girder had to span a full 137.2 m (450 ft) – later changed to 140.2 m (460 ft), have the strength and rigidity to carry railway loads, and be constructed without obstructing the navigation of the strait. Such a structure under these adverse conditions was simply unprecedented.
3. Lastly, Stephenson's initial simplification of the tube for purposes of analysis illustrate the lack of knowledge concerning the analysis of simple beams, let alone trusses. The only texts for analyzing trusses began to be published around 1846-47. By that time it would have been too late to stop the process of development that had already commenced.

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[57] For more detailed description of the trussed girder, see next section.
[58] Rosenberg and Vincenti, p92, note 112.
[59] **Ref. p.A.102-A.104**.

3.6 Evolution of the Iron Beam to 1845

3.6.1 Stephenson's Beam of Unprecedented Scale

In February 1845, Stephenson announced that wrought iron was the best material for the bridge over the Menaï Strait. He further stated that he would use this material to fabricate a hollow girder or tube.[60] As previously stated, this beam was to be of unprecedented scale, more than doubling the longest wooden truss spans in America; and being four to five times the length of the longest iron beams then being built in England. Moreover, Stephenson was going to use wrought iron, which was still a new material to bridge engineering in England at the time. The largest structures previously built of that material were ships. Stephenson himself employed wrought iron in the Ware Bridge, built in 1841. The girders of that bridge were made of wrought iron plate and angles riveted to form an asymmetrical **I**-section as shown in **Figure 3**. The following section will examine the development of beam design in England to 1845 and show why Stephenson's Ware Bridge beam looks as it does. This section will illustrate the extent Stephenson's knowledge of beam theory and strength of materials when he undertook to design the tubes of the Britannia and Conway Bridges.

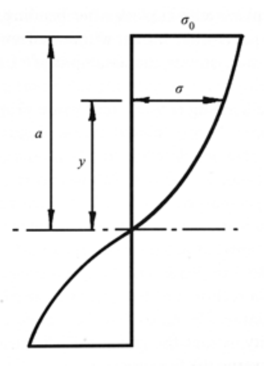

Fig. 19: Hodgkinson's distribution of bending stress. (Heyman)

3.6.2 State of Beam Theory

By 1845, the basic theory of how beams resist bending caused by vertical loads had been accurately established by French engineers and scientists and introduced into England by the lectures and textbooks of Henry Moseley.[61] In 1826, C.L.M.H. Navier, the French engineering theoretician, had published his *Leçons* in which he correctly located[62] the placement of the neutral axis in elastic bending through the center of gravity of the cross-section. The British experimental engineer Eaton Hodgkinson published two memoirs in 1824 and 1831. In these he set out to determine the distribution of bending stress based on the work of Charles-Augustin Coulomb. (**Fig. 19**) His work was an improvement on Thomas Tredgold's method (1822) of determining the neutral axis that only applied to symmetrical sections.[63] Hodgkinson created formulas for calculating the internal stresses in beams with different cross-sectional shapes from this theory. However, these formulas were based on specific assumptions that were not always applicable. Rosenberg and Vicenti write that "it was not at all clear at the time, however, how these calculated stresses should be used in estimating the ultimate strength… of a beam of specified material."[64]

Dempsey's account of the construction of the Britannia and Conway bridges gives an overview of knowledge at the time concerning beam dimensioning. It was generally

[60] Dempsey, p61.
[61] Rosenberg and Vincenti, p13.
[62] Navier actually rediscovered what Coulomb had published in 1773.
[63] Heyman, p27-29.
[64] Rosenberg and Vincenti, p13.

understood that the strength of a beam is proportional to its depth. It was further known that there is compression in the convex side of a beam in bending with corresponding tensile stress in the concave side, and that a symmetrical section was only appropriate for beams with equal resistance to compression and tension. This knowledge came chiefly from Hodgkinson's experiments in 1827 to determine the best form of a cast-iron beam. However, Dempsey does not distinguish between the relative *structural* advantages of a symmetrical **I**-section and a rectangular section regardless if a material is isotropic or not. It was also understood that the stress varies along the length of a beam, with the maximum stress in the center of a uniformly loaded beam.[65] Therefore, it was possible to reduce the sectional area of the beams from a maximum at mid-span to a minimum at the supports. Shear was not well understood until the work of D.J. Jourawski in 1856 and A.J.C.B. de Saint-Venant in 1864. As will be seen later, buckling in beams would be discovered for the first time by Fairbairn in his first tube experiments for the Britannia Bridge. Beam sizing was largely empirical, relying on experiments to determine material coefficients for particular loading cases and section forms. The practical application of strength of materials knowledge was very limited.

3.6.3 Iron Beams

The iron beam developed simultaneously and independently in four different technological fields: rails for the railroad, shipbuilding, buildings, and bridges. While there are important examples of wrought iron used in French building construction during the late 18th century[66], these developments had little impact in England where cast iron was the dominant structural material.

In England, the iron industry was organized around the production of cast iron because of limits Elizabeth I imposed in the 16th century concerning the use of timber for charcoal production. Charcoal was used as a fuel in blast furnaces.[67] Use of charcoal as a fuel in iron production produces wrought iron directly. Elizabeth's decree severely crippled the English iron industry until the use of coal, and subsequently coke, was perfected as a fuel, first accomplished by Dud Dudley and later perfected by Abraham Darby, Sr. in the early 18th century. The use of coal as a fuel produces cast iron. It was only after the introduction of Henry Cort's puddling process at the end of the 18th century, and Hall's subsequent improvements to it in the 1830s, that economic production of wrought iron was possible using coal as a fuel. English iron engineering practice was founded on cast iron when Ironbridge was built at Coalbrookdale, England, in 1779.

3.6.4 Beam Development in Buildings

The first significant iron building structures were fabricated in France using wrought iron in the late 18th century. Jacques-Germain Soufflot and Jean-Baptiste Rondelet heavily reinforced the masonry with iron in the Église de Sainte-Geneviéve, Paris, built in 1870. (**Fig. 20**) This structure is an antecedent to later reinforced concrete. Two roofs were constructed with wrought-iron framing in 1786 and 1789 respectively. Victor Louis designed the first for

[65] Dempsey, p64-69.
[66] Steiner.
[67] Goodale, p40. The reason for this restriction was to preserve the rapidly depleting forests for naval construction.

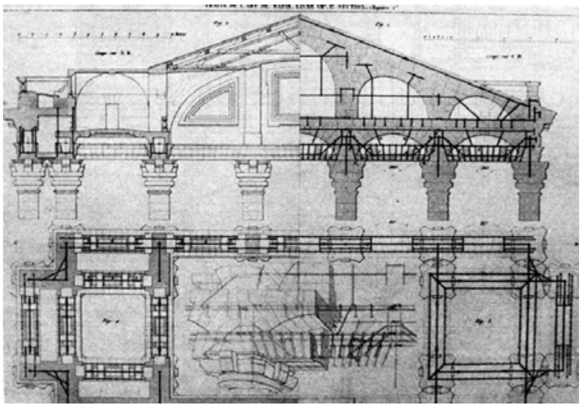

Fig. 20: Eglise de Sainte-Geneviève, Paris, Jacques-Germain Soufflot and Jean-Baptiste Rondelet, 1770. Illustration shows wrought-iron reinforcement of masonry. (Steiner)

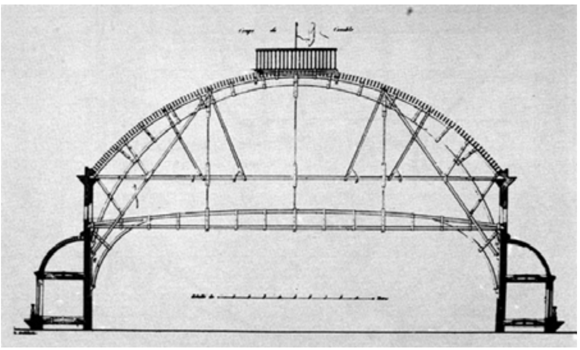

Fig. 21: Wrought-iron roof structure of the Théâtre Français, Paris, Victor Louis, 1786. (Steiner)

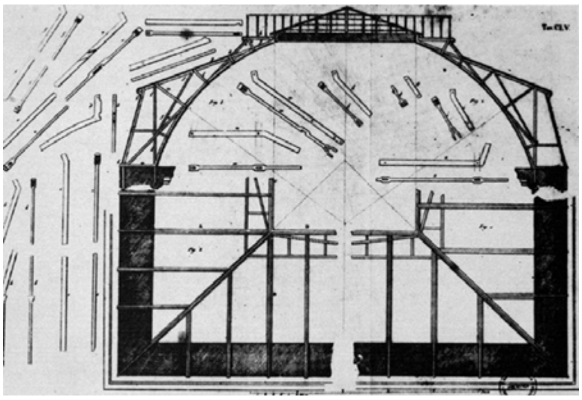

Fig. 22: Wrought-iron roof structure of the roof over the Salon adjoining the Grande Galerie of the Louvre, Paris, Auguste Rénard, 1789. (Steiner)

the Théâtre Français in an effort to reduce the risk to fire that was then prevalent in theaters. (**Fig. 21**) Auguste Rénard built the second roof, which covers the Salon adjoining the Grand Galerie of the Louvre in Paris. (**Fig. 22**) Both roofs employ an arch principle in their construction and it is probable that their respective designers were influenced by the construction of Ironbridge. The French Revolution in 1789 effectively halted building construction and it was several years before construction recommenced.[68]

The first step in the evolution of iron beams in England occurred in the construction of warehouses and textile mills where frequent fires necessitated the development of some form of fire-resistant construction. William Strutt, an important English textile manufacturer, was a pioneer in fire-resistant construction. In 1793, he finished construction of the first so-called 'fire-proof' building, the Millford Warehouse at Belper. In this warehouse, Strutt used cast-iron bars with cruciform sections for the columns. The timber floor beams were protected by plaster on their bottom face. The other faces were protected by the masonry of the floor system employed at the time. (**Fig. 23 and 24(a)**) In 1785, the French reintroduced hollow-pot arches, which date back to the Romans, before this construction method was adopted in England.[69]

Charles Bage fabricated the first cast-iron beam for the construction of a five-story flax-mill at Shrewsbury in 1797. Bage was assisted by his friend William Strutt. William Hazledine cast the iron structure for this building.[70] The beam, shown in **Figure 24(b)**, is an example of *translational substitution* from Strutt's timber form to Bage's cast-iron beam. Both beams incorporate skewbacks to support the intermediary brick arch floor. While the

[68] Steiner, p26.
[69] Johnson and Skempton, p182.
[70] Johnson and Skempton, p193.

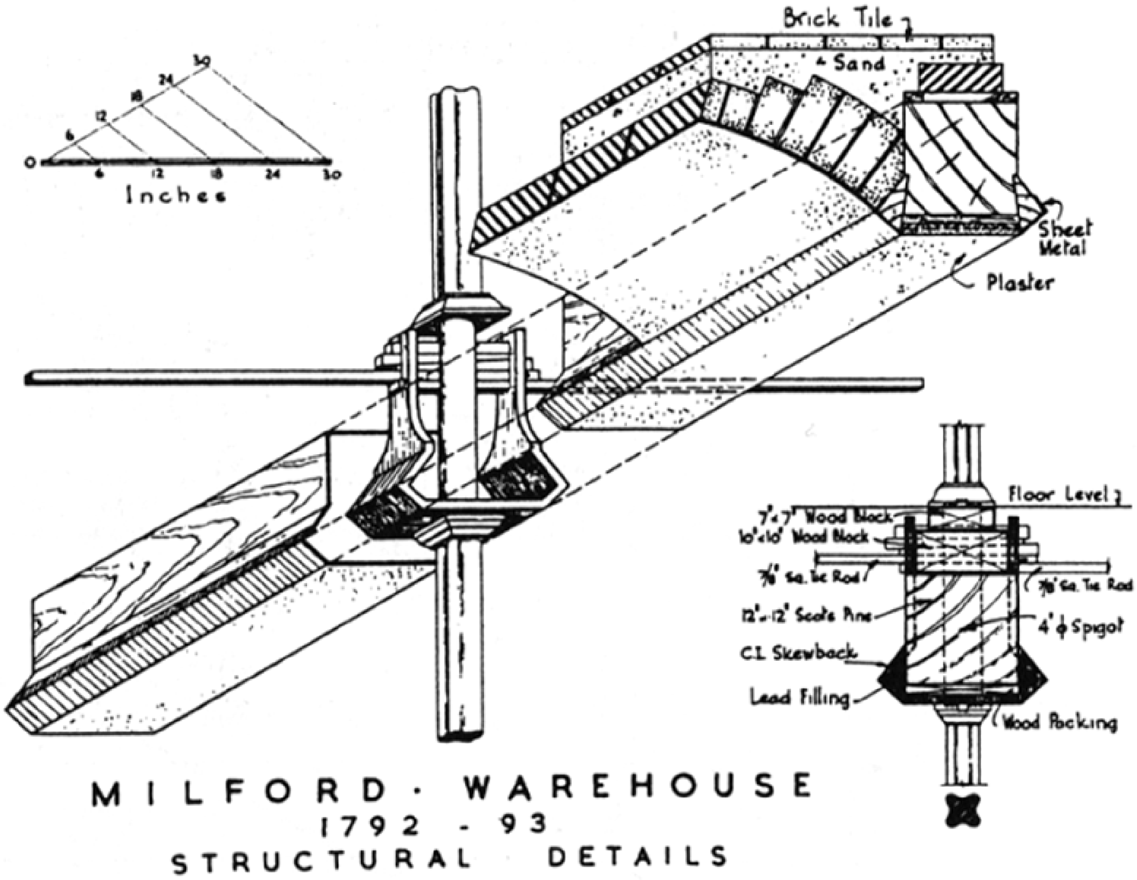

Fig. 23: Structural detail of Milford Warehouse, William Strutt, 1793. (Johnson and Skempton)

inverted-**T** form is appropriate for cast iron, which is much weaker in tension than compression, there is no evidence to say that the form employed was chosen for any other reason than to copy the timber system. Additionally, the beam was continuous over the columns without any modification to the section to account for the negative moment imposed on the beam there. This does not discount the possibility that Bage, Hazledine and Strutt appreciated cast iron's strength characteristics, but it does show how the behavior of a continuous beam was not then understood.

The skewback was simplified by Bolton and Watt at Salford in 1801. This beam has a more typical flange form. (**Fig. 24(c)**) This was an important advance because it was more economical to produce but it was still continuous over the columns.[71]

In 1802, Simon Goodrich submitted a winning design to rebuild a mill that had been destroyed by fire. Goodrich's design was explicitly based upon Strutt's. But he noted in a report accompanying his drawings: "The beams might be of cast iron, but the preference is given to wood, since being fully secured against fire, it is cheaper and more to be depended upon for strength in this position than cast-iron." Thus, we see that in 1802 there were already doubts about the suitability of cast iron for beams, but after it was used in Bage's

[71] Jewett, p.355-356.

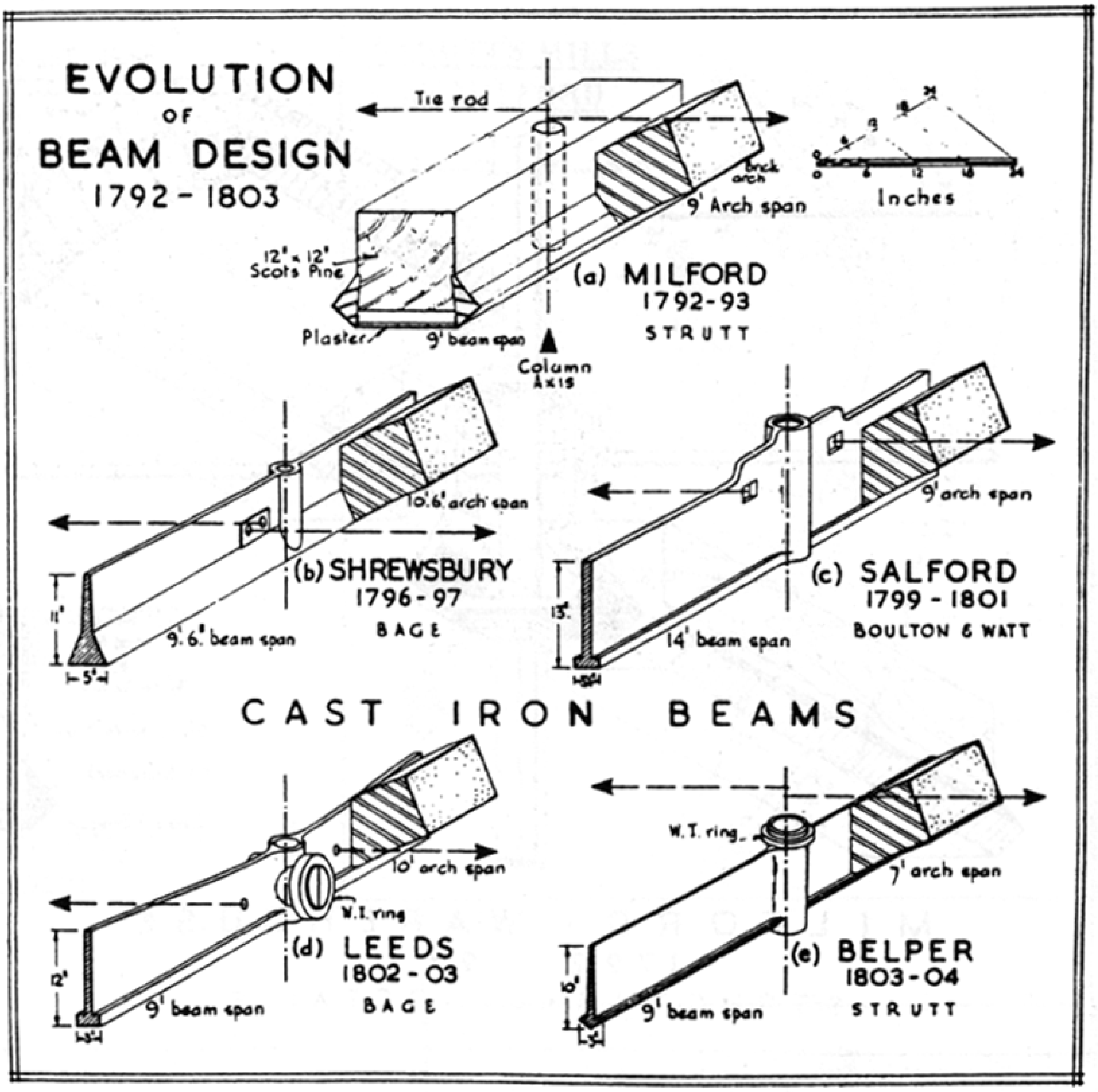

Fig. 24: Evolution of cast-iron mill beam design, 1793-1804. (a) Strutt, 1793. (b) Bage, 1797. (c) Boulton and Watt, 1801. (d) Bage 1803. (e) Strutt, 1804. (Johnson and Skempton)

Shrewsbury mill it was employed in all subsequent mills until the mid-nineteenth century. Then wrought iron took the place of cast iron.[72]

In 1803, Charles Bage tested beams with inverted-tee sections then being used in mill construction. From these experiments, he developed a formula to size the beams used in a mill at Leeds. Bage later verified his formula by full-scale tests. Skempton notes that the equation was worked out *after* the flange was already on the beam.[73] Bage's formula closely resembles the formula later published by Eaton Hodgkinson in 1830, after his experiments to

[72] Johnson and Skempton, p193.
[73] Skempton, p1030.

determine the 'ideal' section of a cast-iron beam. Therefore, by 1803, the form of the cast-iron beam was rationalized and there was a theoretical basis to analyze its design.[74]

In July 1803, Bage and Benzons further improved the form of cast-iron beams by introducing a turtle-back longitudinal section in the Meadow Lane flax mill at Leeds. This profile corresponds to the moment diagram of a simply supported beam and may represent the first such beam to have been fabricated.[75] (**Fig. 24 (d)**) Unfortunately, Bage and Benzons designed a collar to secure the individual beams to the column that effectively made them semi-continuous, thus the introduction of negative moments was not entirely eliminated. William Strutt followed these developments closely and incorporated these improvements into his North Mill at Belper, in 1804. (**Fig. 24(e)**)

The development of the cast-iron beam for mill construction during this initial period culminated in Strutt's South Mill at Belper, built in 1812. Here, Strutt fabricated turtle-backed, simply supported beams as shown in **Figure 25**. These beams were up to 5.2 m (17 ft) long, which was bold for the period. The tie-rods end in an eye that fits over the column spigot and no wrought-iron ring exists to bind adjacent beams together, thus achieving greater freedom of movement.[76]

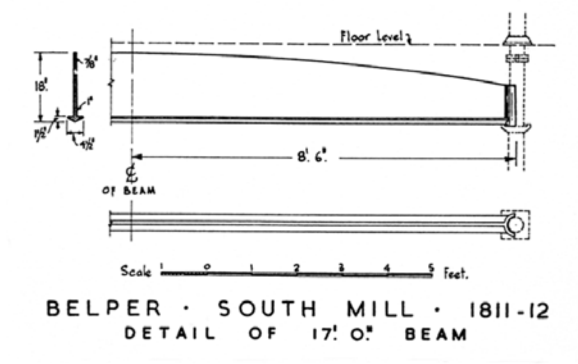

Fig. 25: Detail of 17' cast iron beam of South Mill at Belper, Strutt, 1812. (Johnson and Skempton)

3.6.5 Development of Iron Rails for the Railroad

The development of iron rail is significant because it was for that purpose that the first wrought-iron **I**-sections were *rolled*. The first rails were simple flat bars on a wood stringer. The first all-iron rail was of cast-iron and had a fish-belly longitudinal section. (**Fig. 26(a)**) The rail had a lower flange that was bulb shaped at

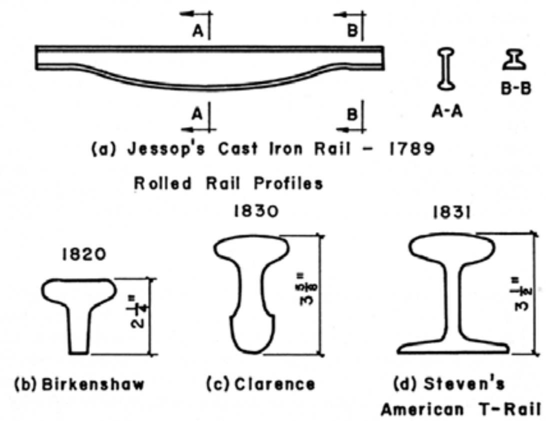

Fig. 26: Evolution of railroad rail. (Jewett)

74 Johnson and Skempton, p194.
75 Johnson and Skempton, p194.
76 Johnson and Skempton, p194-200.

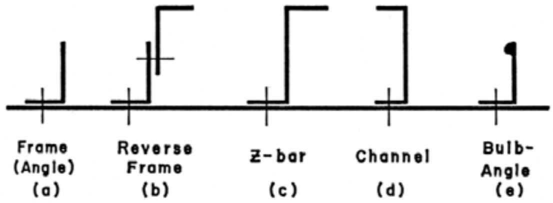

Fig. 27: Framing of iron ships. (Jewett)

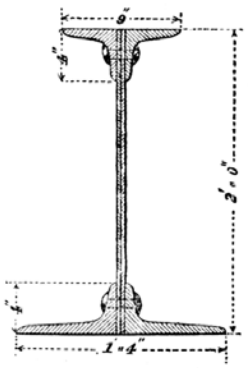

Fig. 28: Built-up wrought-iron asymmetrical I-section for deck beam in shipbuilding, William Fairbairn, c.1839. (Dempsey)

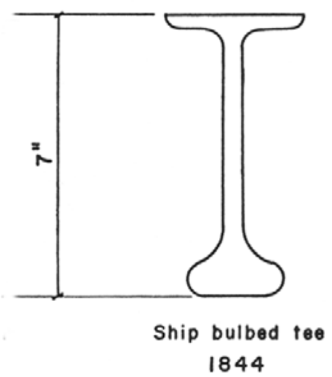

Fig. 29: Bulb-tee, patented by Kennedy and Vernon for use as a deck beam in shipbuilding, 1844. (Jewett)

mid-span and flattened at the ends to seat better upon stone supports.[77]

The first rolled wrought iron rails were introduced around 1811 to address chronic fatigue failures in the cast-iron rails. Plus, wrought-iron rails weighed half that of cast-iron for the same load-carrying capacity.[78] The first wrought-iron rails were simple bars because of limited rolling mill technologies of the time. John Birkenshaw patented the first rolled wrought-iron rail with a more complex form in 1820. Birkenshaw's beam was a type of early **T**-form. (**Fig. 26(b)**) The curved nature of the section and the absence of better-defined corners reflect the limits of the rolling technology, largely due to insufficient motive power. The Clarence rail included a bulb in the bottom flange.[79] (**Fig. 26(c)**)

The final stage in the evolution of rail was the invention of the American T-rail by Robert Stevens in 1830. (**Fig. 26(d)**) While on a voyage to Britain, Stevens whittled out a model of the Birkenshaw rail and, with what Jewett describes as *intuitive perception*, Stevens conceived the idea of an integral continuous bottom flange. The original American T-rail was first rolled in 1831 and was 89 mm (3.5 in) high and weighed 17.4 kg/m (35 lbs/yard). Thus, the American T-rail became the first rolled wrought-iron **I**-beam.[80]

3.6.6 Emergence of Wrought-Iron Structural Shapes in Shipbuilding

The earliest all-iron vessel was built in Scotland in 1818. The ribs were hammered from flat bars into angle-shapes on a special anvil. Various shapes evolved from the basic angle to increase the strength of ship frames. **Figure 27** shows some of these shapes, including the **Z**-section, the channel, and the bulb-angle. Sometime between 1800 and

[77] Jewett, p347.
[78] Johannsenn, p334.
[79] Jewett, p348-349.
[80] Jewett, p349.

1819, a French rolling mill produced the first wrought-iron angles. By 1830, angles were still the only rolled shapes (besides prismatic bars and rounds) and were used principally for steam-boiler construction. Between 1828 and 1830, the tee was rolled. The **T**-section, more difficult to roll than an angle, was used in shipbuilding by the late 1850s. Rolled tee compression members were used for the roof truss in Euston Station, London, in 1838.[81]

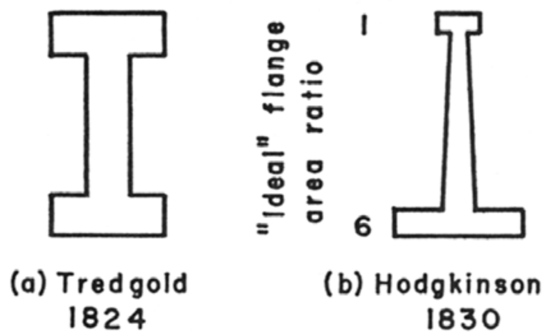

Fig. 30: (a) **I**-section for cast-iron beam, Tredgold, 1824. (b) Hodgkinson's 'Ideal' section for cast-iron beam, 1830. (Jewett)

In the 1830s, William Fairbairn experimented with new rib forms. He created built-up tees by riveting two angles and a plate, and solid tees by forging. Fairbairn was interested in the relative strength between these built-up sections and those formed as one piece. He also fabricated a double-flanged beam composed composed of two bars worked into channel sections and riveted back to back. Fairbairn concluded that the **I**-section would provide the strongest form and experimented with a built-up section as shown in **Figure 28**. Fairbairn developed this section in 1839, but did not publish his findings until 1850.[82]

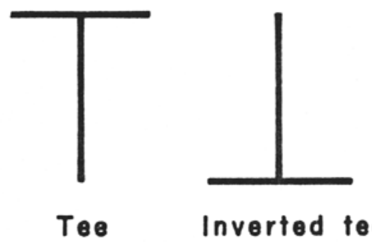

Fig. 31: Orientation of cast-iron **T**-sections tested by Hodgkinson showing that the inverted-**T** is stronger by a ratio of 1:4. (Jewett)

Kennedy and Vernon of Liverpool received a patent in 1844 for the bulb-tee shown in **Figure 29**. It was intended for use as deck beams in shipbuilding. There is a notable similarity between it and the inverted American-T rail. (**Fig. 26(d)**) The patent of Kennedy and Vernon also included a true **I**-shape but there is no record such a profile was fabricated.[83] The bulb-tee not only provided increased strength over existing sections used in shipbuilding at the time, but also facilitated maintenance by making it easier to paint than an **I**-section. The corner of a regular **I**-section, where the bottom flange connects to the web, would be difficult to inspect for corrosion or to ensure it is painted properly. Turner used bulb-tee sections to construct the Palm House at Kew Gardens, between 1846 and 1848. The bulb-tee was useful here for the same reasons as in ship construction – its form made it easy to inspect and protect with paint in the corrosive environment of the hothouse.

3.6.7 The I-Section

In 1820, Alphonse-Jean Duleau, a graduate of the École Polytechnique, Paris, discovered the benefits of the **I**-form while experimenting with a built-up beam of two parallel, flat bars separated by spacers. Duleau found that when the vertical spacing was increased the

[81] Jewett, p350-352.
[82] Jewett, p353-354.
[83] Jewett, p354-355.

strength of the beam increased too. From these tests, he recommended building iron beams of two flanges connected by a web. Unfortunately, his results were not disseminated.[84]

British engineer Thomas Tredgold proposed beams with an **I**-section in the second edition of his book *Practical Essay on the Strength of Cast Iron and Other Metals*, published in 1824. Unlike Duleau, Tredgold's work was widely referenced. Tredgold had conducted experiments on the strength of cast iron that were not too scientific. These tests only measured the deflection of a simply supported cast-iron beam due to its own dead load.[85] From these experiments, Tredgold deduced that the strongest form of the cast-iron beam was a symmetrical **I**-section due to the erroneous assumption that "bodies resist the same degree of extension or of compression with equal degrees."[86] (**Fig. 30(a)**) His conclusions were flawed, but he did recognize the inherent advantage of the **I**-section over a rectangular section with the same area of material in the cross-section. The flanges and web of Tredgold's beam have equal thickness because Tredgold was well aware of the internal strains that could be introduced in metal due to uneven cooling when it is formed.[87] Tredgold credited previous studies conducted by Edmé Mariotte, Charles-Augustin Coulomb, Charles Young, Peter Barlow, Pierre-Simon Girard, and Duleau on rectangular bars as the base of his own explorations. He had not, however, heard of Henri Navier yet because *Leçons…* was not published until two years after Tredgold's second edition.[88]

Fairbairn improved upon the inverted **T**-beam used in mill buildings for one built at Bradford, in 1824. Fairbairn doubted the strength of early cast-iron beams and fabricated a **T**-beam with a larger flange, but still did not introduce a top flange.[89] In 1827, Eaton Hodgkinson conducted tests, sponsored by Fairbairn, to determine the correct proportions of a cast-iron beam. He first tested identical **T**-beams and compared their strengths when oriented as a **T** and when inverted, like those used in the mill buildings. (**Fig. 31**) His results were profound since they decidedly showed that cast iron was at least four times stronger in compression than in tension. Hodgkinson drew on Tredgold's experiments, of which he was highly critical, and added a top flange to the standard inverted tee mill beam. Then it was a matter of parameter variation to increase the size of the bottom flange until the beam failed nearly simultaneously in both compression and tension. From these tests, he determined that cast iron was 5.5 to 6 times stronger in compression than tension. Hodgkinson's beam, shown in **Figure 30(b)**, came to be known as Hodgkinson's "Ideal" beam. Hodgkinson's profile is not ideal however, because the tapered flange is thicker than it theoretically needs to be due to fabrication limits that adversely affected quality beyond a certain thickness when casting iron.[90] In practice, the web was simplified to have parallel sides, which reduced manufacturing costs. Hodgkinson's findings were generally accepted in practice and from then on cast-iron beams had asymmetrical **I**-sections with a top flange to bottom flange ratio of 1:6. From this work, Hodgkinson developed a formula for calculating the strength of cast-iron beams using his sectional proportions. This formula is similar to Bage's determined nearly thirty years before.

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[84] Jewett, p357.
[85] Fairbairn believes that these tests were conducted around 1821 or 1822.
[86] Tredgold, p55.
[87] Tredgold, p55-56.
[88] Jewett, p358-359.
[89] Fairbairn[3], p8-10.
[90] Hodgkinson, p413-444.

Fig. 32: Typical cast-iron girder bridge for railroads in England, early 19th century. (Sealy)

In the early 1840s, the design of cast-iron beams was well established, but when wrought iron began to be introduced problems of strength of materials knowledge re-emerged. Fairbairn introduced a built up **I**-shaped girder to the construction of ship decks in 1832[91] and, in 1841, Robert Stephenson designed the Ware Bridge, one of the first wrought-iron girder bridges to be built. (**Figs. 3 and 28**) Both Fairbairn and Stephenson adopted a section with the proportions of Hodgkinson's ideal cast-iron beam, another example of translational substitution.[92]

Ferdinand Zorés had the first wrought-iron beam for use in building construction rolled in 1849. Zorés, a consulting engineer for French iron mills, had independently developed the I-section because he was dissatisfied with the structural sections then being produced in French mills. He first made a cruciform section that was expectedly found to be weak. Then he forged a **T**-section, to which he finally added another flange. He encountered resistance from French mills to invest in new machinery to roll the new form and only succeeded after securing an order for a certain quantity of the product. He received the silver medal at the French National Exposition of 1849 for his 'invention.' As Rosenberg and Vincenti point out, this empirical development occurred "in the shadow of Navier's Écoles."[93]

3.6.8 Wrought-Reinforced Cast Iron

In the 1830s to the mid-1840s, the dominant structural material in England remained cast iron. The rapid expansion of railroads led to the construction of many short-span cast-iron girder bridges. (**Fig. 32**) These bridges, made of a brittle material highly susceptible to failure from fatigue and impact, soon acquired a reputation for suddenly failing and trust in

91 Dempsey, p21.
92 Jewett, p359-360.
93 Rosenberg and Vincenti, p362.

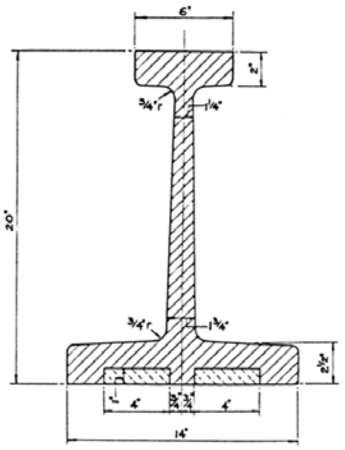

Fig. 33: Wrought-reinforced cast-iron girder for the Reading and Reigate Railway with 7.6 m (25 ft) span, Peter W. Barlow, c.1840s. (Sutherland[2])

the security of railroad bridges declined. In an effort to increase both the span and security of cast-iron girders, wrought iron was used to try to reinforce cast iron.

There were three principle methods tried to reinforce cast iron with wrought iron. The first method was to use the wrought iron compositely like the steel in reinforced concrete. Fielder patented a method to simply bolt or rivet a wrought-iron bar to the bottom flange of a cast-iron girder in 1847, though this was probably tried much earlier. Dempsey mentions another design for strengthening cast-iron beams with corrugated sheet iron in his book published in 1850, but he gives no description of how that system worked.[94] Peter Barlow produced a prototype beam in which he cast wrought-iron bars directly into the bottom flange of a cast-iron girder such that the wrought iron acts similarly to the steel used in present-day reinforced concrete. (**Fig. 33**) Barlow's test beam was 50% stronger than cast iron alone and it was apparently not liable to fracture. Nonetheless, Barlow's idea was not generally adopted.[95]

The second method was not a true composite structure *per se* except in the minds of those who tried it. Various people made tests that showed that the strength of cast iron could be appreciably increased by adding molten wrought-iron scrap to the molten cast iron up to about 40 per cent. In reality, they simply lowered the carbon content, which improved the iron's mechanical properties.[96]

The last method to reinforce the cast-iron girder was to use wrought-iron 'trussing.' Such structures incorporated wrought-iron bars or rods that were fit to the cast-iron girders in such a way intended for the cast iron to only resist compression and the wrought iron tension. (**Fig. 34**) These 'trusses' were limited to the depth of the girder.[97]

Dempsey describes one such girder,

> Many railway bridges were erected with these additions, and were considered safely constructed when each girder was cast in two or more separate pieces, making up, when united, the total width of span, and the pieces being secured together by bolts passing through holes in flanges or projecting plates cast on the ends of each piece. One of these cast-iron compound girder bridges, trussed with malleable-iron bars, erected several years since, to carry the Northern and Western Railway over the river Lea, is formed with girders each 70 feet [21.3 m] in length, and composed of two castings, joined at the centre by bolts passing through vertical flanges. An additional security connection is

[94] Dempsey, p39-40.
[95] Sutherland[2], p70.
[96] Sutherland[2], p70.
[97] Dempsey, p7.

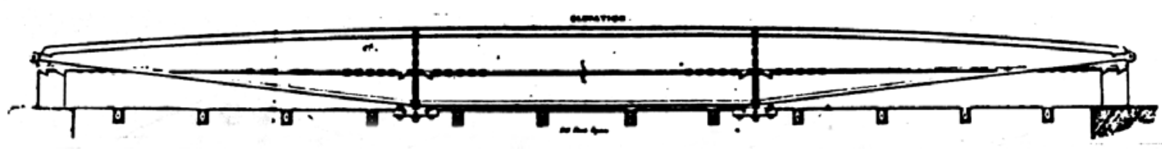

Fig. 34: 'Trussed' cast-iron girder with span of 29.3 m (96 ft) for the Leopold Railway. Six castings with vertical and horizontal joints and wrought-iron ties on each side. (Sutherland[2])

> attained by casting dove-tailed projections or bosses upon the meeting ends of the two castings, and by fixing wrought-iron clips over these bosses. Each girder, thus formed of two castings, is perfectly horizontal from end to end, and the top and bottom lines parallel, the uniform depth being 36 inches [914 mm]; the bearings upon the abutments are 2 feet long [0.6 m] at each end, and the clear span between bearings is thus reduced to 66 feet [20.1 m]. The section of the castings is of the approved form, viz. with vertical rib, and projecting flanges at top and bottom.[98]

Trussed girders were plagued by failures. This was primarily because designers tried to treat the trussed wrought-iron bars as a type of suspension system that unduly stressed the ends of the beams. Ill-advisably, this led designers to think that the 'additional support' in the center meant that the section at mid-span could be reduced. One such girder, shown in **Figure 35**, comes from Robert Stephenson's Dee Bridge. The elevation shows where a girder fractured when it failed in May 1847, killing five workers. Towards 1850, the design of trussed cast-iron girders began to become more rational, but the tragic failure of the Dee Bridge led to the immediate end to that development. The Dee Bridge disaster finally made the public aware of the inherent dangers posed by these trussed girders and led to the organization of the Royal Commission to Inquire into the Application of Iron to Railway Structures. While the trussed girder was one of the least successful structural developments, it did work in many cases and it was prescient with respect to the development of prestressing, which was created in the 20th century in connection with concrete.[99]

3.6.9 State of Beam Design in 1845

On 24 May 1847, Stephenson's Dee Bridge on the Chester & Holyhead Railway partially collapsed during passage of a train and five workers were killed. Its three 29.9 m (98 ft) spans consisted of iron beams each made from three castings bolted together and trussed along the sides with wrought-iron bars. So many of these and other comparatively new types of iron railway bridges had been built in recent years that a public enquiry was set up in August. The results of this enquiry are recorded in the two-volume *Report on the Application of Iron to Railway Structures* that was published 26 July 1849. The Commission included George Rennie, William Cubitt and Eaton Hodgkinson.[100] They concluded:

[98] Dempsey, p7.
[99] Sutherland[2], p72-73.
[100] James[1], p2.

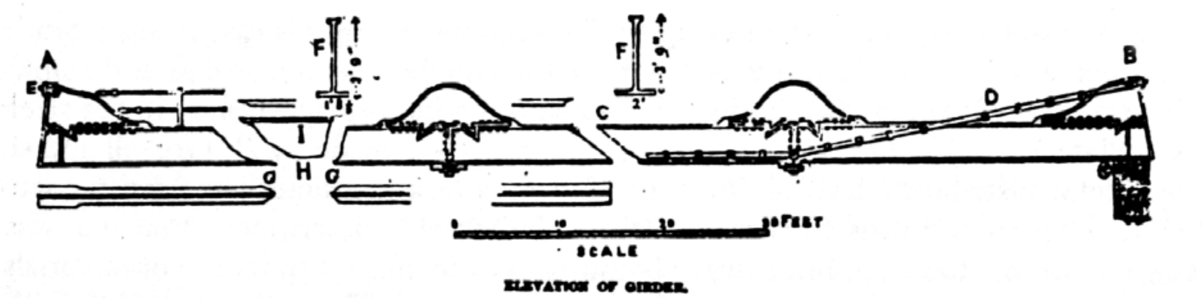

Fig. 35: Elevation of trussed cast-iron girder of Dee Bridge showing where the beam failed. The wrought iron trussing is intended to act in suspension and the ill-conceived longitudinal-section is of reduced depth at center span. Robert Stephenson and Bidder. (Sutherland[2])

- For bridge spans too long for single-casting non-trussed beams, "the cast-iron arch is the best form."
- "For low bridges the bowstring girder is strongly recommended"
- Wrought-iron girders were "of such recent introduction that … we are unable to express any opinion."
- And finally, "Lattice bridges[101] appear to be of doubtful merit."

James writes that the use of long-span, multiple-casting beams ended as a direct result of the accident rather than the report. The report was rather backward looking and at the time it was written, "most of the seeds had been sown while some had already germinated" for the "great period of truss evolution [in] the third quarter of the century." But this was to come only after "the success of the Britannia and Conway wrought-iron tubular girders led to a boom in tubular and box girders for twenty years."[102]

By 1845 in England, the most significant structures in wrought iron were ships. Girder bridges were primarily of cast-iron with the trussed girder reaching spans of 18.3 m to 32.5 m (60 ft to 100 ft).[103] Great progress had been made in the development of the iron beam in the last half-century but knowledge of strength of materials was grossly lacking. Jewett's research "indicates that empirical methods developed the shape [of the **I**-beam] and that the theory was written independently," as opposed to "a well-defined theory of beam flexure" pointing the way to the best beam profile. "The story is one of evolution and practice rather than production from design."[104]

Stephenson's Ware Bridge of 1841 was one of few all-wrought-iron structures. The largest spans then being built on the beam principle were wooden lattice trusses in North America. Wrought-iron lattice trusses were just starting to be built in Europe. In summary, Stephenson had little precedent for the tubes he was considering to build.

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[101] In the Commissioners' terminology, "lattice bridges" meant any form of parallel-flanged framed truss.
[102] James[1], p2.
[103] Sutherland[2], p72-75.
[104] Jewett, p346.

3.7 Wrought Iron, the State of the Art in 1845

3.7.1 Cast and Wrought Iron, Its Production History in England

Until the 16th century, charcoal was the primary fuel used in the blast furnace for iron production. In 1558, Queen Elizabeth passed an act in England to prevent the use of timber for fuel in iron manufacture because severe deforestation was making it difficult to supply adequate timber for the navy.[105] This act crippled the English iron industry for over half a century until the iron-founder, Dud Dudley invented a process to use coal for fuel in the blast furnace in 1619. In 1651, Jeremiah Buck received a patent for the use of coke, a product of coal made in a similar way as charcoal is made from wood, in smelting operations. Unfortunately, Dud Dudley's invention, which was the only practical method then available, was largely ignored because of resistance from charcoal iron masters. In 1735, Abraham Darby, Sr., successfully invented a method for using coke in a blast furnace after five years of experimentation. He had begun his experiments with pit-coal, treated to produce coke, because of low charcoal supplies.[106]

When iron is produced in a blast furnace fueled by charcoal the product is principally wrought iron. Iron produced in a blast furnace fueled by coke is cast iron. In the iron industry, this is called pig iron. Pig iron contains 2 to 5 alloyed percent carbon. It melts at a relatively low temperature, about 1200°C (2,200°F), and, when melted, is readily cast into molds, hence its alternative name – cast iron. Cast iron cannot be worked by hammering or rolling, and it is about 5.5 times stronger in compression than in tension.[107]

Wrought iron contains less than 0.1 percent of alloyed carbon. It has a melting point of 1480°C (2,700°F) and, when heated to redness, is easily worked by hammering, shearing, forging or rolling. It is not suited for casting. Wrought iron is slightly stronger in tension than compression by a ratio of about 6 to 5.[108]

In 1784, Henry Cort patented a process for converting pig iron into wrought iron. In his patent, Cort wrote,

> The iron so prepared and made may be afterwards stamped into plates, and piled or broke, or worked in an air furnace, either by means of pots or by piling such pieces, in any of the methods ever used in the manufacture of iron from coke fineries without pots. But the method and process invented and brought to perfection by me is to continue the loops in the same furnace, or to put them into another air furnace or furnaces, and to heat them to a white or welding heat, and then to shingle them under a forge-hammer, or by other machinery, into half-blooms, slabe, or other forms; and these may be heated in the chafery, according to the old practice; but my new invention is to put them again into the same or other air furnaces, from which I take the half-blooms, and draw them under the forge-hammer, or otherwise, as last aforesaid, into anconies, bars, half-flats, small square-tilted rods for wire, or such uses as may required.[109]

[105] Goodale and Spear, p40.
[106] Goodale and Spear, p40, 43, 46, 55 and 57.
[107] Rosenberg and Vincenti, p78-79, note 5.
[108] Rosenberg and Vincenti, p78-79, note 5.
[109] Dempsey, p12.

Therefore, we see that wrought iron was necessarily more expensive to produce than cast iron because more steps are required in its production than cast iron. However, throughout the early 19th century the cost of wrought iron decreased as demand for both cast iron and wrought iron, particularly in chain, boiler and ship construction, increased. In 1760, not more than 30,000 tons of pig iron was produced in Britain. 325,000 tons was produced in 1818, and over 1,000,000 tons in the late 1830s. By 1847, British production of pig iron had reached 2,000,000 tons.[110] In 1830, Joseph Hall invented the puddling process, which employs a reverberatory furnace.[111] This invention further decreased the price of wrought iron, making it economically viable to for an expanding range of applications that could take advantage of wrought iron's malleability and superior strength characteristics.[112]

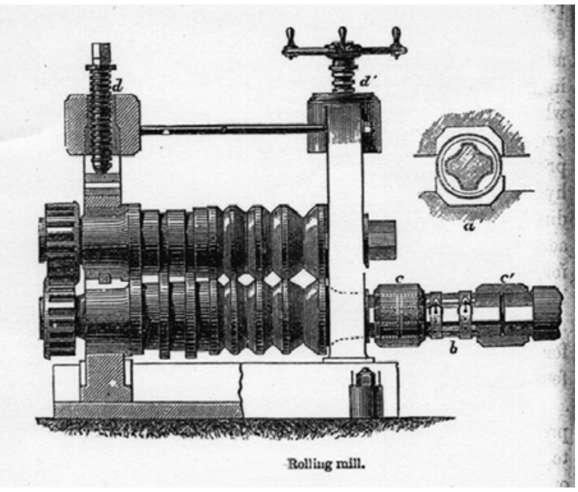

Fig. 36: Grooved rolls for rolling prismatic wrought-iron bars, first introduced by Henry Cort, 1783. (Bauerman)

3.7.2 Material Manipulation

For structural purposes, wrought iron is usually rolled into a variety of forms, though it can also be hammered and forged. The first rolled products were plates rolled between two "smooth" rolls. Bars sheared from these plates were worked into other forms such as rounds, often by hammering. Henry Cort is called the "father of the iron trade" because he rationalized and substantially improved on existing practice to increase productivity and quality of rolled wrought-iron products. In 1754, Cort built the first rolling mill in England. He patented grooved rolls in 1783. (**Fig. 36**) Grooved rolls permitted prismatic bars to be rolled instead of hammered. These forms were limited to squares, rounds and other prismatic forms.[113]

Due to subsequent improvements in the design of rolls and, in particular, more powerful of engines used to run the mill, the rolling of large plates and more complex forms such as angles, channels, tees and, finally, **I**-sections became possible.

Machines for cutting and shearing iron plates also evolved to become more efficient and address problems typical to such procedures that lead to curling or buckling of the plates. Shearing was previously done by a lever-fly, which worked by the lever principle. This machine could also punch holes in the metal. Machines using different means of power replaced the lever-fly; one used hydraulics and the other was the steam engine. The latter was more successful because the hydraulic system worked too slowly to be economical, even in comparison to using the lever-fly.[114]

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[110] Rosenberg and Vincenti, p4.
[111] Oberg and Jones, p12-13.
[112] Rosenberg and Vincenti, p4.
[113] Goodale and Spear, p61 and 73.
[114] Dempsey, p20.

3.7.3 *Early Applications of Wrought Iron Outside of England*

In antiquity, wrought iron was exceptionally employed in flexural members in Greece[115] and India.[116] It was also used in suspension chains in China and Tibet.[117] Otherwise, until the 18th century, wrought iron was principally used in tension for straps and ties.

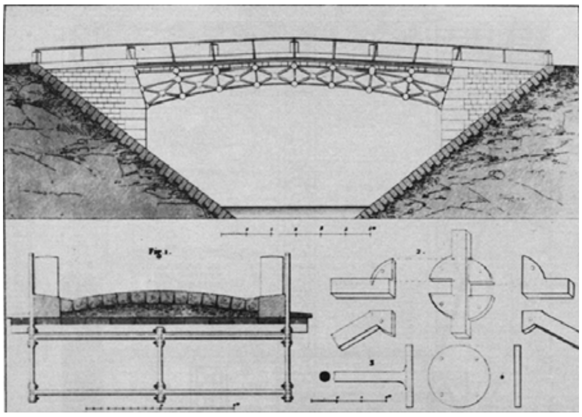

Fig. 37: First all-wrought-iron bridge, Pont sur la Crou, near Saint-Denis, Louis Bruyère, 1808. (Steiner)

The late 18th century saw the start of wrought-iron building construction in France.[118] In the Église de Sainte-Geneviève, wrought iron was used as reinforcement of masonry construction. It was thought to work primarily in tension. (**Fig. 20**) The roofs of the Théâtre Français was constructed of wrought iron in an effort to make the structure less susceptible to fire damage. This roof and the one at the Louvre are both built on the arch principle, probably influenced by the construction of Ironbridge. (**Figs. 21 and 22**) These roofs represent some of the first applications of wrought iron in which material is not chiefly designed to resist tension.

The first all-wrought-iron bridge to be constructed was the Pont sur la Crou, near Saint-Denis, France, in 1808. (**Fig. 37**) Louis Bruyère designed this bridge, which served as a footbridge across the Crou, a tributary of the Seine. Bryère adapted Palladio's design published in 1570 for the timber truss, shown in **Figure 5(f)**, and substituted timber for iron. The Pont sur la Crou has a span of 12 m (39.4 ft) and a rise of 0.9 m (3 ft). It has three trusses spaced 1.75 m (5.7 ft) apart, which are joined by wrought-iron tie-rods. Bruyère designed a similar bridge with a 130 m (426.5 ft) span, but it was not built. Bruyère's bridges had little impact on the development of wrought-iron bridges because they were little known.[119]

3.7.4 *Early Applications of Wrought Iron in England*

As elsewhere, wrought iron was generally used in tension as straps and ties. The first significant development in wrought-iron technology being in the construction of boilers. To increase the power of steam engines it was necessary to build secure pressure vessels. Early boilers were constructed of wood, which was obviously too weak and susceptible to rot and deformation. Boilers of cast-iron plate were brittle and too weak in tension to be appropriate for a vessel under high pressure. Wrought iron was a better alternative because it is strong and ductile. Later developments in shipbuilding and tubular beam construction benefited from the pioneering work done for boilers. Another interesting development in wrought-iron plate construction was its application to caissons or floating gates for the entrances to wet docks.[120]

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[115] Coulton, p148. **Ref. Appendix A-01, p A.20, Fig. 21.**
[116] Hamilton[2], p40.
[117] Peters[3], p18-20. **Ref. Appendix A-02, p A.32-A.33, Figs. 5, 6 and 7**.
[118] Steiner. **Ref. previous section, p A.94-A.95**.
[119] Steiner, p32.
[120] Dempsey, p14.

3.7.5 Wrought Iron in Shipbuilding

Use of wrought iron in the framing and plating of ship hulls played a significant role in the development of the wrought-iron beam, and, reciprocally, the development of the wrought-iron tubular beam played a significant role in future shipbuilding practice. The importance of shipbuilding to the history of the Britannia tube is twofold. First, William Fairbairn was, by 1845, an accomplished and respected shipbuilder who had pioneered various improvements in their construction and in riveting technology. Second, Stephenson used Fairbairn's work in all-iron ship construction to convince the directors of the railway of the feasibility of his tubular idea. An accident while launching the ship *Prince of Wales* in 1847, left the ship supported only by its bow and stern. This convinced Stephenson of the feasibility of his idea. (**Fig. 82**)

Dempsey wrote in 1850 that iron shipbuilding was "an art… still in its infancy."[121] The first all-iron boat appears to have been constructed by Aaron Manby, in 1821, at the Horsely Iron Works, near Birmingham. This boat, named the *Aaron Manby*, was 36.6 m (120 ft) in length and 5.5 m (18 ft) wide.[122] To illustrate the state-of-the-art of shipbuilding at that time, Dempsey describes the construction of the *Grappler*, a steam frigate built by W. Fairbairn and Sons. Dempsey writes,

> In the sheathing or covering plates of iron vessels, which are necessarily weakened at the edges by the close rivet-holes, improvements have been designed to compensate for this weakening, by giving an additional thickness to the plates at the edges. Mr. J.G. Bodmer, of Manchester, some years ago patented a mode of doing this, and designed a reversed covering plate to embrace the thickened edges of the two meeting plates, and thus relieve the rivets of part of the lateral strain to which they are exposed. In the *Grappler*, Mr. Fairbairn adopted sheathing plates rolled with thickened edges, which meet over the center of the T-iron ribs, and are riveted to them. The importance of these thickened edges may be inferred from the results of experiments on this subject,[123] which showed that the strength of a joint to resist a direct tearing strain is, if single-riveted, only 60 per cent. of the strength of the plate; and, if double-riveted, 75 per cent. In the plates of the *Grappler*, the edges are thickened in the proportion of about 5 to 3 of the body of the plate; so that the sectional area through the rivet-holes may be nearly equal to that through the body of the plate. This thickening also affords a great advantage in the external evenness of the sheathing, by admitting the heads of the rivets to be counter sunk, that is, formed conically, and inserted so as to preserve a flush surface.[124]

By 1845, upwards of one hundred British vessels are reported to have been constructed of iron, with frames or ribs and sheathing plates.[125]

[121] Dempsey, p14.
[122] Dempsey, p14.
[123] William Fairbairn performed important experiments on riveted connections.
[124] Dempsey, p15-16.
[125] Dempsey, p15-16.

3.7.6 Connection Technology

Heat welding had been used to make larger wrought-iron products or to make built-up members since ancient times. Examples are the ashlar clamps of the Greeks[126] or the crudely made beam for the Indian temple shown in **Figure 38**. Wrought iron can only be produced in small quantities, so large wrought-plates and structural shapes such as Zorés's **I** -beam had to be made up of a *pile*, that is a stack of smaller bars heat welded and then rolled. **Figures 39 and 40** respectively show several pile configurations for a variety of sections and piles being rolled into structural shapes.[127] An interesting possibility with this method of fabrications is to use different grades of wrought iron, i.e. a higher grade in the flanges and a lower grade in the web. Higher grades of wrought iron are produced by re-rolling the first product of wrought-iron production called the *muck bar* or *puddled bar*, from which other products are rolled directly or from piles of these bars. Large plates were typically made by alternating the orientation of the bars by 90° for each layer much like plywood.[128] (**Fig. 41**)

The first mechanical connections used in wrought-iron construction were similar to the timber-like joints characteristic of the cast-iron Ironbridge. (**Figs. 42 and 43**) This type of construction was superseded by riveting and, to a lesser extent, bolting. Riveting, which was the primary form of structural connection until the early 20^{th} century, was developed in boiler construction in the second half of the 18^{th} century. However, riveting technology significantly predates this period.

Use of plate iron in the construction of boilers and ships led to improvements in rivet manufacture, the machines used for punching the holes and for doing the actual riveting. By

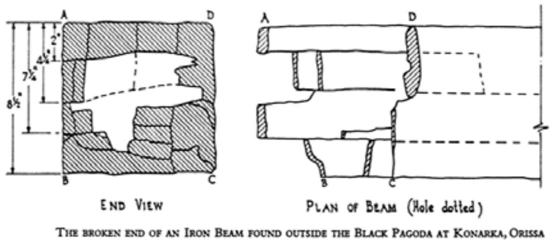

Fig. 38: Broken end of an iron beam composed of heat-welded iron bars found outside the Black Pagoda at Konarka, Orissa. (Hamilton[2])

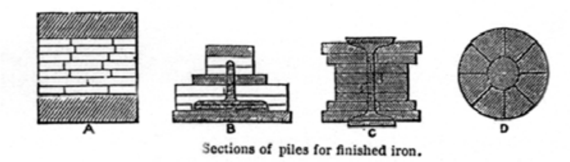

Fig. 39: Representative sample of *piles* composed of wrought-iron bars heat welded together that are then rolled to a desired structural shape larger than can be made from the bars that are made after the initial production of wrought iron. (Bauerman)

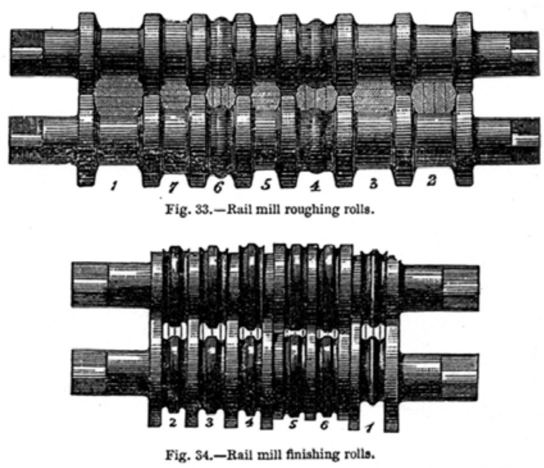

Fig. 40: Piles being rolled into structural shapes. (Bauerman)

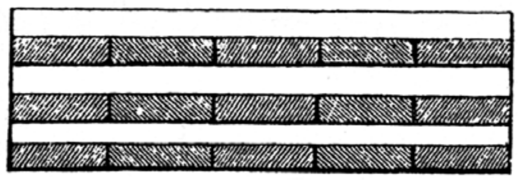

Fig. 41: Pile of bars for rolling plates, each layer oriented 90° to the one below. (Fairbairn[2])

[126] See **Appendix A-01, p A.13, Fig. 13**.
[127] Bauerman, p376-377.
[128] Sutherland[2], p38.

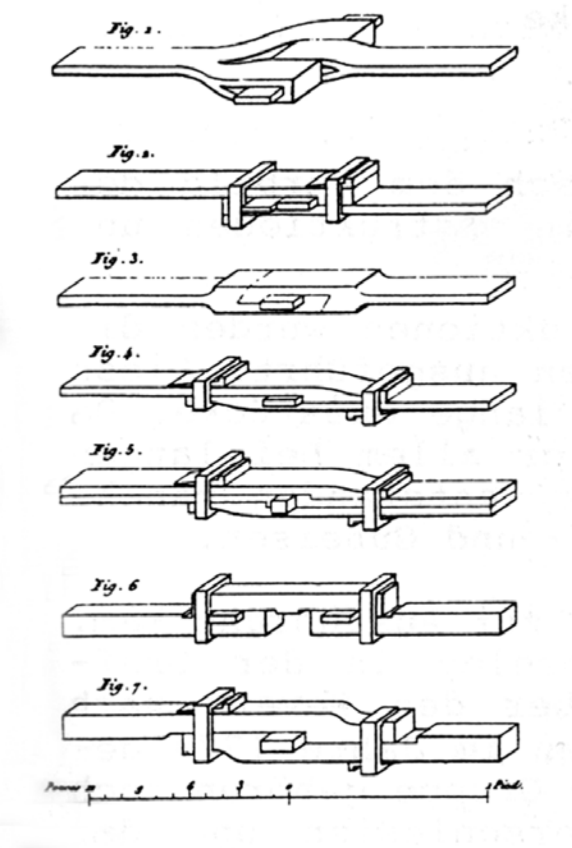

Fig. 42: Variety of early iron connection details that show influence of timber construction. (Mislin)

Fig. 43: Detail of Ironbridge showing timber-like connection. (Brown)

1845, rivets were being manufactured in large quantities by improved machinery that cut a proper length of iron from a rod and formed a precise head with a die. The time for punching the holes was decreased and quality increased by improvements made to the lever-fly. Hydraulic machines were tried but were too slow.[129]

The process of riveting starts with heating the rivet to a red heat. The projecting end is inserted through the hole and then hammered by one man to form a head while the pre-formed head is held in place by another. Pneumatic hammers later replaced mauls for forming the head. This method of riveting results in the shear strength of the rivet being augmented by friction produced when the hot rivet contracts while cooling, clamping the two plates together. An additional benefit is that these joints perform well in keeping water from infiltrating between the plates. William Fairbairn invented a steam powered riveting machine in 1833 that worked well in shipyards but was found to be too unwieldy to use for constructing the Britannia tubes.[130]

3.7.7 The Step-Wise Integration of Wrought-Iron into Bridge Structures

Wrought-iron was first employed on a large scale as a primary structural material in suspension bridge chains starting around the turn of the 19th century. In the 1830s, and perhaps earlier, a footbridge was designed by Robert Stevenson of Edinburgh with wrought-iron underspanned timber beams.[131] (**Fig. 44**)

Sir Marc Brunel experimented with wrought-iron reinforced brick cantilevers in the 1830s. (**Fig. 45**) These tests were performed to compare the tensile strength of two different cements for use in the construction of the Thames Tunnel. Pasley and J.B. White &

[129] Dempsey, p17-18.
[130] Dempsey, p19.
[131] Sutherland[2], p69.

Sons tested wrought-iron reinforced brick beams in the late 1830s, probably influenced by Brunel's earlier experiments. (**Fig. 46**)

In all these uses, wrought iron was mainly thought of as a material to resist tension and in its early structural uses this tensile emphasis remained. The transition from cast iron to wrought-iron beams was a slow one in England. Few people just switched from one material to the other. Fairbairn wrote,

> Before [1845], our knowledge of the properties of wrought iron, and its application to the useful arts, was very imperfect. It had been used in the construction of boilers, steam engines, and water wheels, from a comparatively early period, and even at that time and for some years previous, it was making rapid progress in its application to ship-building. Its properties, distribution, and appliance to beams and bridges, were, however, unknown and unappreciated until the experiments [performed for the Britannia and Conway bridges] proved its superiority over every other material then known for the attainment of objects for which it has since been so largely and so extensively in demand. As a material for the construction of bridges, wrought iron was universally condemned, and some of our ablest mathematicians went so far as to prove its inefficiency in the shape of rectangular tubes composed of riveted plates, as being perfectly utopian, and, to employ the expression then made use of, "*It would crumple up like a piece of leather!*[132]

Perhaps the earliest use of wrought iron for beams in England was the Flitch beams constructed by John Smeaton at the London Docks some time before 1792. Flitch beams were probably used as early as the Roman Empire, and perhaps earlier.[133] Smeaton's beams were comprised of wrought-iron plates sandwiched between timbers. Smeaton may

Fig. 44: Wrought-iron underspanned timber footbridge over the Whitadder at St. Albans, Rathans, Robert Stevenson, c.1830s. (Sutherland[2])

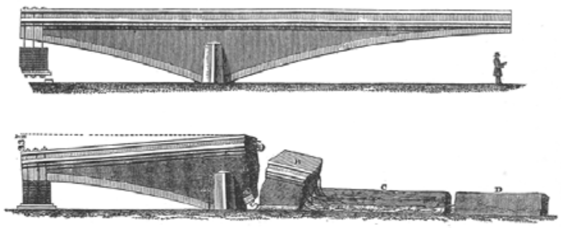

Fig. 45: Wrought-reinforced brick cantilever, Marc Brunel, 1832. (Peters[1])

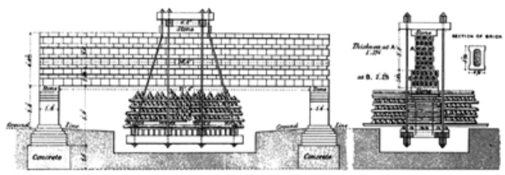

Fig. 46: Wrought-reinforced hollow-brick beam, J.B. White and Sons, 1851. (Peters[1])

[132] Fairbairn[3], p258. (2nd ed., 1857)
[133] Hamilton[2], p37. Flitch beams using bronze plates were used to construct the roof of the portico of the Pantheon in Rome.

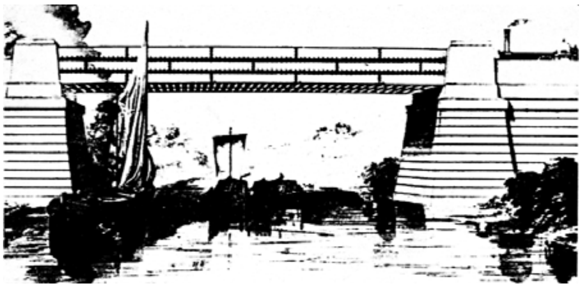

Fig. 47: Built-up cast-iron girder of 36.6 m (120 ft) span, Robert Stephenson, 1845. (Sutherland[2])

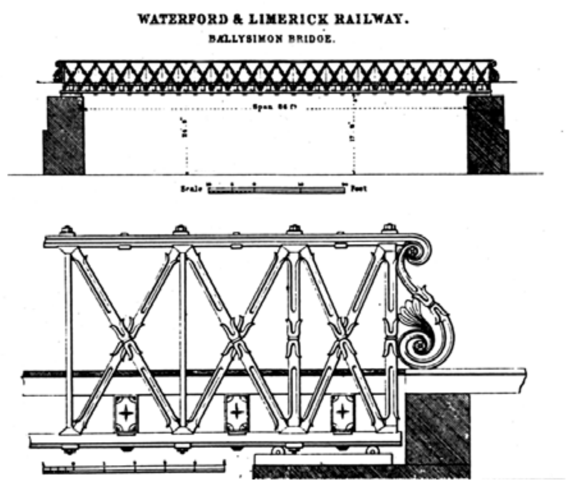

Fig. 48: Combined cast and wrought iron Howe truss with 26.2 m (86 ft) span, Waterford & Limerick Railway, Ireland, c.1840s. (Sutherland[2])

have thought the wood would protect the iron from corrosion if put on the inside. This is probably one of the first applications of wrought iron acting in bending in modern times.[134] Nevertheless, Smeaton's beams had no lasting impact on the development of wrought-iron beams.

As the requirement for longer spans grew with the railroad boom in the 1830s and 1840s, the limitations of cast iron became increasingly problematic. Rather than adopt wrought iron in place of cast iron, engineers sought to overcome the weak tensile strength of cast iron, and its single-cast size limits, by using wrought iron compositely with the cast iron. The application of wrought iron in this period can be summarized as follows:

To overcome the horizontal thrust and grade problems created by cast-iron arches, the tied-arch was developed. The tied-arch was built with an arch of cast iron and the horizontal thrust was resisted by wrought iron in the deck structure that tied the two aches together at their springing.[135] (**Fig. 18**)

To overcome the length limits of single-casting cast-iron beams, built-up beams were developed. These built-up beams were comprised of two or more castings connected by wrought-iron bolts through flanges cast for that purpose. (**Fig. 47**) In 1845, Robert Stephenson designed a built-up beam that was intended to span 36.6 m (120 ft) by itself – more than double the practical limit for single castings. He used 16 sections bolted together in three layers with the joints staggered. This beam was 3.7 m (12 ft) deep and successfully tested with a total distributed load of 108 tons. Edwin Clark mentions provision for wrought iron trussing[136] but it seems that it was not used for the tests. The beam was never used because it used material inefficiently and was

[134] Sutherland[2], p78.
[135] James[2], p185.
[136] Clark, p2.

costly to plane all the joints. This experiment may have been influential in Stephenson's decision to examine the possibilities of fabricating girders using riveted wrought-iron plates.[137]

To overcome the tensile strength limits of cast iron, some particularly interesting developments involved reinforcing cast-iron beams with wrought iron. J.U. Rastrick mentions wrought-iron plates, riveted to the bottom flange of cast-iron beams, in evidence he gave before the Royal Commission inquiry into the application of iron to railway structures. The inquiry report was published in 1849. Peter Barlow tried to improve upon the riveted plate by casting wrought-iron bars directly into the flange of the cast-iron girder, an antecedent to modern reinforced concrete. (**Fig. 33**) For unknown reasons, this idea was not adopted even though a test beam was 50% stronger than cast iron alone and it was apparently not liable to sudden fracture. A final experiment was to add up to 40% of molten wrought-iron scrap to the cast-iron mix. Tests showed that castings were appreciably stronger by this method. Morris Stirling had a patent for "toughened iron" of this type and must have believed that he was indeed reinforcing the cast iron, much as we might reinforce concrete with metal fibers today. In effect, all he really did was reduce the carbon content of the iron, making it closer to steel than cast iron.[138]

To overcome both length limits and the brittleness of cast-iron beams the trussed girder, mentioned in the last section, was developed, as were what we would call normal trusses with triangulated and lattice forms. Sutherland writes,

> Various triangulated forms had been used in timber for a considerable time, but from about 1830 onwards cast-iron struts with wrought-iron ties started to take over. Many roof trusses used cast and wrought iron although some of the earliest ones (at Euston Station dating from the 1830s) were wholly of wrought iron with rolled T-sections for the compression members, and all ties of wrought bar.
>
> In early triangulated bridge forms a combination of wrought and cast iron was also commonly used. Much of the development was done in the United States, and according to Timoshenko, the first all-metal trusses were built there in 1840…. Captain Warren introduced his form of truss in 1846 and the force in these were clearly analysed by Doyne and Blood in 1851.
>
> A typical combined cast iron and wrought iron triangulated railway bridge of the 1840s spanning 86 ft [26.2 m] is shown in [**Figure 48**]. This is a Howe truss derived from the earlier timber form with vertical wrought-iron ties and timber booms and diagonals. Another much larger and later form – a Warren truss – was designed by J. Cubitt over Newark Dyke about 1851. This spanned 240 ft [73.2 m] and had all compression members in cast and all tension ones in wrought iron.[139]

[137] Sutherland[2], p71-72.
[138] Sutherland[2], p70.
[139] Sutherland[2], p75-76.

3.7.8 An Explanation for the Timid Acceptance of Wrought Iron by English Engineers

In 1845, English engineers simply did not know very much about the structural properties of wrought iron. The Fairbairn's deck beams of 1833 and Stephenson's Ware Bridge girders of 1842 clearly illustrate with their inappropriate distribution of material in the flanges that early researches into the bending properties of wrought iron by Navier, Rondelet, Tredgold and others in the 1820s and 1830s did not lead to practical applications.[140] In 1845, Barlow noted the lack of experiments on the bending properties of wrought iron in his book *Strength of Timber*.[141] Barlow ultimately conducted experiments to determine the yield stress of wrought iron.[142] From preceding information of this section and the last, it is clear that in 1845 the English engineering community was firmly committed to the use of cast iron as its primary structural material. Wrought iron had limited application except to augment the tensile strength of cast-iron structures.

3.7.9 The Application of Wrought-Iron to Bridge Structures in 1845

In 1845, the application of wrought iron for bridge building constituted a new branch of the art.[143] From 1830 onwards wrought iron was applied to all-iron structures, either in the form of ties with the bulkier compression members in cast iron, or as a sort of "helper" to cast iron beams. Sutherland writes,

> During the years from 1830 up to abut 1850 countless combinations of wrought and cast iron were tried out – some sensible and some absurd; at first the emphasis was all on the cast iron with the wrought used sparingly, but the accent shifted gradually until in the end no cast iron was left. The great step forward in the middle 1840s was the establishment of wrought iron by itself as a material resistant to bending.
>
> In 1840 cast iron was firmly established as the modern structural material and when people talked of "iron" in bridges and buildings they meant cast iron. At this time wrought iron was rare and for beams virtually unheard of, yet less than 10 years later no engineer would have thought of making a major beam or girder of anything else. By 1850 cast iron had been completely eclipsed.
>
> At the beginning of this period (say up to 1840) the common types of bridge where [the masonry arch, the cast iron arch, the simple cast iron beam, the timber arch, truss or beam, and the suspension chain.]
>
> Not one of these bridge types met all railway needs. Masonry and cast iron arches exerted high horizontal thrusts [and the decks of long span arches had to be too high to be practicable for railways that demanded nearly horizontal gradients.] Simple cast iron beams were limited by casting and handling techniques to spans of 60 ft [18.3 m] or less and … were liable to flaws and sudden collapse under impact loads. Timber, used in a variety of forms in the early railways, was good for impact but liable to burn or rot or warp…. Cable suspension bridges, the only proved means of spanning gaps of 500 ft [152.4 m] or more were soon ruled out for railway because of their instability.[144]

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[140] Sutherland[2], p78.
[141] Barlow, p306.
[142] Dempsey, p70.
[143] Dempsey, p vii.
[144] Sutherland[2], p67 and 69.

In conceiving the tubes of the Britannia Bridge, Stephenson was challenging not only contemporary experience with long span beam structures but also the knowledge and perceptions of a material, wrought iron, that had not theretofore been considered a primary structural material. Iron, either cast or wrought, was used in bridge construction for only about seventy years, an era begun when Ironbridge was completed in 1779. The preceding review illustrates just how ‘new’ wrought iron was to engineering practice in 1845.

3.8 Development of the Tube, 1845-1847

3.8.1 Considering a New Structural Form

Edwin Clark wrote the following to put in perspective the formidable problems confronting Stephenson and his staff in early 1845,

> The natural difficulties to be overcome in crossing such a gulf were … much increased by the requirements of the Act of Parliament, by which the dimensions of the central pier were limited, and the roadway, as at the suspension-bridge, was to be 103 feet [31.4 m] above the water, this clear height or windway being insisted on throughout the entire span. Thus the arch was rejected; scaffolding from below was impracticable; and the navigation was, under no circumstances, to be interfered with. These were the apparently insurmountable difficulties which the engineer had to encounter and to overcome without delay. No existing kind of insistent [sic] structure appeared capable of such fearful extension; and the development of some new principle became imperative.[145]

For lack of better option, Stephenson conceived the idea of a huge tubular bridge, constructed of riveted wrought-iron plates. To Stephenson, “it appeared evident that the tubular bridge was the only structure which combined the necessary strength and stability for a railway, with the conditions deemed essential for the protection of navigation.”[146] Galileo provides an early record of the advantage of the tubular section,

> I wish to discuss the strength of hollow solids, which are employed in art – and still oftener in nature – in a thousand operations for the purpose of greatly increasing strength without adding to weigh; examples of these are seen in the bones of birds and in many kinds of reeds which are light and highly resistant both to bending and breaking. For if a stem of straw which carries a head of wheat heavier than the entire stalk were made up of the same amount of material in solid form it would offer less resistance to bending and breaking. This is an experience which had been verified and confirmed in practice where it is found that a hollow lance or a tube of wood or metal is much stronger than would be a solid one of the same length and weight, one which would necessarily be thinner; men have discovered, therefore, that in order to make lances strong as well as light they must make them hollow.[147]

Rosenberg and Vincenti write, “Such a structure, however, was totally unlike anything that had been previously attempted. Wrought iron had never before been used on so large a scale. The novelty of both the materials and the design was so great that there was no

[145] Clark, p8.
[146] Clark, p29.
[147] Galilei, p150.

reservoir of reliable knowledge or experience upon which to draw in determining feasibility and, above all, safety. Much requisite knowledge had to be fashioned for the occasion since it did not exist at the outset."[148]

Rosenberg and Vincenti continue,

> The unprecedented scale and novelty of the bridge can hardly be overemphasized. The structure (**Figs. 1 and 73**) would consist of two parallel lines of tubes, one for trains in each direction. Each line was made up of four tubes, supported by three masonry towers (the center one on the Britannia Rock) and two end abutments. The intended open distance of 450 feet [137.2 m] between towers (later increased to 460 feet [140.2 m]) was vastly greater than the 31.5 feet [9.6 m] of the previously longest wrought-iron span.[149]
>
> The bridge had three novel aspects:
>
> 1. Previously built long-span bridges were either arches or of the suspension type that exert both vertical and horizontal forces at their end supports. The girder would only exert vertical forces, which is a simplification.
> 2. Although cast-iron beams were being used considerably in various applications, wrought-iron beams had attained only limited utility, mostly in shipbuilding. The Britannia Bridge would be built of this relatively new structural material.
> 3. Experience with beams of both cast and wrought iron had been limited to cross-sections of **I** or **T** shape (with a single vertical element). Moreover, this and the other elements of the cross-section tended to be thick and heavy. The Britannia Bridge would be tubular – with a closed cross-section, as in a pipe – and the walls of the tube would be thin.[150]

The development of Britannia and Conway tubular bridges will now be reviewed. The account of this development is exhaustively recorded in the contemporary accounts of Clark, Fairbairn, and G. Drysdale Dempsey. The history of the development of the Britannia and Conway tubes as recorded in these accounts is rife with dissention as the relationship between the various actors in this affair fell apart towards the end of the project due to contrasting views of who was responsible for what. I will try to avoid this debate. This treatise will rely on Fairbairn's account of the actual research and development program because he was most intimately involved in that process. Stephenson was able to visit Fairbairn's works to view the tests himself only on a couple of occasions over a two year period. It is true that Clark was more directly involved in the process, particularly with respect to constructive details, but it is Fairbairn's work, in partial conjunction with the aid provided by Eaton Hodgkinson, which resulted in the global form of the tubes that was adopted.

[148] Rosenberg and Vincenti, p6-7.
[149] Rosenberg and Vincenti, p9 and 12. Also: Smith, H.S. "Bridges and Tunnels," from Singer et al., *History*, 5:505.
[150] Rosenberg and Vincenti, p12-13.

3.8.2 Stephenson's Initial Conceptual Designs

Initially, Stephenson had conceived a rectangular cross-section for the tube. Stephenson simplified the problem of analyzing the tube by modeling it as two I-sections side by side. Such simplification would have given Stephenson some basis to start from in the design of these tubes since he had no actual precedent to guide him. The method of calculating the strength of cast-iron beams was well established by then so Stephenson understandably felt comfortable viewing his structure in a familiar way. Later, Stephenson thought the rectangular section might be inadequate to resist both vertical loads and lateral wind loads. Therefore, he considered using circular or elliptical sections without completely discarding the rectangular form.[151]

In April 1845, Stephenson consulted with William Fairbairn. With only Fairbairn's confidence in the feasibility of the idea, Stephenson presented his plan to the parliamentary committee responsible. Stephenson's initial proposal defined a bridge of two spans, each 137.2 m (450 ft) long and 7.6 m (25 ft) high, with an intermediary pier on Britannia Rock. There would be 32.0 m (105 ft) clearance between high water and the bottom of the tube.[152] During a public discussion, Stephenson was asked if there was any precedent for a bridge of this kind. Stephenson replied: "No, there is no experience of it; nor was there of the iron vessel some time ago. There is now one building by Mr. Fairbairn, 67.1 m (220 ft) in length, and he says that he will engage that when it is finished that it shall be put down in the stocks at each end, and shall have a thousand tons of machinery in the middle of her, and it will not affect her."[153] On June 30, 1845, the Chester and Holyhead Railway Bill received royal assent to proceed on Stephenson's proposal.

3.8.3 Fairbairn is engaged to Research and Develop the Tubular Concept

Upon receiving Parliamentary approval, Stephenson contacted Fairbairn to begin development of the tubular concept, which Fairbairn had offered to do during their previous meeting. Stephenson was in no position to undertake the research and development himself because he was engaged as chief engineer for many projects which required almost ceaseless travel on his part. Edwin Clark, Stephenson's assistant, was appointed by Stephenson to oversee the day-to-day work of the Britannia Bridge project on his behalf.

Fairbairn was well regarded for both his practical pursuits as well as his commitment to the furthering of knowledge about the properties and applications of iron. At this time, Fairbairn was under the impression that Stephenson wanted to primarily support the tube by chains. "The wrought-iron tube, according to Stephenson's idea, was, indeed, entirely subservient to the chains, and intended to operate from its rigidity and weight as a stiffener, and to prevent, or at least to some extent counteract, the undulations due to the catenary principle of construction."[154] This description is counter to what Stephenson wrote himself whereby Stephenson arrived at the tubular beam concept after revisiting the idea of a suspension bridge. It is true, though, that Stephenson considered using suspension chains as an auxiliary supporting device through much of the development of the tubes. This issue will be discussed separately below. Whether Fairbairn had just misunderstood Stephenson's intent

[151] Clark, p27.
[152] Dasgupta, p81.
[153] Clark, p50.
[154] Fairbairn[1], p2-3.

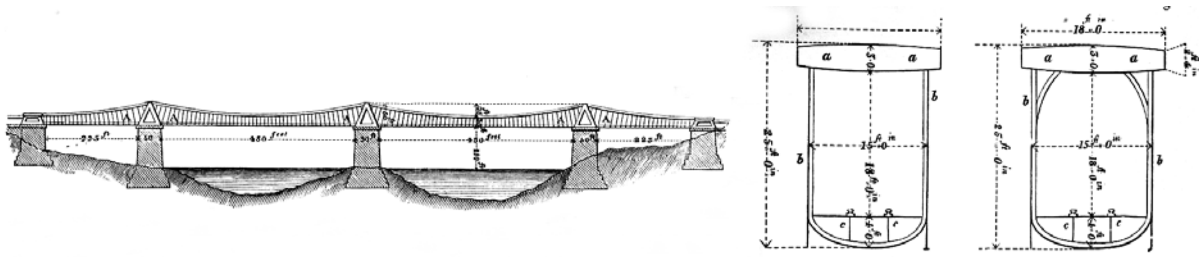

Fig. 49: Fairbain's conceptual proposal for a tubular bridge conceived to be suspended from its top flange with deck suspended from the side-plates. (Fairbairn[1])

is inconsequential because Fairbairn began his studies on the premise that the principle supporting action of the structure should be in tension.[155] This assumption is consistent with how wrought iron was then employed in bridges in England and the state of strength of materials knowledge.

From the beginning, Fairbairn was opposed to the superposition of a large rigid tube and a flexible chain. He feared that such an arrangement would be injurious to the tubes due to oscillations introduced by the chains. In an initial conceptual design from May 1845, Fairbairn proposed to Stephenson to try to get the top of the tube to act in tension whereby the whole of the top flange would in effect act as a suspension element and the sides would suspend the deck and provide the desired rigidity to the structure.[156] (**Fig. 49**) If we look at his ingenious method of constructing this structure using the balanced cantilever method, we see how Fairbairn may have conceptually viewed the top flange acting in tension because it is in tension while cantilevered during construction. (**Fig. 50**) However, once the two sides are connected the structure would act like a beam and therefore his initial premises were wrong.

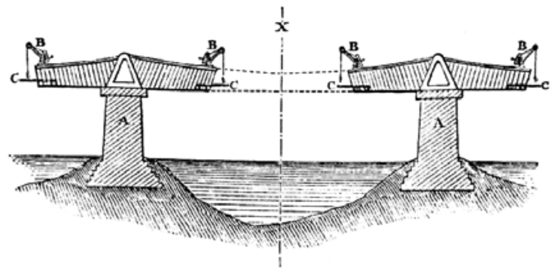

Fig. 50: Fairbairn's proposal to build a bridge using balanced cantilever erection method. (Fairbairn[1])

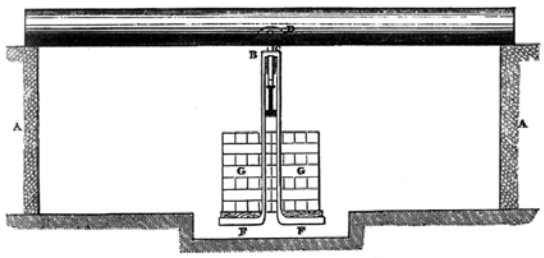

Fig. 51: Circular tube being loaded in Fairbairn's test apparatus. (Fairbairn[1])

3.8.4 Fairbairn's Initial Experiments and the Discovery of Flange Local Buckling

As per Stephenson's wishes, Fairbairn began his tests on tubes with circular and elliptical tubes. (**Figs. 51 and 52**) His first series of tests were on circular tubes for which he varied the distance between supports, tube diameters, and plate thickness. During these first tests, Fairbairn observed that the two smallest tubes with the thinnest plates failed in the compression flange. Seven other tubes failed in tension along the riveted joint of the tube

[155] Fairbairn[1], p5-7.
[156] Fairbairn[1], p5-7.

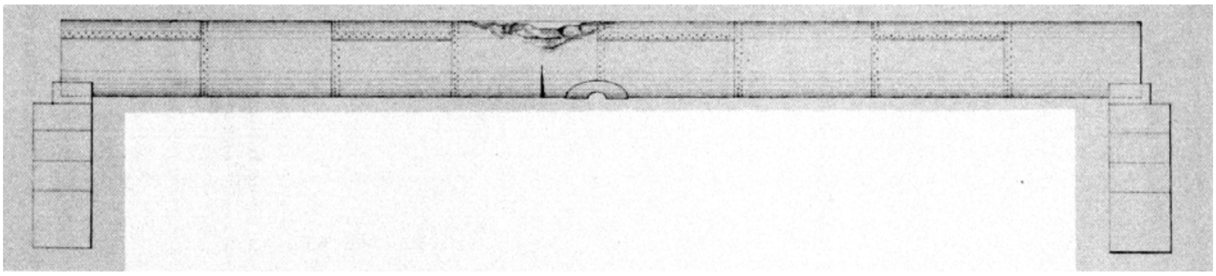

Fig. 52: Elliptical test tube after failure showing buckling in top flange and rupture in the bottom, 17 ft. span. (Fairbairn[2])

that Fairbairn had placed on the bottom. At this point, Fairbairn thought he was not reaching the full capacity of the material's strength, in either compression or tension, because of poor construction details.[157]

In the second series of tests, Fairbairn tested elliptical tubes with the riveted joints now located at the center of the beam. Fairbairn's tests on the elliptical tubes yielded enlightening results. Fairbairn found "that the whole of these experiments indicated weakness on the top side of the tube, which, in almost every case, was greatly distorted by the force of compression acting in that direction. It is probable that those of cylindrical form would have yielded in the like manner, had the riveting at the joints been equally perfect on the lower side of the tube… As the thinner of these tubes yielded by being crushed on top, it would appear that the extreme thinness of the plates about 1 mm (0.04 in) caused the distortion of the tube before the virtual strength of the metal was called into action."[158] Clearly Fairbairn has discovered buckling and, in particular, its application to thin walled tubes. This is the first instance that this failure mode had been identified in beams.

3.8.5 The Rectangular Tube and the Cellular Flange

The third series of experiments were made with rectangular tubes. Fairbairn was impressed by the superior behavior of the rectangular form for the tubes. In his Report of February 1846, Fairbairn recorded that the rectangular form exhibited "considerably increased strength when compared with the cylindrical and elliptical forms; and considering the many advantages which they possess over every other yet experimented on, I am inclined to think them not only the strongest, but the best adapted (either as regards lightness or security) for the proposed bridge."[159]

Through parameter variation Fairbairn discovered that the strength of the tube increased when only the thickness of the top plate was increased. Conversely, increasing the thickness of the bottom plate did not increase the capacity of the beam. It may seem obvious to us now that if the beam failed by buckling before, increasing the thickness of the bottom flange would not help, but this was, to emphasize, a brand new phenomenon and Fairbairn did well not to discount any possible mode to achieve better performance. These experiments showed clearly that their was a problem with realizing the full compressive

[157] Dempsey, p74.
[158] Dempsey, p79. Quoting Fairbairn.
[159] Dempsey, p80. Quoting Fairbairn.

strength of wrought iron, and that the material did not share the same strength characteristics as cast iron, a previously held assumption.[160]

If Fairbairn had kept increasing the thickness of the top plate, as his initial results indicated would be helpful, he would have found that the resistance of the plate to buckling increased as a cube to its thickness. This discovery may have retarded or killed the subsequent development of girders with tubular flanges. However, there were two concerns that may have led Fairbairn to look for another solution. First, his beams were not designed for the strength thicker plates would have provided. Fairbairn had evidently determined the strength he wanted for the bottom flange, and he felt that the amount of material in the top flange should be sufficient to bear the compressive stress. Second, since he was very much focused on keeping the beam as light as possible, he actually sought to solve a more complex problem, trying to change the form of the flange to be stiffer while keeping cross-sectional area of material the same.

3.8.6 Structural vs. Material Failure

These early tests were significant because they made the design and engineering community at large aware of the distinction between *material failure* and *structural failure.* Dempsey incorrectly equates structural failure with *constructive failure* in grouping the buckling failure with the failure of a riveted connection under tension. This is really a material failure.[161] Therefore, it fell to Fairbairn to determine how to obviate local failure in the top flange before reaching the theoretical limit strength of the material.

3.8.7 Origin of the Cellular Flange

Fairbairn wanted to strengthen the flange without thickening it by using a more rigid form than a flat plate. In August 1845, Fairbairn built rectangular and elliptical tubes with a single cell on top. The cell apparently was intended to constrain a piece of timber that Fairbairn thought would brace the flange. (**Fig. 53**) When Fairbairn found that it was almost impossible to make the timber fit the cell perfectly, he decided to leave the timber out and try the tubes with hollow cells.

At this point Fairbairn asked for Eaton Hodgkinson's assistance to explain the buckling phenomena he had observed; and to determine a method for calculating the strength of a tube subject to such a failure mode. Eaton Hodgkinson joined Fairbairn in time to test the tubes shown in **Figure 53** in September 1845. These tests clearly demonstrated the superior performance of a tube with a cellular top. In a letter to Stephenson about the tests on the tubes with the 'fin,' dated September 20, 1845, Fairbairn wrote, "The defective powers of resistance of all the tubes of this shape have suggested a new arrangement and distribution of the metals; it being evident from the experiments, that the tube will resolve itself into a huge hollow beam or girder, leaving the two resisting forces of compression and extension as wide apart as possible."[162] In this letter, Fairbairn included sketches of two possible cross-sections for the Britannia tubes. (**Fig. 54**) These sketches show a remarkable similarity to the final tube designs.

[160] Dempsey, p82.
[161] Dempsey, p83.
[162] Fairbairn[1], p16.

Fairbairn and Hodgkinson tested elliptical and rectangular tubes and concluded that the rectangular form was best suited to the purpose.[163] As Rosenberg and Vincenti state, "Design engineers, especially in a pioneering effort, often select a particular configuration because it lends itself readily to thought and analysis."[164] The failure behavior of the elliptical tubes in particular was probably too complex to analyze because in such form it is difficult to determine where the flange ends and the web begins.

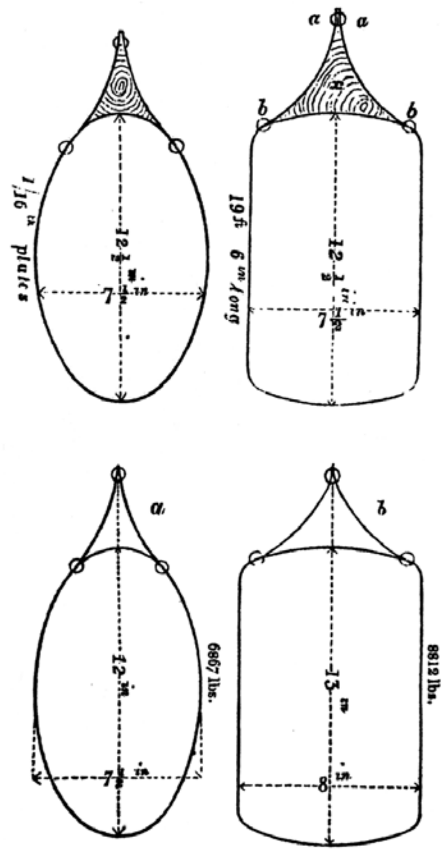

Fig. 53: Single-cell tubes with and without fir timber filler intended to stiffen plate from buckling. Timber was ineffective. Designed August 1845 by Fairbairn, tested September 1845 with Hodkgkinson. (Fairbairn[1])

In October 1845, Fairbairn and Hodgkinson constructed a rectangular tube. They used two corrugated-iron plates to form two circular cells in the top flange of the beam.[165] (**Fig. 54**) This form was probably influenced by Eaton Hodgkinson's experiments on the buckling behavior of cast-iron columns. From these experiments he determined that a hollow section column resisted buckling better than one with a solid section of equal cross-sectional area.

This beam was nearly twice as strong as tubes with simple plate flanges. Fairbairn observed that this form resisted the "puckering," or buckling, that was characteristic of previous tests. This allowed the full compressive strength of the section to be used. The superior strength of the tube with corrugated top flange influenced Fairbairn to adopt the cellular structure of the top of the tube.[166]

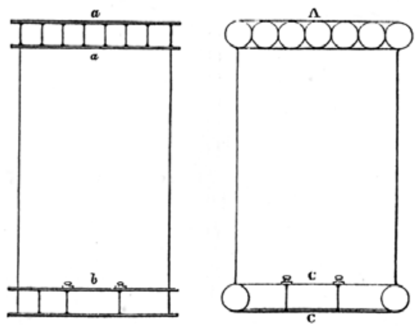

Fig. 54: Sketches reflecting conclusions of Fairbairn and Hodgkinson that tube will act as a beam, therefore section is rationally proportioned as such, incorporating this new idea about using a cellular flange to avoid this new failure mode, buckling. The tube with rectangular cells is remarkably similar to the final design. September 1845. (Fairbairn[1])

3.8.8 The Discovery of Web Local Buckling

Having solved the problem of flange local buckling, a new failure mode appeared. Fairbairn observed the buckling of the web, which tore away from the top and bottom flanges. Appreciating the value of this new knowledge and seeing the importance of

[163] Fairbairn[1], p15-19.
[164] Rosenberg and Vincenti, p19.
[165] Fairbairn[1], p19.
[166] Fairbairn[1], p12.

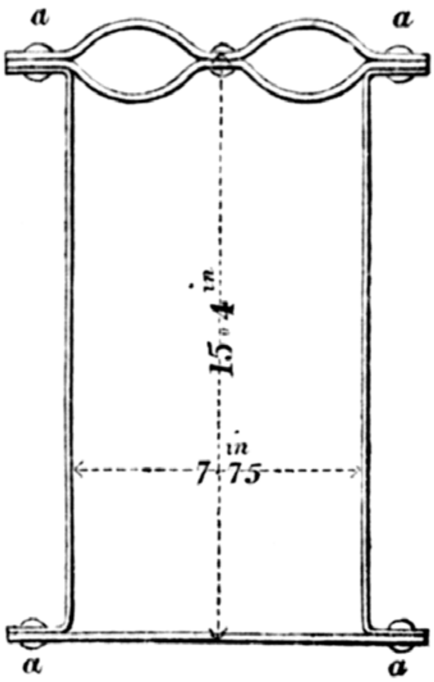

Fig. 55: Test section that convinced Fairbairn and Hodgkinson of the merits of the cellular compression flange. Cells made from two corrugated iron plates riveted together. October 1845. (Fairbairn[1])

understanding these phenomena, he said,

> Strength and lightness are desiderata of great importance, and the circumstances above stated are well worthy the attention of the mathematician and engineer. For the present we shall have to consider not only the due and perfect proportion of the top and bottom sides of the tube, but also the stiffening of the sides with those parts, in order to effect the required rigidity for retaining the whole in shape.[167]

3.8.9 No Knowledge / New Knowledge

In a letter to Fairbairn from Hodgkinson, dated Sept. 26, 1845, Hodgkinson concluded that there appears "to be a great want of fundamental information on the subject" of buckling in the top flange of girders."[168] Hodgkinson referenced Navier's *Leçons*, but it was not helpful to him.[169]

Writing to Stephenson in a letter dated October 15, 1845, Fairbairn reported,

> Our experiments of yesterday were the best and most satisfactory we have yet made; and agreeable to expectation, the form, as per [**Fig. 55**], gave not only the greatest strength, but what was of equal importance, there was a near approximation to an equality of the forces on the top and bottom sides…
>
> Some existing formulae of "Navier" and others can be applied, but they are not satisfactory; and before my friend Mr. Hodgkinson can satisfy himself on the mathematical part of the case, some further experiments must be made on *exceedingly small* and *greatly enlarged* tubes, with certain functions, calculated to establish the law which governs, not only the strength of the present, but of all future forms of tubes.[170]

[167] Dempsey, p85. Quoting Fairbairn.
[168] Fairbairn[1], p19.
[169] Fairbairn[1], p19.
[170] Fairbairn[1], p19-20.

Fairbairn and Hodgkinson conducted similar experiments on tubes with more cells, such as the one shown in **Figure 56** from December 1845. They found that the beam failed nearly simultaneously in compression and tension when the ratio between the areas of the top and bottom flanges was nearly 5:3. Subsequent trials with better quality control of the fabrication process gave improved results. (**Fig. 57**) Fairbairn felt that these experiments "were highly satisfactory and supplied sufficient data on which to proceed, with some degree of certainty, in preparing the designs and sectional drawings for the large tubes."[171]

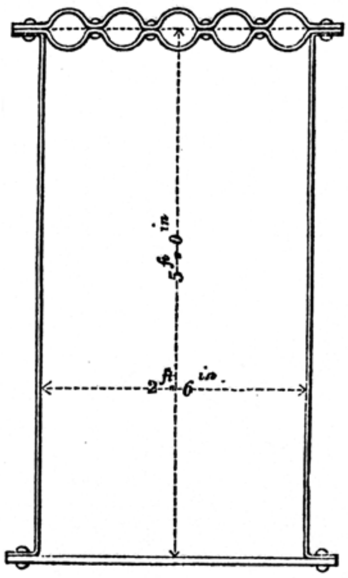

Fig. 56: Section as of December 1845. (Fairbairn[1])

Fairbairn concluded,

> So far as our knowledge extends, and judging from the experiments already completed, I would venture to state that a tubular bridge can be constructed, of such powers and dimensions as will meet, with perfect security, the requirements of railway traffic across the straits, … and although suspension chains may be useful in the construction in the first instance, they would nevertheless be highly improper to depend upon as the principal support of the bridge. Under every circumstance, I am of opinion that the tubes should be made sufficiently strong to sustain not only their own weight, but, in addition to that load, 2000 tons, equally distributed over the surface of the platform – a load ten times greater than they will ever be called upon to support.
>
> In fact, it should be a huge, sheet-iron, hollow girder, of sufficient strength and stiffness to sustain those weights; and provided the parts are well-proportioned, and the plates properly riveted, you may strip off the chains, and leave it as a useful monument to the enterprise and energy of the age in which it was constructed.[172]

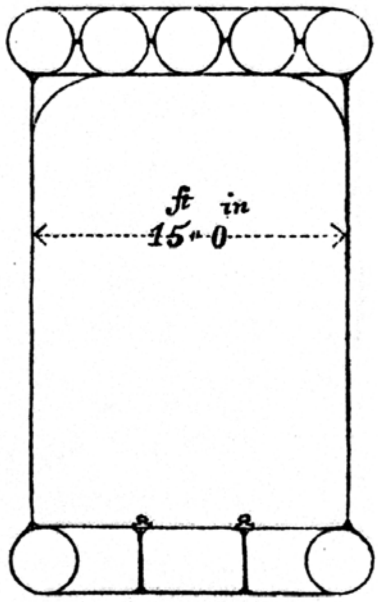

Fig. 57: Section as of February 1846. (Fairbairn[1])

[171] Fairbairn[1],p20-21.
[172] Fairbairn[1], p41-21. From his portion of the *Report* to the Directors of the Chester and Holyhead Railroad, 9 February 1846.

It is apparent then, that Fairbairn considers the construction process an integral part of his conceptual design. He also refers to the fact that the masonry piers were already under construction and designed to accommodate suspension chains. This is an early example of a Design-Build project delivery method, much like that employed in the Menaï Suspension Bridge. The design of the Britannia Bridge seems to be much more organized.

In Hodgkinson's part of the *Report* to the directors of the railway, his involvement and his impressions to that point were reviewed. He made clear that "any conclusions deduced from received principles, with respect to the strength of thin tubes, could only be approximations; for these tubes usually give way by the top or compressed side becoming wrinkled, and unable to offer resistance, long before the parts subjected to tension are strained to the utmost they would bear."[173]

3.8.10 Theory vs. Practice

After the basic principles of the cellular flange were established, Fairbairn and Hodgkinson proceeded with two different kinds of tests. Fairbairn built and tested a 1:6 scale model of the cellular-flanged tube as then envisioned. Hodgkinson tested plates and simple tubes of varying sizes and plate thickness to determine their buckling characteristics under compression. From these a formula to calculate the strength of a full-sized tube was created. These were the first such tests on thin walled tubes and compressed plates ever performed.[174]

Hodgkinson's expected formulae did not appear in time to be of any service in proportioning the parts of the large tubes.[175]

By Feb. 1846, the Directors were putting enormous pressure on Stephenson to complete the bridge and it was essential that tube construction should begin. The problem was that they had been waiting six months for Hodgkinson to report back with results of his investigations and a means to calculate the proportions necessary for the bridge, but Hodgkinson was not having great success in this regard.[176] As Peters writes, "Hodgkinson's analytical approach to structural behavior was more common in France than in Britain at the time. The standard British method was trial and error. An example is Telford's preference in 1819 for suspending a full-sized chain to measure the sag in the chains he proposed for the Menaï Bridge rather than calculating it with a simple catenary equation."[177] In the interest of time and money, Fairbairn proceeded to dimension the girder by conventional English practice. He used what theory and knowledge that was available to him, but also relied in good measure on his own engineering judgment formed from many years of experience.[178]

[173] Fairbairn[1], p43. From Hodgkinson's portion of the *Report* to the Directors of the Chester and Holyhead Railroad, 9 February 1846.
[174] Timoshenko, p162.
[175] Fairbairn[1], p22.
[176] Fairbairn, p53 and 54.
[177] Peters[1], p162-163.
[178] Fairbairn[1], p66-67.

3.8.11 The Top Flange: Ideal vs. Implemented Form

By April 1846, Fairbairn had calculated the section shown in **Figure 58**. He determined this best resisted the buckling in the top flange; and made most economical use of the material in terms of its strength in both tension and compression. Fairbairn was certain that any formulae that Hodgkinson might come up with would confirm his own results.[179]

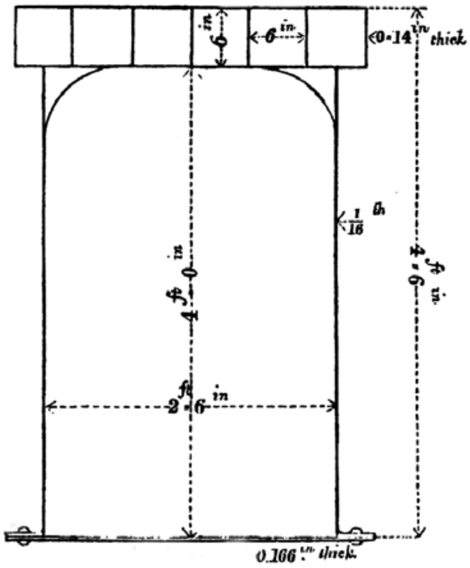

Fig. 58: Section as of April 1846. (Fairbairn[1])

However, Hodgkinson insisted that a circular form should be used because his tests showed that the stiffest cell form was that of the circular cells.[180] (**Fig. 59**) In a letter to Stephenson, dated October 27, 1846, Fairbairn wrote,

> The circular cells are no doubt preferable to the squares, but I doubt the propriety of their application. The rectangles or squares may not be so strong, but they are much easier of execution; and, practically speaking, they are infinitely preferable. Besides, the rectangular cells, as constructed in the model tube, are of the first order: as yet, we have never been able to crush them; and the great convenience of flat straight plates with easy access to any part, is a consideration we must on no account lose sight of.[181]

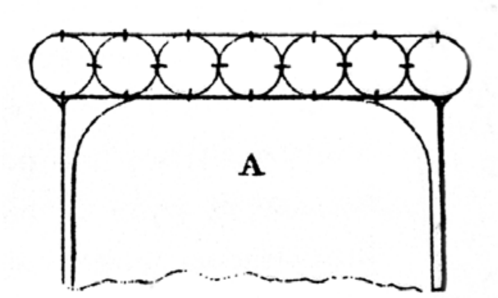

Fig. 59: Detail of circular cell structure proposed by Hodgkinson, October - December 1846. (Fairbairn[1])

Here Fairbairn is making a point about the relative merits of ideal versus implemented form. Writing back to Fairbairn in a letter dated December 19, 1846, Stephenson states that the more he thinks "of the circular cells, the more the practical difficulties increase. If you can see your way to riveting the rectangular cells effectually, I have no question about their being the most eligible, notwithstanding their comparative weakness."[182] Therefore, both Fairbairn and

[179] Fairbairn[1], p71-72.
[180] Fairbairn[1], p12.
[181] Fairbairn[1], p116.
[182] Fairbairn[1], p122.

Stephenson, with great practical experience between them, recognize that the best form is not always the structurally ideal form. Both see inherent difficulties in the actual fabrication and maintenance of the circular forms and are satisfied enough with the performance of the rectangular cells that they are willing to compromise some amount of strength for the benefits of simplified construction and maintenance that in turn translate into economy of time and money.

Hodgkinson was finally convinced of the merits of the rectangular cells, not because of the aforementioned constructive reasons, but because subsequent testing showed there were "new elements of resistance in the angle-iron which connects the square cells." Fairbairn explained to Stephenson that Hodgkinson had found that the "angle-iron in each corner gave great resistance to a crushing force, and in may respects rendered it an approximate, if not nearly equal to the cylindrical forms." [183]

Further exploration by Hodgkinson produced two other proposals for the top flanges, both of which he made too late in the design process to be considered. The first proposal was to use cast iron, which Fairbairn strongly opposed because of safety issues that had arisen in composite wrought-iron and cast-iron structures.[184] This proposal indicates some ignorance on Hodgkinson's part about contemporary practice and the design of the trussed cast-iron girders. Additionally, it ignores the fact that when the beams are made continuous the negative moment over the piers would introduce tensile stress into cast iron, but I do not know if they understood the behavior of continuous beams yet. W. Pole's research on continuous tubes was only incorporated late in the process when the tubes were connected.

In further tests, Hodgkinson found that a single, thick plate could replace the cellular top flange. Hodgkinson had discovered a thickness-cubed increase in the buckling strength of plates with increasing thickness. However, the plate would have had to have been too thick to be practically fabricated at that time.[185] It was simply easier to construct cellular flanges. This new knowledge is significant, however, to the future development of the plate girder. It is interesting to note that the initial fears about the stability of wrought-iron plates in compression led to the experiments of the cells which in turn led back to the conclusion that single plates are ok as long as they are thick enough, which the cubed law makes easier to realize than if the relationship were linear.[186]

3.8.12 The Bottom Flange and Constructive Effects on Form

By May 1846, Fairbairn in collaboration with Clark, had concluded that the tube should have a cellular bottom flange as well, as seen in **Figure 60**.[187] As Rosenberg and Vincenti explain,

> The importance of practical requirements in the final design is apparent in the cellular construction of the bottom of the tube. If the bottom had been made in one layer as in Fairbairn's model, this layer would have had to be three inches [76 mm] thick to provide the required cross-sectional area. Since single plates of this thickness could not then be produced, such a layer would have had to be

[183] Fairbairn[1], p131-132.
[184] Fairbairn[1], p138-139.
[185] See next sub-section on design of the bottom flanges.
[186] Rosenberg and Vincenti, p34.
[187] Fairbairn[1], p78-79.

> built up by riveting several thinner plates together. Because of uncertainty about the action of the long rivets that would be required, this procedure was not practical, and the bottom as well as the top was made cellular. An additional reason for the cellular bottom in the Britannia Bridge was that in this bridge (though not in the single span Conway tubes) the four spans in each line were to be joined solidly together over the towers to form a continuous beam on multiple supports. This continuity provided additional strength but meant that the beam would bend concave downward near the towers, putting the bottom rather than the top into compression.[188]

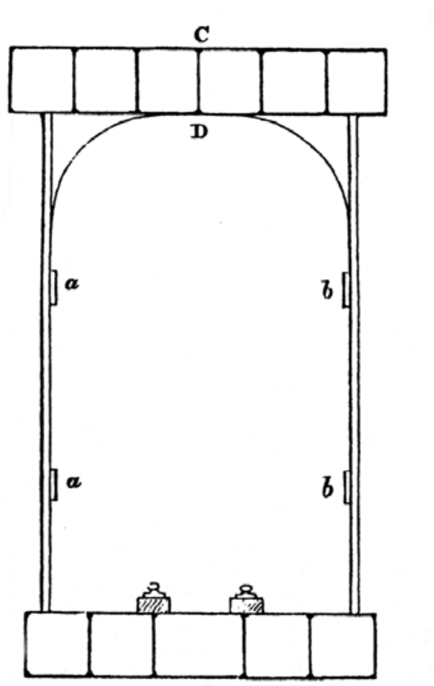

Fig. 60: Section as of May 1846. (Fairbairn[1])

3.8.13 The Local Web Buckling and Stiffeners

One important aspect Hodgkinson did contribute to was with respect to the design of the webs. He advised that care be taken in the construction of the webs of the tubes because these would be critical to the stiffness of the tube. He warned, "If this is not attended to, the tube will give way by bending *laterally*, a tendency very commonly observed in experiments on wrought iron.[189] Fairbairn studied these effects. (**Fig. 61**) The tests until then had treated the flanges as quasi-independent of the web. Rosenberg and Vincenti describe the problem of this new failure mode,

> The web is essential to the structural functioning of the beam. In particular, the magnitude of the tension and compression in the flanges varies continuously form one position to another along the beam. The web performs the essential function of balancing the variation in compression along the top flange against the variation in tension along the bottom. As a result, the flanges in return impose a so-called shearing load on the web, with the top of

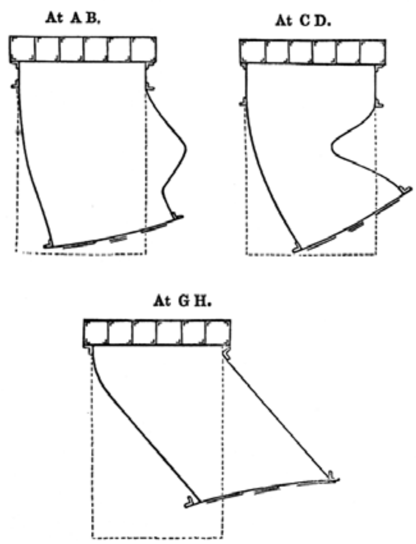

Fig. 61: Web buckling tests. (Peters[1])

[188] Rosenberg and Vincenti, p31-32.
[189] Fairbairn[1], p56.

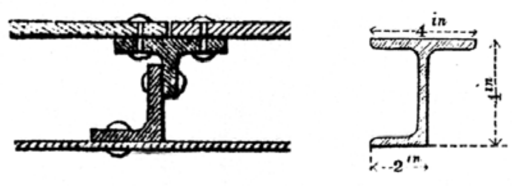

Fig. 62: (a) Initial design for stiffeners fabricated from built-up sections. (b) Fairbairn's proposal to have special section rolled to eliminated need for expense of built-up sections. May 1846. (Fairbairn[1])

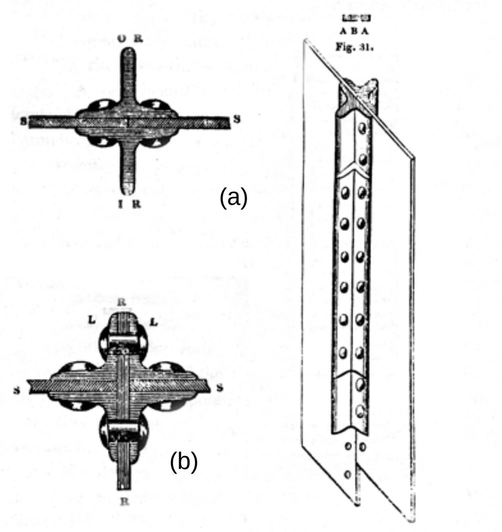

Fig. 63: Final web stiffener design. (a) typical detail, section and axonometric view. (b) detail of first and last 30 stiffeners on either end of each tube segment. 1847. (Dempsey)

the web being pushed toward one end of the beam and the bottom toward the other. If the web is too thin or is unrestrained laterally, such loading can cause it too to buckle, involving phenomena even more complicated than those encountered in the buckling of the top flange. The buckling problem is thus not in fact limited to the top. Although these matters are clear today, hardly anyone understood them at the time.[190]

An initial design for the stiffeners is shown in **Figure 62**. This example shows that Fairbairn always sought to simplify. In this case the initial design required a built-up section. Fairbairn proposed to Stephenson to see if it could be rolled directly, thus saving time and weight in the final construction.[191] The Britannia and Conway Tubes were of such scale that tooling costs to produce a special profile probably could have been justified but it was finally decided to simply use standard **T** and **L** sections. (**Fig. 63**)

D.J. Jourawski, a Russia theoretician, criticized the design of the Britannia Bridge. He had made important discoveries about the nature of shear stress, which he published in 1854.[192] His work was not known in Western Europe while Stephenson, Clark and Fairbairn were determining the form of their stiffeners. Thus, no theoretical basis was available for analyzing the web buckling they observed in the model tests.[193]

Jourawski showed that web buckling is caused by internal compressive forces acting diagonally at 45 degrees to the horizontal and that stiffeners inclined in this direction would be considerably more effective even if made with smaller cross-sectional area.[194] Certainly it is unreasonable for Jourawski to be so

[190] Rosenberg and Vincenti, p27-28.
[191] Fairbairn[1], p78-79.
[192] Rosenberg and Vincenti, p59; Timoshenko, p141.
[193] Rosenberg and Vincenti, p27-28.
[194] Rosenberg and Vincenti, p33.

critical of a project that was completed *before* publication of his findings. Moreover, the designers of the Britannia and Conway tubes had to deal with many more unprecedented problems than just shear. Even if the designers of the Britannia Bridge better understood shear, it seems unlikely that they would have used the theoretically correct form because it would entail increased construction complexity, and, by extension, increased costs. Like the rectangular cells, vertical stiffeners were found to be sufficient for their purposes and they were willing to compromise structural idealism for constructive and economic pragmatism. Most large girders today are indeed fabricated with vertical rather than diagonal stiffeners for the same reason. As Rosenberg and Vincenti observed, “With this later theoretical knowledge Stephenson and Fairbairn could have built a better bridge, but without it they built a reasonable and sound one – a typical situation in the progress of engineering knowledge.”[195]

3.8.14 Final Form

In a letter to Stephenson dated October 9th, 1846, Fairbairn wrote,

> So far as regards myself, I am perfectly satisfied; and exclusive of all theoretical inquiry, I have no hesitation, from the experimental facts before us, to give an unqualified assurance that the work can be done at a moderate expense, and that successfully. If you concur with me in this opinion, I will set my shoulder to the wheel, complete the work, and then ask the opinion of the mathematicians after the whole is finished and at work. The present discussion on the subject in different journals is mere moonshine. *I will stick to the facts before the figures*; and although I am far from despising theoretical knowledge, I would nevertheless infinitely prefer solid facts, and my own judgment in reasoning from such data, for the construction and consummation of such work.[196]

This letter expresses confidence in the design that he, with Clark’s aid, had arrived at without significant theoretical input from Hodgkinson. Hodgkinson was not happy that design of the tube had preceded his results. Stephenson was compelled to remind him that “the Directors consider me as their engineer, responsible for the works of the railway being ready for the public by the time specified in their reports to the shareholders, and I have no choice [but to proceed with the design]; some risk must be incurred, or the pecuniary loss to the Company must be very serious.”[197]

Fairbairn’s scale model indicated the most advantageous distribution of material in the flanges and helped determine an effective arrangement of vertical stiffeners to protect the web against bucking, but it also gave further information about the best arrangements of plates and rivets.[198]

In Fairbairn’s final series of tests, he determined that the ratio of the cross-sectional areas of the top and bottom flanges was 1.2:1. He determined this in the same way Hodgkinson determined the ideal distribution of material in a cast-iron beam. Fairbairn kept the top flange a constant cross-sectional area, which he had oversized to ensure that the material would fail before buckling, and then he increased the thickness of the bottom flange until both flanges

[195] Rosenberg and Vincenti, p59.
[196] Fairbairn, p108-109.
[197] Fairbairn, p60.
[198] Rosenberg and Vincenti, p27-28.

failed nearly simultaneously. The remaining higher area in the compression flange resulted from the fact that wrought iron fails at a slightly lesser load in compression than in tension. Fairbairn had succeeded in increasing the ultimate load from 353 to 857 kN (79.4 to 192.7 kips), an increase of two and a half times, at the expense of only 20% more material.[199] In Clark's words, "The magnificent model…failed…after carrying a greater weight than even a double line of locomotives throughout its entire length. Nothing could be more satisfactory than this result."[200]

When the design of the Conway tube was completed, Hodgkinson was asked to calculate the maximum load the bridge would carry and the deflection that this would produce. Timoshenko writes,

> Although the problem was a simple one, requiring merely a calculation of the maximum stress and maximum deflection of a simply supported and uniformly loaded beam of constant cross section, this kind of analysis was beyond the power of Fairbairn and Clark. It required assistance from the "mathematician." (Hodgkinson) This analysis was considered so important that, with all its details, it was included in Clark's book and in the Report of the Commissioners… It was reproduced in Weale's "Theory, Practice, and Architecture of Bridges." Hodgkinson found that, with the dimensions taken in the design and allowing a maximum compressive stress of 8 tons per in.^{2} [12.4 MPa], the load concentrated at the middle could be as much as W = 1,130.09 tons [1148.22 metric tons] when the corresponding deflection would be δ = 10.33 in [262.4 mm]. For a uniformly distributed load, *W* could be doubled and the deflection multiplied by 5/4. The modulus of elasticity in this calculation was taken as 24x10^6 lb per in^2 [16.5x10^4 MPa]. After the completion of the bridge, the deflections produced by a load applied at the center were made and a deflection of 0.01104 in. per ton [0.2760 mm/metric ton] was found. This is about 20 percent higher than that which the calculations predicted.[201]

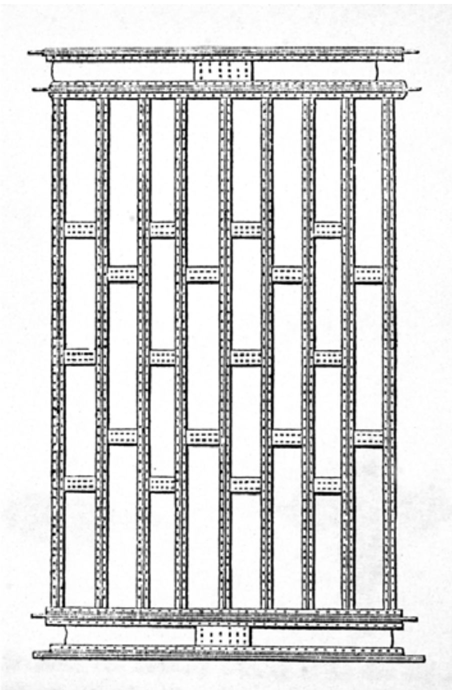

Fig. 64: Final web construction detail. (Dempsey)

The final form of the cross section of the Britannia Bridge is shown in **Figure 1**. The cross section of the tube is rectangular throughout. The webs are vertical and parallel but their height varies from end to end. The external height is 9.1 m (30 ft) at the center over the Britannia tower and 6.9 m (22 ft 9 in) at the abutments, the bottom line being horizontal, but the top line forming a parabolic curve.[202]

[199] Rosenberg and Vincenti, p27.
[200] Clark, p184.
[201] Timoshenko, p160.
[202] Dempsey, p102.

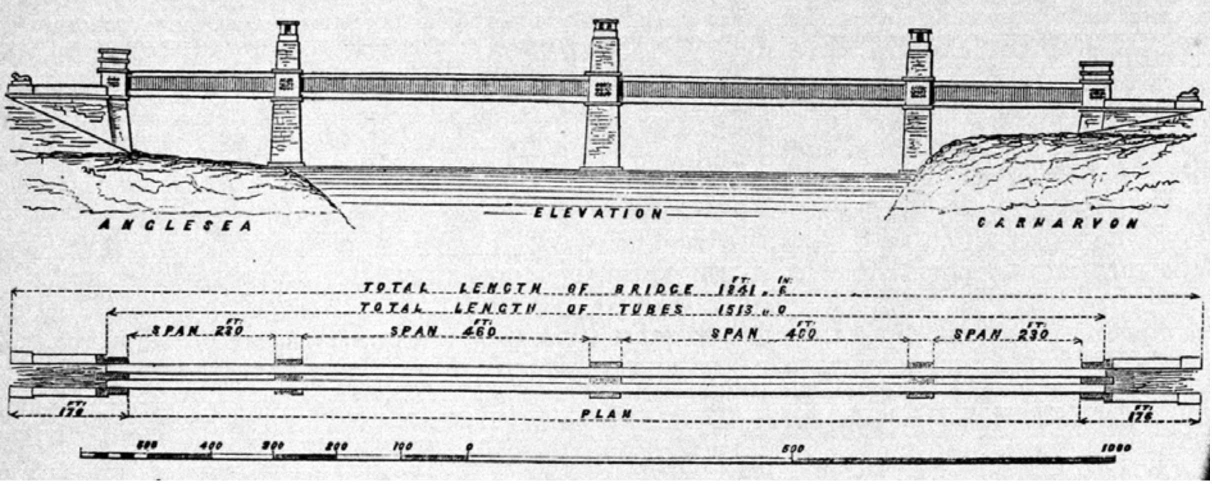

Fig. 65: Plan and elevation of final design. Each tube totals 461.2 m (1513 ft) long. (Dempsey)

Plate thickness varies along the tube's length. Plates forming the sides are reduced in thickness from the ends towards the middle of the tube, and those forming the top and bottom are increased in the same direction. The top and bottom flanges are both made of cellular construction. The top flange is comprised of eight cells and the bottom flange of six cells.[203] The relative proportion between top and bottom flanges was determined to be 24 to 22.[204] Webs are constructed as shown in **Figure 64**. Note that the horizontal joints are not continuous to guard against a progressive failure of the joints in that plane. The sides are stiffened by **T**- and **L**-bar ribs arranged vertically. See **Figure 63** for various constructive details of the stiffener / web construction.[205]

Each main tube weighs 1600 long tons. Of these 1600 tons of wrought iron, 500 are disposed in the bottom, 500 in the top, and 600 in the sides. The four separate tubes were made continuous during construction so that instead of eight separate tubes there are two parallel tubes measuring 461 m (1513 ft) long each.[206] (**Fig. 65**) This will be discussed in greater detail below.

3.9 Construction

3.9.1 Construction Process Was Integral Part of Design

From the earliest date that Stephenson announced his intention to design tubes of unprecedented scale using wrought iron, he had thought about the construction and erection of the tubes. When Stephenson made his announcement in February 1845 to use wrought iron and develop a tubular structure, he made a point of explaining that he would obviate the need for scaffolding by manufacturing the tubes on a staging area and then positioning them into place *en masse*.[207]

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[203] Dempsey, p102-103.
[204] Fairbairn[1], p146.
[205] Dempsey, p104-107.
[206] Dempsey, p101 and 111.
[207] Dempsey, p61.

Fairbairn, also, considered construction process an essential part of his conceptual thinking. He wrote, “From the very commencement of the inquiry, the means which should be employed in erecting and raising such a large structure over a broad and rapid stream, was the subject of deep reflection; and the point seemed indeed surrounded with impediments not easily overcome.”[208]

Fairbairn’s first proposal for a tube “suspended” from its top flange included a plan to use the balanced-cantilever method to erect the tubes. (**Fig. 50)** In such a plan, the tubes must be designed to act as cantilevers while under construction and as a suspended structure when connected. Of course, Fairbairn was not able to actually make the tube act in tension. If this scheme had been pursued then the tubes would have acted as a beam, which would have required the flanges to be designed to equally resist compression and tension to accommodate the inversion of the stress distribution between the top and bottom flanges when the cantilevers were finally connected and they began functioning as a single beam.[209] Even when Fairbairn’s initial bridge form was abandoned the idea of building on the cantilever principle remained an option. By April 6, 1846, the erection process still had not been settled and Fairbairn asked Stephenson if he had determined how it would be possible to build the tube using the balanced cantilever method.[210] On further reflection, Fairbairn expressed pessimism that such an erection technique could be accomplished. He stated that “it appeared next to impossible to maintain the balance of so great a mass upon the pier as a fulcrum, and so to keep both ends in an exact line (as regards their horizontal and lateral position), as to cause them to meet in the middle. The plan was therefore abandoned for another of a more tangible kind.”[211]

Another possibility was to use suspension chains to support falsework upon which tubes could be fabricated *in situ*. These so-called auxiliary chains were a subject of much discussion among Stephenson, Fairbairn, Hodgkinson, and Charles William Pasley, the Inspector General of Railways. Even when Fairbairn had more or less convinced Stephenson that the chains were not necessary, he still considered using them for construction because the alternative required floating the tubes into the strait to be put in position. Stephenson was understandably apprehensive about loosing a costly tube to the bottom of the strait. To build the tubes on the suspended falsework Stephenson planned to use ballast to achieve a uniform distribution of load on the chains during construction. The ballast would be removed as the length of the tube advanced. Hence, there would be minimal differences in the deflection of the chains that would interfere with the precise construction of the tubes. One can imagine the logistical difficulties of moving material onto the suspended falsework and constructing the tubes in such a confined work area. Additionally, the threat of instability due to wind during construction had to worry Stephenson just as much as loosing a tube into the strait. To make matters more complicated for Stephenson, Palsey had insisted that the chains be put in place as an auxiliary support for the tubes. Palsey was not convinced that this novel new structure would function as intended.[212]

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[208] Fairbairn[1], p7.
[209] Fairbairn[1], p5-7.
[210] Fairbairn[1], p71.
[211] Fairbairn[1], p8.
[212] Sutherland[2], p79.

When tests on Fairbairn's model tube proved so successful, two of which Stephenson was present for, Stephenson felt more assured that the chains would be unnecessary. Fairbairn finally convinced Stephenson and the Directors to dispense with the chain all together, which saved an enormous amount of money on their manufacture.[213] In his arguments against using the chains, Fairbairn said, "I would suggest that you give the subject your serious consideration; as, in case we can accomplish this, a saving of time and one-half of the cost of the bridge may be obtained."[214] Peters cites Fairbairn's comment concerning technology and saving time as the earliest explicit statement he has yet discovered that consciously makes technological or organizational method responsible for saving construction time.[215]

There appears no record of a proposal to launch the tubes across the strait. It seems it would have been an appropriate solution.

The erection scheme finally proposed was to fabricate the landside tubes on traditional scaffolding and the center tubes on a plateau beside the strait. For this purpose, Fairbairn introduced mobile workshops that moved as the manufacture of the tubes progressed.[216] (**Figs. 66 and 67**) This way the tube did not have to be constantly moved for a stationary workshop. The plan was to float the beams to the piers from where they would be lifted by an hydraulic device. In an early scheme, Fairbairn proposed to float the tube using their own buoyancy by sealing the ends. It was decided that water-tightness could not be assured and that the potential damage to the tubes by salt-water infiltration of the joints was unacceptable. Therefore, barges were designed to float the tubes into place. (**Fig. 68**)

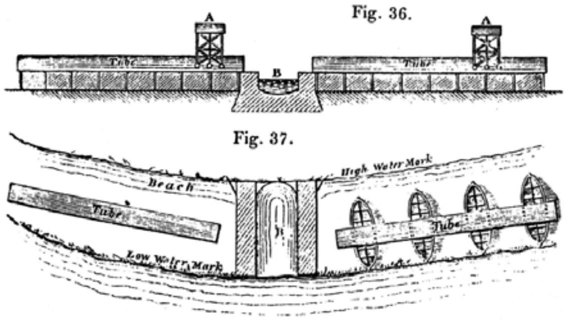

Fig. 66: Schematic design for mobile workshops that moved as length of constructed tube increased, Fairbairn, August 1846. The tubes were to be transferred onto the boats and floated into position with the tide. (Fairbairn[1])

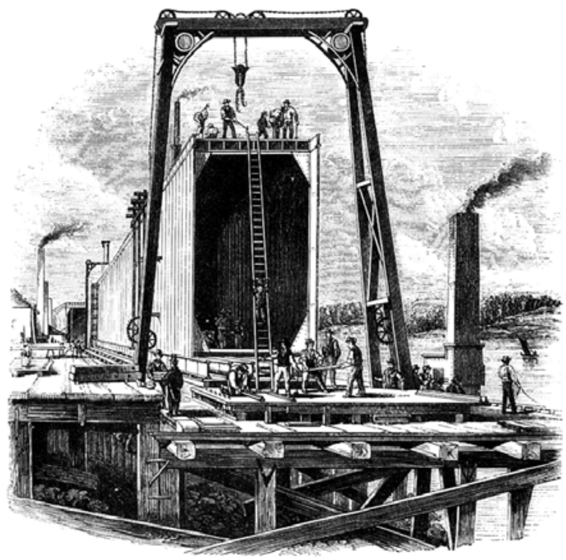
Fig. 67: Britannia tubes under construction on assembly platform with mobile gantry cranes. (Peters[1])

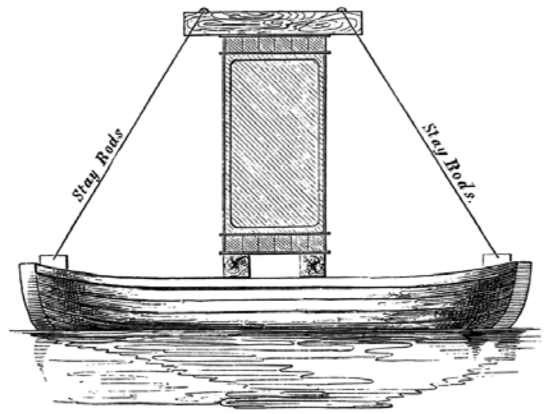

Fig. 68: Conception of barge to float the tube into place at the base of the towers. (Fairbairn[1])

[213] Dempsey, p99-101.
[214] Fairbairn[1], p90-91.
[215] Peters[1], p171.
[216] Fairbairn[1], p98.

Fig. 69: Riveted plate detail, Fairbairn. (Fairbairn[1])

3.9.2 Fabrication of the Tubes – Riveting

Riveting was the most critical task in ensuring that the tubes would function properly. **Figure 64** shows how the tubes were fabricated with thousands of relatively small plates. All of these plates had to be joined to ensure that the tube acted as a monolithic beam. Fairbairn relates that Stephenson was anxious about the quality of the riveting on the construction of the Conway tubes "as everything depended upon their security and soundness."[217]

Fairbairn, who had pioneered many aspects of riveting technology in shipbuilding and boiler fabrication, followed boiler-making practice at the outset. However, he quickly found that practices for the joining and riveting of plates that were satisfactory for boiler making were not adequate for structural purposes. He therefore immediately adopted improved methods.[218]

Fairbairn developed connection details that addressed failure in the riveted joints he had observed in his early tube experiments. For the bottom plates, he calculated that if the plates were to be 1" thick, they should "be composed of two, ½ inch plates with transverse joints resting upon the solid plate on the one hand, and suspended from it on the other, as the case may be. This will give great uniformity of strength, when treble riveted, with a covering plate of the same thickness, well attached with tail-rivets to prevent curling at the ends, and perforating with as few holes as possible the solid plates, the same as" shown in **Figure 69**.[219]

Dempsey describes the riveting process as follows,

> The rivets are a full inch in diameter, and arranged in rows. The spaces between the centres of the rivets are 3 inches [76 mm] in the vertical joints, and 4 inches [102 mm] in the horizontal joints. The rivets are heated in portable furnaces, which are moved form place to place as the work proceeds; from these furnaces they are taken up with tongs and placed in the holes punched for them, and the ends firmly clenched or riveted, before cooling, with heavy hammers. The rivet-head thus formed is then finished by hammering a steel cup-shaped tool upon it, and the contraction of the length of the rivet in cooling draws the plates closely together with a considerable force. The number of rivets is said to be 327,000 in each of the main tubes, and about 2,000,000 in the entire bridge. A very beautiful machine, which is nearly self-acting, the precise distances and intended positions for the holes are very truly observed, and the design displays a more skilful arrangement of parts, being in this respect similar to many other contrivances which have emanated from the same clever machinist.

[217] Fairbairn[1], p147.
[218] Rosenberg and Vincenti, p14 and 16.
[219] Fairbairn[1], p68.

> It is almost needless to observe that in the preparation of the plates, and execution of the whole work, judicious means and practical contrivance shave been adopted in facilitating the construction, and rendering it uniform and correct in all its details. Thus, templates were prepared for the plates and ribs, &c., and all the holes truly marked with unerring precision and certainty, so that all the parts, when presented to their intended positions, should be found to 'fit.' Large portions of the plating for the tubes were thus put together partially on the platforms, and being raised to their places with the stages and tackling described, were speedily fixed in their true positions, and required the straightforward work of riveting only to complete their connections.[220]

This erection technique is quite elaborate, involving an assembly line that uses prefabricated, modular construction. In this way, multiple segments could be under construction at the same time, and if managed correctly, segments could be ready for "just-in-time" delivery on the main line. Such a situation would allow a faster production schedule while minimizing the need to stockpile enormous quantities of material on a job site subject to tidal flows.

The first rivet for the tubes was put in on August 10, 1847.[221]

3.9.3 Placing the Tubes

The first tube to be placed was the one for the Conway Bridge. The construction of the Conway tubes began in April 1847, slightly preceding that of the Britannia tubes. The first train passed through in April 1848.[222] L.T.C. Rolt described the building of the Conway Bridge as "the scene of a dress rehearsal for the larger and more difficult undertaking at the Menaï."[223]

As described above, the method of placing the tubes was not determined until quite late in the design process. Ultimately it was decided to float the tubes in place. For this purpose special barges were fabricated. Temporary lateral bracing was installed inside the tube to ensure that it would not deform due to twisting.[224] The Conway tube was placed with little problem in the relative calm of the Conway River, though a mishap while lifting the tube led Stephenson to introduce new security measures for the Britannia tubes.

The Menaï Strait presented more significant challenges because of the swiftness of the flow and tide in the strait. Stephenson had a scale model of the Menaï Strait built to test the floating operation. As Peters describes, "The engineers had to coordinate their movements with the tide and the currents, and the site personnel practiced again and again to develop strategies that would cover all eventualities… The caution with which the team had practiced the floating method paid off in the tidal strait. They used their carefully studied emergency procedures twice before the tubes were all put in place."[225] (**Fig. 70**)

Once the tubes were in place at the base of the piers, bar-chains were attached at either end that were connected to a hydraulic lifting apparatus. (**Fig. 71**) The beam was elevated in six foot increments, with timber and masonry underpinning being built up beneath the tubes

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[220] Dempsey, p108-109.
[221] Dempsey, p131.
[222] Rosenberg and Vincenti, p82-83.
[223] Rolt, p307.
[224] Fairbairn[1], p98-101.
[225] Peters[1], p171-172.

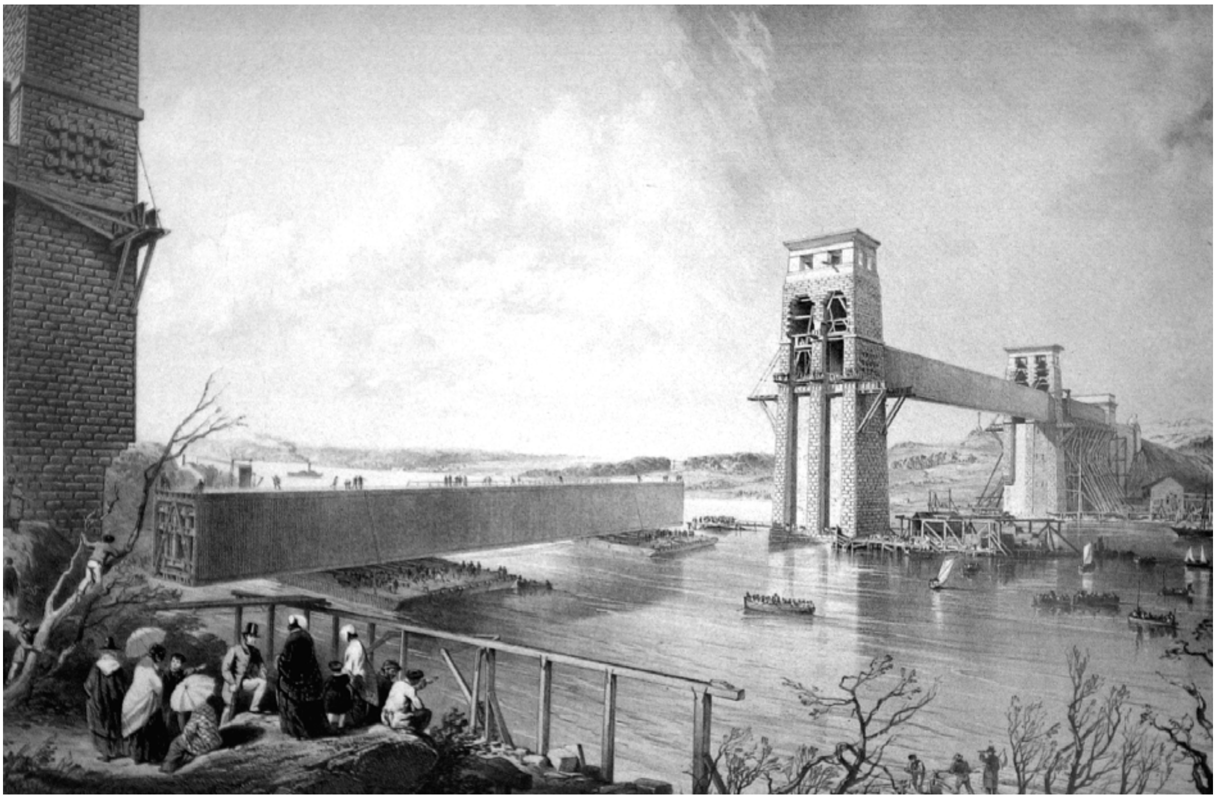


Fig. 70: A tube being moved into place on the Menaï. (Walker)

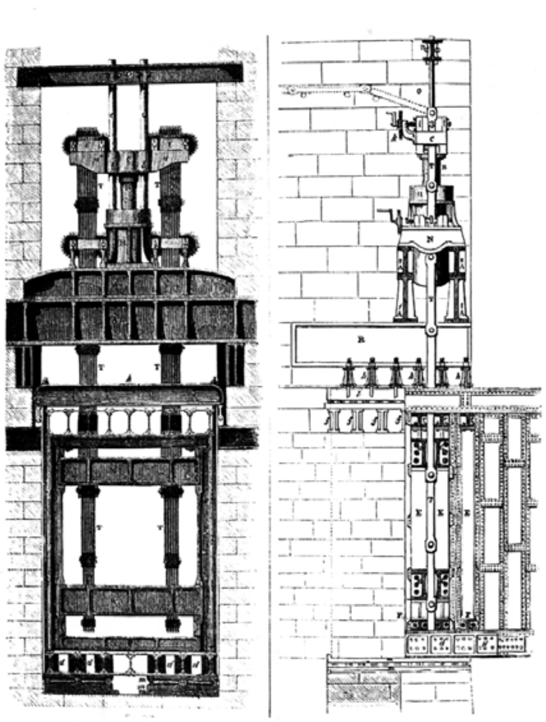


Fig. 71: Hydraulic lifting apparatus. (Peters[1])

during each lift. Stephenson required a maximum of two feet between the masonry and timber underpinning at all times in order to prevent the tube from being destroyed if one of the chains or the hydraulic powered lifting device failed.[226]

3.9.4 Making the Tubes Continuous

Once the individual tubes were placed in their final positions, all that remained was to secure them. Fairbairn had considered the possibility of making the beams continuous after he determined that it would be impossible to make the tubes act like a suspension structure. "The tubes," therefore, "were taken as a common girder or beam, and that without reference to the attainment of tension on the upper side caused by the deflection of the tubes over the piers."[227]

[226] Rosenberg and Vincenti, p40.
[227] Fairbairn[1], p109-110.

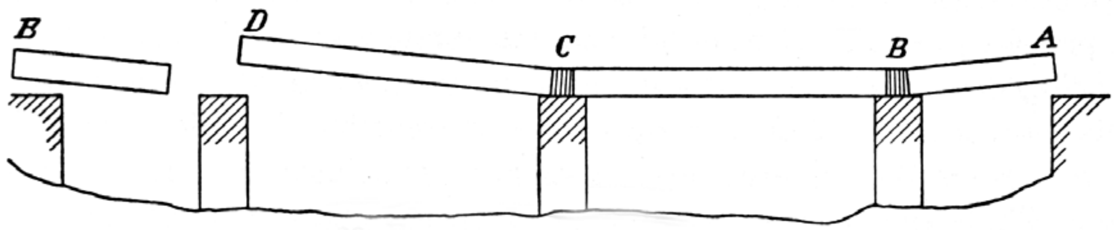


Fig. 72: Schematic of how tubes were placed (angles exaggerated) in order to prestress the bridge such precamber would be introduced. This lowers the stress at mid-span under service loads. (Timoshenko)

In their final calculations, Fairbairn, Clark and Hodgkinson assumed their beams to be simply supported. In Clark's book, he shows that the engineers clearly understood the effects on a uniformly loaded continuous beam over the supports.[228] Clark writes,

> We have to observe that the object in view was not to place the tubes in their place in precisely the same conditions as though they had been originally constructed in one length and then deposited on the towers, for in such a continuous beam (supposed of uniform section) the strain would have been much greater over the towers than at the center of each span, whereas the section is nearly the same; the object, therefore, was to equalise the strain.[229]

The negative moment was actually exploited to minimize the amount of material at mid-span of the two center spans, where the effect of dead load is most important. The tubes were connected using a special procedure. Stephen Timoshenko describes the process,

> The large tube *CB* was placed in position first. In making the joint *B*, the adjacent tube *AB* was tilted to some angle as shown, under which condition the riveted joint was made. Now, when the portion *AB* was put into the horizontal position, a bending moment at the support *B* was induced, the magnitude of which depended upon the amount of tilt. A similar procedure was adopted in assembling the joints *C* and *D*. In selecting the proper angles of tilt, the engineers were assisted by W. Pole, who investigated the problem of bending of continuous beams by using Navier's method.[230] (**Fig. 72**)

Because of this procedure, the center tubes were prestressed such that the stress at mid-span was reduced. The local reversal of strain over the piers was recognized explicitly in the design of the top cells where the proportioning of the plates and rivets over the piers was modified to withstand the tensile strain.[231] There is no corresponding indication, however, that the bottom cells were modified to resist compression. When explaining the design of the bottom flange, Clark said, "The bottom of the tube may be regarded merely as a chain of plates."[232] Rosenberg and Vincenti observe that "such failure to integrate ideas completely and consistently is not unusual in engineering – as in other endeavors – when so many of the ideas are new and the participants are working under such pressure."[233]

[228] Timoshenko, p160.
[229] Clark, p766, vol. 2.
[230] Timoshenko, p161.
[231] Clark, p557 and 559.
[232] Clark, p560.
[233] Rosenberg and Vincenti, p36-37.

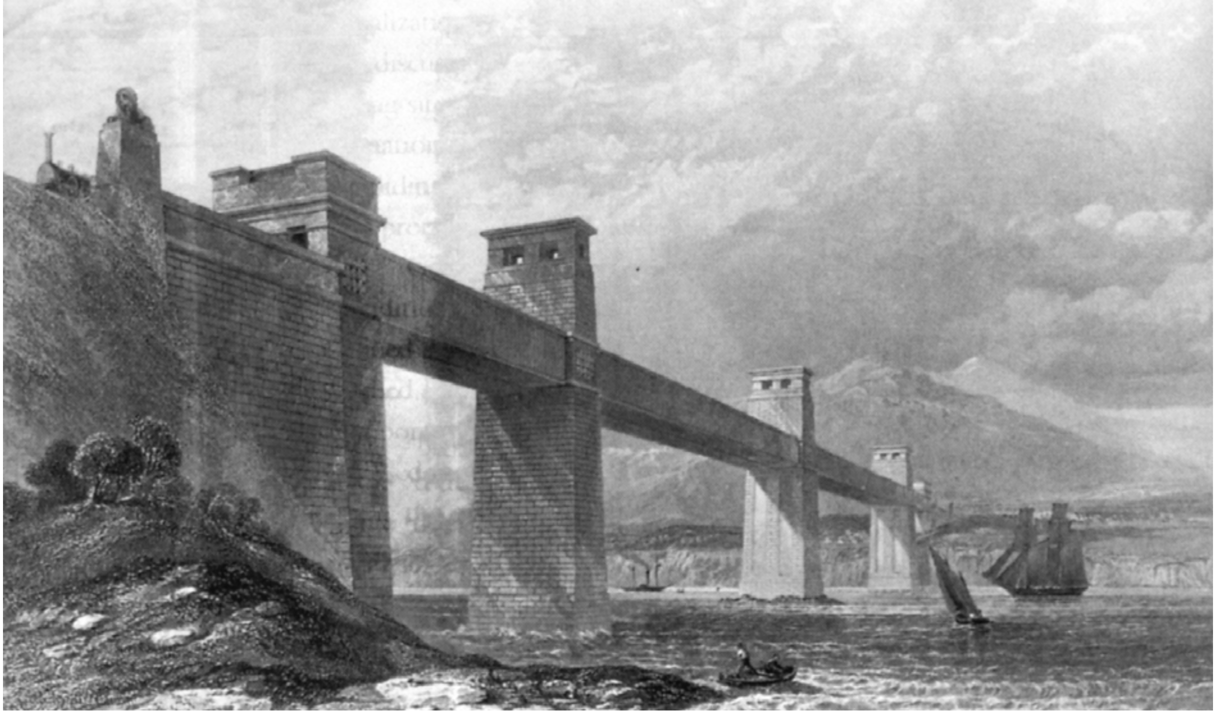


Fig. 73: Perspective of the completed Britannia Bridge, 1850. (Peters[1])

3.9.5 Thermal Expansion

One of the pioneering investigations done during the development of the Britannia Bridge was to determine the effects of temperature on the structure. Each tube was 461.2 m (1,513 ft) long. J. Frederic Daniell had performed experiments showing that a variation of 24.4°C (76°F) produces a change in a bar of wrought iron equal to 1/2000th of its length. His results were published in the *Philosophical Transactions* of 1831. By Daniell's method, I calculate that each tube would have about 229 mm (9 in) of movement do to thermal expansion. The actual measured valued was closer to 305 mm (12 in). The engineers mitigated this problem by only fixing each tube to the center pier on Britannia Rock. The other bearings were rollers that let the tubes contract and expand freely along their total length. Therefore, the tubes had a net movement of 153 mm (6 in) at each abutment.[234]

The first tube of the Britannia Bridge was opened to traffic in March 1850. The second tube opened in October 1850. (**Fig. 73**)

3.10 Influences

3.10.1 The Influence Goes Beyond Tubes

The value of the Britannia Bridge to future structural development goes far beyond the actual creation of a tubular beam. In fact, the tubular beam was only used on a few projects over twenty years following the construction of the Britannia tubes. The true value of the history surrounding Britannia Bridge is the knowledge it produced about structural behavior and

[234] Dempsey, p101-102.

theory, and developments made in construction methodology. This section will review some of the principle influences of the Britannia Bridge on future developments in structural engineering and construction.

3.10.2 Strength of Materials

Before Fairbairn and Hodgkinson performed their tests, wrought iron was principally used for tensile members. When it was used as a flexural member, such as in Fairbairn's deck beams and Stephenson's Ware Bridge, the structure was designed with proportions that were typical of cast-iron construction. Because of Fairbairn and Hodgkinson's tests the compressive and tensile strengths of both cast iron and wrought iron, as well as wrought iron's behavior in bending, were better understood.

3.10.3 Buckling and the Behavior of Thin-Walled Structures

Fairbairn's discovery of flange local buckling in beams, and the subsequent discovery of web local buckling was entirely new. This discovery led to the fundamentally new realization of the difference between *material failure* and *structural failure*. Hodgkinson's experiments laid the foundation for theoretical understanding of this behavior that resulted in methods to calculate the resistance of a beam subject to such failure modes. Timoshenko states that Hodgkinson's experiments are the first to address the structural characteristics of thin-walled tubes. This knowledge led to developments in the construction of airplanes and submarines[235], which in turn produced their own knowledge that was passed back to civil engineering.

3.10.4 Riveting Technology

The Britannia tubes had to be built up with plates riveted together on a scale never before attempted. The success and failure of the bridge was dependent upon the quality of the riveting to ensure that all the individual plates would act monolithically as one beam. Fairbairn conducted many tests that built upon previous experimental research and development in riveting technology. For the Britannia Bridge, Fairbairn conducted shear tests of the individual rivets as well as determined the strength of various types of riveted joints. His work also resulted in better understanding of the beneficial clamping stress introduced when a rivet cools, which was evidently unknown until then.[236] Todhunter and Pearson claim these experiments to be among the earliest such experiments.[237] Fairbairn's riveting machine was only used to fabricated the Conway tubes. The contractor considered the machine too cumbersome so he exclusively built the Britannia tubes using mauls to form the rivets. One improvement made was to increase the weight of the maul.

3.10.5 Knowledge Transfer

Because, as Peters puts it, Stephenson, Clark and Fairbairn "washed their dirty linen in public,"[238] we have an unparalleled record of the process of developing a historically novel artifact. All the knowledge gained during that process was therefore passed onto the

[235] Rosenberg and Vincenti, p84, note 20.
[236] Rosenberg and Vincenti, p30.
[237] Todhunter and Pearson, p791.
[238] Peters[1], p165.

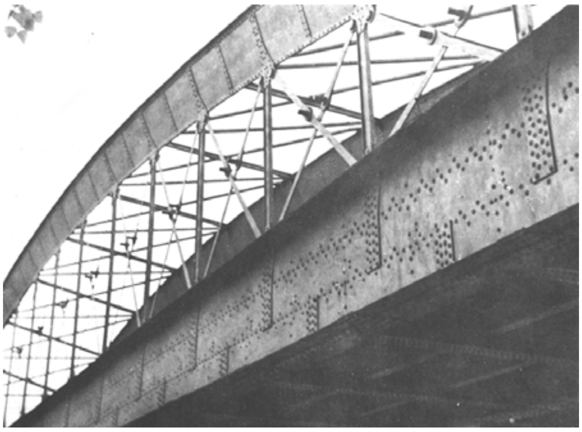


Fig. 74: Windsor Bridge, tied-arch, compression tube with triangular section, I.K. Brunel, 1849. (Sealy)

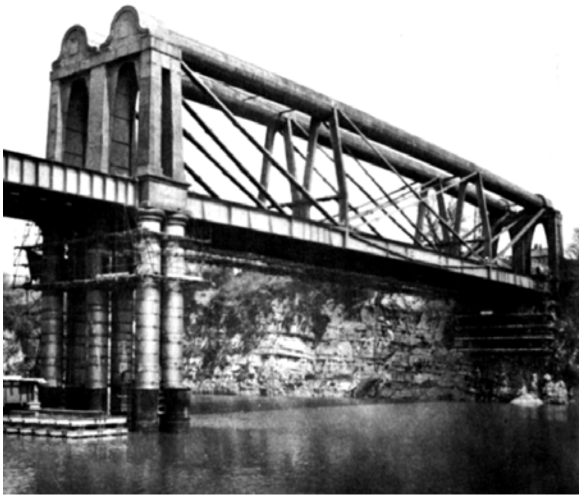


Fig. 75: Chepstow Bridge, suspension system stiffened and strutted by compression tube with circular section, I.K. Brunel, 1852. (Sealy)

"general stream of knowledge of structural engineering."[239] Rosenberg and Vincenti write, "Outside Britain the construction of the tubular bridges attracted great interest, and the results of the experiments were widely used and quoted in the engineering literature. Karl Culmann[240] in particular was impressed by the newly completed bridges and discussed them in his writings, which 'greatly influenced the growth of theory of structures and of bridge engineering in Germany.' The contributions from the bridge experience were also 'frequently referred to [in France] by Saint-Venant in his edition of Navier's *Leçons*.'"[241]

3.10.6 Influence on Bridge and Building Construction

The tubes of Britannia Bridge are antecedents to the modern plate girder, a development largely the responsibility of William Fairbairn. But it is unlikely that plate girders would not have developed anyway out of the concurrent development of the I-beam. Hybrid tubular systems were adopted by I.K. Brunel for several bridges[242] in which he used the tube as a compression member. (**Figs. 74, 75 and 76** Knowledge created by Fairbairn and Hodgkinson about thin-walled tubes certainly aided J. Fowler and B. Baker build the Firth of Forth cantilever bridge in 1890. This bridge had tubular compression struts 3658 mm (12 ft) in diameter. (**Fig. 77**)

Rosenberg and Vincenti write,

> Through Fairbairn, whose extensive technical writings exhibit a marked influence from the bridge project, the experience from that project had a broad effect in building design. Giedion credits Fairbairn with a major role in "the decisive change in methods of industrial

239 Rosenberg and Vincenti, p58-59.
240 Culmann's first name is variously spelled Karl or Carl.
241 Rosenberg and Vincenti, p58. Also: Todhunter and Pearson, *History of Elasticity*, p795; C. Tomlinson, ed. (London, 1869), 1:239-52.
242 The Windsor, Chepstow and Saltash (Prince Albert) Bridges.

construction that is to be observed shortly before the middle of the century," including the introduction of wrought iron as a building material.[243] According to Carl Condit, Fairbairn's *Application of Cast and Wrought Iron to Building Purposes*, which drew heavily on the bridge experience, "quickly became the gospel in the British construction industry."[244]

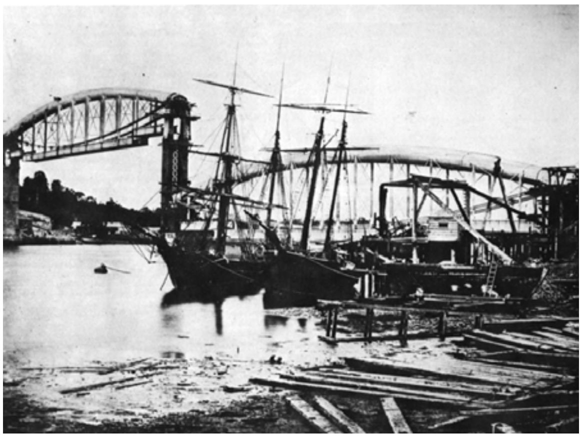


Fig. 76: Saltash Bridge, lenticular truss, compression tube with elliptical section, I.K. Brunel, 1859. (Sealy)

3.10.7 Technology Transfer

The development of the tubular design had far reaching implications in fields other than bridge construction. Fairbairn improved the common crane by using a curved tubular arm that made it possible to raise bulky articles, which was impossible with conventional cranes used in the 1840s that employed straight-line jibs that could be elevated only to an angle of 40 or 45 degrees. (**Fig. 78**) Within a few years, 60 ton capacity cranes of the "Fairbairn" type were being fabricated.[245]

An even greater contribution of the tubular concept was made in ship construction. Fairbairn introduced the tubular model for ship construction in his book on shipbuilding published in 1865.[246] Fairbairn simplifies the ship hull to be a beam and developed two worst-case load conditions. (**Figure 79)** Fairbairn also proposed to model the section of a ship as a large **I**-beam with an intermediary flange that represents an intermediary deck. (**Fig. 80**) I.K. Brunel modeled the design of the *Great Eastern* on the tubular beam concept he translated directly from the form of the Britannia Bridge tubes. (**Figs. 81**) The *Great Eastern* was the largest ship ever built at that time.

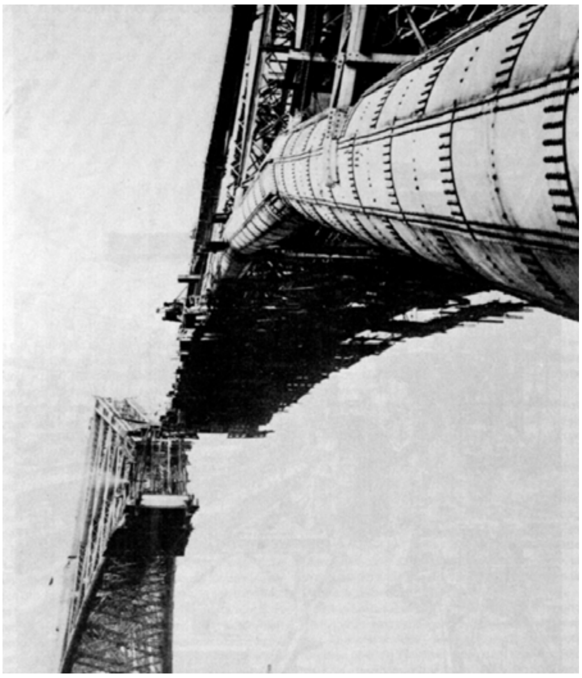


Fig. 77: Firth of Forth Bridge, long-span cantilever, 366 cm (12 ft) diameter struts with circular section, Fowler and Baker, 1890. (Fachtmann)

[243] Sigfried Giedion, *Space, Time and Architecture*, 5th ed. (Cambridge, Mass., 1973), p.194.
[244] Rosenberg and Vincenti, p49; Carl Condit, "Buildings," from *Technology in Western Civilization*. M. Kranzberg and C.W. Pursell, Jr., editors. (New York, 1967), p376.
[245] Rosenberg and Vincenti, p49.
[246] Fairbairn[4].

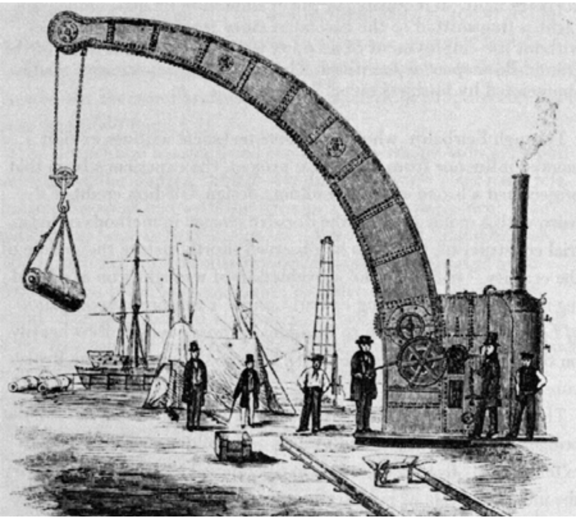
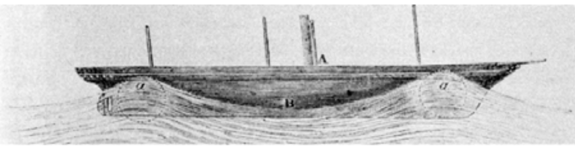


Fig. 78: Fairbairn type crane with tubular structure. (Rosenberg and Vincenti)



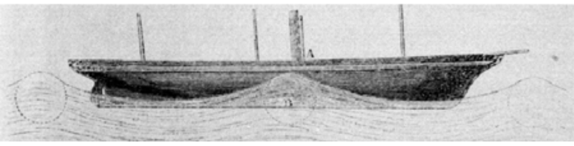


Fig. 79: Extreme load cases ship hulls are subjected to, as defined by Fairbairn. (Fairbairn[4])

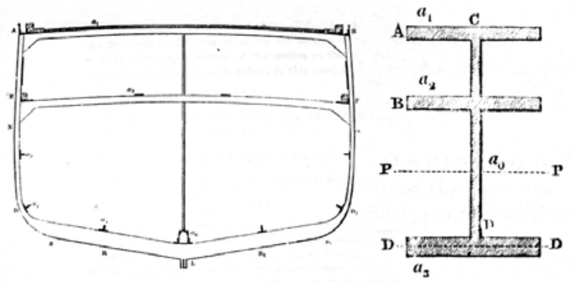


Fig. 80: Section of a wrought-iron ship hull with corresponding beam model for calculating area of material needed in each section. (Fairbairn[4])

Rosenberg and Vincenti described the reciprocal influence of interdisciplinary technology transfer. They wrote,

> The bridge-building experience… exhibits… interesting ways in which separate technologies interact and mutually influence one another. The relation of iron-ship building to bridge building demonstrates a recurring phenomenon of technological history: the reciprocal, constructive interaction of technological innovations in separate industries. Although the building of the Britannia Bridge generated a great deal of invaluable technological knowledge for shipbuilding, technological knowledge drawn from iron ships had also played a critical role, at several junctures, in the building of the bridge. Indeed, when Stephenson was brooding over the feasibility of a large, wrought-iron tubular bridge, his confidence in the project was decisively confirmed by information drawn from an accident in the launching of an iron steamship, the *Prince of Wales*. (**Fig. 82**) Although the hull of the ship was accidentally subjected to immense strains, it suffered only trivial damage. Stephenson stated: "The circumstances here brought to light were so confirmatory of the calculations I had made on the strength of tubular structures, that it greatly relieved my anxiety, and converted my confidence into a certainty that I had not undertaken an impracticable task."[247]

Today, at the CCLab we are trying to find material-adapted structural forms and building methods. Surely there is much knowledge we can similarly apply from the various transportation industries that have been using these materials for decades.

[247] Rosenberg and Vincenti, p70; Clark, p21.

3.10.8 Project Delivery Methods and Project Management

Stephenson's decision to build the piers before design of the tubes had been defined is an early example of what we would call today the Design Build project delivery method. This method requires good teamwork from different disciplines. In the case of Britannia Bridge it was between the chief engineer Robert Stephenson, the project manager Edwin Clark, the resident engineer William Evans, the consulting engineers William Fairbairn and Eaton Hodgkinson, and the contractors. This early example of such team-based organization was rudimentary and ended with some partners feeling disenfranchised or offended. Nonetheless, the case of the Britannia Bridge serves as an excellent example for illustrating the complexity of such an undertaking. It illustrates the enormous managerial responsibilities required to keep the team functioning well and the project progressing smoothly.

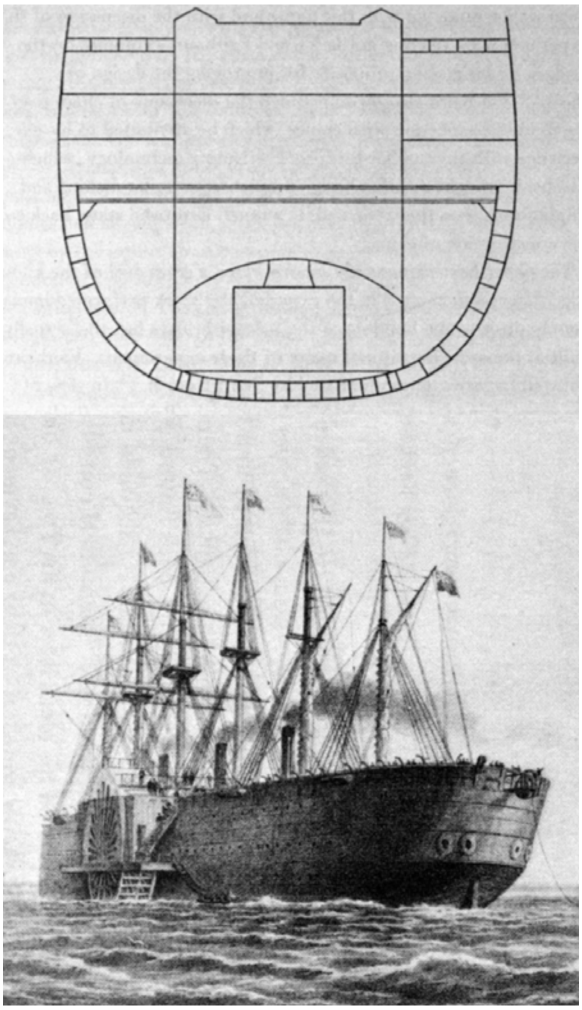


Fig. 81: Section and illustration of *The Great Eastern*, designed on tubular concept of Britannia Bridge, I.K. Brunel, 1858. (Rosenberg and Vincenti)

An interesting aspect of this building method was Stephenson's recognition that more effort put in one part of the work, the piers, would save him time later and give him the maximum number of options to address any problems that might be encountered with the undefined tubular design. Such thinking is an early example of critical path planning, which is common today but a wholly new development then. Peters explains this development as being the product of changes in the financing of constructed projects in England.[248] In the past, large projects were funded by a small number of subscribers who could typically afford delays and accepted them. The railroad boom changed the economics of everything, including construction. Instead of subscribers there were now shareholders. These shareholders demanded returns on their investments in a timely manner. It is for this reason that Stephenson could not wait for Hodgkinson to come up with his theoretical formulas and instructed Fairbairn to get the job done.

3.10.9 Process, A New Aesthetic Consideration

In Peters' book *Building the 19th Century*, he devotes a whole section to the definition of a new aesthetic by which to gauge constructed works, the aesthetic of process. This section is devoted to examining the often criticized pier height of the bridge. (**Fig. 73**) As we now

[248] Peters[1], p159.

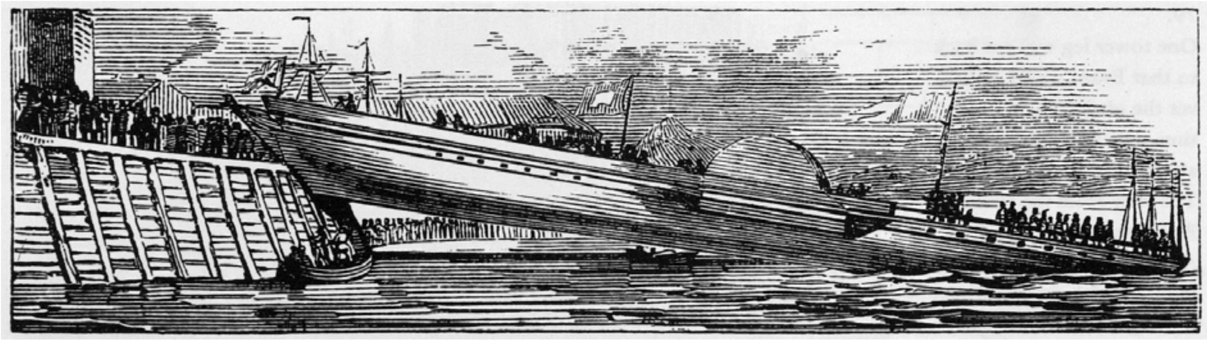


Fig. 82: Accident while launching the *Prince of Wales* at the Millwall Dock led the boat to be supported only at the ends, leaving the center free of any support. This event convinced Stephenson and the railway directors that the tubes would support themselves without auxiliary chains. (Peters[1])

know, these were intentionally built to accommodate chains in the event that the tubes, which had not yet been designed, needed to be supported by some auxiliary means. It was also considered to use chains to support a platform upon which the tube could be built *in situ*. Chains were never used and the towers thus stood in Fairbairn's words as "a useful monument of the enterprise and energy of the age in which it was constructed."[249]

Peters concludes,

> The often-repeated contemporary opinion that the Britannia Bridge was an aesthetic failure is quite unjustified from a procedural standpoint. We cannot judge engineering structures solely by the standards we apply to buildings as pristine and isolated objects. But engineering design and practice deal primarily with processes in their context rather than with products. If we accept the premise that engineering aesthetics should include issues that engineers consider important, we need an "aesthetics of process" to judge engineering works, and not a variant form of "aesthetics of product." The decision Stephenson made to raise the Conway and Britannia piers above the level of the beam supports must be seen it its procedural context and recognized for the elegant invention it is.[250]

3.10.10 Cast Iron to Wrought Iron

As Rosenberg and Vincenti conclude, "The most broadly influential aspect of the tubular-bridge experience, economically as well as technologically, was its role in the shift from cast iron to wrought iron as the material of choice for engineering structures."[251] The impact of the Britannia and Conway tubes dramatically shifted the English engineering community from one that was cautiously conservative and ignorant in their use of wrought iron to one that would not think of using anything else. This transition took about ten years, from 1840 to 1850; though the abandonment of cast iron took only a few years after the Britannia Bridge was built. Wrought iron remained the predominant structural material into the 20th century when technology finally made steel economically competitive with wrought iron; and reinforced concrete began to mature in its application such that it started to take market share from all-metallic structures.

[249] Fairbairn[1], p42.
[250] Peters[1], p178.
[251] Rosenberg and Vincenti, p60-61.

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ALUMINUM, PLYWOOD AND RIGID AIRSHIPS

A-04

4.1 Why Rigid Airships are Germane to this Project

The great era of the rigid airship began with the first flight of Count Ferdinand von Zeppelin's LZ-1, in 1900, and ended with the fiery destruction of the Hindenburg in 1937. LZ-1 and the Hindenburg were respectively 128 m and 245 m long and were constructed around aluminum airframes that structurally function like beams subject to three-dimensional dynamic loading. In contrast, the greatest single span length of the Britannia Bridge was only 140.2 m (460 ft) and did not have to fly. When the LZ-1 was launched, the aluminum industry was, for all practical purposes, only twelve years old and few significant structures had been realized using that material. The airships of Luftschiffbau Zeppelin broke new ground in the development of dirigible flight, aluminum structures, light spatial structures in general, thin-walled structural forms, and in numerous other technological areas.

In 1908, another German airship company was formed called Luftschiffbau Schütte-Lanz with the intention of competing against Zeppelin's monopoly in the rigid airship market. Schütte-Lanz chose to use plywood as its principle structural material. This decision was a bold one because there had been little exploration of plywood's structural potential due to the lack of sufficient water-resistant glues. Most texts about the history of plywood begin with the developments made in World War II, particularly in airplane construction. Outside of the airshipophile community, the airships produced by Schütte-Lanz during the First World War remain obscure and unknown. This thesis presents the first introduction of this development I have found in the general history of technology, including the general history of flight, with respect to the revolutionary use of structural plywood in the construction of these enormous flying structures.

The story of the rigid airship is germane to this project because it presents two examples of two distinctly different structural materials employed for identical applications. In the case of Zeppelin, his ships were unprecedented in scale and, except for one small-scale experiment, the rigid airship did not theretofore even exist. Both of the structural materials used by Zeppelin and Schütte-Lanz, aluminum and plywood, were brand new in large-scale structures. Therefore, there was little precedent for either of them to draw from in their fabrication and application.

The history of the Zeppelin and Schütte-Lanz airships provide a unique case study of the introduction of new structural materials in relation to their application and the forms in which they were employed. Additionally, both structural materials are the unique product of man. Aluminum does not exist naturally in its metallic state and plywood was specifically designed by man to overcome the natural deficiencies of unmodified wood. Zeppelin invented new structural forms globally and locally while Schütte-Lanz adapted those global forms to the different properties of plywood at a local level. The knowledge produced by these two companies transcended the field of airships and found application in airplane construction as well as in the development of structural forms and analytical tools for the general construction field. For all of these reasons, the rigid airship developments of the early 20th century are germane to this project.

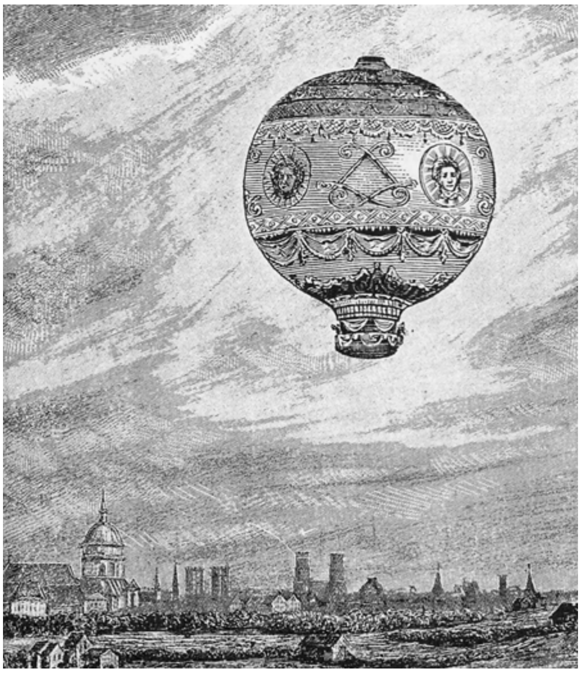

Fig. 1: Free balloons predating the era of the dirigible airship. (Knäusel)

4.2 Brief Introduction to Airship Technology

4.2.1 From Free Balloon to Dirigible

Airships are the descendants of the free balloon, which had been flown since the second half of the 18th century. (**Fig. 1**) Airships are the result of man's desire for dirigible flight, which is the ability to maneuver in the air against the whim of natural air currents.

The first successful dirigible airship was *La France*, built in 1884 by Charles Renard. (**Fig. 2**) *La France* was a non-rigid type airship, or blimp. A non-rigid airship is essentially an elongated free balloon that has motive power. *La France* was powered by a 9 hp electric motor.[1] Count Zeppelin cites the successful flight of *La France* as one motivation for his work in airships.

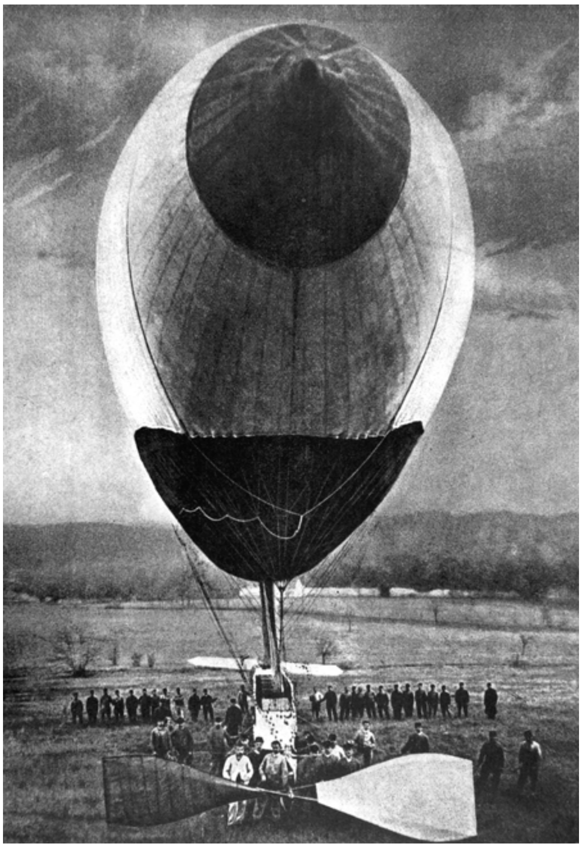

Fig. 2: *La France*, first dirigible airship. Non-rigid type. Charles Renard, 1885. (Knäusel)

Airships can be classified as being *non-rigid, semi-rigid,* and *rigid*. It is typical today to mistakenly call any airship a *zeppelin* or a *blimp*. Most airships today, typically used for advertisements and media broadcasts of sporting events, are non-rigids, or blimps, which are simply airbags from which a control car is suspended. The common usage of the term *zeppelin* to describe any airship is a testament

[1] Hartcup, p31.

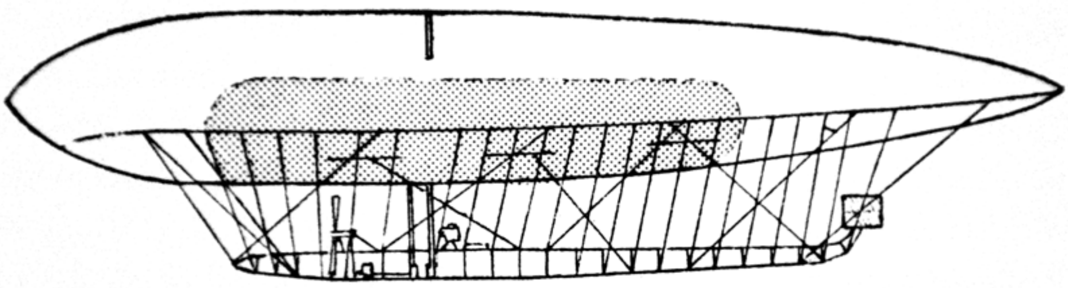

Fig. 3: A Clément-Bayard non-rigid airship with ballonet. (Hartcup)

to the success and influence Count Ferdinand von Zeppelin and Luftschiffbau Zeppelin had on airship technology, but it can be misleading when speaking technically of different airship types since historic Zeppelin's are characterized by an aerodynamic envelope stretched over a rigid metal airframe in which the airbags are located. Likewise, it is incorrect to call a rigid airship a blimp. This chapter only uses the term Z*eppelin* to describe the rigid airships produced by the airship manufacturer Luftschiffbau Zeppelin.

4.2.2 Non-Rigids

The principle developments in airship technology by the early 1890s were made with the non-rigid type airship. A non-rigid airship, such as *La France*, is simply an elongated gasbag from which the gondola, engine and control surface are suspended. The non-rigid is light and capable of absorbing shocks, which facilitates ground handling operations.

Non-rigid airships rely on the internal pressure of the lifting gas to maintain the shape of the envelope. The density of the lifting gas significantly changes depending on temperature (both ambient and local due to heating by the sun) and altitude. As an airship rises the air becomes less dense, expanding and increasing pressure. At a certain altitude, called the pressure height, it is necessary to relieve the pressure in the gasbags to avoid rupture. Releasing some lifting gas relieves the pressure. The pressure height limits the maximum altitude and range of an airship.

Releasing the gas is not only wasteful but can put the airship in jeopardy when the airship descends to a lower altitude where relatively more lift is needed. If there is too little gas the ship may descend at an unsafe rate. Another effect of loosing too much gas is that the internal pressure in the gas bag at lower altitudes may become too low to maintain the airship's aerodynamic shape and the envelope's resistance to shear and moment stresses is correspondingly reduced.

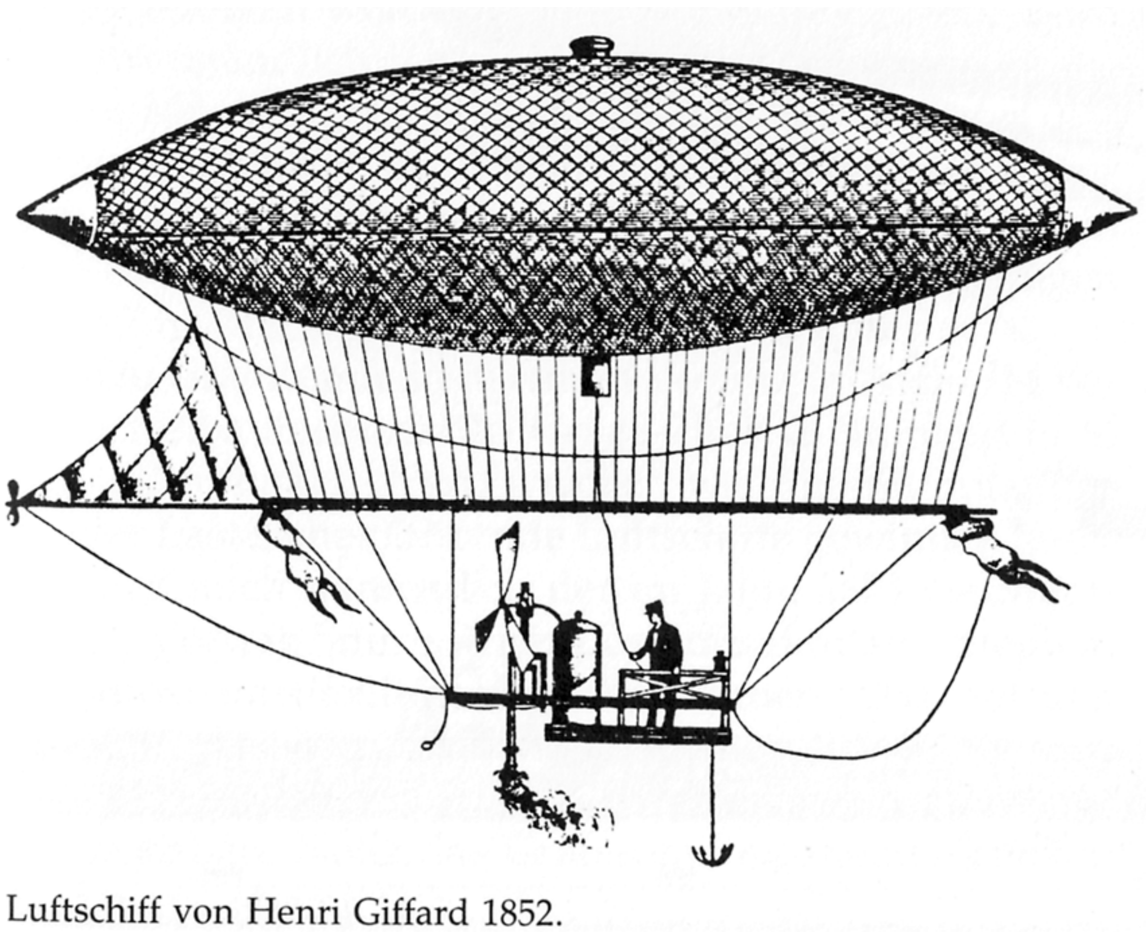

Fig. 4: Net suspension system designed to better distribute load of gondola to the gas envelope of a non-rigid airship. Forward-looking design by Henri Giffard, 1852. (Knäusel)

An airbag, called a ballonet, was introduced inside the gas envelope to address these problems. The ballonet is capable of being inflated with normal air, or deflated, to maintain pressure inside the hull. The intake for the air is usually placed just behind a propeller. (**Fig. 3**) While the ballonet does solve the pressure problem in the airship at low altitudes, it does not help very much in increasing the pressure height. In fact, the ballonet takes up some space in the envelope, which decreases the volume of the lifting gas and decreases useful lift. The only way to increase the pressure height and useful lift is to make larger airships.

A problem with the nose of the airship being blown in by wind pressure was related to the gas pressure. Adding the ballonet did not satisfactorily alleviate this problem. The nose being blown in was not debilitating but it did adversely affect the performance and maneuverability of the airship. This problem was resolved by reinforcing the nose with a radial arrangement of stiffeners made of some type of rigid material such as wrought iron, wood or aluminum. While weight is a consideration the stiffeners do not have to be especially robust to resist wind pressure. Today's non-rigid airships, such as the well-known Goodyear blimps, have heavily reinforced noses to accommodate docking points for the airship.

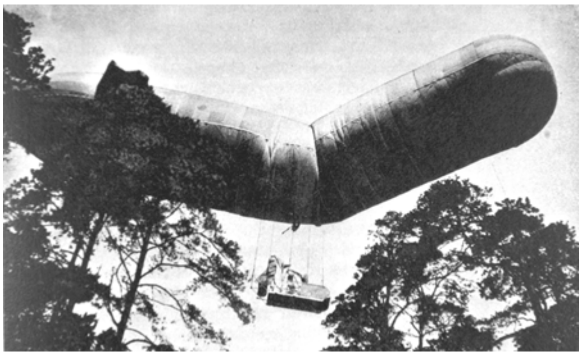

Fig. 5: Buckled envelope of a non-rigid due to moment stress in the gas envelope. German aeronaut August von Parseval designed the pictured ship. (Knäusel)

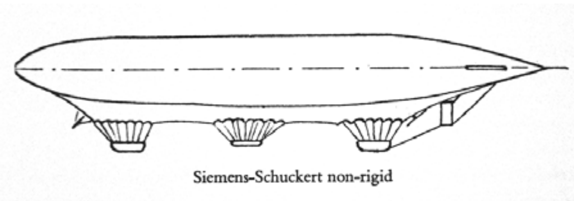

Fig. 6: Siemens-Schuckert airship, 1911. Specially reinforced textile keel, similar in principle to the semi-rigid type ship. (Hartcup)

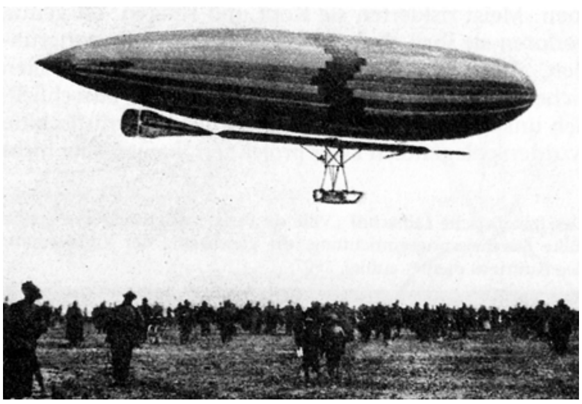

Fig. 7: A semi-rigid type airship with rigid spine on underside of envelope. Shown is a German Gross-Basenach airship. (Knäusel)

Another problem with the non-rigid aircraft is to properly distribute the weight of the gondola, engines, steering surfaces, etc., to the gas envelope without compromising the aerodynamic shape of the bag. Many solutions have been tried to overcome this. Most solutions comprised of some arrangement of ropes either wrapped over the top of the envelope like a net or attached at multiple points along the length of the envelope to distribute the load. (**Fig. 4**) Nonetheless, the envelope had to resist moment forces and occasionally failed when the loads were not properly distributed or the gas cell pressure fell too low. (**Fig. 5**)

Perhaps one of the better non-rigid designs was the German Siemens-Schuckert airship built in 1911. The Siemens-Schuckert had a keel made of specially reinforced fabric that evenly distributed the loads to the envelope, but this arrangement was not adopted in other non-rigids. (**Fig. 6**)

4.2.3 Semi-Rigids

The Siemens-Schuckert design is actually a non-rigid adaptation of another development in airship technology that dates to the 1890s. This development was the introduction of a rigid spine that was hung below the envelope. The gondola, engine pods, and control surfaces could be attached to this spine, or keel, which evenly distributed the load to the envelope. This type of ship is called a *semi-rigid*. (**Fig. 7**)

The semi-rigid incorporates a rigid framework – sometimes it was a singular linear element but mostly some type of truss arrangement – that extended most of the length of the envelope. The spine not only distributes the loads to the envelope but also resists the moment force acting in the airship. The keel of the semi-rigid airship increases the inherent stability of the gas envelope, thus the internal gas pressure can decrease, which increases the pressure height. Additionally, the diameter of the ship can be reduced and the length made longer because the keel decreases the likelihood of the envelope bucking. The slimmer airships showed greatly improved aerodynamic behavior.

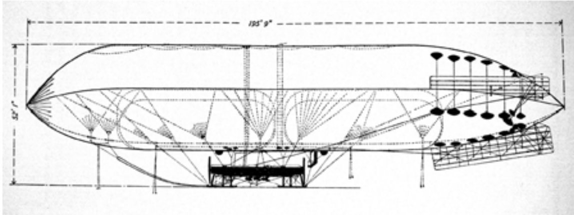

Fig. 8: Semi-rigid airship designed by Enrico Folanini in 1909. The gasbags are inside the aerodynamic envelope and attached to a rigid girder on the bottom. The envelope is pressurized with air and used as a ballonet. (Hartcup)

One problem of all airship types is that the lifting force acts upon the top of the envelope while most of the loads are applied to the bottom. This makes it necessary to transfer this load to the top of the envelope around a curved surface.[2] Enrico Forlanini, an aeronautical engineer who worked for the Sociéta Leonardo da Vinci in Milan, found a novel solution to this problem in a semi-rigid built in 1909. He jointed the gasbag to an internal rigid girder that ran from bow to stern. (**Fig. 8**) Fitted to the girder were the car, elevating and stabilizing planes, and rudders. The circular envelop consisted of a double skin with an intervening air chamber that served as ballonet. This arrangement maintained the aerodynamic envelope while also isolating the gasbag from the effects of the sun and atmospheric disturbances.[3]

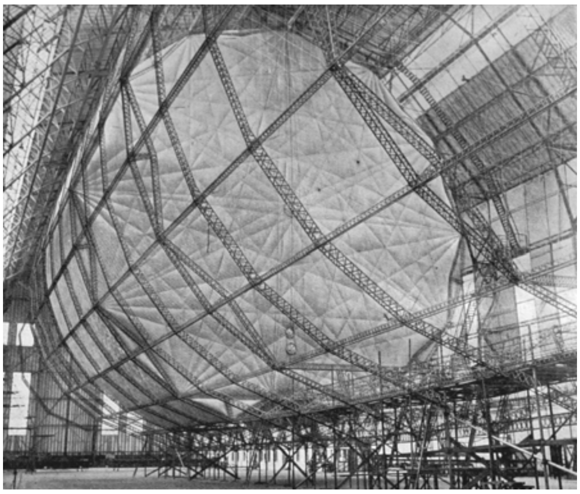

Fig. 9: General view of a Zeppelin-type rigid airframe. Image shows one inflated gasbag. The design is based on a compartmentalized gasbag system. (Dürr)

Today, there is a resurgent interest in airship technology. Invariably, these airships are either non- or semi-rigids. New engineered textiles make it possible to make ships on the scale of the late Zeppelins without the need for such an onerous, heavy airframe structure. The lack of materials for, and knowledge of, stressed membranes in the late 19^{th} and early 20^{th} centuries limited the development of non- and semi-rigid airships. Thus, for a time, the gigantic Zeppelin's reigned supreme in the skies because they were the only flying craft with significant useful lift capacity and range. The non- and semi-rigid types were of limited value to spotting, reconnaissance, scientific experiments and the like but were not effective cargo carriers because of their low lifting capacity and range.

4.2.4 Rigids

The rigid-type airship, as characterized by the *Zeppelin*, is comprised of a framework, called an airframe, over which an envelope is stretched. The gasbags, separate from the envelope, are inside the airframe. (**Fig. 9**) The gondola, engine cars and control surfaces are attached to the airframe, which distributes and transfers the load to the gas cells. The airframe made

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[2] In the case of the Zeppelins, these forces were transferred by the airframe, particularly the transversal cable nets that stabilized the transversal ring frames.

[3] Hartcup, p47.

Fig. 10: View of ground handling procedure requiring many men. (Knäusel)

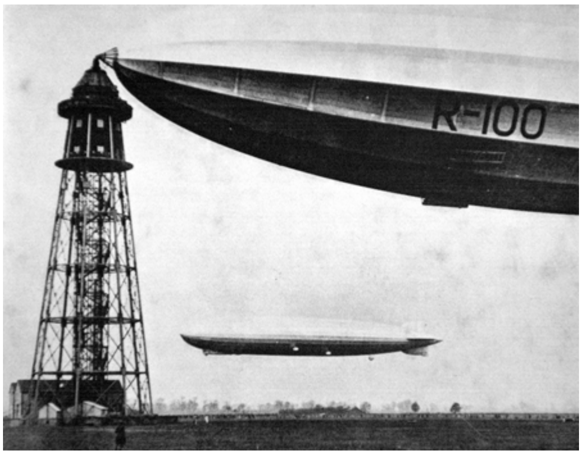

Fig. 11: The mooring mast, a British invention, allows the airship to turn 360° with the wind while moored. (Hartcup)

it possible for the first time to attach the control surfaces in more suitable positions on the hull of the airship rather than suspended beneath the gas container. The airframe also made it easier to distribute the load due to equipment and service compartments to better stabilize the airship.

The aerodynamic shape of the airship's envelope is maintained by the airframe, not the pressure of the gas cells. A ballonet is unnecessary in the rigid-type airship because the gasbags are free to expand and contract inside the airframe. Additionally, the airframe resists the moment forces of the ship, thus relieving the gas cells from the task. Therefore, the gas cells of a rigid type airship can be thinner than those of a non-rigid because less gas pressure is necessary in the gas cells to maintain the aerodynamic shape and the cells do not have to resist moment forces.

The Zeppelin-type airship is further distinguished by the use of several gasbags instead of one. In this way the ship can remain aloft should a gasbag or two be compromised and lifting gas is lost. Some non- and semi-rigid ships did incorporate multiple gas cells but this also means that more than one ballonet is necessary for conventional designs.

Rigid airships had a great advantage in their useful range because they could hold more gas and there was less need to release gas wastefully as there was with the non- and semi-rigids. The gas cells in a rigid airship can be partially filled such that the pressure height will be higher. The pressure height of Zeppelin airships reached upwards of 20,000 feet during World War I.

Rigid airships had an economic advantage over other types of airships because of their size. The rigid airships could carry larger useful loads than their non-rigid counterparts could. Interestingly, the relative cost of motive power did not increase linearly with the size of the airship because aerodynamic drag does not increase proportionally with size. Therefore, the engine cost per horsepower was less expensive per kilogram of useful lift in the rigid-type than its non- and semi-rigid counterparts.

A significant disadvantage of the rigid type airship was handling it on the ground. (**Fig. 10**) The size of these ships made them highly vulnerable to crosswinds. Any sort of impact easily damaged their rigid structure. A number of Zeppelins were blown from their moorings and destroyed. During World War I, the rigid airships of Germany required a battalion of soldiers at each airship field just to handle the airships in and out of their hangers – perhaps not the best use of limited personnel resources during a war. The British invented a mooring mast to solve this problem. (**Fig. 11**) Once secured to such a mast the ship was free to swing 360° with any change in wind currents.

4.3 Aluminum: Its History, Properties and Manufacture

4.3.1 The Reduction of Aluminum from Its Ores

Aluminum makes up 7.57% of the earth's crust. In contrast, iron is only 4.7%. Good grade ore, containing large percentages of alumina, is rather limited however. The principle aluminum ore is bauxite and can be found in plentiful quantities in Europe, North America, Australia and tropical climates.[4]

Aluminum does not naturally exist in its metallic state. To isolate aluminum from alumina requires specific chemical reactions that would not normally occur either naturally or by the accidental action of man. All the known metals until the nineteenth century, such as copper, silver, gold and iron, could be found in their metallic state or easily observed when a rock containing such elements was subject to high temperatures such as in a fire. Aluminum could not be observed in its metallic state until the development of modern chemistry advanced sufficiently to determine the chemical reactions necessary to isolate aluminum. Aluminum is the first structural material to be a product of modern science.

The existence of aluminum was first determined in 1760 by T. Baron de Henou-ville, a professor of chemistry in Paris, but he was unable to isolate it. In 1809, Sir Humphrey Davy of Britain tried to use a battery made of 1000 zinc plates to isolate aluminum. He had successfully isolated potassium and sodium using such a battery. When Davy fused iron to whiteness using an electric arc in contact with alumina the iron later showed it contained aluminum when dissolved in acids. Thus, the fact was established that alumina could be decomposed while fluid in the electric arc.[5] Davy was ahead of his time by some sixty years in his use of an electric arc to reduce aluminum from its ore. Electrolytic reduction was introduced as the principle means of producing aluminum around 1886 due to the development of the dynamo. Electrolytic reduction is still the principle means of producing aluminum today.

In 1827, a German professor of chemistry at the University of Göttingen, Federick Wöhler, became the first person to isolate aluminum, though his specimen was impure and the aluminum was only a powder. In 1845, Wöhler improved upon his results, producing larger and more pure pinhead size samples of the metal. (**Fig. 12**)

[4] Nichols, p59.
[5] Richards[2], p4.

Fig. 12: Test tube holding aluminum particles reduced by Friedrich Wöhler, 1845. (Nichols)

In 1854, Saint-Claire Deville, a Frenchman, was the first to produce significant quantities of aluminum and determine aluminum's properties. Deville used chemical reduction to produce his first quantities of aluminum. He worked the rest of his life to industrialize and commercialize the production and application of aluminum.

4.3.2 Industrialization

Deville isolated aluminum both chemically and electrolytically. He would have liked to use the electrolytic process but the cost of battery cells at the time was prohibitive in comparison to chemical processes, which were also expensive. In the absence of a viable power industry that could produce large amounts of cheap electricity necessary for aluminum's reduction, chemical reduction was the only viable option.

Deville first used potassium to isolate aluminum but switched to using sodium, which was less expensive to produce. The focus of much of Deville's ensuing work was to bring down the cost of aluminum. He saw the possibilities of aluminum as being a material useful for everyday goods that would benefit the general population. Towards this end, Deville's efforts resulted in a reduction of costs from FFr. 6,000 per kilogram in 1852 to FFr. 59 per kilogram in 1888.[6]

Deville and his peers achieved such great cost reductions by doing value analyses of the entire aluminum production process. This included how the bauxite ore was obtained and how the sodium was produced. The cost of sodium production was identified as a principle contribution to the cost of aluminum production. As a result Deville worked with members of the sodium industry to develop more efficient and economical processes for producing sodium, thus reducing costs not only for producing aluminum but all industries that used sodium.

In 1888, Deville died and shortly thereafter his process was abandoned for the new and more economical electrolytic process.

The introduction of the electrolytic process, invented concurrently by Charles Martin Hall in America and Paul T.L. Héroult in France between 1886 and 1888, dramatically reduced the cost of aluminum further. By the end of 1891, when Count Zeppelin decided to use aluminum in his airships the cost had fallen to FFr. 6.15 per kilogram. In 1899, the year Count Zeppelin began construction of his first airship, the cost had decreased further to FFr. 2.70 per kilogram. The cost of aluminum eventually bottomed out in 1911 at FFr. 1.45 per kilogram, after which the cost rose again to just over two French francs per kilogram due to the looming signs of World War I.[7]

[6] Joliet, p69.
[7] Joliet, p69 and 80.

4.3.3 Aluminum and Its Alloys

Aluminum was first made as pure as possible. Deville's successful isolation of aluminum was actually an accident because he had been trying to make aluminum proto-oxide. Deville's chemical process was conducive to the production of pure aluminum. Commercially pure aluminum normally contains between 1% and 8% of impurities, most frequently iron and silicon.

Early developments using the electrolytic process, as represented by the works of Grätzel in Germany, Kleiner in Zurich, and Cowles in America, made it easier to make alloys rather than pure aluminum.[8] One alloy that became prominent in the commercial aluminum market was bronze-aluminum, which is an alloy of 90% copper and 10% aluminum. This alloy, being harder than pure aluminum, was popular in such products as tableware, architectural ornament and other such articles, but obviously does not take advantage of one of aluminum's most coveted properties, its lightness. While Hérault and Hall developed similar electrolytic processes conducive to the production of pure aluminum, Hérault was more interested in exploiting the already established market for aluminum alloys while Hall focused on the unexploited commercial possibilities of pure aluminum.[9]

In structures, where lightness was a principle requirement, aluminum based alloys were developed commonly using nickel, zinc or copper. These alloys typically had better strength properties than pure aluminum but were susceptible to other problems such as brittleness or reduced corrosion resistance.

The most important aluminum alloy to be developed during the airship era was Duralumin. Alfred Wilm, a German metallurgist, invented Duralumin in 1909 for the German company Duerener Metallwerke. Duralumin is an alloy of aluminum, copper, manganese and magnesium with any impurities of iron minimized. Duralumin, which was manufactured in three different varieties, has a range of tensile strength from 372 MPa to 450 MPa[10] (54 ksi to 65 ksi[11]), better than some high strength steels (Fe E 235[12] = 360 MPa) but only one-third of the weight. It was not employed in the Zeppelin airships until 1914 because of quality control problems with the alloy and the time it took to establish the special procedures necessary to form the metal.

4.3.4 Material Properties

Aluminum is an anisotropic metal, characteristically light, and resistant to oxidation and other forms of corrosion. Aluminum's corrosion resistance is due to the metal's natural coating of aluminum oxide. A number of its alloys are less resistant to corrosion then others. Pure aluminum has a tensile strength of 147 MPa to 216 MPa[13] (21 ksi to 31 ksi) but aluminum alloys such as Duralumin can be stronger than Fe E 235 steel. Generally, aluminum and its alloys have a specific weight around 2.71 g/mm^3 and a Young's elastic modulus of 70,000

[8] Richards[2], p24-28.
[9] Nichols, p77.
[10] Joliet, p66.
[11] 1 ksi = 1 kilo-pound per square inch. 1 ksi = 6.9 MPa. 1 kip, or kilo-pound = 1,000 pounds = 453.6 kg
[12] Fe E 235 (SIA) = Fe 360 (EU) ≈ ASTM Grade 50
[13] Dürr, p32.

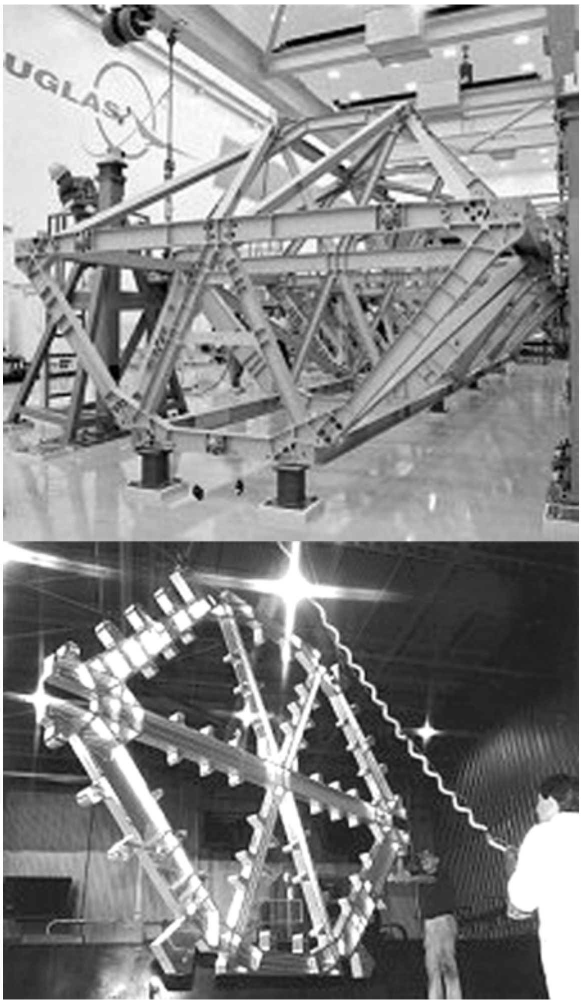

Fig. 13: International space station. (a) General view showing trussed spine made of aluminum. (b) Detail showing one transversal bulkhead of the space station truss machined from one cast aluminum block measuring 3.4 m x 4.3 m, 115 mm thick and weighing 5,450 kg. (Fortner)

N/mm^2. Aluminum is a ductile material, attaining 17% elongation in 50mm, comparable to steel's 23%. Cold rolled Duralumin is less with 14% to 16% elongation.[14]

Aluminum can be cold and hot worked. Cold working strain hardens the metal, increasing strength but reducing ductility. Best practice requires frequent annealing when cold working, though aluminum has to be annealed less often than steel. The final passing or forging stages can be done without annealing to benefit from the strain-hardening effect. Aluminum can be formed by a wide variety of processes. Some of the primary processes are listed in the following sub-section.

4.3.5 Material Manipulation Technologies

By 1900, all but one of the principle means or working and manipulating aluminum were established. The main process that seems not to have developed by the turn of the 20th century is extrusion, which is so pervasive in the creation of all manner of complex profiles of constant cross-section today.

Casting

Deville is quoted as saying: "Castings of great beauty may be obtained, but it is not advisable to conceal the fact that to be able to succeed completely… requires for aluminum, as for other metals, a special familiarity with the material which practice alone is able to give." Casting molds were made of sand or metal. Castings of high tolerance and large scale could be made, such as teakettles with a wall thickness of 1.6 mm (1/16 in) or bathtubs 1.8 m long, 0.6 m wide, 0.6 m deep (6 ft x 2 ft x 2 ft), with a wall thickness of 9.5 mm (3/8 in) and weighing 63.5 kg (140 lbs).[15]

Today, cast aluminum can be made on a much larger scale with high quality. The transversal member of the International Space Station truss, shown in **Figure 13**, is machined from one cast block of aluminum measuring 3.4 m x 4.3 m, 115 mm thick and weighing 5,450 kg (11 ft x 14 ft, 4.5 in thick, and 12,000 lbs). Each bulkhead costs several hundred thousand dollars and requires a week for machining.[16]

[14] Joliet, p66.
[15] Richards[2], p445-447.
[16] Fortner.

Freeing aluminum metal from impurities

Impurities can be removed from aluminum while it is molten but it is difficult to control by appearance. It is more effective to have a high level of chemical control over the process of reducing alumina to aluminum and remove known impurities through chemical reaction.[17] The Luftschiffbau Zeppelin Company found that it was possible to increase the homogeneity of aluminum and largely neutralize the negative effects of impurities by cold working the metal into thin sheets. This was especially true of the aluminum alloys. The resulting product was more consistent in quality and cold working improved the strength of the metal because of strain hardening. The basis of new structural textiles lies in the knowledge that the thinner we work a material, in this case the smaller diameter we can reliably manufacture the individual fibers use to make the textiles, the less impurities and other flaws compromise the potential strength of a material.[18]

Annealing

Aluminum can be annealed by heating it to a low redness and quenching in water. Aluminum gets soft and looses significant strength after annealing. Where strength and stiffness are required it is beneficial to cold work the metal again to its final form. Annealing allows aluminum to be worked into thin sheets and fine wire. Rolled elements should be annealed after each pass until the plate thickness is less than 3 mm, after which it can generally be rolled with few annealings.[19]

Hardening

Hammering, rolling or drawing hardens aluminum. This process makes the metal “sensibly” stiffer. Castings can be hardened by being cast slightly larger than their final dimension and then drop forged. Hardening increases aluminum’s resistance to wear and it can become elastic enough for hairsprings or watches.[20]

Rolling

Before rolling an aluminum bar, the metal should be softened by annealing and the leading edges should be tapered down by hammering - aluminum forges well under the hammer at a low red heat. Aluminum rolling stock is best when cast into plates and the surfaces planed to remove irregularities. Cold rolling is not difficult but a large amount of power is required, about the same for hot steel. Aluminum hardens quickly when worked; therefore it must be annealed frequently. Rolls warmed to 100° - 150°C work better than cold rolls.[21]

Drawing

Fine aluminum wire can be drawn because of that material’s ductility. Aluminum tubes, round or square, can be drawn from sheet that has been soldered together or from cast rings of required section.[22]

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[17] Richards[2], p447-452.
[18] Dürr, p57.
[19] Richards[2], p452.
[20] Richards[2], p452-453.
[21] Richards[2], p453-454.
[22] Richards[2], p454-455.

Fig. 14: Aluminum baby rattle fabricated to commemorate the birth of Napoleon III's son, one of the first artifacts to be made of that material. Charles Rambert, designer, 1856. (Nichols)

Fig. 15: Aluminum bracelet, Victor Chapron, c.1865. (Nichols)

Stamping / Spinning

Aluminum can be pressed and stamped when cold. This process was important to the creation of the complex shapes developed by Luftschiffbau Zeppelin. Aluminum can also be spun on a lathe into all sorts of round and hollow forms.[23]

Welding

Aluminum can only be welded with difficulty. As of 1900 and well into the 20th century, welding was impractical and extremely difficult to execute.[24] Today, the welding of aluminum is commonplace but still requires great care and quality control to do it well.

Soldering

Soldering was much more commonplace around 1900 than welding. Still, soldering aluminum was difficult because of some of the same reasons it is so hard to weld that metal. The thin layer of alumina makes it difficult to expose a 'clean' surface of aluminum. It is nearly impossible to remove the alumina because when it is removed a new layer forms almost instantaneously. Aluminum's electro-negativity and high conductivity of heat requires high temperatures when welding or soldering. As of 1896, a lead-tin solder was used most by industry, including the large works at Neuhausen, which figure into the story of the Zeppelin airship. Richards states that soldering could provide a joint that would not fail before the base metal.[25]

4.3.6 Historical Applications of Aluminum Prior to 1900

From Precious Metal to Kitchenware

When Deville first began producing aluminum commercially the cost was 6,000 French Francs per kilogram. Not long after he had reduced that price by half, but at FFr.3,000/kg,

[23] Richards[2], p455.
[24] Richards[2], p457.
[25] Richards[2], p457-469.

aluminum still could only be treated as a precious metal.

The first article made of aluminum was a baby-rattle for the birthday of Napoleon's son in 1856. (**Fig. 14**) As a precious metal, aluminum was used to make jewelry and sculpture. (**Fig. 15**) There was even an effort to use aluminum for minting coins.

As Deville improved his process and production volume increased the cost decreased. By 1888, aluminum produced by the Deville-Castner process was selling for FFr.59/kg. Corresponding to this plunge in price, aluminum began to be used for applications such as military equipment (canteens, belt buckles, buttons, etc.), tableware and window frames. (**Figs. 16**)

Fig. 16: Aluminum-bronze spoon set. Société Paul Morin & Cie., c.1880. (Nichols)

The introduction of electrolytic reduction dramatically reduced the cost of aluminum to FFr.6.15/kg by the end of 1891. It was around this time that Zeppelin decided to use aluminum and cost must certainly have been a factor. With the lower costs of aluminum the material was used for increasingly mundane products. One of the largest early commercial industries was the manufacture of aluminum pots and pans. The low cost of aluminum now made it more interesting in wider application, particularly those that needed large quantities of the material, such as structures. With no great advantage to be had using the much more expensive aluminum in lieu of iron or steel in static structures the first structural applications were movable structures, where the weight advantage and corrosion resistance could be exploited most effectively.

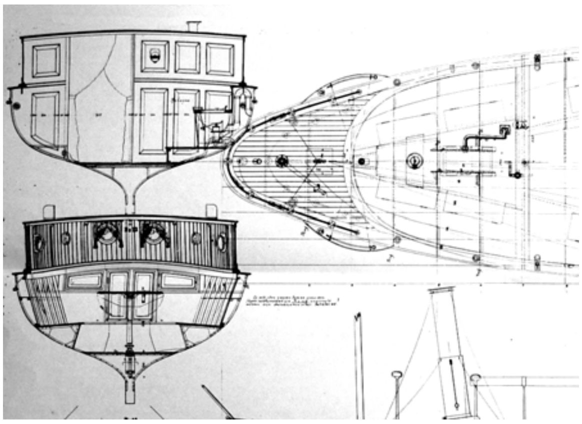

Fig. 17: All aluminum naphtha boat design, Escher-Wyss of Zurich, c.1889. (Escher-Wyss)

Naval Applications

While some wagons and horse racing sulkies were manufactured of aluminum, the first significant structural applications were in boats. Escher-Wyss of Zurich fabricated the first all-aluminum boats prior to 1892, probably around 1889 just after the Neuhausen works became established on the Rheinfall. The first Escher-Wyss boat was a naphtha motor launch. Even the smokestack and rigging was of aluminum.[26] (**Fig. 17**)

Escher-Wyss exported their boats worldwide, even building a 13.1 m (43 ft) long yacht for Alfred Nobel, the inventor of dynamite. Nobel's yacht, the *Mignon*, had its keel, stern and sternposts made of forged aluminum with a section of 178 mm by 25.4 mm (7 in x 1 in). The frames are angles 25.4 mm wide and 25.4 mm deep (1 in x 1 in), 1.6 mm (1/16 in) thick. The aluminum sheet covering the hull is 2.4 mm (3/32 in) thick. Connections are made with aluminum rivets. The rigging is aluminum wire. All of the machinery is of aluminum, including the propeller, except the cranks and shafting.[27]

Drawings provided to me by Sulzer-Escher-Wyss of Zurich show another Escher-Wyss boat built in 1894. The drawing clearly shows a structure that would be characteristic to iron construction in naval applications, so the forms of these boats were not novel. When I contacted Sulzer-Escher-Wyss they informed me that there are literally only a few documents in their archives concerning the Escher-Wyss boats so the information available about these important boats is very limited. Escher-Wyss offered all of their boat models with hulls made of wood, "diagonal" wood (boards run diagonally top to bottom versus longitudinally), galvanized steel and aluminum. It would be interesting to compare the fabrication drawings to examine how their constructions differed but it seems that this is not possible.[28]

Throughout the 1890s aluminum boats were manufactured in other countries as well. The American yacht *Defender*, which defended the American Cup against the English yacht Valkyrie III in 1895, was made of two different aluminum alloys. Its hull was of aluminum-bronze (10% aluminum - 90% copper) below the waterline and aluminum plates alloyed with 3% nickel above the water line. The upper frames and deck beams were also of the nickel alloy.[29]

Up until 1896, the largest vessel then constructed of aluminum was a French torpedo boat with a length of 18.3 m (60 ft), and constructed to be carried by the warship *La Foudre*. The torpedo boat was built by Yarrow and Company and first sailed in 1894. The aluminum was strengthened by alloying with 6% copper, giving a tensile strength of 240 MPa (35 ksi). All the frames were made 25% larger than if they were steel, and therefore weighed exactly one-half as much.[30]

Construction

In Joseph W. Richards' 1996 edition of *Aluminium*, he makes clear that there were few applications to be found in construction. As long as aluminum cost more pound for pound than steel, there was no advantage to using it in static structures except where lightness or resistance to oxidation was the first conditions to be filled. For this reason, he puts forward his own prediction that aluminum will generally remain structurally limited to movable structures, such as torpedo boats, railway carriages, cabs, wheelbarrows, etc. As he writes, "The substitution of aluminium for steel is simply a question as to whether the lightness

[26] Richards[2], p476.
[27] Richards[2], p476.
[28] Escher Wyss, catalog and construction drawing.
[29] Richards[2], p478.
[30] Richards[2], p477.

gained compensates for the increased cost."[31] This is the situation today concerning fiber-reinforced polymers, which are used principally in dynamic structures and their main disadvantage in static structures is their cost.

More research may reveal that aluminum was used in other structural functions. Perhaps the aluminum used in the staircase and elevator enclosures of Burnham and Root's Monadnock Building in Chicago (1889-1892) may have served a structural as well as decorative function. (**Appendix A-09, p.A.379, Fig. 31**) Burnham and Root also specified aluminum in the Venetian and Isabella buildings, completed in 1891 and 1893 respectively. In 1898, *The Aluminum World*, an industry trade publication intended to disseminate information concerning the growing number of uses for aluminum, reported Daniel Burnham's assessment of the new metal. Burnham wrote,

> I like the metal because it does not rust, but looks well and when used awhile it turns a dull color and does not shine like nickel-plated brass or other metals. It also lasts where you put it. There will be a future for aluminum in replacing wood in certain buildings. However, that will be largely a question of price and conditions. I believe aluminum is a useful metal for architects at present and that it will be more useful in the future.[32]

In the intervening decade from the time the first Escher-Wyss motor launch was built until the construction of LZ-1, Zeppelin's first airship, in 1900, the largest structures built of aluminum were the French torpedo boat at 18.3 m (60 ft) long and a rigid airship built by David Schwarz, which was an antecedent to Zeppelin's own development. Schwarz' ship was 38.3 m (126 ft) long with a height of 18.2 m (46 ft). In contrast, LZ-1 was 127 m (417 ft) long with a diameter of 11.65 m (38 ft). In other words, no aluminum structure built before 1900 compared to the scale of Zeppelin's realized vision of a rigid airship. We will see that while Schwarz' ship may have provided some knowledge to Zeppelin, it was of such limited value structurally that he and his engineers were really constructing a structure without precedent and without parallel in scale for its application to dirigibles.

4.3.7 Aluminum Alloys Used in the Zeppelin Airships

When Zeppelin first started his project in earnest the Neuhausen works at the Rheinfall in Switzerland was the largest aluminum producer in the world, with the great amount of electricity required being provided by harnessing the energy of the largest waterfall in Europe. Most production was in aluminum-bronze. Zeppelin initially made a contract with Neuhausen around 1892. It is unclear if Zeppelin could have ordered a lighter aluminum alloy or commercially pure aluminum. Ultimately, nothing came of this early contact because Zeppelin still had to overcome many problems in his design before he could begin construction.

The first aluminum rigid-type airship, built by David Schwarz, was constructed of an alloy invented by Carl Berg called Viktoria aluminum. Carl Berg, son of a successful iron founder in Germany, owned and operated his own aluminum works. I have not been able to find what exactly the composition of Viktoria aluminum is but it was not used in any of the Zeppelin ships.

[31] Richards[2], p487.
[32] Nichols, p86-87.

Legierung	Anlieferungszustand	Streckgrenze + 0,2 % bleibende Dehnung kg/mm²	Zugfestigkeit kg/mm²	Dehnung $l = 11{,}3\sqrt{F}$ %	Querschnittverminderung %	Kerbschlagfestigkeit cmkg/cm²	Kugeldruckhärte Brinell	Tiefungsfähigkeit mm	Biegewinkel r = 2 ⁄ 3 d °	Elastizitätsmodul kg/cm²
681 B ⅓	veredelt	24 bis 27	38 bis 41	18 bis 21	18 bis 30 durchschnittl. 26	140 bis 158	115	7,50	180	650 000 bis 720 000
	kalt nachverdichtet Härte ½	30 bis 32	40 bis 44	14 bis 16	12 bis 28 durchschnittl. 22	115 bis 145	122	6,40	170 bis 180	650 000 bis 720 000
681 B	veredelt	26 bis 28	38 bis 42	18 bis 20	15 bis 30 durchschnittl. 24	132 bis 149	118	7,00	180	710 000 bis 740 000
	kalt nachverdichtet Härte ½	32 bis 34	43 bis 46	12 bis 15	11 bis 27 durchschnittl. 19	105 bis 116	125	6,25	170 bis 180	710 000 bis 740 000
Z	veredelt	27 bis 29	41 bis 44	17 bis 19	14 bis 28 durchschnittl. 22	100 bis 113	120	6,60	180	710 000 bis 740 000
	kalt nachverdichtet Härte ½	33 bis 35	44 bis 47	10 bis 14	10 bis 26 durchschnittl. 18	88 bis 100	128	6,00	160 bis 170	710 000 bis 740 000

Table 1: Different duraluminum alloys and their properties. (Joliet)

Zeppelin's first airship, LZ-1, was constructed of commercially pure aluminum supplied by Carl Berg. The angle and T-profile sections were produced by rolling, similar to iron profiles. The aluminum elements had a material thickness of 1 to 4 mm. The rolling procedure strain hardened the aluminum, thus it was possible to have tensile strengths of 15 to 22 MPa (2.2 to 3.2 ksi) with 5% to 10% elongation.[33]

Aluminum-zinc and aluminum-zinc-copper alloys were used in Zeppelin airships from LZ-2 to LZ-25.[34] There were quality control problems with these alloys because their properties were not homogeneous. By LZ-7 these problems were satisfactorily overcome through refinement of production methods but the product was still not ideal. Ultimately it was possible to have tensile strengths from 35 MPa (5.1 ksi) with approximately 12% elongation.[35]

Duralumin is composed of aluminum alloyed with copper, manganese and magnesium. Iron impurities are minimized. Duralumin has tensile strengths of 40 to 44 MPa (5.8 to 6.4 ksi) when cold rolled as opposed to 38 to 41 MPa (5.5 to 5.9 ksi) when not worked at all.[36] Luftschiffbau Zeppelin immediately considered using Duralumin but technical difficulties with the material kept it from being using until 1915. The material was susceptible to becoming unacceptable brittle when worked using normal methods of the time. Processes had to be refined to accommodate the particular characteristics of the Duralumin alloy.

Luftschiffbau Zeppelin's chief engineer, Ludwig Dürr, describes Duralumin as an *einwandfreies Baumaterial*– a perfect building material. Dürr specifically was interested in

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[33] Durr[1], p32.
[34] All Zeppelin airships ever constructed were given unique identification numbers starting with 'LZ-' that were assigned in sequential order of design with few exceptions. Some airships were never constructed and some were constructed before others.
[35] Durr[1], p32.
[36] Joliet, p66.

the possibility of developing new construction forms using this material because of its impressive strength to weight properties. The challenge was to exploit the excellent characteristics of the material without sacrificing the rational building method theretofore developed by Luftschiffbau Zeppelin for the construction of metal airframes.[37]

Duralumin was first used to construct the airship LZ-26, which was put into service in 1915. Different Duralumin alloys had to be used for different applications depending on the peculiarities of the production process needed to fabricate the different parts of the airframe. (**Table 1**) Cold drawn or rolled structural profiles were fabricated using Duralumin alloy 681 A. Alloy 681 B was used for components that needed less strength, including sheet metals. Alloy 681 B was also used to fabricate parts with complex forms such as the propellers. Duralumin alloy N was specially designed for rivets. Its hardness was reduced to make it less likely that damage would be caused to the elements being connected when the rivet was struck.[38]

4.4 Zeppelin, an Introduction

4.4.1 Prologue

In 1899, when Count Ferdinand von Zeppelin and his partner Carl Berg, a German aluminum manufacturer, began construction on the first zeppelin airship the aluminum industry was, for all practical purposes, only twelve years old. Aluminum was only first produced commercially in 1853, at a cost of some 6,000 French francs (FFr) per kilogram. Until 1888, the price of aluminum, although it had dropped dramatically to FFr. 59 per kilogram, was still too high to be used in structures. After 1888, electrolytic reduction processes largely replaced chemical processes for the production of aluminum metal and the cost reduced to a level that was sustainable for wider use, including in some structural applications, particularly boats.

Count Zeppelin had decided on using aluminum for his airship in 1891. By this date very few structures had been built of aluminum. The most significant were relatively small motor launches built by Escher-Wyss & Company in Zurich. Zeppelin lived in Manzell, on Lake Constance, separating Germany and Switzerland. He was probably aware of these boats being built in Zurich since a factory production list shows that boats were shipped for use on Lake Constance as well as internationally. In fact, the boat pictured in a photo showing Count Zeppelin's engineers testing air propellers looks very similar to an Escher-Wyss model listed in their 1912 catalog[39], though the style and painting scheme could reflect a regional tradition rather than one manufacturer's own mark.

From the beginning of his project, Count Zeppelin knew that his airship had to be large if it were to be successful. Surprisingly there is no evidence that he gave any great consideration to the possibility of building a smaller trial model before the full-scale ship. While there was one rigid airship completed in 1897 by a man named Schwarz with the aid of

[37] Dürr[1], p32.
[38] Dürr, p32-33.
[39] The manufacturer's list is provided in an English translation of 1912 Escher-Wyss & Co. *Motorboote und Motoryachten, 1912 Katalog*. The English translation focuses on the steam-powered boats listed in the original catalog. The photo is in Hans Knausel's *LZ-1, Der Erste Zeppelin…* (1985), p 98.

Carl Berg, there was no other structural precedent for a zeppelin airframe, certainly none in aluminum. Count Zeppelin's first ship, at 127 m long, was to be over three times as long as the Schwarz ship and of much lighter construction.

An airframe such as a zeppelin's is highly redundant, up to twenty times so[40]. At the time, there did not exist an adequate method of analysis for such a structure. Graphic statics, while adequate for individual components and simple truss work was insufficient to accommodate such redundancy. As a result, Luftschiffbau developed largely empirical methods of calculating the strength and behavior of the airframe.[41] Only computers can possibly calculate a structure with such redundancy, and these were not available to the Zeppelin engineers. The British, relying on theoretical calculations, took up to two years to complete the calculations for one of their rigid airships built in the 1920s.[42]

In summary, the zeppelin airframe was an unprecedented structure with no adequate methods of analysis available to its engineers. The material, aluminum, was largely new and of unknown behavior in such a large structure. Additionally, it can be claimed that aluminum marks the first material discovered by man made possible through modern science.

Over the forty years or so Count Zeppelin and Luftschiffbau Zeppelin constructed their airships, the use of aluminum advance greatly, achieving greater strength and stiffness with decreasing amounts of material. In particular, the zeppelin development coincided with the development of thin walled cold-formed structural elements. Such structural elements were also new and required the development of theory and analysis methods to adequately model and calculate their structural behavior.

As a material, aluminum was first exploited and advanced by the naval and aeronautic industries, similar to the fiber reinforced polymers (FRP) of today. Therefore, the mechanisms of technology transfer of this case study are important to the development of that material, though this analysis is outside the purview of this project.

The zeppelin legacy clearly influenced the field of aeronautical engineering, in both lighter-than-air (LTA) airships and heavier-than-air (HTA) aircraft. The development of thin walled structural members and spatial structural systems influenced structural engineering in general as well.

All of these developments, which were new and characterized more by what was unknown than that which was known, provide an opportunity to analyze how a new structural material can be developed. All of the questions of this thesis can be addressed by this case study. For all of the above reasons the zeppelin case study is germane to the project.

4.4.2 Brief Biography of Count Ferdinand von Zeppelin

Count Ferdinand von Zeppelin was born 08 July 1838 in the city of Constance, which is located at the end of Lake Constance, with is bordered by Germany, Switzerland and Austria. His father, Friedrich, was councilor to the court of the Duke of Hohenzolern-

[40] Redundancy figure given by Johann Schütte's engineer, Fritz Gentzke, in Schütte's *Der Luftschiffbau Schütte-Lanz 1909-1925* (1926), p93.
[41] Hartcup, p177. This information was provided to some British airship designers by the former LZ engineer Paul Jaray.
[42] Hartcup, p177.

Sigmaringen and his mother, Améli, came from a family of textile manufacturers by the name of Macaire in Constance. The Macaires came from Geneva and were an old emigrant family of Southern France origin. Améli's grandfather, Jacques Louis Macaire de l'Or, had been encouraged by the Emperor Joseph II to establish a calico factory at Constance with a view of introducing the manufacture of cotton into Germany. Zeppelin and his siblings spent their childhood in Girsberg, near Constance, and were mainly educated by private tutors who lived with the family.[43]

Zeppelin's early interest in labor management and machines was recorded in a letter written by his mother to a relative when he was just five and a half years old. His mother wrote, "He…always knows exactly in which field the farm-hands are working, and is unusually interested in such things as new ploughs and seeding machines."[44] We can assume that Zeppelin would have been further exposed to complex machines and production processes at the cotton textile-making factory owned by his mother's side of the family.

In 1853 Count Zeppelin left Girsberg to enroll in the Realschule in Cannstatt near Stuttgart. A year later he changed to the Polytechnic Academy of Stuttgart. In 1855 he became a cadet of the military school in Ludwigsburg and decided to take up a military career as an officer in the army of Württemburg.[45] Zeppelin's early interest and family background surely influenced his decision to enroll at Tübingen University in October 1858 to study mechanical technology, inorganic chemistry, economics and history. In May 1859 he was forced to quit his studies because he was called back to service. Perhaps Zeppelin may have pursued a professional path in the mechanical arts had his social and political position not left him with few honorable career paths to follow.

Over the next two years, Zeppelin spent his military service with engineering units posted at Ulm and with the Quarter-Master General's staff at Ludwigsberg. During training exercises in Verona Zeppelin records observing "interesting exercises carried out by the engineers."[46] While his academic background in engineering was extremely limited, Zeppelin's two years of exposure to engineers and engineering in the military must have given him an appreciation of how engineers worked and how to work and communicate with them.

Zeppelin began keeping a diary around 1862. With occasional breaks, Zeppelin continued to make entries into his diary until 1917, near the end of his life. As a result, there is a stack of thousands of large quarto pages.[47] This voluminous record reflects on Zeppelin's character. He was persistent, disciplined, dedicated and filled with seriousness of thought. Hugh Eckener, Zeppelin's biographer and an influential employee of Luftschiffbau Zeppelin, is the only person outside of the family known to have had unrestricted access to the diaries. Unfortunately, Zeppelin's heirs have restricted access to the diaries ever since. Therefore, the quotes recorded by Eckener in his book coming directly from Zeppelin's diaries are the best source of information now available.

In 1863 Zeppelin took a leave of absence for several months in order to study military strategies in the American Civil War. He traveled to America and participated in the war on

[43] University of Konstanz, *Pictures*, p1 and Eckener, p7.
[44] Eckener, p3.
[45] University of Konstanz, *Pictures*, p1.
[46] Eckener, p34-35.
[47] Eckener, p19.

the side of the Northern Army. He later undertook an expedition to the springs of the Mississippi River. In St. Paul, Minnesota, Zeppelin had the opportunity to ascend in a balloon owned by a Professor Steiner. That was August 19, 1863.[48] Eckener makes clear that in neither Zeppelin's diaries nor their personal conversations did Zeppelin ever claim that he had thought of airships prior to his balloon ascent. The ascent did not even suggest the idea in his mind.[49]

Zeppelin was appointed aide-de-camp of the King of Württemberg after his return to Germany. Zeppelin was promoted to Captain, and, in 1866, he took part in the Austro-Prussian War and from 1870-1871 he fought in the war between France and Germany. During these conflicts Zeppelin was decorated for his service and became known in wider circles for the first time as a result of a daring reconnaissance mission he led behind French lines. This event became known as the *Schirlenhof* event and was often cited in later years as an example of Zeppelin's daring character. After the war, Count Zeppelin served in different cavalry regiments in Strasburg, Ulm and Stuttgart.[50]

Zeppelin married Isabella Freiin von Wolff from Livonia in 1869. Their only daughter, Hella, was born in 1879 in Ulm. In 1885, Count Zeppelin was appointed plenipotentiary officer of Württemberg in Berlin, where he moved to with his family. Zeppelin found that he did not gain favor with the Prussians because of his critical views of the military structures and of the role of the states within the German Empire.[51] This political clash would have long lasting repercussions in his life.

After Zeppelin's service in Berlin as a diplomat of Württemberg ended in 1890, he returned to military service as commander of a cavalry brigade in Saarburg. Under the pretext of a disagreement about the evaluation of his command during autumn maneuvers in 1890, Zeppelin was forced to submit his final resignation from military service after his promotion to a lieutenant general. Henry Cort Meyer suggests that this event was simply used by the Prussian leadership to push Zeppelin out of the military as a penalty for his outspoken criticism of Prussian leadership.[52] Zeppelin was 52 years old.

Meyer focuses on Zeppelin's forced resignation as an event that caused a fundamental psychological change in Zeppelin. Meyer argues that this change, one based in the psychology of lost honor, led to Zeppelin's persistent and dedicated travail of conceiving and building not only an airship, but various technological industries that were necessary to produce the specialized parts needed in airship construction.

In 1890, Count Ferdinand von Zeppelin suddenly found himself with no useful purpose in life, having been maneuvered out of the military into early retirement from a military career that he had devoted his whole adult life to. Having sporadically pondered the possibilities of lighter-than-air vehicular flight in the past, Zeppelin recovered from what must have been a demoralizing and humiliating event, and devoted his life to the creation and development of the airship. Zeppelin saw both the commercial and strategic advantages such craft might have for the *Vaterland* and committed himself to the long and arduous task of building a

[48] University of Konstanz, *Pictures*, p1.
[49] Eckener, p54.
[50] University of Konstanz, *Pictures*, p2.
[51] University of Konstanz, *Pictures*, p2.
[52] Meyer[1], p37-38.

dynamically flying craft without precedent for himself, to restore his honor, and for his homeland, to fulfill his desire to be of significant service to his country. The evidence of these intentions are well presented and supported in Henry Cord Meyer's *book Count Zeppelin, A Psychological Portrait*.

Zeppelin's arduous journey from airship conception to construction is treated in detail in the following sections. To summarize here, Zeppelin began construction on his first airship in 1899, nearly ten years after starting this task in earnest. The first Zeppelin airship, LZ-1, took off for the first time on 02 July 1900, from its floating hanger at Manzell on Lake Constance. This success was quickly met by failure when the joint-stock company Zeppelin formed in partnership with the aluminum manufacturer Carl Berg, the Aktiengesellschaft zur Förderung der Luftschiffahrt, was liquidated in December 1900.[53]

Zeppelin continued to develop the airship, primarily funding the effort with his own money. Between 1906 and 1908, Zeppelin constructed three other airships. The unfortunate crash of LZ-4, which was then on its final leg of a twenty-four hour endurance test flight, resulted in an outpouring of public support and financial aid. On this financial basis, the company Luftschiffbau Zeppelin GmbH was founded in Friedrichshafen in 1908. Alfred Colsman, a Westphalian industrialist and son-in-law of Carl Berg, was appointed the executive manager. From this time forward, Count Zeppelin steadily retired from the business.[54]

In autumn 1909, the Deutschen Luftschiffahrts-Aktiengesellschaft (DELAG) was founded to establish the first scheduled air transport service. Orders for ships from DELAG and increasing orders from the military ensured Luftschiffbau Zeppelin's future existence.[55] During World War I, Zeppelin airships did not perform spectacularly but their psychological impact on Britain was effective.

Count Zeppelin lived to see the mass production of 'his' airships during World War I, though he did not live to see their temporary end as a result of the Treaty of Versailles, their spectacular re-emergence in the skies as passenger carriers, and their fiery end at Lakehurst, New Jersey when the Hindenburg exploded in flames on 06 May 1937. Count Ferdinand von Zeppelin died 08 March 1917 in Berlin. He is buried in the Pragfriedhof Cemetery in Stuttgart.[56]

4.4.3 The Source of Zeppelin's Motivation and Success

Count Zeppelin could well appreciate the military application of dirigible flight after having participated in the Franco-Prussian War and seen observer balloons used in the American Civil War. In fact, one of the most significant events that made Zeppelin think it imperative that Germany have its own dirigibles was the successful flight of Renard's *La France*, which flew around the Eiffel Tower in 1886.[57] Zeppelin rightly realized that it would strategic folly to ignore the emergence of such a radical technology in neighboring France given the history between the two countries. Zeppelin is recorded as having used the flight of *La France* to encourage the Prussian government to fund his work.

[53] University of Konstanz, *Pictures*, p3.
[54] University of Konstanz, *Pictures*, p3.
[55] University of Konstanz, *Pictures*, p3.
[56] University of Konstanz, *Pictures*, p3.
[57] Eckener, p158.

Undoubtedly, Zeppelin being a career soldier and having fought the French would have recognized the military applications of his invention but I do not believe this purpose overwhelmed his pursuit. The evidence I have seen indicates that he was more motivated by the creation of a commercially viable airship that could "cross the Atlantic in four days." Supporting this point of view is a well known speech he gave in 1914 to the Association of Airship Builders in which he, in the midst of a restless Germany gearing up for war, makes no mention of the airship's military value, only saying in passing that the flight of *La France* did indeed make him believe that Germany was falling behind the French in military technology. In that same speech, Zeppelin makes quite clear that his chief motivation was for civilian and commercial purposes. Zeppelin records an interest in dirigible flight after a lecture given in 1874 by the Postmaster-General on the possibility of transporting the mail by air. This lecture made Zeppelin dream not of military hardware but of passenger and cargo transport. Indeed, while Zeppelin sought funding from the military in the early years, a logical source of funding for this radical new technology, the Zeppelin airship company started its life and was primarily funded as a passenger carrying enterprise.[58]

Zeppelin's success is primarily attributable to his perseverance (i.e. stubbornness) and his managerial skills, though it was in no small part due also to his political and financial influence inherited though social position. In a letter to the chairman of the Daimler motor works, Zeppelin wrote,

> You appear to have less confidence than when I was first permitted to broach my plans to you. That is natural enough and does not surprise me. Once engaged upon the matter, you are influenced by the unfavorable opinion generally passed upon the plan of building airships, by the reports of how hundreds of its devotees, among them highly-skilled technicians, have failed and how millions of money have been swallowed up in it. How then can Count Zeppelin have solved the problem? I have solved it, not because I knew any more than my rivals, but by simple, sober thinking of a serious man, whom nature has endowed with common-sense; and by coordinating already established data on the subject, while making use of the latest discoveries and inventions.[59]

Eckener explains that the fact that Zeppelin made some technical mistakes in his work is beside the point. Zeppelin was not a trained engineer and had to gradually learn his subject. Eckener writes that Zeppelin always spoke "very humbly of his preliminary training and of his natural aptitude for the task he undertook."[60] I think the key to his success was his ability to recruit and delegate responsibility to a talented team of engineers. Zeppelin, though considered a difficult man, seems to have been a fair and just boss, giving credit where it is due. In a letter to General von Lindequist dated 29 December 1893, Zeppelin wrote,

> The machine, which we have the honour to submit to you to-day, is my own invention only so far as concerns its fundamentals. The whole merit for its completion, which can challenge comparison with such modern engineering

[58] Eckener, p156-157.
[59] Eckener, p163-164.
[60] Eckener, p163.

> problems as the Channel tunnel, the Eiffel Tower, etc., belongs to my engineer, Herr Kober.[61]

My research shows that Zeppelin's respect for his employees never abated. Indeed, when there were a series of unfortunate mishaps with LZ-2 through LZ-6, Zeppelin never placed blame on anyone when the accident was caused by inexperience.[62] Zeppelin well understood that the task of creating a dirigible rigid airship was fraught with unknowns. Over the course of his tenure at Luftschiffbau Zeppelin, Count Zeppelin oversaw the creation of many new companies, all founded to support the primary task of building an airship. Zeppelin rewarded loyal employees, such as Claude Dornier, with managerial responsibility and, ultimately, ownership of one of these companies, an airplane manufactory that later took Dornier's name. For workers on the lower rungs of the organization, Zeppelin built a well-designed housing community that his workers could afford to live in. Today, these homes are much sought after in the city of Friedrichshafen, where Luftschiffbau Zeppelin made its headquarters.

4.4.4 Kernel of the Idea

First Thoughts

Count Ferdinand von Zeppelin was obliged to leave the military, where he made his career, in 1890, at the age of 52. For the next ten years Zeppelin committed himself to the construction of a rigid airship. At the time, there was no precedent for such an airship as all airships then being built were of the non- or semi-rigid type. In this section I will explore the evidence of why Zeppelin became interested in dirigible transportation and how his idea grew.

Zeppelin's first recorded experience with lighter-than-air craft was in 1863 when he ascended in the free balloon of Professor Steiner in St. Paul, Minnesota after his study of military strategies in the American Civil War. This experience is not known to have been a significant contributing factor to his quest to build a dirigible airship.

According to his diary, Zeppelin was first inspired by the possibilities of dirigible flight by a lecture held in Strasburg by the Postmaster General, Heinrich von Stephan in 1874. The lecture, titled *Weltpost und Luftschiffahrt*, led Zeppelin to make an entry in his diary that indicates that he reflected for the first time on the problems of airship construction.[63]

Interestingly, the Postmaster General's lecture was given just one year after a Frenchman, Joseph Spiess, published a proposal for a rigid-type airship in 1873 but was unable to secure funding for its construction.[64] I do not know what information Spiess published or Stephan presented in his lecture so I do not know the extent to which the ideas recorded by Zeppelin in his diary are his own. I think we can conclude that Zeppelin's ideas were not formed in a vacuum and that he was, as a minimum, building off the ideas of Spiess, Stephan and other balloon developments at the time.

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[61] Eckener, p187.
[62] Eckener, p266-267.
[63] University of Konstanz, *Pictures*, p2.
[64] Hartcup, p89.

In this diary entry about *Ballonfahrzeug*, Zeppelin envisioned the possibility of transporting the post, freight and passengers, perhaps directly reflecting the content of the Postmaster General's lecture.[65] In a subsequent diary entry dated 25 March 1874, Zeppelin recorded his "Thoughts about an Airship." Zeppelin wrote,

> The machine must have the dimensions of a big ship. The gas-chambers so calculated as to carry the machine except for a slight overweight. Elevation will then be obtained by starting the engine, which will drive the machine, as it were, towards the upward-pointed wings. Arriving at the desired height, the wings will tend to flatten out so that the airship remains on the horizontal plane. To drop, the wings will be flattened out still further or the speed be reduced… The gas-chambers should, whenever possible, be divided into cells, which can be filled and emptied separately. The engine must always be able to replace gas… Parachutes, if they can be used at all, could be attached as part of the ceiling of the passenger compartment and be detachable by the application to them of a person's weight.[66]

This entry shows an early understanding on Zeppelin's part the necessity for size to overcome the problem of creating sufficient lift capacity when using a rigid-type airframe. His description about maneuvering the ship is perhaps simplistic but also quite visionary – he could easily be describing the future design of submarines nearly three decades before their time. His thoughts about the gas cells and parachutes reveal that he was concerned about operational and safety issues. Where Zeppelin lacks technical competence in this early conceptual idea he makes up for with a sound appreciation for the operational and technological complexity of the problems to be faced to create such a machine.

From this diary entry, Eckener concludes that Zeppelin had sketched out the global idea of his airship in 1875. Zeppelin had determined that the dimensions should be that of a "big ship", the ship must be dirigible, and the gas container should be divided into multiple cells.[67] How much of this idea is really that of the Postmaster General's is neither clear nor addressed by Eckener or any other source I have referenced.

Zeppelin made another diary entry about airships 04 April 1875 in which he wrote,

> I should like in advance to point out a not unimportant change that aviation will effect in transport. The manufacturing cost of airships will, it is true, be high, but running expenses and maintenance very small. At first, therefore, aviation will be, partly, a luxury sport, and, partly, will only be introduced where land or water communications between two points are especially difficult and call for permanent easy, safe, and rapid means of transit (by air). [68]

This entry demonstrates that Zeppelin has a clear appreciation for the economics of such an endeavor as constructing airships. This entry also shows his interest in civilian applications for the airship, though it is suspected that Eckener may well have omitted Zeppelin's more militaristic thoughts out of his interest during the inter-war years to protect the Zeppelin Company from dissolution and to promote the Zeppelin airship as a benign product eminently qualified for civilian applications.

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[65] Luftschiffbau Zeppelin…, p1.
[66] Eckener, p156-157. Translated quote from Zeppelin's personal diary.
[67] Eckener, p157.
[68] Eckener, p157.

In the years following his April 1875 entry, Zeppelin made only a few vague references to the airship problem, showing that his mind continued to be occupied by the idea. On 29 November 1877, Zeppelin writes, "Should not ascent and descent be operated by two screws on a vertical axis? The wings (for dynamic movement) could then be dispensed with." And on 09 June 1878, Zeppelin lists specific material and treatment details when he describes making the "balloon envelopes of Chinese silk, very light and, if varnished, almost entirely gasproof."[69] These entries show the Count preoccupied with the issue of keeping the airship airborne and how the airship would be maneuvered. The infancy of aeronautics is revealed by Zeppelin's use of the word *screws*, a distinctly marine term, for *propellers*. Zeppelin is also moving beyond conception and beginning to solve detail problems such as how to make a leak-proof container for the gas.

La France

In the autumn of 1884, two Frenchmen, Captains Renard and Krebs, piloted their non-rigid airship *La France*, successfully completing a short circular flight. Though there is not record in his diary, Zeppelin said in later years that this event made a great impression upon him. In the following years, Zeppelin continued to think about the idea of an airship. In October 1886 Zeppelin made a diary entry in which he first describes, according to Eckener, the possible political and military applications of the airship. Zeppelin wrote, "With favorable wind dirigibles could be used for inter-communication between countries in Central Africa or for rationing armies in the field…."[70] In May 1887, Count Zeppelin sent a memorandum to the King of Württemberg entitled *Notwendigkeit der Lenkballone*. In this document Zeppelin said that the deficiencies of the tethered balloon had convinced the war ministries in countries such as France that only dirigible balloons could constitute a serious factor in war. Zeppelin stated that Germany was behind France, citing the success of *La France* as an example. Zeppelin therefore suggested that the only way to make dirigible flight practical for military purposes was to build airships with very large capacity.[71]

Zeppelin Commits to Building an Airship after his Resignation from the Military

In 1890, Count Zeppelin was forced to resign from his career in the military. Henry Cord Meyer records in detail how this event must have significantly affected Zeppelin's determination to prove his worth to his country and to himself by committing himself fully and uncompromisingly to making Germany a leader in aeronautics.[72]

In 1890, Count Ferdinand von Zeppelin embarked on an ambitious mission. Zeppelin was convinced that the only practical airship was a necessarily large one if it were to have any useful lifting capacity at all. Zeppelin's approach was one of "all or nothing." I have found no record that Zeppelin ever intended to make any small working models. Rather, Zeppelin intended to directly build an airship with an airframe of unprecedented scale and lightness for any type of structure. Not only would his first airship be nearly half the height of the Eiffel Tower in length, it would float and move through the air. Eckener explains that the source of Count Zeppelin's strength and confidence came from his unswerving belief in the soundness of his idea. "With the optimism of all inventors, [Zeppelin] firmly believed in the advancement

[69] Eckener, p158.
[70] Eckener, p158.
[71] Eckener, p158.
[72] Meyer[1].

of technique and in its ability to solve all the problems set by his ship…. He was convinced that his own airship scheme was the best. 'If airships are possible at all,' [Zeppelin] wrote in his diary at the time, 'then mine is possible!'"[73]

4.5 Zeppelin, 1891 to 1899: Conception to Construction

4.5.1 Engineer Theodor Gross, 1891: Materials Tests and other Research

Zeppelin's fundamental idea to build a rigid framework with gas cells inside was criticized by both aeronauts and engineers. The aeronauts criticized the fragility of the structure because it would be susceptible to damage when landing.[74] This criticism was made with good reason, but to Zeppelin this was not a critical question. Zeppelin believed that technology could overcome such problems. He was not going to stop his endeavor before he first determined how to first build a rigid airship that could fly dirigibly.

The engineers whom Zeppelin consulted were of the opinion that a rigid ship, if built strongly enough, would be too heavy to rise, and, if built lightly enough to rise, would not be sufficiently strong. In response to this criticism, Zeppelin commenced a period of intense study and experimentation to answer many of the technological questions that his idea had given rise to. In early 1891, Zeppelin hired Theodor Gross, an engineer recommended to him by the Daimler Motor Works, to assist with this research.[75]

Gross conducted a technical review of the technologies necessary to build the airship. This review included making materials tests. Zeppelin wrote in his diary, "All kinds of materials for the frame-work, for the outer cover and for the gas-cells were tested in respect of weight and stability."[76] I have not found what kinds of tests Gross did but he gave a lecture in 1914 in which he states that they had considered wood, steel and aluminum.[77] Leigh Farell, who translated Hugh Eckener's biography of Zeppelin into English, incorrectly translated *stability* for *stiffness.*

In addition to materials tests, Zeppelin had engine technologies of the time assessed for weight, and consumption of fuel and water. He tried to have all of these values reduced as far as possible by working with the engine manufacturers. Zeppelin had air propellers tested using a boat because no one knew how effective they would be in providing thrust in the air. Supplies of hydrogen gas were not pure enough and Zeppelin attempted to obtain better and lighter gas from the existing works. Gross and Zeppelin addressed many questions at this time, receiving help from numerous firms and private persons.[78] The shear number of unknowns and technological problems must have been daunting. There was little or no precedent.

[73] Eckener, p162-163.
[74] Eckener, p165.
[75] Eckener, p165-166.
[76] Eckener, p166.
[77] Zeppelin[2], p1.
[78] Eckener, p166.

4.5.2 Confidence, Despair, Confidence Restored

In June 1891, Zeppelin, encouraged by knowledge acquired to date, wrote the King of Württemberg's principal private secretary. He said,

> In view of the great interest His Majesty has taken for some years in the progress of aviation, I have the honour to ask you to inform His Majesty that I am intending shortly to build airships, which I am satisfied will be dirigible even in strong wind. Passengers and goods will be carried in special vehicles attached to the machine. My airships are designed to facilitate rapid transport independent of terrestrial obstacles, and will therefore be of the first importance for military and, especially, naval purposes.[79]

A few days after writing the King of Württemberg in June 1891, the Zeppelin wrote to Count Alfred von Schlieffen, the Prussian army's Chief of the General Staff. He said,

> My machines, carrying fuel for twelve hours and having a device which allows of uniform gas expansion under different air-pressures, without loss of gas, will be able to make quick long flights. They will not… fly alone, but have carriages attached to them for passengers and freight. Their load is reckoned for the present at 500 kg each. The increase in air-resistance is slight, so that it should be possible to build a fairly long.[80]

Zeppelin requested that von Schlieffen send an expert to Stuttgart to review his designs and estimates. Schlieffen ordered Captain von Tschudi, commander of the Prussian airship service, to contact Zeppelin. However, one day after Zeppelin sent his letter he decided to give up after realizing that his calculations for the air resistance on the ship where greatly underestimated. He calculated the air resistance based on the mistaken assumption that the cross-section of the airship was the primary component of air resistance and that the ship's length is a secondary factor.[81] Eckener writes,

> At that time, aerodynamic experiments and data hardly existed. Now we know that the skin resistance of the whole machine amounts too much more than its head resistance. This knowledge came to Zeppelin through his contacts with Herr [August] Riedinger, a well-known balloon manufacturer of Augsburg, and with Herren [Hans Bartsch] von Sigsfeld and [August] von Parseval, both expert balloonists. These gentlemen had no belief in the dirigible balloon, convinced the future of dirigible travel lied with the aeroplane. They tried to convince Zeppelin of this and, combined with Zeppelin's knowledge of the severe limitations of engine technology at the time, nearly put a halt to his endeavor.[82]

Zeppelin reconsidered his position with two thoughts. He believed that the airship would be successful because it would be able to freely take off and land anywhere. At the time, von Riedinger was thinking of catapulting his airplane into the air with a sling shot device to overcome the thrust limitations of the engines then available. Zeppelin had observed that this would mean that the airplane could only take off from locations equipped with this device. Secondly, Zeppelin had received information that von Sigsfeld was producing twenty horse-power gas engines that weighted only 110 kg, or 5.5 kg/horse-power, which was very

[79] Eckener, p167.
[80] Eckener, p168.
[81] Eckener, p168-169.
[82] Eckener, p170. Von Parseval later became a successful builder of non- and semi-rigid airships.

light for the time. Zeppelin thought he could simply use four such engines to produce the eighty horsepower that was estimated to be necessary to make his airship design dirigible.[83] With von Tschudi's encouragement and offer of assistance, Zeppelin decided to continue.[84]

4.5.3 Engine Power

It turned out that Zeppelin's information about the characteristics Sigsfeld's new engine came from an unreliable source. Von Tschudi told Zeppelin that the weights mentioned for Sigsfeld's engine were too low. Undiscouraged, Zeppelin turned to the Daimler works to develop the required engine. Zeppelin was motivated by nationalistic impulses to make sure that Germans did not fall technologically behind the French. Zeppelin had read a recent article that described tests carried out in Paris with a dirigible balloon that featured a new high-power, low-weight engine.[85] After reading the article, Zeppelin wrote to Gross,

> You may have seen from the newspapers that the French military air service has been experimenting before the Minister of War at Chalais-Meudon with a dirigible airship driven by a small but very powerful engine. You will realise how we Germans must hurry up, if we are not to be left behind…. [Max von] Duttenhofer must use his influence with Daimler, to get a move on.[86]

4.5.4 Disorganization, Termination of Theodor Gross

Zeppelin had reassured himself that his was the correct path. However, his momentary lack of confidence put his organization into disarray. Duttenhofer, who had been Zeppelin's closest ally, threatened to withdraw his support, which I assume was partially financial, and wrote to Gross that Zeppelin was being over optimistic about the ability to develop adequate motor power and should not therefore spend any large sums of money. Gross did not apparently answer this letter firmly enough for Zeppelin's liking and was dismissed.[87] About Gross' dismissal, Zeppelin wrote Duttenhoffer saying: "In view of the natural prejudice against me as a layman it is doubly necessary that my engineers should support me, instead of being, as Herr Gross often is, an obstacle in my path…."[88] Eckener observed that Zeppelin could not afford any room for "safety first" men."[89] Which means that Zeppelin needed people surrounding him who believed in the fundamental soundness of his ideas and to pursue solutions, not complain about problems. There were too many problems to resolve to become thwarted by any one of them.

4.5.5 Principle Design Elements Determined, 1891

1891 was a productive year for Zeppelin. By the end of the year, even though his organization was in disarray, the principle elements of Zeppelin's airship had been determined. The basic idea was that of a rigid framework with gas cells contained within. The rigid framework would be an aluminum structure, giving solid form to the whole ship and

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[83] Eckener, p171-172.
[84] Eckener, p173. The Airship Service (later the Airship Battalion) later opposed and discouraged Zeppelin's efforts.
[85] Eckener, p174.
[86] Eckener, p175. Duttenhofer was an industrialist and early supporter of Zeppelin.
[87] Eckener, p176-177.
[88] Eckener, p177.
[89] Eckener, p177.

providing a platform from which such things as control surfaces, engines and the gondola could be attached. This allowed the loads on the ship to be better distributed than if they were supported by a non-rigid envelope like a free balloon or non-rigid, or blimp type, airship.

The structure would be covered in fabric that to function as an aerodynamic envelope and to protect the gas bags. There would be numerous gas cells, providing some redundancy in the system should a gas cell leak. The gas cells would be free to expand and contract, “breathe” as Zeppelin described, obviating the need for ballonets as the gas pressure changed during ascent, descent and varying weather conditions. A ballonet is a bag that is open to the atmosphere and with which the pressure inside the gas container is regulated. Ballonets were previously developed for non-rigid airships. Characteristic of the ballonet system is the need to take-off with full gas cells and then release the gas when reaching altitude, thus wasting precious lifting capacity and significantly limiting their flight endurance. The Zeppelin system did not have this problem except at much higher pressure-heights than could be achieved by non- or semi-rigids.

Zeppelin proposed that the ship could be enlarged as demand required by extending the frame and adding gas cells. The idea of a modular system indicates a high level of systems thinking and the preliminary technological reviews conducted by Zeppelin support this view. Zeppelin saw the entire ship as a system composed of sub-systems. I think this approach contributed greatly to Zeppelin’s ultimate success. In this way, each problem could be treated individually while retaining perspective of the whole.

4.5.6 Why Zeppelin Chose Aluminum

By the time Count Zeppelin made a decision on what material he was going to use he had already conceived the global form of the airframe structure. The basis of his idea lies in the simple arrangement of transversal ring girders connected by longitudinal girders. Based on this scheme, Count Zeppelin could evaluate the materials available to him. Zeppelin says he considered using wood, steel or iron. These materials were tested in Stuttgart under the direction of then chief engineer Theodor Gross. Based on the results of these tests, Zeppelin decided to use aluminum for its combination of lightness and strength. Wood was considered too difficult to work even though it had the same stiffness as aluminum. Steel sheet was too thin for the required strength. When the sheet was thickened for stiffness the material was too heavy.[90]

I have no evidence to suggest that Zeppelin considered both plywood and normal wood. Plywood would have been discounted because it performed poorly in adverse conditions due to there not being a suitably waterproof adhesive. By 1891, plywood had earned a reputation for poor quality, subject to rot and delamination. Without plywood, unmodified wood products would probably have been too heavy. The biggest problem with wood is the fabrication of

[90] Zeppelin[2], p1. “Ob ich aus Holz, Stahl oder Aluminium baen sollte, war nicht leicht zu entscheiden. Ertst nach langen Versuchen unter der Oberleitung des Professors, jetzigen Staatsrats von Bach in Stuttgart, entschloß ich mich für Aluminium als Baumaterial, welches sich bei gleicher Festigkeit nicht unerheblich leichter als Holz, besonders wie man es damals anzuwenden wußte, erwies. Von dem Stahl wurde abgesehen, weil für die notwendige Festigkeit shon zu nünne und deswegen an den Kanten nicht anfaßbare und nicht fest zu nietende Bleche ausgericht haben würden, die Verwendung stärkerer Bleche aber wieder zu großes Gewicht ergeben hätte.”

adequate connections, particularly joints subject to tension. Complicated joints would have been necessary that would have resulted in weight penalties.

4.5.7 Theodor Kober, 1892-1900

In March 1892, Ferdinand von Zeppelin published his airship idea under the title *Denkschrift über das lenkbare Luftschiff*, which he distributed to various authorities. Zeppelin hired a young engineer named Theodor Kober to further design and construct his airship. Kober began work in May 1892.[91]

Not dissuaded by the loss of Duttenhofer's support and Gross' dismissal, Zeppelin had already begun to make arrangements with various manufactures to fabricate parts for his airship. For tubing Zeppelin contacted the Mannesmann-Röhre works. Interestingly, Zeppelin had contacted the aluminum works at Neuhausen, Switzerland to produce the aluminum parts. The Neuhausen works, whose dynamos were powered by the largest waterfall in Europe, was then the largest aluminum producer in the world. Unfortunately, I have not found any more information about the arrangements made between Zeppelin and Neuhausen. At the time, Neuhausen was chiefly producing aluminum-bronze, a bronze alloy with only 10% aluminum, which would have been too heavy for Zeppelin's purposes. I do not know if Neuhausen was equipped to produce pure aluminum or some aluminum based alloy. Zeppelin also contacted a British firm of exhauster manufacturers to supply the best flywheels.

When Kober entered Zeppelin's service he began careful testing and detailed design of Zeppelin's airship. The aluminum airframe was to be composed of transversal, cable-braced rings and longitudinal members. This arrangement is characteristic of all Zeppelin's built to the end of the airship era at the end of the 1930s, though the system was greatly refined and improved over time.[92]

Zeppelin and Kober made significant progress in addressing the multitude of technological problems posed by the airship. Under Kober's direction, materials were tested at the Royal Material's Testing Office at Stuttgart.[93]

Research from England about the resistance of moving surfaces was used to justify the scale of Zeppelin's ship. This research showed that resistance does not increase in proportion to size, whereas the area of the circle increases in proportion to the square of its radius. Zeppelin concluded that if an airship is given a cylindrical form, its carrying capacity increases far more quickly than the resistance of the air to its forward momentum.[94] While these conclusions are not technically accurate, it is true that the airships size and cargo carrying capacity can be increased faster than the resistance, thus supporting Zeppelin's contention that only very large airships are practical.

[91] Eckener, p178.
[92] Eckener, p182.
[93] Eckener, p187-188.
[94] Eckener, p179-180.

For engines, Zeppelin was then using a 30 horsepower model that weighted 2250 kg, or 75 kg per horsepower. Experiments showed that the flywheels applied 70% of the engine's power to the propeller shafts.[95]

4.5.8 Zeppelin's Management Style

In a letter to General von Lindequist describing his progress and asking for support, Zeppelin wrote,

> Calculations and drawings as well as various tests of machinery and material have been made by skilled and experienced engineers in my service. The machine, which we have the honour to submit to you to-day, is my own invention only so far as concerns its fundamentals. The whole merit for its completion, which can challenge comparison with such modern engineering problems as the Channel tunnel, the Eiffel Tower, etc., belongs to my engineer, Herr Kober.[96]

This passage reveals Zeppelin's management style. He gives credit where credit is due and is probably the single most important reason why those who worked for him were so loyal and committed to the success of his enterprise.

4.5.9 Zeppelin Requests a Committee be formed to Review his Design

By 1893, Zeppelin felt confident enough with the progress made on the design of the airship to submit it to review by a committee of military experts from the Prussian Airship Service. However, it took until 10 March 1894 for a committee to be finally convened. The organization of the committee took so long because there was great reluctance to give Zeppelin, considered an amateur by the so-called experts, the benefit of credibility that formation such a committee would bestow on his idea. Eckener explains that if it were not for Zeppelin's social position, character and military record, he would never had been able to get a committee organized at all.[97]

A review of this committee meeting is recorded below. Chronologically, it is important to review a related history that fits into the period between Zeppelin hiring Kober and the convening of the committee. That story is about David Schwarz, who designed and built an aluminum, rigid-type airship before Zeppelin.

4.5.10 David Schwarz, a Curious Footnote to the Zeppelin Achievement

The story of David Schwarz and his aluminum rigid-type airship is one that seems to be treated as a footnote to the better known and abundantly written about the development of Count Ferdinand von Zeppelin's giant airships. Zeppelin's airships of course gained infamy for being the first passenger carrying aircraft in the world, introducing aerial bombardment of civilian population centers, and the Hindenburg's fiery destruction that effectively ended the great era of airships. David Schwartz, on the other hand, has the distinction of being the first person to actually build any rigid airship, which happened to have been constructed of aluminum as well. While David Schwartz' efforts ended in failure, his ship had successfully

[95] Eckener, p180.
[96] Eckener, p188.
[97] Eckener, p182-184.

lifted of the ground, proving to the skeptics, well before Zeppelin's first ship flew, that it was indeed possible to build a rigid type lighter-than-air vehicle.

This airship, designed and constructed by David Schwarz, in collaboration with the German aluminum producer Carl Berg, was the first rigid-type airship to be built. Prior to Schwarz, only the proposals made in the early 1870s by the Frenchman Spiess and the German Postmaster General Stephen as well as two documents written by Count Zeppelin prior to 1892 offer any evidence of the development of a rigid-type airship.

The story of Schwartz and Zeppelin is curiously intertwined but it appears only to have been seriously addressed by Cvi Rotem[98], who was interested in recounting the biography of David Schwarz so that his life and contribution to the history of airships is not forgotten. I have not read Rotem's text, which is in German, but I have been able to determine fairly well how Schwarz' work may or may not have influenced Zeppelin's development from a review of Rotem's book written by airship historian Douglas H. Robinson and from other sources.

Everyone involved in airship history knows that both Schwarz and Zeppelin worked with the same aluminum manufacturer, Carl Berg. Normally, historical accounts start at the point when Schwarz' second ship crashed in 1897 and Zeppelin signed a contract with Berg to build his own design just months afterward. Ever since, there has been a debate on whether or not Zeppelin stole Schwarz' design, but always with the implied understanding that Zeppelin would have gained knowledge of Schwarz' design only after teaming up with Berg. In fact, the historical record shows that it is possible that, while Zeppelin and Schwarz may have never had any direct contacts with one another, their simultaneous development of the aluminum airship led to numerous opportunities when Schwarz' activities in particular may have indirectly affected Zeppelin's progress. In the following I will trace this history and examine what Zeppelin may or may not have learned from Schwarz' work, and when, in the context of Zeppelin's own work that was advanced with the aid of two engineers named Gross and Kober. I will also show how Schwarz may very well have indirectly helped those opposed to Zeppelin and his idea to frustrate and slow down Zeppelin's work.

4.5.11 The Independent Developments of Zeppelin and Schwarz

Zeppelin first recorded his interest in the possibility of dirigible technology as early as 1874. He only began intensive development in 1890 after his "early" retirement from military service, evidently forced upon him by petty politics.[99]

The earliest date I have found indicating when Schwarz began his work is 2 May 1892; when the Russian War Ministry agreed to allow Schwartz to build his craft at St. Petersburg, at Schwartz's own expense. Before that, the Austro-Hungarian War Ministry rejected Schwarz' proposal out of hand when he approached the ministry for funding.[100] Allowing for some time to lapse between the time he approached the Austro-Hungarian War Ministry and the Russians as well as some amount of time he spent pondering the problem, Schwartz probably began his work in earnest sometime in early 1891.

[98] Rotem, Cvi. 1983 manuscript, Indiana University Press.
[99] Eckener, p151.
[100] Robinson, p2.

Aluminum was Schwarz' preference from the earliest point that his work could be traced. I have found no evidence indicating why Schwarz chose to use aluminum. Robinson writes,

> While his wife called him an "engineer," he was self-taught and relied heavily on Carl Berg's engineers, particularly M. von Watzesch-Waldbach, who signed the drawings preserved today in the Deutsches Museum. From the beginning Schwarz' design included a rigid, internal framework of aluminum girders, covered with riveted aluminum sheets 0.2 mm. in thickness; a gondola rigidly connected to the gas container; and a gasoline engine driving a least one propeller in the horizontal plane to provide upward or downward thrust.[101]

Schwarz began construction of his first airship around December 1892. Schwarz' aluminum supplier was a Westphalian industrialist, Carl Berg. Schwarz and Berg signed a contract for the Russian ship on August 23, 1893. Schwartz was to provide the ideas and supervise construction and flight trials while Berg would provide the aluminum sheets and girders, and the necessary funding.[102] Douglas Robinson, who reviewed Cvi Rotem's biography on Schwarz, writes that Berg was the *obvious source* for the aluminum structure [emphasis added]. I do not know whether these are Robinson's or Rotem's words but it is not clear why Berg was the obvious choice. How did David Schwarz even make contact with Berg? Schwarz was formerly a timber merchant in what is today Croatia. He was a subject of the Austro-Hungarian Empire. He looked for funding support in Vienna, first with the Austro-Hungarian authorities and then at the Russian consulate. How did he end up in Westphalia? Why not the works at Neuhausen, Switzerland on the Rheinfall, then the largest aluminum producer?

Zeppelin had already made agreements with Neuhausen, thus explaining why, if Schwarz had contacted Neuhausen, he did not work with them. Given the importance of Neuhausen at the time and the fact that there were not that many aluminum producers at the time, it is possible that Schwarz did contact Neuhausen at some point. Neuhausen could have refused his proposals without informing him they were already working with Zeppelin on such a project, but it is possible that contacts in Neuhausen informed Zeppelin that there was another person expressing interest in constructing an airship using aluminum. This is speculation on my part. Nonetheless, the question remains, why Berg?

Cvi Rotem writes that both Schwartz and Berg insisted on secrecy, Schwarz wanting to protect his ideas and Berg then being a contractor to the Prussian War Ministry for aluminum mess kits and canteens – his relationship with Czarist War Ministry might not have been received well by the Prussians.[103] These reasons seem strong enough to believe that the work on Schwarz' first airship was not known in Germany outside of Berg, thus Zeppelin would not have been privy to the development except perhaps to the rumor that someone was indeed seeking to build a rigid airship out of aluminum.

Regardless of the similar time frames in which Zeppelin and Schwarz began their work, it is clear that Zeppelin was unaware of the specifics of Schwarz' work until sometime after

[101] Robinson, p1-2.
[102] Robinson, p1 and 5; Berg[2].
[103] Robinson, p2.

Schwarz obtained a contract to build an airship for the Imperial German Government (or Royal Prussian Government) on December 16, 1894.[104]

4.5.12 Fabrication and Failure of Schwarz' First Airship, 1892-1894

Meanwhile, in 1892, Berg's works were busy producing aluminum castings, profiles, and tubes for girders, as well as metal sheets for the envelope of the Schwarz ship.[105] Having examined the photos and construction plans I have at my disposal, I cannot determine what the tubes were used for. The castings made were probably for various fittings and perhaps the cable anchors and tensioning buckles. I do not know if the cables are aluminum, wrought iron, or steel. I believe it would have been possible at that time to produce aluminum cables, but for pure tension members steel may be more appropriate even considering weight. The sheets were 0.2 mm thick and were riveted to the airframe. Aluminum sheet was also used to clad the gondola but it appears to be a heavier gauge. The girder members are comprised of angles and tees. I presume they were rolled hot and possibly work hardened by cold rolling the finishing passes. It would have been necessary to do frequent annealing if the aluminum was just cold rolled.

On 10 November 1892, Eveking sent the first of seven crates to Vienna, and by 14 December all parts ordered had been delivered to Vienna for a total weight of 1579 kg (3481 lb), including 100 sheets of 0.2 mm. aluminum, and 269,000 rivets weighing 45.4 kg (100 lb).[106]

The ship was completed 22 March 1894. There was 2341 kg (5161 lb) of material in the ship, and the total empty weight was 2525 kg (5566 lb). The hull volume was 3280 m^3 (115,960 ft^3) and the gross lift was 958 kg (2068 lb). The four-cylinder engine weighted 298 kg (657 lb), water and fuel 170 kg (374 lb), for a total of 468 kg (1031 lb). The total weight of equipment and three persons was 385 kg (849 lb), and the excess lift was 85 kg (187 lb). The gondola was 2.6 m (6.5 ft) long and 2.3 m (5.75 ft) wide. While the engine developed 10 hp. at 480-rpm, the propellers, 2.6 m (6.50 ft) in diameter, turned at 250 rpm. The rubber belt drive was regarded as very unreliable and, of course, there was a minor problem with the propeller brackets being too short.[107]

This first Schwarz ship never left the ground. Schwartz, it seems, had always planned to use the aluminum-clad airframe as the primary gas container. Curiously, he did not include a ballonet inside the envelope to allow for expansion and contraction of the lifting gas. At the time, Russian Engineer Kowanko pointed out this omission in the Schwarz design: without a ballonet open to the atmosphere, intolerable pressures would be put on the rigid metal hull with expansion of the gas in ascent and its contraction during descent. Schwartz dismissed this criticism, though it was not his only problem.

The riveted aluminum sheet was not airtight. As a result, filling the airship with gas was a problem, never mind keeping it in! Schwarz had planned to fill bags inside the hull with hydrogen and then, when all were full, release the gas into the hull and remove the bags through small openings in the bottom of the hull. This did not work because the bags

[104] Robinson, p4.
[105] Robinson, p2.
[106] Robinson, p2.
[107] Robinson, p3.

themselves leaked – Schwarz blamed the manufacturer and the manufacturer blamed Schwarz' process. Ultimately, Schwarz decided to just pump the airship full of gas directly. This having been accomplished, the crew went to lunch only to be alerted in the course of eating that the airship was bursting. In fact, the aluminum sheeting was imploding, probably due to a drop in temperature. Futile attempts to correct the problem yielded no positive results and the airship was ultimately abandoned.[108]

4.5.13 Zeppelin's Independent Development

Zeppelin's modular approach to airship construction was distinctly different from that of Schwarz' and was not applicable to the non-rigid and semi-rigid type airships. The idea of a modular system indicates a high level of systems thinking and the preliminary technological reviews conducted by Zeppelin support this view. Zeppelin saw the entire ship as a system composed of sub-systems. I think this approach contributed greatly to Zeppelin's ultimate success. In this way, each problem could be treated individually while retaining perspective of the whole.

Schwarz, on the other hand, did not think through details such as the possibility of damaging the air bags when installing or removing them through small openings. He also miscalculated the length of the propeller brackets, which were too short on the hull to port and starboard, and a pocket or indentation had been built into the metal hull to accommodate the propeller tips![109]

4.5.14 Schwarz' Second Airship, 1894-1897

Carl Berg obtained a contract to build an airship for the Imperial German Government 16 December 1894. Berg presented himself as being the builder and David Schwarz, with whom he signed a contract 21 December 1894, as having provided the ideas.[110]

Engineer Tenzer, of the Airship Battalion, and Carl Berg's engineers von Watzesch-Waldbach and Weisspfennig prepared the drawings (undated) for the German ship under Schwarz' direction.[111] (**Fig. 18**) Schwarz, who had had no formal engineering training, was a thoroughly opinionated person. Despite criticism of the Russian design by the officers of the Czarist War Ministry, the German ship was almost identical with its predecessor. The only significant changes were the addition of a fourth propeller orientated horizontally for ascent and decent, and the use of "Victoria aluminum," an alloy developed by Berg in 1895.[112] I have been unable to find out what the composition of Viktoria aluminum is or what its physical properties are.

Work on the second airship took place at Tempelhof Field, in the hangar of the Prussian Airship Battalion. It is likely that Schwarz became acquainted with officers of the Airship Battalion: Major Nieber, the commanding officer; Major Gross, an opponent of the rigid airship concept; Captain Kussmann; Captain von Tschudi; First Lieutenant von Sigsfeld. It is not clear how much help the officers of the Airship Battalion gave Schwartz. Schwarz, who

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[108] Robinson, p3.
[109] Robinson, p3.
[110] Robinson, p4.
[111] Robinson, p4.
[112] Robinson, p4.

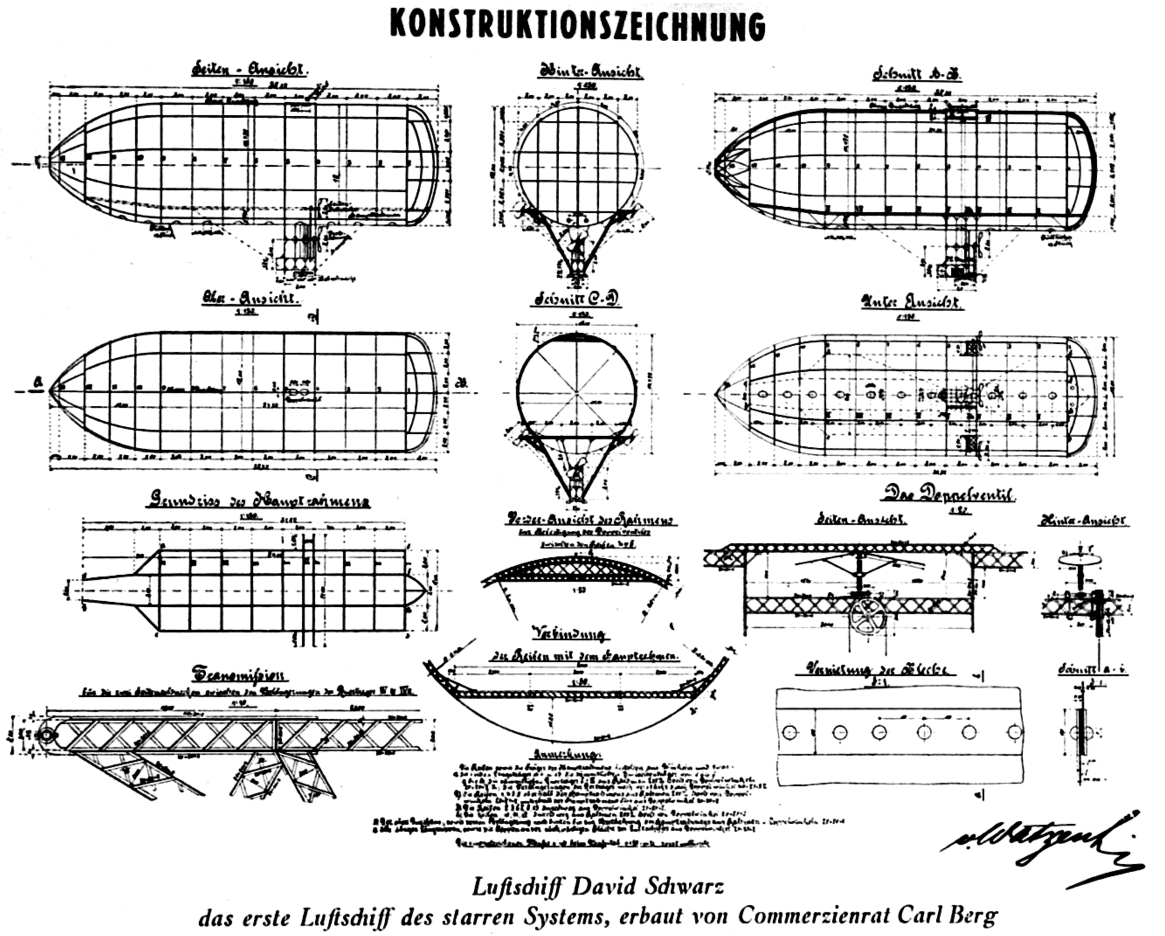

Fig. 18: Plan of David Schwartz's rigid airship, 1895. (Knäusel)

was surely aware of Zeppelin's work by this time, evidently insisted on secrecy and released little technical data.[113]

Berg shipped the first parts from his plant in Eveking to Berlin in July 1895. Schwartz' progress on the ship encountered delay after delay. Schwarz complained of the German workmen: "I have to stand over them with a whip, it was easier to get them to work in Russia." What roles do labor issues play in the design process? What role *should* it play?

4.5.15 First Committee Review of Zeppelin's Airship Design, 4 March 1894

The committee that Zeppelin requested be formed in 1893 to review his airship design finally convened 10 March 1894, and included, among others, H. Müller-Breslau. The commission agreed in Zeppelin's presence that Zeppelin's plans were "worthy of notice… and recommended the Royal Ministry of War to take steps to secure the invention for the Reich." The commission specifically criticized arrangements for landing and anchorage of the airship. Müller-Breslau questioned whether the framework was strong enough to meet lateral resistance. Importantly, Müller-Breslau said at the time that the necessary reinforcement

[113] Robinson, p4.

would not be a very serious matter and should not prevent the ship from rising and keeping in the air. Zeppelin agreed to revise his model in light of Müller-Breslau's criticism.[114]

In June 1895 the committee, reserving final judgment, recommended to the Royal ministry of War that it should guarantee the possibility of further work by granting the minimum funds necessary to provide continuity between the preparatory work, the experiments already carried out and starting with the construction of an airship.[115] This recommendation would be curiously retracted only one month later.

At a meeting held on July 14, to which Count Zeppelin was not invited, the committee unanimously rejected the model, mainly on the grounds of Professor Müller-Breslau's report recommending the need of "further very substantial reinforcement, far exceeding the present buoyancy capacity."[116] The reasons for Müller-Breslau's change of opinion are not evident, however taking events into context I think this situation represents an instance where socio-political influences can have a negative impact upon the development of technology. Zeppelin's request for funding was probably rejected because the Prussians, one, did not want Zeppelin to succeed, and, two, Berg had just shipped the first components for the Schwarz ship to Tempelhof Field. It is possible the powers-that-be determined that they could test rigid-airship technology much sooner than Zeppelin could promise and the scale of the test probably struck them as more reasonable. Furthermore, this could have been a power play by some Prussians to neutralize the Schwabian Zeppelin by supporting the development of a rival. Clearly, any inventor-developer, in addition to creativity and imagination, needs to have a sense of diplomacy and/or an obstinate will to succeed.

I do not believe Müller-Breslau issued his report on purely technical grounds and, even if his findings were made in good faith, his findings were wrong, being based on his knowledge and experience that could not have been equipped to deal with the rigid airship. Surely, Müller-Breslau's familiarity with pioneering development of spatial structures by Johann Wilhelm Schwedler and August Föppl and his own development of advanced analysis methods for truss structures applying the method of virtual displacements and Henneberg's method[117] would have given him some confidence in analyzing Zeppelin's airframe. However, Müller-Breslau's conclusions were ultimately proved wrong. In retrospect, Eckener reports, that Müller-Breslau and other experts were incorrect about their assumptions about wind-pressure – the pressure did not increase in proportion to surface, as they had presumed, but in a much smaller ratio. They were also incorrect on one very important structural factor.[118]

In a letter requesting that the committee be reconvened, Zeppelin wrote,

> Annexed to its letter of August 1st of this year [1895], the Royal Ministry of War communicated to me Professor Müller-Breslau's report on the static conditions governing the dirigible airships I have designed. The faults he finds are

[114] Eckener, p189-190.
[115] Eckener, p190.
[116] Eckener, p191-192.
[117] Timoshenko, p309.
[118] Eckener, p192.

> admitted, if, in judging the airship's stability, only the metal structure itself is considered, and not the added strength derived from gas expansion.[119]

This letter explicitly states that Zeppelin and his engineers recognized that the inflated gas cells stiffened the airframe, though at the time it was not possible to calculate its contribution with any exactitude. Eckener explains that the contribution of the gas bag to the strength of the airframe was "very seriously weighed [as of 1938], but which for several reasons cannot always be determined with mathematical exactitude in the case of a moving ship and which at that time was still further from the mind of an exact expert in statics."[120] This shows the rapidly growing need for more powerful modeling and analytic tools. Existing limits of structural understanding were not adequate to the task of analyzing the airship structures, particularly the air-supported structural behavior of the gasbags. Müller-Breslau did not have the knowledge necessary to make the evaluation he did. Who did? Again, the airship development is characterized by all of the unprecedented problems encountered and ultimately dealt with.

Zeppelin did not regain the committee's support. Apart from the technical and aeronautical objections, the War Ministry argued, "the ratio of cost to utility of such machines is so unfavorable that the military authorities could not be asked to furnish considerable sums for the execution of these designs. Only if the machines were already in service and had proved their value in civil transport could any attempt be made to use them for military purposes." Eckener writes: "A bureaucratic government is always parsimonious and averse to outside plans and schemes; a pioneer and inventor must have faith in progress, he must take risks and not count the pennies."[121]

Supported by what turned out to be inaccurate calculations pertaining to the possible speed attainable by Zeppelin's airship, Zeppelin continued to work. Once a start was made, Count Zeppelin counted upon technical progress to meet all requirements in matters of building material and engine power as time went on.[122] This is the 'Can-Do' attitude that is an integral part of technological thought. Sometimes false hope, information, analysis, or results can encourage a development to go on, exposing the original faults in the project but providing the necessary information to make the requisite corrections to the design. An analysis of the 'Can-Do' attitude would probably reveal that its effectiveness is linked directly to funding. If enough money is thrown at the problem, solutions are found. This is really the basis of the current missile defense program in the United States, a reason for the success of the Manhattan Project, and any number of other wartime developments of technology. However, in this case, it was one man with financial means, who was funding such an ambitious project. What does it take personally to succeed under such conditions?

Coming back now to Schwarz, Berg and the Prussians, it is very likely that some person in Prussia informed Zeppelin of the goings on at Tempelhof Field. This is speculation on my part but Zeppelin spent much time in Prussia as a soldier and as representative of the King of Württemburg. He cannot have made enemies of every person he met. It may be possible that Captain Tschudi passed word to Zeppelin himself. It seems certain Zeppelin would have

[119] Eckener, p193.
[120] Eckener, p193-194.
[121] Eckener, p195.
[122] Eckener, p199.

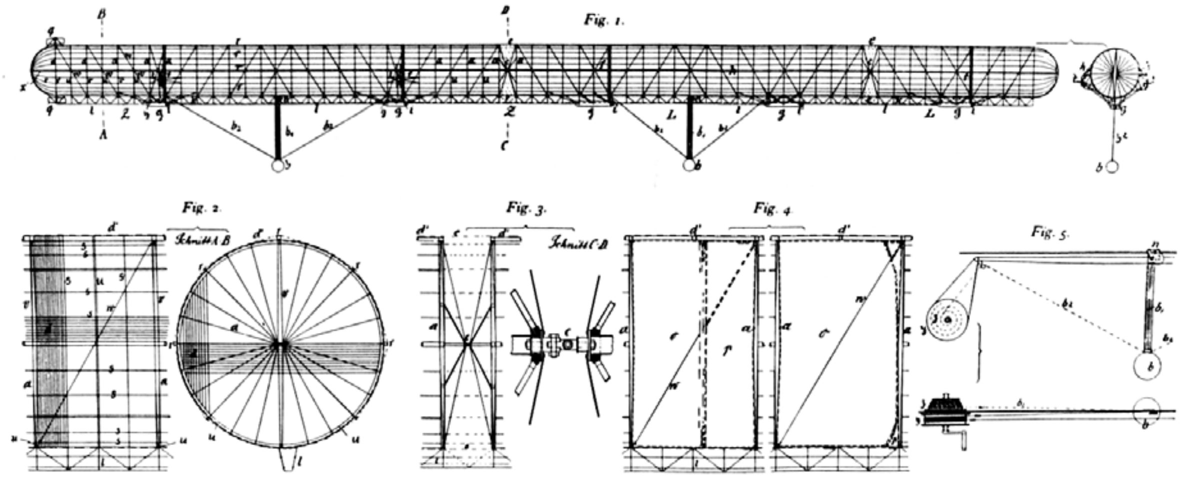

Fig. 19: Zeppelin's 1895 patent drawing. (Knäusel)

tried to obtain information about the Schwarz design. However, I do not think he would have benefited greatly. In fact, Zeppelin probably would have reacted with disdain upon learning that the Schwarz ship was a metal-clad airship operating without independent gas cells or a ballonet to regulate gas pressure – Zeppelin was resolute in his confidence in his own design, it was superior to all.

4.5.16 Zeppelin's 1895 Patent

Zeppelin received a patent in August 1895 for a dirigible airship, or Luftfahrzug.[123] Theodor Kober did the technical planning according to the fundamental ideas outlined by Zeppelin. The timing of this patent conveniently coincides with the activity on Schwarz' second ship in Prussia, though Eckener denies Schwarz' project had any influence upon Zeppelin's work except for the contributions that Zeppelin's later partnership with Berg provided.[124] It is likely that Zeppelin took his patent to protect his own design. Berg had made the contract with the Royal Prussian Government in December of 1894 and his works shipped the first parts to Tempelhof Field in July 1895.

The drawing accompanying the patent shows a regular framework composed of transversal, cable-braced ring girders connected by longitudinal girders. (**Fig. 19**) Diagonal members are included that clearly are conceived to function as tension diagonals meant to both stiffen the airframe and, primarily, to transfer the load of the gondolas and the transfer truss located along the longitudinal axis on the bottom of the airframe to the top of the airframe where the load would be transferred to the supporting gas bags. No circumferential bracing is shown in this drawing, indicating that the Zeppelin and Kober had not incorporated the developments made by Schwedler and Föppl. (**Figs. 28 and 29**) We can also observe in the patent drawing that Zeppelin was still influenced by the ideas presented by the post-master general in 1874 for an air-train. The patent shows that Zeppelin's airships could be linked together to give airships any length. This idea was subsequently abandoned.

[123] Zeppelin[1]
[124] Eckener, p211.

The structural details of the actual construction are not defined in Zeppelin's patent. It is, however, consistent with his verbal descriptions of the airframe that can be date to as early as 1891 or 1892. The differences in design principles between the airships of Zeppelin and Schwartz can be compared in **Figures 18 and 19**. Besides scale, Schwartz' airframe is radically different from Zeppelin's. Schwartz' structural model seems to be that of a tubular system while Zeppelin's is a composite structure composed of the transversal rings that seem influenced by the spoked wheel, and the transversal girders that connected the rings. The general principle seems to be based on a truss, though there does not appear to be sufficient torsional stiffness since the diagonals are in the interior of the airship. Zeppelin and Kober were probably relying on the pressure of the gasbags themselves to brace the airframe.

4.5.17 The Post-Mortem Completion of David Schwarz' Airship, 1897

David Schwarz died suddenly in Vienna 13 January 1897, before completing the second airship. Carl Berg, cynical in the face of delays and evasions, required confirmation of Schwarz' death, even suspecting that he might be fleeing to Paris to sell his "secrets" to the French government.[125]

Schwarz' death nearly meant the end of the project but Berg, having invested so much already, decided to continue the work in an awkward partnership with Schwarz' wife. The Airship Battalion continued to offer its assistance and was instrumental in making final preparations for its test flight and making one significant modification to the airship, the installation of a pressure-relief valve fitted into one of the many small openings provided for on the bottom of the hull to insert and extract the filling bags. Berg and the Airship Battalion completed the second Schwarz ship at Tempelhof field sometime in the autumn of 1897.[126]

4.5.18 Technical Specifications of the Schwarz Airship

The technical specification of the second Schwarz ship were as follows: gas volume 4610 m^3 (162900 ft^3); overall length 38.32 m (26 ft)[127]; the cross section was elliptical, 15.4 m (39 ft) wide and 18.2 m (46 ft) high – indicating the airframe was designed analogous to a tubular beam; and the cross sectional area was 132 m^2 (1420 ft^2). The pointed nose was 14.2 m (36 ft) long, while the parabolic plate at the stern was 3.9 m (10 ft) deep. The 12 hp Daimler engine weighed 508 kg (1113 lb) and had an incandescent platinum tube ignition. There were three pusher propellers 2.6 m (6.5 ft) in diameter, one between the gondola and the hull, and two on the sides of the hull, while a horizontally mounted one under the gondola floor provided vertical thrust.[128]

Schwarz' ship was all aluminum, constructed of a rigid aluminum framework around which an envelop of aluminum sheet 0.2 mm thick was riveted.[129] It is not fully known why Schwarz insisted on using aluminum sheet but later developments of airship technology considered the benefits of metallic envelopes as being more resistant to shear and torsional effects.

[125] Robinson, p5.
[126] Robinson, p5.
[127] Berg[2], p19.
[128] Robinson, p4.
[129] Robinson, p4.

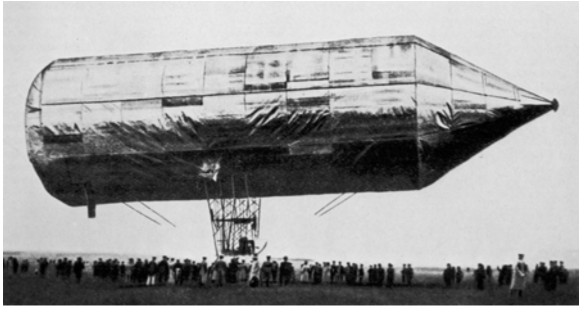

Fig. 20: Flight of the Schwartz airship. Notice the tension fields in the aluminum sheet covering the hull. (Knäusel)

From reviewing the construction drawings and a number of photos of the Schwarz ship it is evident that the airframe is indeed designed as a tube, somewhat like the Britannia Bridge, with a heavier compression section at the top of the hull. (**Fig. 18; Appendix A-03, p.A.77, Fig. 1**) The lower flange of the tube is not in the extreme curved surface of the vertically oriented elliptical section but higher, forming a horizontal chord through the oval cross section, about 1.5 m. from the lowest point of the oval.

Construction drawings show that only six stabilizing cables stabilize the transversal ring structure. The cables are oriented in a simple radial arrangement. Cables transferred the weight of the gondola to the lower chord of the tubular airframe.

The shear strength of the ship is questionable. Diagonal tension fields can be seen in the middle panels of the envelope in a picture depicting the 1897 trial flight. The drawings do not show any diagonally oriented cables around the circumference of the airframe, such as in Zeppelin's ship. This means that the 0.2 mm aluminum sheeting must take shear stresses as is evidenced by the photo of the ship in flight. (**Fig 20**)

Every major intersection of the airframe, including the gondola support structure, shows evidence of an attempt to make rigid connections. This is particularly strange for the propeller supports because struts rather than ties support the cantilevered brackets. Ties could have reduced valuable weight. Perhaps it was thought that the more rigid looking arrangement would better maintain stability of the belt transmission system used to power the propellers from the engine located in the gondola. Nonetheless, these rigid connections increase weight. Zeppelin opted to use a complex arrangement of stabilizing cables in lieu of struts, which has the added benefit of limiting the transfer of vibrations from the engines to the airframe.

In the 1930s, at the end of the airship saga, the USS Akron and the British R101 both employed rigid transversal ring frames to eliminate the transversal cable arrangements, thus allowing larger gas cells to be used. Eliminating the cables and minimizing the number of gas cell sides reduced weight. Of course, in those cases the scale was enormously different, the Akron having been 310 m (785 ft) long with similar if a somewhat longer length for the R101 compared to Schwarz' 38.3 m (126 ft). While the final Zeppelin airships were comparable in size, they did not take advantage of the better quality gas-cell materials then available that would have allowed them to reduce the number of gas cells if they had wished to. This was partly because Ludwig Dürr, Luftschiffbau Zeppelin's chief design engineer since the completion of LZ-1 in 1900 was extremely conservative. It was actually one of his former engineers who moved to the United States to work for Goodyear-Zeppelin and introduced the rigid frame construction into airships there. He had not been able to convince Dürr to try his ideas. An additional reason for seeking to reduce weight as much as possible in the Akron was that it used helium instead of hydrogen, meaning it had to be relatively lighter than a Zeppelin ship of comparable useful lift. This conservativeness in engineering design is a major influence in how materials, as well as structural systems, are introduced and accepted into wide spread use in the industry.

4.5.19 Schwarz' Second Ship Takes Flight, 1897

In the autumn of 1897, Ernst Jägels, an Airship Battalion mechanic, was commissioned to test fly the Schwarz airship. The Airship Battalion inflated the ship 2 November 1897, using the bag filling method, though there was a lot of leakage. When filled, the bags were removed and all the manholes in the bottom of the ship were closed, except for one, which housed the automatic pressure relief valve. On 3 November 1897, the ship was topped off with gas and Jägels took controls of the craft for its first test flight.[130]

Jägels lifted off at three o'clock in the afternoon. He soon disconnected the horizontal propeller because there was an excess of lift. A tethered flight was evidently intended but the ship broke from the grasp of the ground crew. As the ship rose to 130 m (330 ft) the rubber belt fell off the left propeller sheave, the craft turned broadside to the wind and the forward tether broke free. The ship drifted away, rising rapidly up to 510 m (1300 ft) and the belt fell off the right propeller sheave. With no way of steering the craft back to Tempelhof field Jägels opened the top valve and landed in a convenient open space. Jägels jumped out unhurt as the hull lay over on its side and collapsed. David Schwarz' dream ended in a crumpled mass of aluminum.

Berg, well aware of the faults of the Schwarz "metal box" design, wrote in a letter to a an acquaintance a week after Schwarz' death, "If I could get my money back, I would take part in a project which would promptly attain my goal: I would retain the rigid attachment of the gondola to the hull, also the aluminum framework, but would cover it with silk or other fabric for ease of filling and gas-tightness." He added that he was working on the design and might offer it to the War Ministry. Berg did not have the opportunity to develop his ideas because by 3 November 1897, Count Zeppelin was trying to set up a meeting with Berg in Berlin, and in a letter to Berg of 8 December 1897, the Count agreed that Schwarz' "metal box system" was not capable of development.[131]

4.5.20 Zeppelin's Partnership with Carl Berg and Funding

Zeppelin and Berg subsequently formed a partnership to build Zeppelin's airship design. Berg, under contract to supply aluminum for airships exclusively to the Schwartz undertaking, had to obtain release from this contract by an arrangement with Schwarz' heirs before his firm could deliver aluminum to Count Zeppelin.

To fund their project, Zeppelin had secured the written support of the Union of German Engineers to help him secure investors for the project. The Union of German Engineers issued a statement 21 December 1896, stating,

> Count Zeppelin's model promises, as compared with previous performances by dirigible airships, if not an increased speed, at any rate a substantially longer duration of flight… The successful completion of the design presupposes the solution of certain preparatory questions, the answers to which through

[130] Robinson, p6.
[131] Robinson, p6.

> experimentation are in themselves of importance to the development of aviation that the committee recommends that further steps be taken to realise the scheme. The problem to be solved [is] one for technical science… The manufacture of serviceable aircraft has only quite recently come within the field of the engineer's work. Very many engineers are still indifferent towards and even skeptical about everything pertaining to aviation. Comparatively few have studied improved means of transport by air sufficiently deeply to recognize in it one of the biggest technical problems which our century is bequeathing to the next. Theoretically, it is agreed that natural laws present no obstacle and that existing technical resources are sufficient to meet the static and dynamic requirements of airship construction. In the opinion of distinguished physicists and engineers the difficulties and objections are no greater than those which faced existing technique before the days of modern shipbuilding and railways.
>
> The object of these endeavors is: safe transport in the air, independent of all kinds of roads, at speeds hitherto unattained. Distant as this goal may seem to-day, everyone who thinks it a technical possibility will think it worth much effort and sacrifice. The goal must be approached step by step, in the opinion of eminent experts, which we share, would be the construction of an airship based on the Zeppelin model and on preliminary experimental investigation.
>
> It is out of the question that the substantial sums required for this work should be expended in accordance with purely economic considerations, that is, with a view to immediate financial gain by individuals and companies. For the technical results will no doubt be common property and not such as could be monopolised to the advantage of individual promoters.[132]

This statement succeeded in its effects, later resulting in the formation of a joint stock company for the promotion of air navigation under the management of a number of German industrialists. The Gesellschaft zur Förderung der Luftschiffart was formed in May 1898 with 800,000 Marks in capital, of which Zeppelin had to subscribe almost half because there was so much distrust and so little desire to participate. Among the shareholders were Phillipp Holzman and Carl Berg.[133]

4.5.21 The Influence of David Schwarz

Zeppelin's design did not copy that of Schwartz, but it is a certainty that he learned useful information and gained valuable knowledge through Schwarz' failure and Berg's role in that development. For instance, before 1890, Zeppelin had thought that a horizontal propeller would be necessary for dynamic control of the airship. Jägels test flight may have confirmed that this was not necessary.

In other ways, Schwarz' activity could have had an immediate impact on Zeppelin's work. Schwarz' relationship with the Prussian Airship Battalion could have enabled the Airship Battalion to rebuff Zeppelin even after offering assistance a couple years prior. The unfavorable opinion of the committee that reviewed Zeppelin's design in 1894 any chance for funding at the same time that Berg began shipping parts to Tempelhof field. In fairness, Schwarz' project may be considered to seem more reasonable in scale for such an unprecedented machine and structure. Zeppelin's plans were to build a full-scale ship more

[132] Eckener, p204-209. Excerpts from statement published by the Union of German Engineers (VDI).
[133] Eckener, p209-210.

than three times as large right from the start. This must have seemed, and still does, a daunting task. Nonetheless, Schwartz' failure also served to strengthen the position of those within the Airship Battalion, such as Major Gross, and other persons who were antagonistic towards the rigid airship concept.

Eckener admits "that Count Zeppelin benefited from certain structural experiences of the Berg firm, just as Schwartz' machine did, too. Moreover, Herr Berg did exceedingly valuable work for Count Zeppelin."[134] Eckener never actually gives Berg any credit for specific developments in the design or construction process of Zeppelin's airships or any credit to Schwartz for providing his ideas to Berg.

If there was any knowledge to be gained from the Schwarz experience then Berg and his staff were best positioned to analyze and provide that knowledge. I do not think Berg's engineers were particularly brilliant but they did have the experience of collaborating on the design *and* construction of an actual rigid airship with aluminum airframe. This was a source of knowledge available to Zeppelin and it seems hard to believe he would have ignored it. For all of Zeppelin's experiments, he did not have the experience of actually trying to build an airship.

While I have not found any written evidence so far, I think the following thoughts on Berg's contribution to Zeppelin's design are fair. LZ-1 was evidently designed to use aluminum tubes in the construction of the airframe.[135] Eckener mentions this but does not elaborate on what form or configuration these tubes were to be utilized, or how they would be connected together. LZ-1 was instead built with lattice girders quite similar to those employed by Schwartz except for the important alteration of removing a large number of the lattice panels such that each girder had X's connecting the top and bottom chords only every 1.5 meters or so. (**Fig. 21**) The lattice girder form does not show up on any drawing, including Zeppelin's 1895 patent, before his partnership with Berg. If, as Eckener says, Zeppelin and Engineer Kober had planned on tubes, then it is clear that Berg had a significant part in the decision to employ those lattice girders in LZ-1, even if it was perhaps at the cost of structural performance.

One reason why they may have changed to the lattice girder was difficulties in developing connections for the tubular system originally planned. The lattice system was surely cheaper than tubes and the enterprise had the benefit of Berg's experience fabricating the lattice girders.

Likewise, Berg and his engineers had the experience of actually constructing a rigid airship. This experience must been valuable to the development of construction methods for at least the first two Zeppelin ships, thus establishing the basis for all further developments in construction process. Berg's method for riveting the members together must have been particularly valuable.

Had Schwarz not existed I do not believe Zeppelin's technological developments would have differed significantly based on the historical record I have found. However, if Schwarz had not existed the order of events and the characters involved may have changed greatly. Schwarz gave Berg the distinction of having the only works with experience building rigid

[134] Eckener, p211.
[135] Eckener, p214.

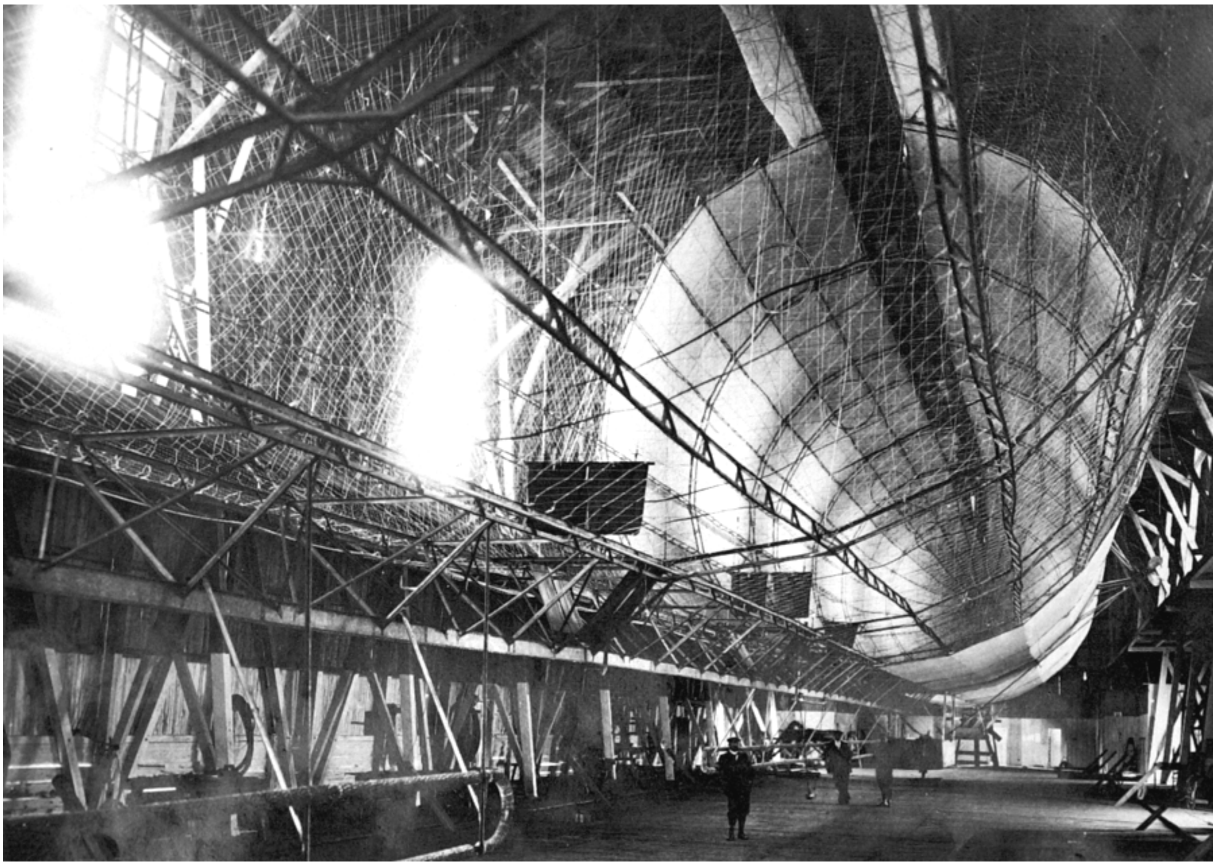

Fig. 21: Airframe of LZ-1. (Knäusel)

airships. Therefore, Schwarz was the seed for a development of knowledge, and while the actual useful knowledge gained may not have been overwhelming, it existed nonetheless, perhaps saving Zeppelin more failures than he had.

4.5.22 Assembly of Zeppelin's First Airship Begins, 1899

In 1898, Hugo Kübler, an engineer, was employed as technical manager. On 17 June 1899, construction was begun on the first ship in a floating hanger on Lake Constance at Manzell, Germany, near Friedrichshafen. Ludwig Dürr, who would later become chief engineer for the Zeppelin airships, was employed as a construction manager.[136] Eckener writes that commencement of the construction of LZ-1 "ended the initial, perhaps most difficult and at times apparently hopeless stage of the whole struggle for [Zeppelin's] idea. Now the work had to speak for itself – as it did. The battle against human distrust and human intrigues was won. There now began instead the fight against the difficulties inherent in inanimate objects."[137]

LZ-1 was completed in the summer of 1900.

[136] Eckener, p210; University of Konstanz, p4; Luftschiffbau Zeppelin GmbH…, p1.
[137] Eckener, p210.

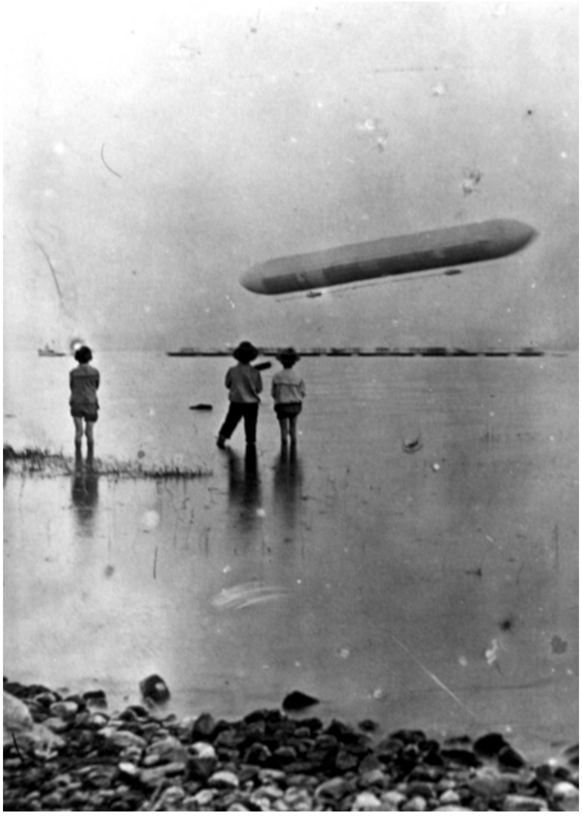

Fig. 22: First flight of LZ-1, July 1900, Lake Constance. (Knäusel)

4.5.23 The First Idea Greatly Shapes all Future Developments

In a speech given in 1914, Zeppelin began by saying, "Each development begins with the idea."[138] The first idea greatly shapes all future developments because it creates priorities in the mind. It creates a hierarchy of what is important. While this hierarchy might change with time the first idea has an effect on those changes because it is the kernel of the form that will ultimately be constructed. Zeppelin's basic idea was formulated by 1891. He conceived of a skeletal framework of transversal rings and longitudinal girders inside of which would be 17 gas cells.

The fundamental ideas incorporated into the first Zeppelin airship were preserved throughout the development of ever larger, better performing airships. As Zeppelin also said in his speech: "The first rigid airship, with all its weakness and faults, nevertheless formed a sound basis for developing the rigid system."[139] In the following pages I will review the significant improvements made to the first Zeppelin and how these improvements resulted in the construction of airships nearly twice the length. The development of the aluminum structure shows a linear, evolutionary process of refinement and innovation. That one company made such great progress in the development of structural form is a significant achievement.

4.6 Zeppelin Airship Production, 1900-1937

4.6.1 LZ-1 to Liquidation, 1900-1901

Before reviewing the technical development of the Zeppelin airframes, it would be helpful to briefly recount the events that led to the success and ultimate demise of the era of Zeppelin airships.

LZ-1, the first Zeppelin airship was completed by July 1900. It took off from Lake Constance at Manzell 2 July 1900, to much fanfare. (**Fig. 22**) The first flight was twenty minutes long. Five passengers were carried on the first flight. Two 14 hp motors powered the airship.

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[138] Zeppelin^2, p1. "Jede Entwicklung beginnt mit der Erzeugung."
[139] Zeppelin^2, p1. "Dieses erste starre Luftschiff hat mit allen seinen Schwächen und Mängeln doch eine sichere Grundlage für die Ausbildiung des starren Systems gebildet."

After the first flight, the apparatus from which the ship was suspended in the shed broke, and it took two weeks to repair the damage done to the ship. (**Fig. 23**) Two subsequent test flights were made 17 and 24 October, for one hour fifty minutes and twenty-three minutes respectively.[140]

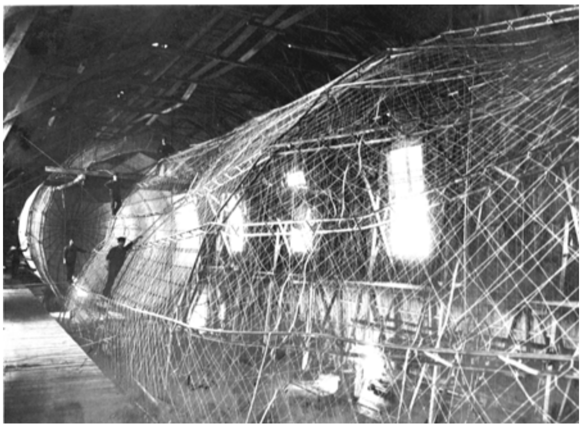

Fig. 23: View of damage done to LZ-1 when an apparatus in its shed broke. (Knäusel)

Two main questions, whether the construction was stable enough and whether its speed was sufficient for practical requirements, could not be definitely answered. During the trials, small details, such as the breaking of a crankshaft and a jammed lateral rudder, prevented normal flying. What could be definitely concluded was that the stability and steering controls needed to be improved.[141] The mishap in the hanger also illustrated the fragility of the rigid airframe, which required great improvement both in the construction of the airship and the process for handling the ship on the ground.

Further experimental flights had to be abandoned for lack of funds. The first trials did not convince the initial shareholders to invest more money in the enterprise. In December 1900, the Aktiengesellschaft zur Förderung der Luftschiffahrt was liquidated.[142]

Zeppelin was determined to continue the experiments. He argued that LZ-1 was a success from a technical point of view because it had kept in the air without veering and had responded to manipulation of the control surfaces, proving that his idea was practical and functional. Zeppelin was convinced that the necessary improvements to the structure and steering apparatus could easily be made and would follow in course of time.[143]

Zeppelin, determined to continue, bought the ship and its accessories from the company so that he could continue his experiments.[144] However, Zeppelin did not conduct further air trials for lack of funds and LZ-1 was dismantled in 1901.[145]

4.6.2 Refinement, Time Trials and the Founding of Luftschiffbau Zeppelin, 1901-1908

In 1903, Zeppelin published two papers[146] with the intention of receiving financial support to construct further airships. Over the course of the next two years Zeppelin secured enough funding to build the second zeppelin airship, LZ-2. The airframe of LZ-2 was strengthened and incorporated an important innovation to the construction of the girders that Ludwig Dürr, now chief engineer, credits to Zeppelin. LZ-2 was first tried 30 November 1905. Eckener

[140] Luftschiffbau Zeppelin GmbH…, p1 and 4.
[141] Eckener, p214.
[142] Eckener, p215.
[143] Eckener, p215.
[144] Eckener, p215.
[145] Luftschiffbau Zeppelin GmbH…, p1.
[146] *Aufruf an das deutsche Volk* (June) and *Notruf zur Rettung der Flugschiffahrt* (October)

says the ship never actually took off; rather it was carried along the surface of the water far out into the lake before being recaptured.[147]

LZ-2 took flight on a second trial held 17 January 1906. The ship was forced to make an emergency landing and was so damaged that it had to be dismantled at the accident site.[148] Undeterred, Zeppelin published another paper entitled *The Truth about my Airship*[149]*,* in February, to dispel criticism of his airship that increased with each failure.

After overcoming many fund raising difficulties construction began on LZ-3.[150] LZ-3 was nearly identical to LZ-2 save for two significant improvements. LZ-3 included the addition of horizontal stabilizing wings at the stern of the ship that checked the ship's tendency to pitch. Horizontal veering was later corrected by the addition of vertical stabilizers.[151] From these changes, we see that Zeppelin and Dürr were learning from each airship and each failure. However, public criticism was based on the visible failure of their ships being dismantled or destroyed.

LZ-3 flew on 9 and 10 October 1906 for two hours at a time. Steering maneuvers were performed successfully.[152] Based on this success, the Schwabian government permitted a lottery in Württemberg to support Zeppelin's project and for the first time granted financial support of its own.[153]

After anther floating shed was completed September 1907, LZ-3 was flown for a period of eight hours. Impressed, the Reichsregierung paid Zeppelin 2,6 million Marks to purchase the existing ship and as an advance towards another one to be built. The terms of the agreement were dependent upon the successful completion of a continuous 24-hour endurance flight.[154] We see from this success that the question of feasibility was settled with finality. The new target was reliability and endurance, without which the military could not have an interest. At this time, the military seemed to be the only available market willing to invest the necessary money in an unproven technology.

At this time, Zeppelin tried to form company to ease his financial burden, as the money promised by the Army would be withheld until Zeppelin had passed the endurance flight. However, Zeppelin's negotiations with the Krupps of Essen and with Carl Berg's firm failed because of objections to preferential prices demanded by the German government for its perceived burden of risk involved in investing in Zeppelin's airship. As a result, Zeppelin remained sole owner.[155]

LZ-4 was completed 20 June 1908. This ship incorporated numerous improvements over LZ-3. The strength of the airframe was increased by continuing the spine at the bottom of the structure for the whole length of the ship. Flying stability was improved by the addition of large horizontal and vertical fins, while vertical and lateral maneuverability was made more efficient by larger and better-placed wings. The building material was of better finish and

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[147] Eckener, p223.
[148] Luftschiffbau Zeppelin GmbH…, p1.
[149] *Die Wahrheit überr mein Luftschiff*
[150] Luftschiffbau Zeppelin GmbH…, p4.
[151] Eckener, p226.
[152] Luftschiffbau Zeppelin GmbH…, p4.
[153] University of Konstanz, p5.
[154] Luftschiffbau Zeppelin GmbH…, p4.
[155] Eckener, p233.

quality and the size of the ship was increased from 11,300 to 15,000 cubic meters, permitting of an increase of useful lift to approximately 2000 kg in contrast to the 500 kg design load of LZ-1.[156]

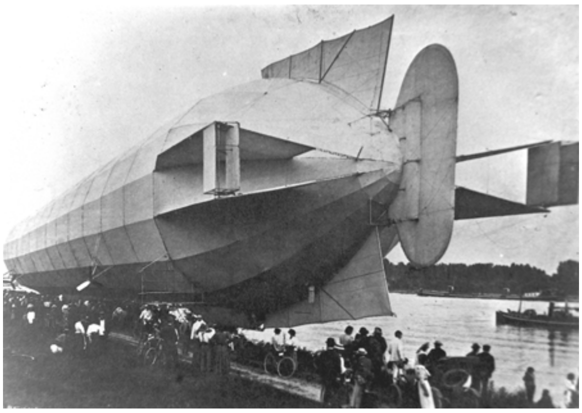

Fig. 24: General view of LZ-4, identical to LZ-5. (Knäusel)

A 12-hour flight was made in LZ-4 over Switzerland 1 July 1908 with twelve persons on board. The success of this flight, which was not publicized by attracted great media attention, gave Zeppelin's team the confidence to attempt the 24-hour endurance flight. In the meantime, Zeppelin celebrated his 70th birthday 8 July 1908.

The endurance flight was begun 4 August 1908.[157] The ship made it past the half way point before it was decided to land in a field to make repairs to a motor that had stopped working. Unfortunately, a sudden wind tore the ship from its moorings. The ship crashed into a tree, ignited on fire, and was destroyed. The event sparked an unexpected outpouring of support from the German public, and 6.5 million Marks were collected.[158]

From this windfall, Luftschiffbau Zeppelin GmbH, a joint-stock company, was founded at Freidrichshafen in 1908. From this date, Zeppelin steadily retired from the day-to-day business, letting Alfred Colsman run the business while Ludwig Dürr led the technological development of Zeppelin's airship.[159]

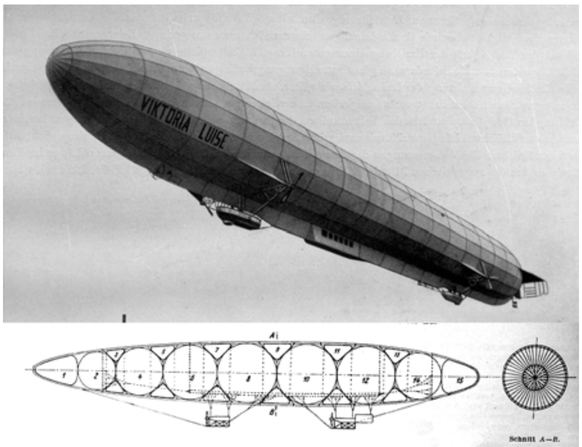

Fig. 25: Comparison of LZ-11 (top), completed early 1912, with Schütte-Lanz airship SL-1 (bottom), completed late 1911. (Knäusel; Schütte)

4.6.3 Competition, Improvement and Performance, 1909-1917

One internal and two outside forces markedly influenced the period of development at Luftschiffbau Zeppelin between 1909 and 1917. In autumn 1909, the Deutschen Luftschiffarhts-Aktiengesellschaft (DELAG) was founded to provide the first regularly scheduled passenger

[156] Eckener, p234.
[157] Luftschiffbau Zeppelin GmbH…, p2.
[158] Luftschiffbau Zeppelin GmbH…, p2; University of Konstanz, p5.
[159] University of Konstanz, p5.

air service in the world. DELAG was founded by Alfred Colsman to create a non-military market for Zeppelin airships. Orders from both the army and DELAG ensured the future solvency of Luftschiffbau Zeppelin.[160] By 1914, DELAG had made 1588 flights carrying 10197 passengers using seven airships without an accident.[161]

In early 1909 LZ-5 was completed. This ship preserved the cigar shaped hull that was characteristic of all previous Zeppelins. (**Fig. 24**) At the same time, the new competitor, Luftschiffbau Schütte-Lanz began to publish its competing design whose hull form was based on the most recent knowledge of air resistance and stream lining. It also had markedly improved control surface design and the bottom load distribution truss was put inside the envelope, unlike the zeppelin that still had the spine on the outside. The first Schütte-Lanz airship was not completed until 1911. While that ship, with its radical hull structure made of plywood, was not a practical success because the structure was not strong or stiff enough, the technological advances it incorporated were far ahead of Zeppelin's current design. (**Fig. 25**) However, Luftschiffbau Zeppelin had already hired two talented engineers, Claude Dornier and Paul Jaray. Jaray focused on aerodynamics while Claude Dornier was responsible for further development of the thin-walled aluminum structural elements and connection plates then used in constructing the airframe.

In successive generations of Zeppelin airships, the hull became progressively more streamlined and the arrangement of control surfaces simplified. The reason for Zeppelin's apparent reluctance to fully streamline was because of the competing concerns of economy and performance. The cigar shape afforded a high number of respective sections that sped construction and reduced construction cost. There was a balance to be made between how much should be spent on streamlining and how much performance could be increased with the limits of the current motor technology. As motor power increased, the Zeppelin ships became progressively more streamlined to maximize performance.

In summer 1909, a flight to Berlin was made with LZ-6. This was the last airship to be built in the floating hanger on Lake Constance by Manzell. The works at Manzell were razed and a new works was built in Friedrichshafen. In 1910, LZ-7 was built at the new shipyard in Friedrichshafen. It was followed by the LZ-8 and the LZ-9. A better motor for airship travel was developed at the Motorenbau GmbH in Bissingen. The first airship to be powered by the improved motors was the LZ-10 *Schwaben*. With every new construction the performance of the ships improved considerably.[162]

By beginning of First World War the previous civil airships were put into military service on the frontlines and new ships for this purpose were developed. Altogether over 100 airships were built during World War I.[163] In 1915, LZ-26 was the first ship built of Duralumin, am aluminum alloy with strength properties of steel but one-third the weight.[164]

In 1916, ships with a gas volume of 36000 m^3 and 17000 kg of useful lift were being constructed. The maximum altitude and air speed improved with new developments, particularly with respect to engine technology. In 1917, LZ-90, an army airship, flew 101

[160] University of Konstanz, p5.
[161] Luftschiffbau Zeppelin GmbH…, p2.
[162] Luftschiffbau Zeppelin GmbH…, p5.
[163] University of Konstanz, p5.
[164] Density of aluminum alloys = 2.7 Mg m^{-3} and steel = 7.8 Mg m^{-3}, from Ashby and Jones, p55.

hours non-stop, LZ-101, a naval airship, reached a record altitude of 7600 m, and LZ-104, also a naval airship, flew 95 hours from Africa to Bulgaria with 95 metric tons of cargo. By the summer of 1918, Zeppelin airships were being constructed with gas volumes of 62000 m^3 and 44000 kg of useful lift. Seven Maybach-Motoren motors producing 260 hp powered these ships.[165]

When war broke out, Count Zeppelin was 76. He had just begun to retire somewhat from active work. Zeppelin died in Berlin 8 March 1917. He lived to see the mass production of airships that were the product of his ideas and perseverance. The airship company that he founded in turn founded numerous enterprises that specialized in specific products and technologies related to the construction of airships. The zeppelin airship pushed the limits of technology and Luftschiffbau Zeppelin took it upon itself to ensure the availability of necessary products if they were not to be found in existing markets.

Perhaps the most important technology necessary for the success of the airship was a high power, low weight motor. Zeppelin had worked for years with Daimler and other manufacturers to improve their motors but with mixed results. To secure better motors, Zeppelin brought in Wilhelm Maybach and formed the Maybach-Zeppelin Motor Company, which was succeeded by a daughter company in 1918, called Maybach Motorenbau.[166] This company still exists under the name Motoren- und Turbinen-Union Friedrichshafen GmbH. Under the technical direction of Karl Maybach, engine weights were reduced from 26 kg/hp. in 1900 to 4 kg in 1905, 3 kg in 1910 and 2.5 kg in 1913. At the same time fuel consumption was reduced form 500 to 225 gr/per hp/hour.[167]

Zeppelin became interested in airplanes relatively early. In fact, he had financed Theodor Kober to develop an airplane. In 1915, Zeppelin commissioned one of his structural engineers, Claude Dornier, to plan and construct a flying boat. This led to the founding of the Dornier Division of Luftschiffbau Zeppelin GmbH in 1914 and that company's eventual independence under the name Dornier Metallbauten GmbH. Dornier became an influential figure in the development of heavier-than-air flying technology and continued to refine the use of thin walled, folded aluminum structural systems. These developments were introduced back into the design of the airship airframe.[168]

Zeppelin founded the Zahnradfabrik GmbH Friedrichshafen, also in 1915, for the production of transmissions, axels, and steering systems. The first director was Alfred von Soden-Fraunhofen. The company became a joint stock company in 1921. Today, the ZF-Group employs around 25,000 workers worldwide. Other daughter firms that were founded under Luftschiffbau Zeppelin are: the Ballonhüllen-Gesellschaft, Berlin; Die Zeppelin-Hallenbau GmbH, Berlin; and the Zeppelin-Wohlfahrt GmbH, Friedrichshafen.[169]

165 Luftschiffbau Zeppelin GmbH…, p3 and 6.
166 Eckener, p261.
167 Eckener, p261-262.
168 Luftschiffbau Zeppelin GmbH…, p2 and 5.
169 Luftschiffbau Zeppelin GmbH…, p2 and 5.

4.6.4 Optimization and Disaster, 1918-1937

In the post-war period, Hugo Eckener, who became an accomplished airship captain and promoter of Luftschiffbau Zeppelin, took it upon himself to rebuild the Zeppelin civilian passenger service and to likewise keep the company alive.[170] The company was lucky to have been spared liquidation by the Versailles Treaty. In return, Luftschiffbau Zeppelin had to build 'reparations' ships for Italy, France and the United States.

Eckener reestablished DELAG and passenger service recommenced in 1919. LZ-120, the *Bodensee*, flew a regular schedule between Friedrichshafen and Berlin. This was the first fully streamlined Zeppelin airship. After 101 flights the transportation arrangement was augmented with the LZ-121 *Nordstern*, which serviced flight routes to Italy and France.[171] Ultimately, the Bodensee and Nordstern were given to Italy and France respectively for reparations.

In 1924 the 'reparations' ship, LZ -26, also known as the ZR III or *Los Angeles*, was completed and flown to America by Dr. Eckener to be handed over to the US Navy. The LZ 126 departed Friedrichshafen 12 October and landed at Lakehurst, New Jersey USA 15 October, successfully completing the first transatlantic crossing by air.[172]

Eckener used the public lottery system to finance the construction of a large airship for world transportation. The Zeppelin-Eckener-Spende raised 2.5 million Marks and enabled the construction of LZ 127, the *Graf Zeppelin*.[173] LZ 127 was 236.6 m long, had a volume of 105000 m^3 and 30 tons of useful lift capacity. In comparison, LZ-1 was 128 m long, had a gas volume of 11300 m^3, and a 500 kg of useful lift.[174] LZ 127 first flew 18 September 1928. In 1929, the LZ 127 circumnavigated the world from 15 August to 4 September, in four stages. The airship traveled a total distance of 35200 km.[175]

The *Hindenburg*, LZ 129, was the largest airship ever built when completed in 1936. This airship, and its sister ship, LZ 130, were 245 m long, had a gas volume of 200000 m^3, 60 tons of useful lift, and could travel at 131 km/h.[176] The Hindenburg established global passenger travel service. In 1936, it began a regularly scheduled service between Frankfurt and Lakehurst USA. On 07 May 1937, the *Hindenburg* caught fire while landing at Lakehurst, destroying the airship and effectively ending the era of air travel by the gigantic airships. Thirteen passengers and twenty-two crewmembers perished, ending the period of hydrogen filled airships. The Hindenburg had traveled 337181 km before its fiery end.[177]

LZ-127 and LZ-130 continued to fly until 1939, primarily for research expeditions. LZ-130 had been designed to be inflated with helium but had to be inflated with hydrogen when the United States refused to ship supply the helium. In 1940, Hitler's Airship Minister ordered the two ships to be dismantled.[178]

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[170] Luftschiffbau Zeppelin GmbH…, p6.
[171] Luftschiffbau Zeppelin GmbH…, p3 and 6.
[172] Luftschiffbau Zeppelin GmbH…, p3 and 6.
[173] Luftschiffbau Zeppelin GmbH…, p6.
[174] Schiller, p157 and 209.
[175] Luftschiffbau Zeppelin GmbH…, p3.
[176] Schiller, p209.
[177] Luftschiffbau Zeppelin GmbH…, p3 and 7.
[178] Luftschiffbau Zeppelin GmbH…, p3 and 7.

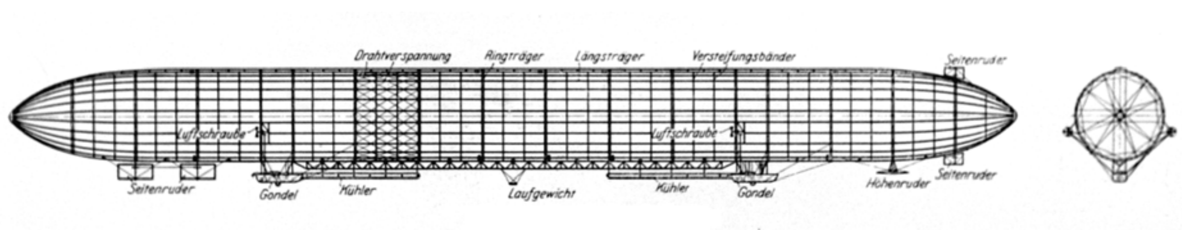

Fig. 26: Wire frame design of an early Zeppelin airship, longitudinal and transversal sections.

4.7 System Form Development in Zeppelin Airships[179]

From about 1891, Zeppelin had determined to construct the airframe of his airship with a hybrid structural System composed of transversal rings and longitudinal girders. (**Fig. 26**) Zeppelin's 1895 patent drawing clearly shows that Zeppelin intended to stiffen the transversal rings and transfer load through them using cables. In the patent drawing, these cables are arranged radially as in a spoked wheel. Zeppelin could surely have been confident that a wheel-type structure of that size was practical because of the 76.2 m (250 ft) diameter Ferris Wheel, designed and constructed by George Ferris, exhibited at the Chicago Columbia Exposition in 1893. (**Fig. 27**) Other cable patterns were later adapted to better address the fact that the principle load was always acting downward on the same segment of the ring, unlike a bicycle or Ferris wheel in which the load is constantly being transferred to different segments of the wheel as the wheel turns.

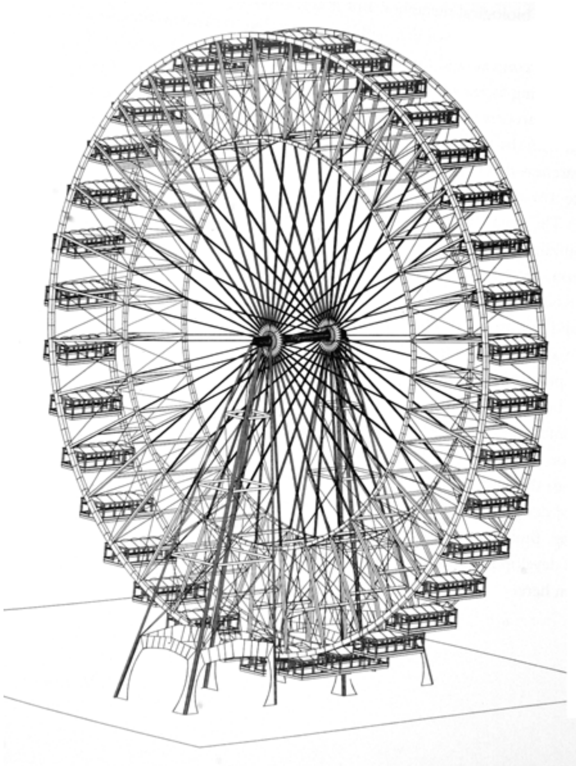

Fig. 27: George Ferris' Ferris Wheel, with diameter of 76.2 m, exhibited at the Chicago Columbia Exposition in 1893. (Peters)

Zeppelin and his engineers surely made use of the knowledge of spatial structures and analytic methods developed by the German engineers Johann Wilhelm Schwedler, August Föppl, and Heinrich F.B. Müller-Breslau[180]. Zeppelin may not have utilized this knowledge before Ludwig Dürr became chief structural

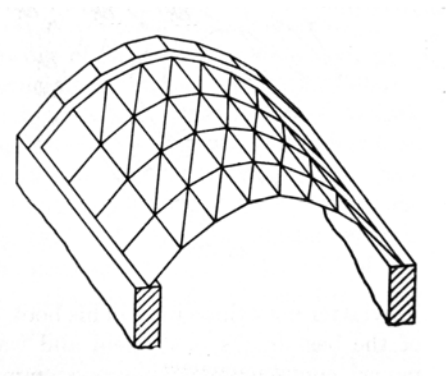

Fig. 28: August Föppl's design for a lattice shell, 1892. (Timoshenko)

[179] Main sources for this section: Dürr, p.20-31; Schütte, p1-7 and 137-150.
[180] See bibliography for references of Schwedler, Föppl, and Müller-Breslau.

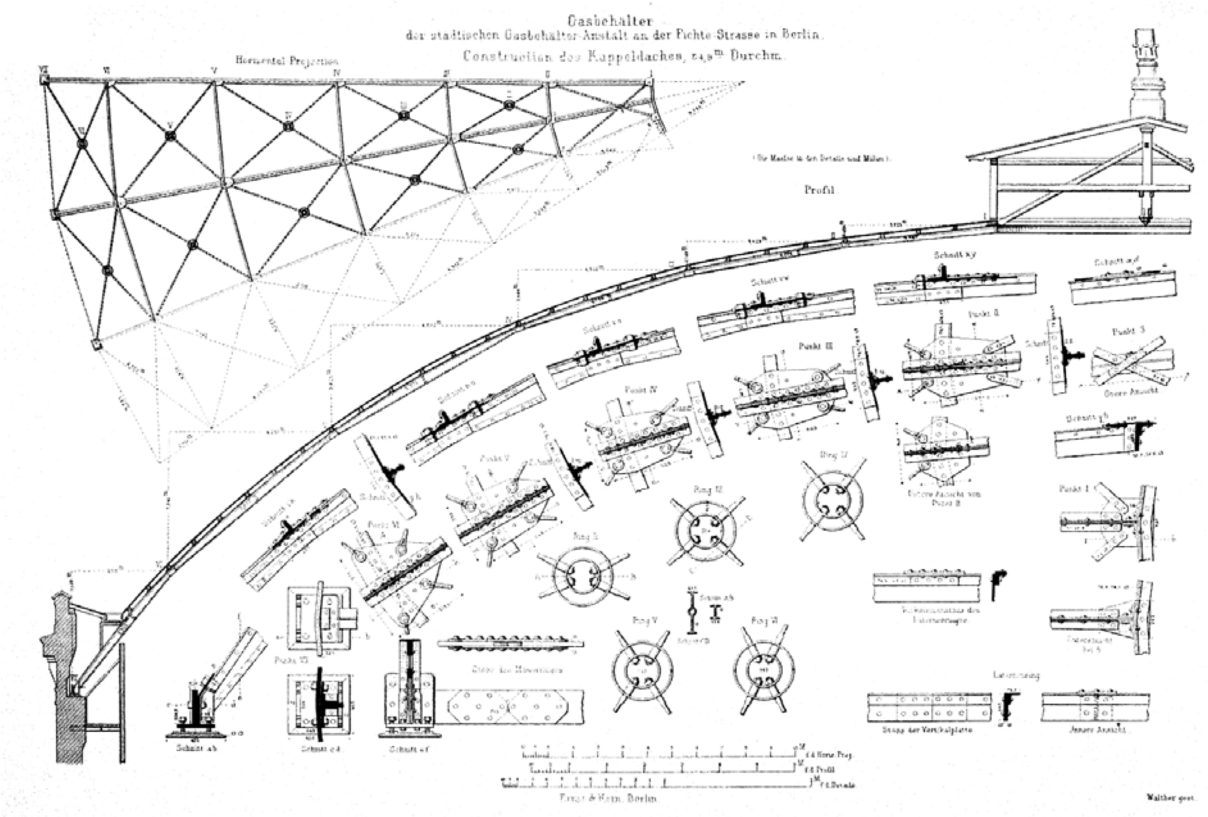

Fig. 29: Design of a Schwedler Küppel. Johann Wilhelm Schwedler, 1877. (Schwedler)

engineer because both the patent drawing and LZ-1 lacked sufficient diagonal bracing wrapped circumferentially around the airframe. (**Figs. 28 and 29**) Dürr has written that the extreme redundancy of the airship airframe prevented existing structural analysis methods from being practically employed. Instead, Dürr and his assistants developed empirical analysis methods that they refined over time by incorporating the knowledge that the behavior of each subsequent airship provided.

The basic stiffening ring and longitudinal girder arrangement remained constant in all of Luftschiffbau Zeppelin's airframes. The global form of the airframe was first cigar shaped. This form was first justified because of incorrect knowledge about aerodynamics and drag. It was first thought that aerodynamic drag was largely a function of the cross-sectional area of the ship and that drag along the length was inconsequential. It later became known that this was untrue. However, Luftschiffbau Zeppelin continued to use the cigar shape for constructive reasons. A fully aerodynamic form, such as that in LZ-126, means that most components, rings and joints will have different geometries and sizes. (**Fig. 30, bottom**) This introduces great constructive complexity. Luftschiffbau Zeppelin resisted full streamlining for this reason until Schütte-Lanz introduced in 1911, and competition forced Luftschiffbau Zeppelin to improve the performance of their ships.

The performance of airships depended on motor power. Zeppelin personally led such motor manufacturers as Daimler to improve the power-to-weight ratio of their motors. Even if Luftschiffbau Zeppelin had fully streamlined their ships before the second decade of the twentieth century, the performance gains would have been limited because of the motor power available. Therefore, the shape of the hull was a compromise between constructive cost and performance.

World War I changed the parameters of this compromise because in war performance far outweighs cost. Therefore, the airframe of the Zeppelin airships became more streamlined while preserving a central segment of the hull with parallel sides to control costs. Again, Zeppelin justified this because of the limited performance gains that could be had with engine technology at the time. Zeppelin could also increase the size of a standard hull quickly and efficiently by inserting more modules in the middle. This was demanded throughout the war to improve range, pressure height, and lift capacity.

Other streamlining and structural ideas from Schütte-Lanz were incorporated into Zeppelin airships. In 1913, Luftschiffbau Zeppelin put the lower spine of the airframe into the main volume of the envelope, thus improving aerodynamics. (**Fig. 30**) Luftschiffbau Zeppelin's complex system of control surfaces was replaced for a more simple, though structurally demanding, cruciform arrangement of surfaces that are stayed cantilevers.

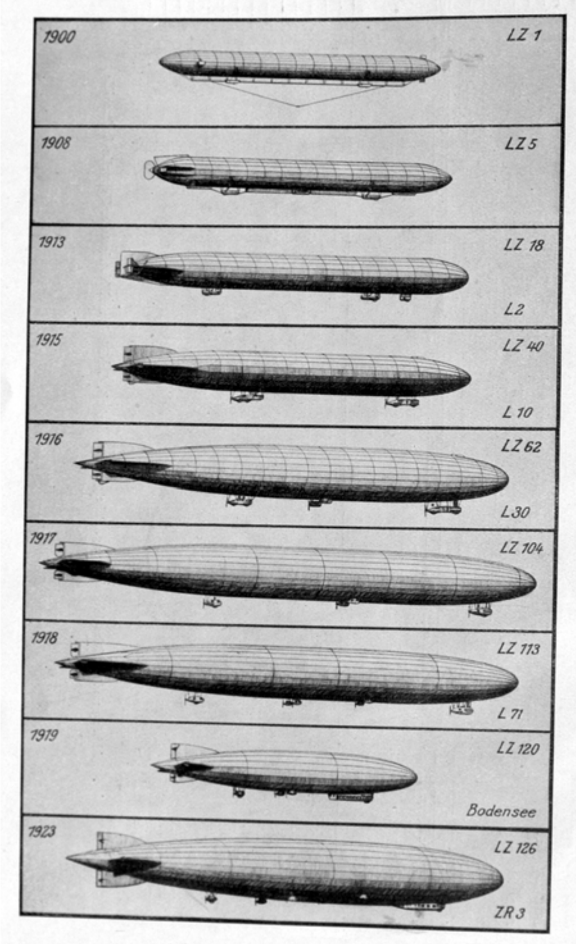

Fig. 30: General development of the Zeppelin airship from 1900 to 1923. (Dürr)

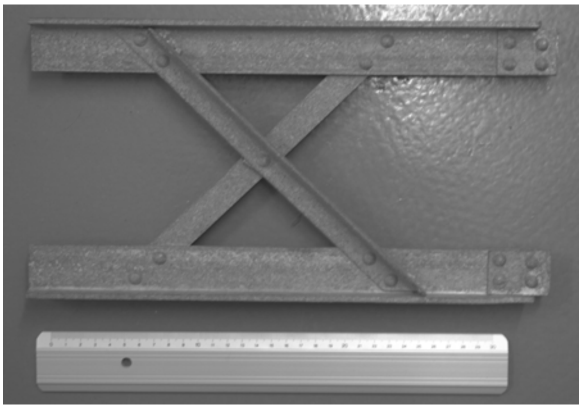

Fig. 31: Girder, LZ-1. (Dooley)

The development of the System Form of Luftschiffbau Zeppelin airframes was driven by a number of competing factors. The first factor was to enclose the requisite volume of space for the gasbags. The aerodynamic envelope was shaped to reflect both performance and economic concerns. The cigar-shaped hull made production of the airframe simpler by using repeating, modular parts. This form became less straight sided as engine power improved and wartime requirements made performance costs more important relative to construction costs. The competition given by Schütte-Lanz spurred innovation within the Luftschiffbau Zeppelin Company, but only to a certain extent. Ludwig Dürr, the chief design engineer, was unreceptive to the suggestion

Fig.32: Girder, LZ-2. (Dooley)

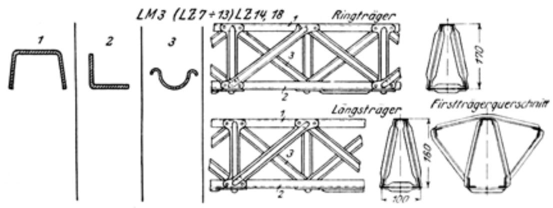

Fig. 33: Girder, LZ-7. (Dürr)

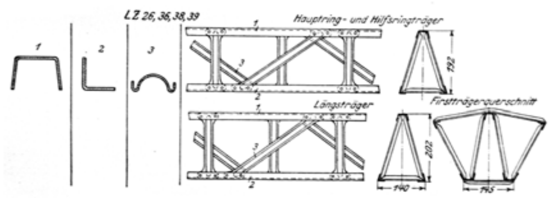

Fig. 34: Girder, LZ-26, first Zeppelin airframe made of Duraluminum. (Dürr)

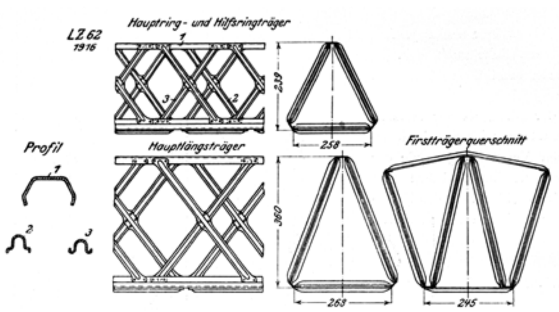

Fig. 35: Girder, LZ-62. (Dürr)

that the basic configuration of the airframe should be changed. This basic System Form had become institutionalized within the organization and the entire development was as refinement of that basic configuration. The scale limit of the System became apparent only on the Component Level when design began on the Hindenburg. This will be addressed in the next section.

4.8 Component Form Development in Zeppelin Airships[181]

The girders of LZ-1 were copied from riveted truss beams similar to those used in the Schwarz airship. (Fig. 31) It must be assumed that Carl Berg had an influence in their use and design. These girders are made with rolled **T**- and **L**-profiles. The flanges were made with the **T**-profiles and the **L**'s were used for the diagonals. The lateral stiffness of these girders was small. Their weakness is apparent in the damage caused while LZ-1 was being placed back in its hanger after its first trial flight. (Fig. 23)

Zeppelin and Dürr designed a triangular, spatial lattice girder for LZ-2. (Fig. 32) Dürr attributes the idea of the triangular girder to Count Zeppelin in a letter to Zeppelin dated 23 March 1903.[182] The triangular form is far more rigid and resistant to torsion. This basic Component Form was used in all succeeding Zeppelin airships. Rolled U-profiles were used at the apex of the girder, and opened angles were used at the corners at the base of the section. The sharp corners of these elements were not satisfactory because they abraded the gasbags. The diagonals were hollow tubes, pressed flat at the ends and riveted to the flange sections.

[181] Primary source of this section: Dürr, p32-38.
[182] Letter copied in Knäusel, p193-196.

The girders of LZ-7 were the first to exhibit the characteristics of the future development of shaped, thin-walled structural components in all succeeding Zeppelin airships. The girder of LZ-7 is made of profiles that are fabricated from drawn metal strips that are then shaped by a combination of rolling, punching and pressing. (**Fig. 33**) This processing procedure improves the physical properties of the aluminum, making it stronger and stiffer by strain hardening. The diagonal's curved form and rolled edge stiffeners make it a very strong component while using minimal material. The form of the member and the stiffening edges increases the buckling resistance of thin-walled components. This processing method made it possible to build extremely efficient and economic airframes.

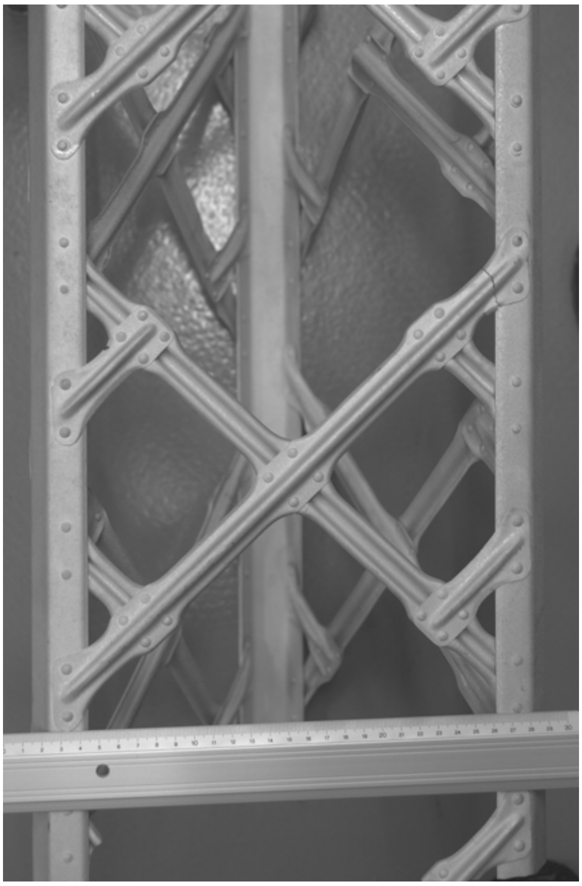

Fig. 36: Girder, LZ-127. (Dooley)

A **W**-section was introduced in LZ-7 for use in the top ridge of the airframe and in the lower spine of the airframe. The **W**-section is made my combining a few triangular Component profiles. This reduces the number of part sizes used to construct the airframe and keeps the thickness of the aluminum sub-Components within a maximum favorable thickness.

LZ-26 is the first Zeppelin airship built using duralumium. The superior strength to weight characteristics of this alloy compared to the alloys use theretofore is visible in the girder design shown in **Figure 34**.

From LZ-62, an improved girder diagonal form was introduced to allow the diagonals to be crossed. (**Fig. 35**) The profiles of open U-shaped Components are specially formed for the purpose. The need to increase the density of diagonals reflects the design thinking of how to build ever larger ships using the same basic System Form, processing technologies, and construction methods. This new girder configuration helped to substantially stiffen the airframe.

Fig. 37: Girder, LZ-129, Hindenburg. (Dooley)

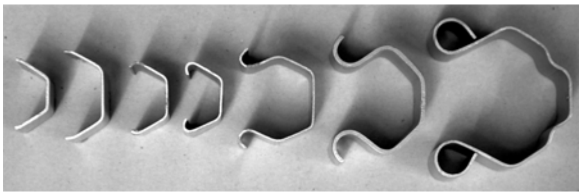

Fig. 38: Evolution of flange profiles in Zeppelin airships. (Dooley)

The principle form of the airframe components did not change until after the construction of LZ 127, the *Graf Zeppelin*. As the scale of the ship increased, supplementary diagonals were added to the lattice girder diagonals to improve their buckling resistance. (**Fig. 36**) It appears to be a rather crude way to address the problem of scale, but it is also logical and as long as the weight penalty was not prohibitive, this system worked fine.

The scale limits of the standard girder Components was finally reached with the design of LZ-129, the *Hindenburg*. This ship was 245 m long, 41.2 min diameter, and had a gas volume of 200,000 m^3. After first resisting, Dürr was convinced that the established way of building was not suited to the size of the new ship as a result. The diagonal form shown in **Figure 37** was designed. The design of this Component was adapted from Claude Dornier's development of similar forms, with their characteristic lightening holes, in his aircraft manufactory.

The development of the girder Components of the Zeppelin airframe is a study in optimization and adaptation. The Component Form was adjusted incrementally as the scale of the System Form increased. (**Fig. 38**) Only in the last series of airships did the scale factor cause a major change in the form of the girder sub-Components. The System Form stayed essentially the same however. Dürr was conservative about making any significant changes to the basic system, preferring to make the System more robust than trying to find more efficient forms. Dürr had good reason. The design methods of Luftschiffbau were largely developed in-house and base more on experience than theory. Developments in the United States by the Goodyear-Zeppelin Company (which had no formal relation to Luftschiffbau Zeppelin in Germany) tried eliminating the stiffening cables of the transversal rings by making rigid rings. This development was of questionable success. The rings required heavy connections, which offset the weight savings gained by eliminating the cables.

4.9 Plywood: Its History, Properties and Manufacture

4.9.1 What is Plywood?

The structural quality of natural wood is compromised by various natural flaws as knots, checks and splits. As a biological product, the properties of the same species of wood can vary due to growth patterns affected by climatic hazards and possible sickness during the life of the tree. One of the chief concerns in using wood is that its dimensional stability is highly sensitive to moisture content. When wood dries out too much it will shrink, often causing cracks and warping. The natural swelling and shrinking of wood, a dimensional change that is most significant perpendicular to the grain, can be detrimental to a structure. As a structural material, wood is anisotropic, having little appreciable strength transversally to the grain relative to its strength parallel to the grain. Reconstituting wood from smaller units glued together can neutralize all of these faults. Such products as glue-laminated timber, oriented-strand board, wafer board, particleboard, and plywood represent what is generally called engineered lumber today.

Plywood is made by gluing together at least three sheets of veneer plies, where, if the plywood was made of three plies, the middle ply is oriented with the grain 90° to the parallel outer plies. (**Fig. 39**) The thickness of the middle ply is typically equal to the total thickness of the two outer plies. Such an arrangement changes the structural characteristics of wood such that there is approximately equal strength in at least two directions (the plies can be theoretically orientated to any angle within a plane to augment the strength in other directions) and there is great dimensional stability. In any single direction on the plane of the plywood sheet the strength is less than for a normal piece of natural timber in the direction of the grain, but the strength in the transversal direction is significantly better. Plywood is less susceptible to the effects that checks or knots can have on the strength of normal timber because plywood is made of several smaller units of wood that will compensate for flaws in other layers as long as faults in different layers are not aligned. A simple mechanized process now repairs knots. The knot is cut from the ply and a wood wafer is inserted. The wood wafer is lozenge-shaped to minimize stress concentrations.

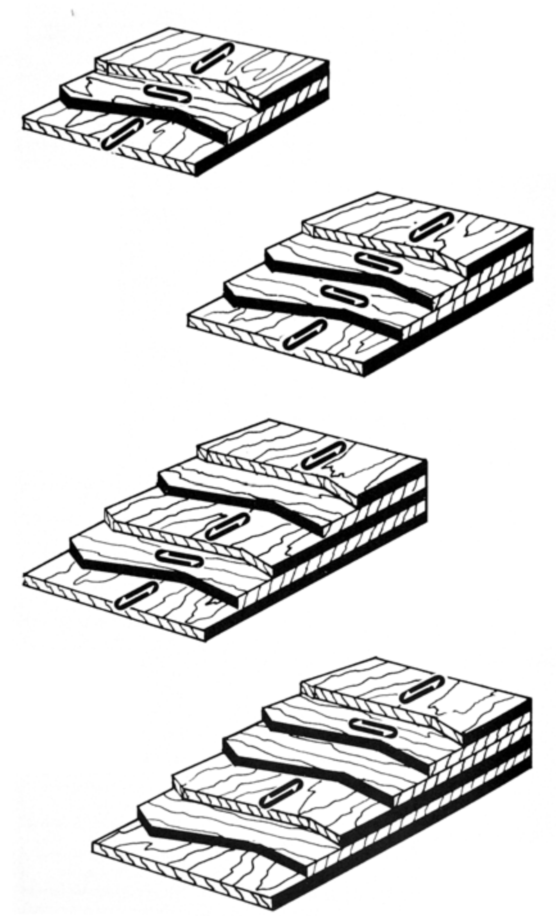

Fig. 39: Counter-orientation of plywood plies, typically 90°.

4.9.2 The Antecedents to Modern Plywood

Plywood technology dates back to the time of ancient Egypt where veneering was employed. The Egyptians used veneers of exotic woods to decorate coffins. Exceptionally, the sides of one coffin were made of six different layers of wood glued together.[183] (**Fig. 40**) The Greeks and Romans used veneering techniques,

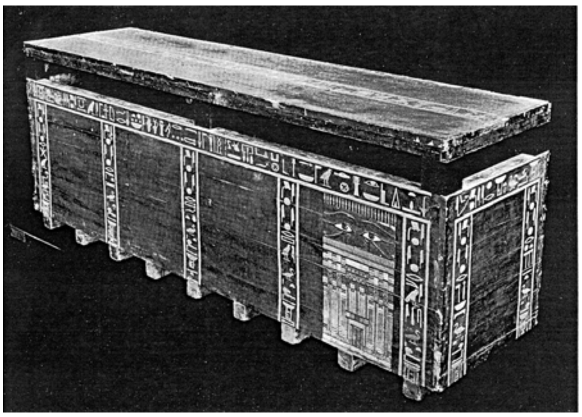

Fig. 40: Egyptian coffin made with veneered wood. (Wood)

[183] Wood, p2.

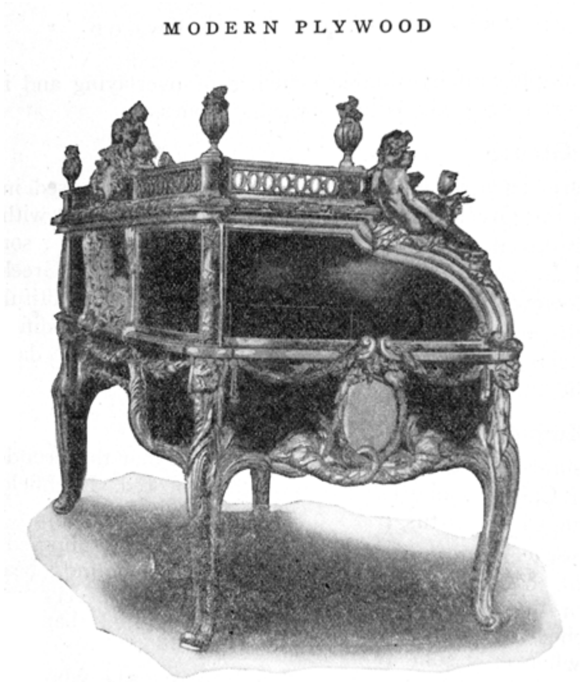

Fig. 41: Bureau du Roi, built for Louis XV using veneered wood construction, 1769. (Perry[1])

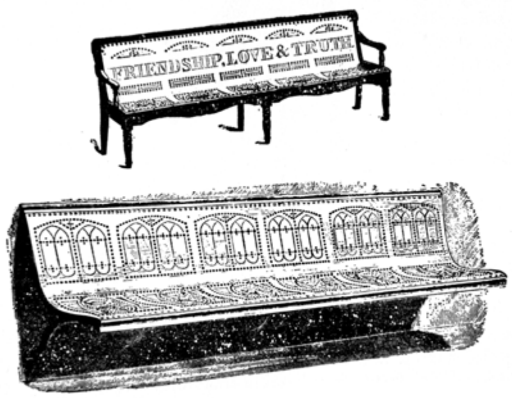

Fig. 42: Molded plywood railway station bench, nineteenth century. (Perry[1])

probably learned from the Egyptians, but only for decorative work. Pliny the Elder's *Natural History*[184], Book XVI, devotes a whole chapter to veneering. Writing of the economy of veneering, Pliny wrote: "In order to make a single tree sell many times over, laminae of veneer have been devised."[185] In the Dark Ages, veneering skills were lost and were only reintroduced in the Post-Renaissance period for furniture making.

Furniture making reached a high state of refinement during the time of Louis XV in France and in Elizabethan England, as is evidenced by the exquisite *Bureau du Roi*, built for Louis XV, in 1769 at a reputed cost of more than one million francs. (**Fig. 41**) The work by such artisans as Chippendale reveal that these furniture makers recognized how the non-desirable characteristics of wood, such as warping and splitting, could be controlled by cutting the wood into smaller units and reassembling them in specific ways so that the orientation of grain in one piece would counteract the movement of the pieces adjacent to it. Furthermore, with curved members these craftsmen learned to layer the wood such that there were no clear planes of weakness.[186]

4.9.3 The Emergence and Early Application of Modern Plywood

Modern plywood was first used around the 1830s by the piano industry. Thin sawn maple was cross laid into pin planks or wrest planks which gave the shank of the tuning pins constant contact on all sides against the ends of the wood fibers such that the pins could not slip but the pins could still be turned to tighten or loosen the wires. Steinway & Sons Company introduced laminated wood construction for the rims of grand pianos

[184] Pliny the Elder's writings date from about 72 BC.
[185] Perry[1], p21.
[186] Wood, p3.

around 1860.[187] This technology demonstrated the ability to create complex shapes by forming thin plies of wood and gluing them together while firmly clamped in the desired form. Such molded-plywood construction became prominent in furniture design in the mid-nineteenth century and such products as plywood benches, sewing machine covers, and sleighs were manufactured in molds. (**Fig. 42**) Molded plywood was used extensively for the fabrication of structural elements for the Schütte-Lanz airships.

During the latter half of the 19th century, plywood was primarily limited to applications that were protected from excessive moisture, which meant that exterior applications were very limited. By the end of the 19th century, plywood was used in products such as paneled doors, desktops, organs, chairs, sewing machines, and other furniture items.[188] Where tried, the exterior use of plywood typically ended up with the plywood rotting or delaminating. This situation, combined with plenty of products produced with bad workmanship, led to low popular opinion of the material that stigmatized the industry well into the late 20th century. The reason why the plywood was unsuitable to exterior applications was because the animal and starch based adhesives that were then available were not resistant to water and were highly susceptible to fungal and bacterial attack.

4.9.4 Material Properties

Particular properties of plywood

The properties of plywood can only be generalized because specific qualities like stiffness and strength are dependent upon the type of wood used and orientation of the fibers in the various plies. Furthermore, there are variant plywood products that have cores of either normal wood or various other materials like polystyrene or honeycomb made of any number of materials.

The chief characteristics of most plywood products are dimensional stability and the ability to fabricate the wood in dimensions that are not practical or possible using unmodified timber stock. Technically, the limiting size of a piece of plywood is only the length of veneer that can be cut and the size of the press. The width of a ply is limited by the waste incurred due to bending when veneer is cut from long logs on a rotary veneering lathe.

All dimensional and mechanical changes in wood due to moisture occur below the fiber saturation point, which is around twenty-five per cent. After that, no further swelling takes place and the additional water simply adds considerable weight to the wood.[189] Dimensional stability in plywood is due to the fibers running in one direction resisting the change in dimension of the fibers orientated 90° to those in the adjacent plies. Generally, the two outer plies are oriented in the same direction and the section is symmetrical (i.e. comprised of an odd number of plies) to balance the internal stresses and prevent warping of the plywood sheet.

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[187] Perry[1], p28.
[188] Sellers, p7.
[189] Gordon[2], p145.

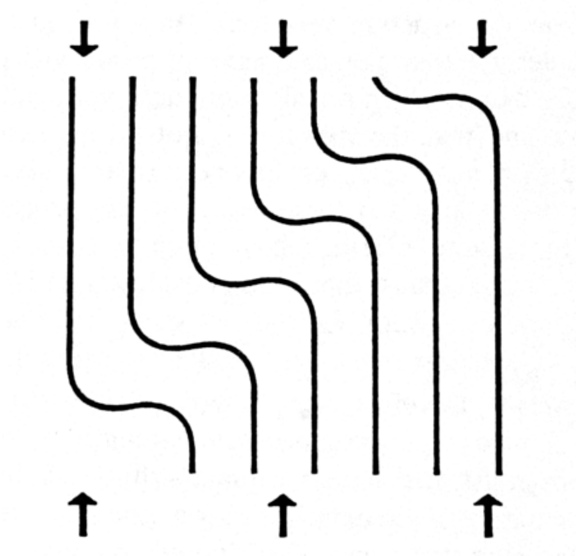

Fig. 43: Compression failure of wood grain. (Gordon[2])

The most typical ply architecture is composed of veneer layers oriented 90° to one another with the two surface plies oriented in the same direction. Ideally, the center ply is the same thickness as the sum of the thickness of each outer ply, but this is not standard because of the expense of cutting veneer to different thickness and the increased complexity in fabrication. Therefore, construction plywood typically has a higher strength capacity in one direction versus another. This difference is most apparent in standard three-ply plywood, but this disparity decreases when the number of plies is increased.

Besides the above-mentioned qualities, plywood's properties are similar to unmodified wood.

Tensile and compressive strength

Wood's strength is greatest in the direction of the fibers, which resist fracture very well. The tensile strength of spruce is around 120 MPa (17 ksi) with an elastic strain of about one per cent when carefully measured. In contrast, mild steel strains elastically about 0.15 per cent. J.E. Gordon states that "weight for weight, the tensile strength of wood is equivalent to that of 300,000 psi [2070 MPa] steel, which is four or five times the strength of steels in common use," though in practice "it is not very easy to make effective use of the high tensile strength of timber."[190]

Transversally, the fibers are very easy to crush or pull apart such that the lateral tensile and compressive strength is only about 2 MPa (300 psi). However, the lateral weakness of wood fibers is also advantageous. Gordon writes, "It is just because the fibre tubes can be crushed locally that wood can be nailed and screwed without splitting, provided we do not abuse the wood too much. Incidentally, nails and screws of reasonable size, put in with reasonable care, do not weaken the wood, as a whole, in any measurable way, in other words wood is astonishingly resistant to stress concentrations."[191]

Wood is weakest in compression along the grain. Wood fails in compression when one of the fiber tubes buckles, which leads to progressive failure of adjacent fibers. (**Fig. 43**) Spruce has a compressive strength of approximately 30 MPa (4.4 ksi), far less than its tensile strength but is weight for weight still quite good.[192]

The strength properties of plywood are dependent upon the type of wood used and the ply architecture. Plywood being used in a structure should always be tested to determine precise properties. The manufacturer normally does this but it is a good idea to conduct

[190] Gordon[2], p138-139.
[191] Gordon[2], p138-139.
[192] Gordon[2], p139.

independent tests, especially in critical structures. The strength of standard 90° oriented veneer plywood can be estimated by calculating the effective section comprising only of those plies with fibers running parallel with one another. Obviously, plywood is weaker in any given direction than a piece of normal wood of the same thickness because the wood fibers in the plywood do not all run in the same direction.

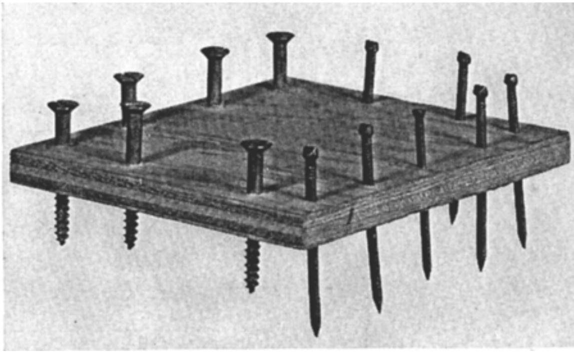

Fig. 44: Plywood is tougher than normal wood, it resists splitting better. (Wood)

Effect of moisture on strength

Wood that is very wet may have only a third of the strength and stiffness of completely dry wood. Gordon explains that “biological materials always operate in the saturated state: this gets rid of the problem of shrinkage and swelling at the expense of a reduction in strength. In engineering, cellulose is never used in the completely dry condition so that the range of strength and stiffness is not quite as bad as it sounds.”[193]

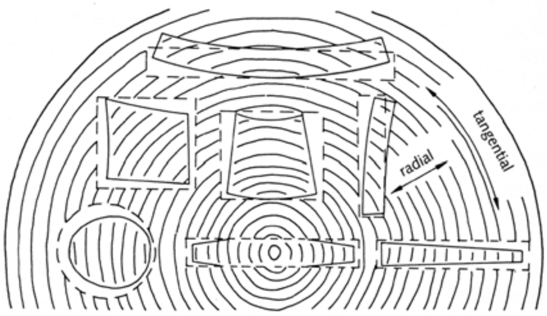

Fig. 45: Shrinkage characteristics of different cuts of wood from trunk. (Perry[2])

Bending / Shear / Stiffness

Wood is most often used in bending because it naturally produces a linear structural element whose fibers are ideally oriented to take compression and tension. One interesting quality of a normal wood beam is that as the fibers in the compression side of the beam are crushed load is transferred to the tensile side. As a result, the nominal stress in the beam can be up to twice the true compressive stress before the beam actually fails.[194] Plywood, on the other hand, is more often used in plate applications as a shear wall or stressed skin member. When designing a wood or glue-laminated beam, stiffness often controls the design rather than strength. The low stiffness of wood is partly due to the parallel structure of the fibers, which reduces the shear resistance of the material. Engineered lumber I-beams often use plywood or wafer-board for the web because these materials have a superior shear resistance due to the crossed arrangement of fibers that transfer the diagonal shear stresses more efficiently.

	Ultimate Tensile Strength	*Specific Gravity Dry*	*UTS/SG*
Normal Plywood			
Birch or Beech	13,200	.67	19,700
Phillippine Mahogany	10,670	.53	20,500
Spruce	5,600	.43	13,000
High-density Plywood			
Birch or Beech	28,500	.97	29,400
Philippine Mahogany	20,000	.95	21,100
Steel, Heat-treated	100,000	7.75	12,900
	125,000	7.75	16,100
	150,000	7.75	19,300
	175,000	7.75	22,600
Aluminum, Various Alloys, etc.	40,000	2.81	14,200
	50,000	2.81	17,800
	60,000	2.81	21,400

Table 2: Strength to weight ratio for several structural materials. (Perry[1])

[193] Gordon[2], p143.
[194] Gordon[2], p140.

$$E = \textit{Modulus of Elasticity}$$
$$= 1{,}400{,}000 \text{ for spruce plywood}$$
$$= 2{,}250{,}000 \text{ for birch plywood}$$
$$= 10{,}000{,}000 \text{ for aluminum}$$
$$= 30{,}000{,}000 \text{ for steel}$$

$$I = \textit{Moment of Inertia} = \frac{b\,h^3}{12}$$

where b = width = 1 in. in all cases
h = thickness at uniform weight of .43 lb. per sq. ft.
= .153 in. for spruce plywood
= .098 in. for birch plywood
= .0295 in. for aluminum
= .01075 in. for steel

$$EI = \textit{Stiffness Factor} = E \times \frac{b\,h^3}{12} = E\,\frac{h^3}{12}$$
$$= 1{,}400{,}000\,\frac{.153^3}{12} = 416 \text{ for spruce plywood}$$
$$= 2{,}250{,}000\,\frac{.098^3}{12} = 178 \text{ for birch plywood}$$
$$= 10{,}000{,}000\,\frac{.0295^3}{12} = 22 \text{ for aluminum}$$
$$= 30{,}000{,}000\,\frac{.01075^3}{12} = 3.1 \text{ for steel}$$

Table 3: Stiffness to weight ratio for several structural materials. (Perry[1])

	Veneer		*Lumber*	
	A	*B*	*C*	*D*
Cylindrical Measure				
Total content, board feet	339	339	339	339
Core or waste, board feet	51	26	131	114
Core or waste, percent	15%	8%	39%	34%
Net available product, board feet	288	313	208	225
Net available product, percent	85%	92%	61%	66%
Net Product, by Log Scales				
†Doyle Scale, board feet	196	196	196	196
*possible yield, percent	147%	160%	106%	115%
Scribner Scale, board feet	213	213	213	213
possible yield, percent	135%	147%	98%	105%
Scribner Decimal Scale, board feet	210	210	210	210
possible yield, percent	137%	149%	99%	107%
Spaulding Scale, board feet	216	216	216	216
possible yield, percent	133%	145%	96%	104%
British Columbia Scale, board feet	207	207	207	207
possible yield, percent	139%	151%	100%	109%

Table 4: Yield Comparison, Veneer and Sawn Lumber. (Perry[1])

Weight for weight, timber is competitive or better strength-wise in comparison to other structural materials. It also has almost exactly the same Young's modulus, weight for weight, as steel and aluminum. These qualities make wood very efficient in beams and columns.[195] In practice though, wood's actual Young's Modulus, around 12,000 MPa (1500 to 2000 ksi) for spruce, is low and requires a larger sectional area relative to steel or aluminum to carry the same load, which can be prohibitive from the standpoint of function and cost.

Timber creep

Timber, like concrete or steel prestressing cables, relaxes over time when under constant load. This is called timber creep and is seen in old timber beams that sag. This is less of a problem in plywood, which not only resists such dimensional changes but also is also not generally applied in applications of pure bending or compression.

Rot

Fungi that live parasitically on cellulose cause rot. Rot can be limited by keeping moisture content below 18 per cent. Good ventilation will also help limit the fungal growth even when the moisture content rises above 18 per cent.[196]

4.9.5 Summary of Advantages of Plywood

The advantages of plywood are as follows:

- Solid wood is predominantly strong in only one direction while plywood can have equal strength in a minimum of two directions.
- The cross-wise arrangement of fibers in plywood makes the material more resistant to splitting than solid wood. (**Fig. 44**)
- Plywood has superior dimensional stability compared to solid wood.

[195] Gordon[2], p140.
[196] Gordon[2], p146.

Fig. 46: Drawing showing Egyptian manufacture of wood veneer. (Perry[1])

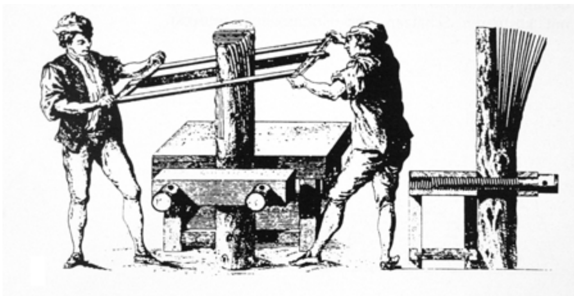

Fig. 47: Gang saw, c.1650.

- Plywood is available in large areas whereas solid timber is relatively narrow, being limited to the radius of the tree, though boards of lumber can be up to 7.3 m (24 ft) long, and of timber up to 18.3 m (60 ft). Wide solid boards tend to warp badly since the rate of circumferential or tangential shrinkage is about double that of radial shrinkage. (**Fig. 45**) The size of plywood is limited to the length to which veneer can be successfully cut and the practical length of the log that can be turned in the lath while minimizing waste. The longer the log the more bending, which makes for uneven cutting and a larger core will have to be discarded.[197]
- Plywood has a favorable strength to weight ratio. (**Table 2**)
- Weight for weight plywood is comparably stiff to steel and aluminum. (**Table 3**)
- Plywood conserves timber by creating less waste and increasing its performance characteristics for a number of applications. (**Table 4**)
- The time to season the wood is shorter because it is cut into smaller

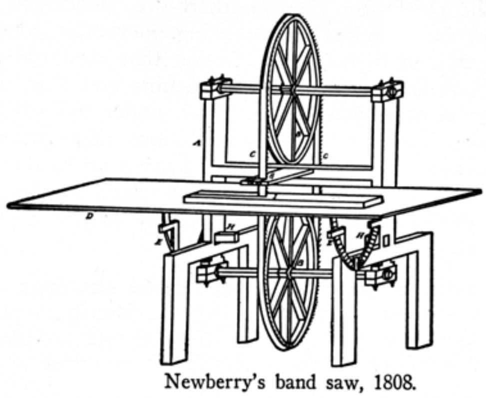

Fig. 48: Band saw, patented in 1808 by Englishman William Newberry. (Perry[1])

[197] Perry[1], p37-38.

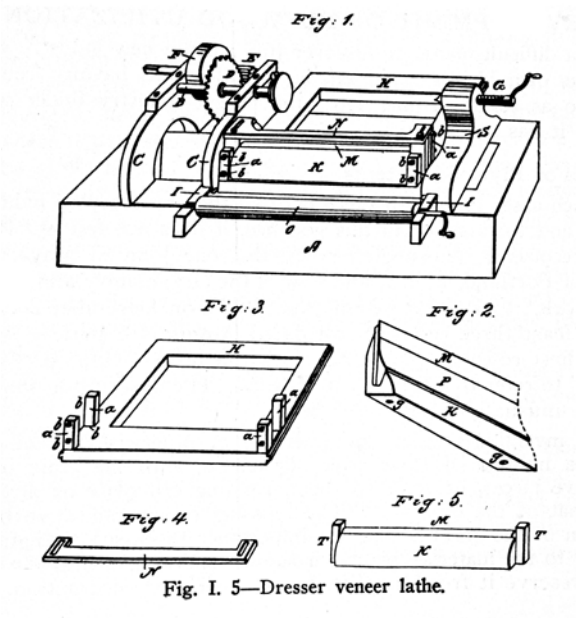

Fig. 49: Veneer radial cutting lathe, patented in 1840 by American John Dresser. (Perry[1])

pieces, which dry more quickly. This reduces cost because it means that the manufacture does not have to keep as much wood in stock over long periods of time as is the case with larger timber.

4.9.6 Manufacture and Manipulation of Form

The manufacture of veneer

The manufacture of veneer is a critical part of making plywood. The veneers used by the Egyptians, Greeks, Romans, and post-Renaissance furniture makers was chiefly for decorative purposes to economize on the use of valuable and rare woods. **Figure 46** shows the manufacture of veneer by the Egyptians.[198] Such a process would be very labor intensive and require a high level of skill. Therefore, we can assume that veneering was used only in valuable works, which is understandable in a country with limited timber stocks.

The first development in mass-producing veneer occurred with the development of the single or gang saw shown in **Figure 47**. This labor intensive process, which emerged around 1650, next gave way to the powered circular saw, which dates to a least 1777, when Samuel Miller received a patent in England for such a machine. Both technologies wasted a great amount of material because the kerf of the saw blades is quite wide to prevent buckling and to ensure the blade stays true.[199]

The band saw, first recorded in an English patent granted to William Newberry in 1808, revolutionized veneer cutting, and the cutting of lumber in general because the saw could be made to accommodate large diameter trees without the need for a wide blade required of the typical circular saws.[200] (**Fig. 48**)

The first device designed specifically for the manufacture of veneer was a modified turning lathe patented by John Dresser in America in 1840.[201] (**Fig. 49**) From this device evolved a number of veneer cutting methods still used today that are shown in **Figure 50**. Most plywood today is manufactured using veneer that is rotary cut. These machines can produce veneer at rates up to 60 lineal m/min (200 lineal ft/min).[202] Only a small percentage of specialty veneer is sliced or sawn. The rotary veneer lathe, developed in the second half of the nineteenth century, provided the basis for increased production, and better adhesives increased the market potential for stock panel plywood products.

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[198] Perry[1], p20.
[199] Perry[1], p22-24.
[200] Perry[1], p24.
[201] Perry[1], p25-26.
[202] Perry[1], p101. Note: this statistic is from 1945.

The manufacture of plywood – sheet pressing and molding

After the veneer plies are seasoned, they are stacked in what is called a pile. There are many variants of plywood architecture, where each ply can be of a different wood and thickness and oriented to any angle. Generally, the plies are oriented 90° to one another in an alternating arrangement with the top and bottom piles having their grains oriented in the same direction. Adhesive is pre-applied and then the whole pile is put into a hot press where it is consolidated and the adhesive is cured. (**Fig. 51**)

Plywood can also be made using moulds in order to make more complex shapes. In such a case the veneers are glued and clamped or pressed under pressure until the adhesive dries. A good example of how wood can be molded is the airplane fuselage shown in **Figure 52**. **Figure 52** shows the concrete mould in which the veneer was pressed to shape after the adhesive had been applied, not dissimilar to contemporary practice in the manufacture of FRP structures for the aerospace, automotive and small boat industries.

Bending Wood

Plywood can also be bent after manufacture in the same way as solid wood is bent. While wet wood is easier to bend than dry, the most effective means to bend wood is by heating it. Steaming, which is a traditional method for bending wood, is a convenient way to heat the wood without drying it out.[203] The rail station benches shown in **Figure 42** are manufactured in this way.

Types of Construction

Plywood can be manufactured in a wide variety of ways besides the typical all-veneer construction. **Figure 53** shows some of the products that can be made by using various

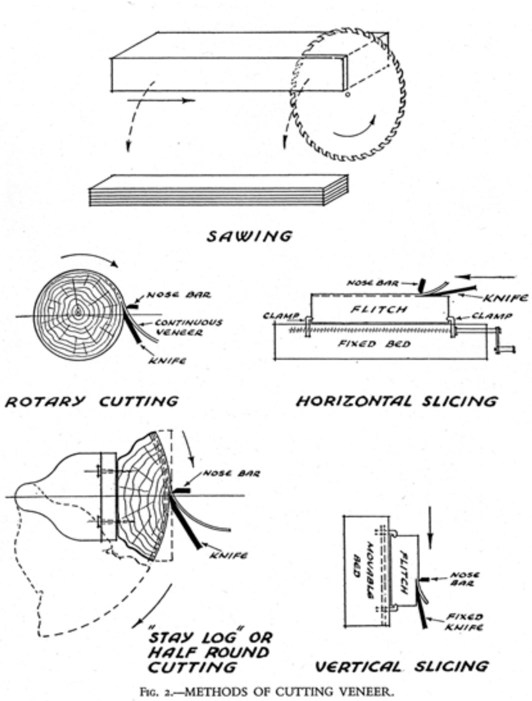

Fig. 50: Different veneer cutting methods used today. (Wood)

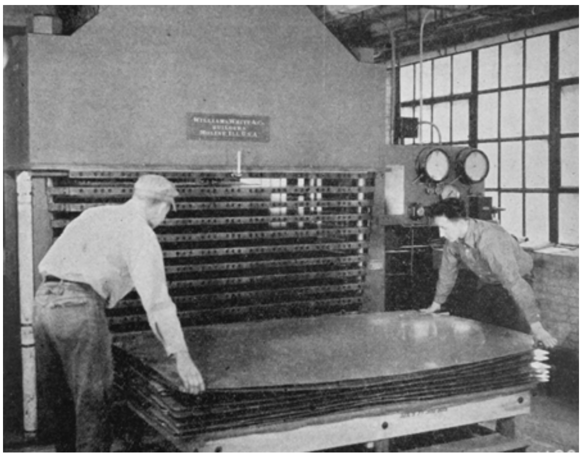

Fig. 51: 10-high hot press. (Perry[1])

[203] Gordon[2], p143.

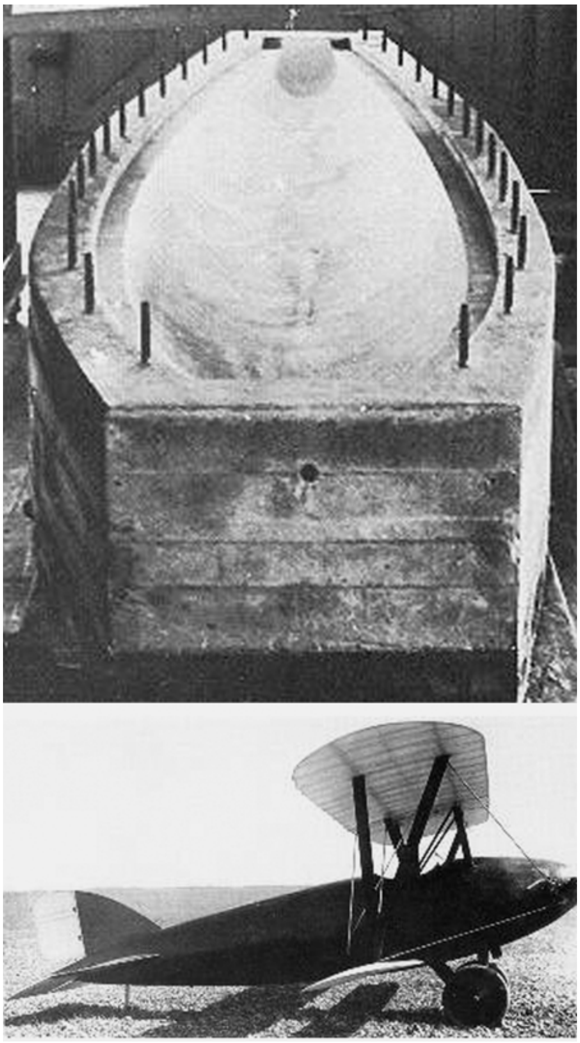

Fig. 52: Concrete mold and molded plywood monocoque fuselage it produces. (n/a)

core materials and configurations. Cores can be of solid wood, honeycomb materials, high-density foam, particleboard and others.

4.9.7 Adhesives

Summary

The most critical and limiting factor to the emergence of structural plywood in the early 20^{th} century was the types of adhesives that were available to manufacture plywood. Before 1900, the only glues available were animal and starchy vegetable based glues. Structural plywood became practical when the adhesive casein was introduced to the plywood industry in the beginning of the twentieth century. Finally, the invention of synthetic glues revolutionized the plywood industry, which was itself revolutionized in lockstep with the development of airships and airplanes.

"Natural" glues

Until the 1920s, the only adhesives available were those of animal and plant origin – hide, bone, blood, casein, and vegetable. Except for casein, which I will cover in more detail below, early adhesives could not resist moisture and were intended for interior use only, where moisture and other wetting influences would not be a factor. In dry, protected environments, veneered products enjoyed an excellent reputation, particularly in furniture. Plywood exposed to wet conditions was liable to delaminate either because the adhesive is water-soluble or because the adhesive would be attacked by fungus or bacteria. Thus, veneered products earned a poor reputation that prevailed through the mid-nineteenth century until the development of moisture resistant adhesives in the early twentieth century, though public opinion still widely associates veneering with poor quality.[204]

Animal based adhesives come from hides, bone, and blood. The blood for the blood-based adhesive typically came from cattle and was used in the United States during World War I but never adopted widely. Hide adhesives were made from the hides of cattle, sheep, goat, horses as well as their tails, ears, sinews, etc. Bone adhesive is made from fresh bones supplied directly from packinghouses and markets. Hide and bone adhesives could be processed to be fast or slow setting. Hide glues are stronger than bone glues. Animal adhesives are sold in different forms, but each requires that water be added and the mixture

[204] Sellers, p8-9.

heated before application.[205] Animal-based adhesives are most likely those used in antiquity. **Figure 46** shows the Egyptian process of manufacturing veneer. The pot over the fire is being heated, which is indicates an animal-based adhesive is being used.[206]

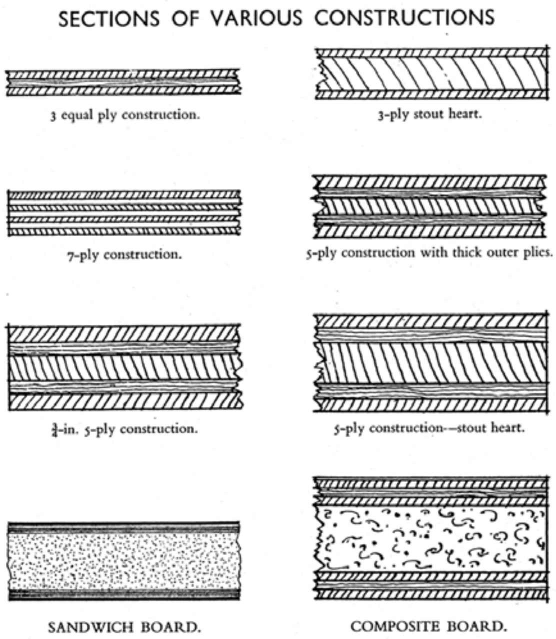

Fig. 53: Different plywood products. (Wood)

Better grades of animal adhesive are too expensive to be used in mass-production veneer and plywood operations, but are still valued by skilled artisans for making high-quality veneer. Animal adhesive is resoluble in water, is not water-resistant, and can turn to liquid if subjected to heat. Additives can be used to make animal glues somewhat water-resistant to cold water, but not hot water.[207]

Vegetable glues are made from starch. According to Perry, these types of adhesives were not used extensively until about 1910. They make strong joints and can be applied cold, though they are unsuitable to some uses because of their high viscosity and tendency to stain some veneers. The principal raw material for such adhesives is cassava. Potato, corn, wheat and rice starches can also be used. These were not generally used as much as the cassava based product.[208]

While hide and cassava based adhesives are strong, their water solubility and resistance to fungal attack made them ill-suited to critical structural applications, particularly if exposed to wet or damp environments.

Casein – The First Engineering Adhesive

In the early 1900s, casein, an adhesive made from the whey of milk, was “rediscovered.” Casein is very easy to use and apply. Casein is an excellent adhesive. It is easily applied at room temperature, is forgiving in the sense that the quality control demands are not very high, and is resistant to water and elevated temperatures.[209]

Casein was used in Germany and Switzerland from the eighteenth century, though it is possible that it was known even in ancient Egypt. It is not known why it took so long for this adhesive to emerge in engineering. Perry records that casein was industrially produced in the United States as early as 1900[210] and used as early as 1870 in Europe, though early casein adhesives were a mixture of sour milk and quick lime, which were quite unlike other casein adhesives.[211]

[205] Perry[1], p55-56 and 61.
[206] Perry[1], p55-56.
[207] Perry[1], p57.
[208] Perry[1], p57-58.
[209] Gordon[2], p155.
[210] Perry[2].
[211] Perry[1], p59.

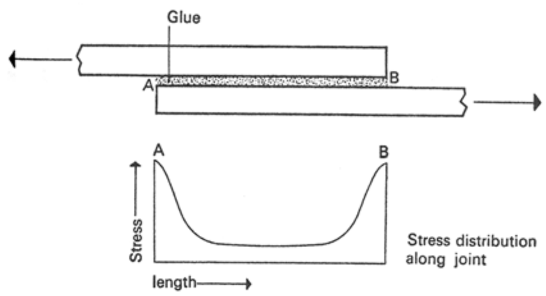

Fig. 54: Stress distribution of an adhesive connection. (Gordon[2])

Precipitating the whey from milk with a weak acid produces casein. The adhesive is sold as a white powder with additives to re-dissolve it in water. Casein is applied as a creamy paste and sets in about two days. Casein is more or less waterproof but will soften when wet. This softening can actually be advantageous.

Casein is hard and brittle when it is dry. If a crack begins to form in the edge of the adhesively bonded joint it will propagate and fail like a crack in glass. Adhesively bonded joints have an uneven stress distribution. (**Fig. 54**) Most of the work of the joint is done by the extreme edges of the glued joint meaning that the strength of an adhesively bonded joint is not dependent upon length but width. When casein is exposed to moisture and softens it will deform plastically, thus redistributing the stresses to the interior areas of the joint normally under little stress.[212]

The single most important disadvantage is that casein is susceptible to fungal and bacterial attack because it is a kind of cheese. As J.E. Gordon describes, "Its last hours are like those of Camembert; it becomes a liquid smelly mess and runs out of the joint, leaving only a dirty mark behind. Curiously, the addition of fungicides to the adhesive is not very effective."[213]

I have found no record that it was used for any significant structural purpose until 1909, when Luftschiffbau Schütte-Lanz started construction on its airship. Nonetheless, there is clearly the possibility that Schütte-Lanz had more knowledge to draw from about the properties and use of this adhesive than the historical record I have found would otherwise indicate. All sources I have referenced on the history of plywood incorrectly state that the first significant structural application of plywood using casein adhesive was in the development of airplanes in the 1920s and 30s[214]. The first Schütte-Lanz airships were built a decade earlier on a far grander scale than the early airplanes.

Synthetic Adhesives

World War I, from 1914-1918, is generally responsible for starting the significant advances in plywood and adhesive technology because the aircraft factories required considerable quantities of thin plywood, and the development of the plywood industry kept pace with that of the airplane. Andrew Wood writes, "Chemists throughout Europe and in America set about the task of perfecting waterproof glues, and as a result of intense research work crammed into the early days of the war a thoroughly sound plywood was marketed."[215] We will see that Schütte-Lanz was actively involved in this development.

During the 1930s, one of he most important innovations in the development of plywood was the introduction of synthetic resin-adhesives, such as ureaformaldehyde (UF) and

[212] Gordon[2], p155.
[213] Gordon[2], p155.
[214] Perry[1], p215 (WWI); Gordon[2], p156 (WWII).
[215] Wood, p3-5.

phenolformaldehyde (PF). Today, most interior hardwood plywood is bonded with UF, and all construction softwood plywood is bonded with the exterior PF resin-adhesives.[216]

About the great advances in the development and acceptance of plywood as a structural material, Wood writes,

> From 1919 onwards, the plywood industry expanded very rapidly. With the outbreak of the Second World War further notable progress in manufacturing technique was made, particularly in England and North America; the resin-bonded plywoods of to-day giving to the designer and user a waterproof substance of uniform strength which can safely be used in structures of the highest importance. The hard experience of the war on land, on sea and in the air, proved the material, if proof were required, and the mid-twentieth century may well become known to woodworkers as the 'plywood era.'[217]

Unfortunately, after World War II, the plywood industry, perhaps distracted by the enormous growth in the general construction market, failed to keep pace in research and development for aeronautical applications. Aluminum largely eliminated plywood from that industry.

4.9.8 Connections

Screws, Bolts, Nails

Plywood can be mechanically connected using screws, bolts or nails. Such connections are generally superior then when used with solid wood because plywood resists splitting. (**Fig. 44**) Nonetheless, such connections are far less efficient than gluing or connections that are lashed or stitched. Wood screws are the least efficient of all joints.[218] A note of caution about nails: the number of nails to use in a connection is often underestimated. It is best to closely space nails to better distribute the stress, as is done with riveted aluminum sheets.

Hollow Rivets

Schütte-Lanz used hollow rivets, in addition to adhesive, to built its plywood structural components. (**Fig. 55**) The rivets doubled as clamps to hold two members together while they were being glued and as an attachment point for stabilizing cables. Hollow rivets are similar to the grommets on the edges of large tarps, which also act as stressed attachment points that transfer concentrated loads to the main structure. I have not seen such connections used elsewhere and wonder if it could be economically applied elsewhere.

Stitching

In his book *The New Science of Strong* Materials, J.E. Gordon mentions that some flying-boat hulls made of plywood were stitched together with copper wire.[219] He cites no reference or gives no additional information. According to Gordon, "In pure strength, apart from their flexibility, the lashings, sewings and bindings used by primitive peoples, and by seamen down to recent times, are more efficient than metal fastenings."[220] Indeed, even today, tall scaffolding structures and other large temporary structures are made of lashed bamboo in most of East Asia. Stitching, like riveting or gluing, distributes the stress evenly along the

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216 Sellers, p8-9.
217 Wood, p5.
218 Gordon^2, p138-139.
219 Gordon^2, p154.
220 Gordon^2, p154.

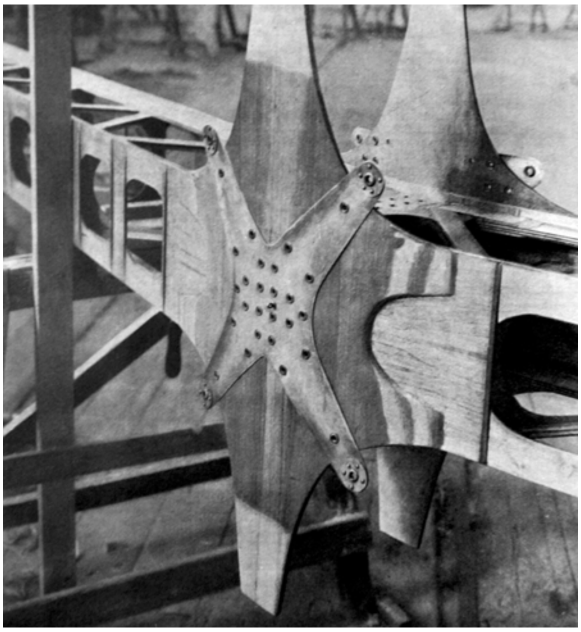

Fig. 55: Schütte-Lanz airframe connection using glue and hollow rivets. (Schütte).

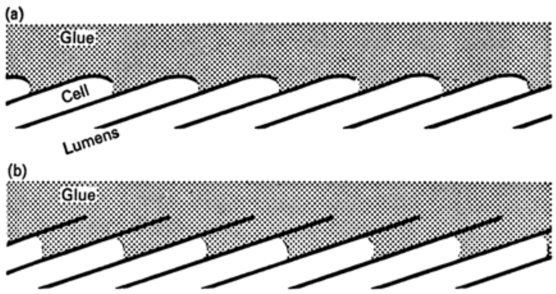

Fig. 56: Glue penetration of wood fibers of plywood face sheets. (Gordon[2])

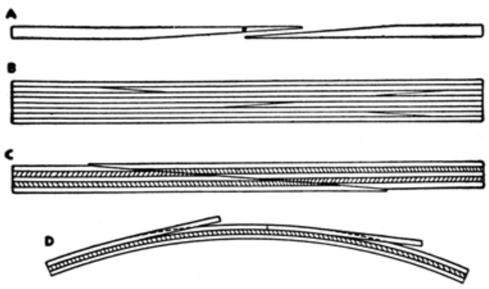

Fig. 57: Scarf joint. (Perry[1])

whole length of a structural connection. These types of connections are particularly suitable for connecting plates together or for making joints where the surface area of the joint is large enough to adequately distribute any stress concentrations.

Adhesive Bonding

Adhesively bonded connections are by far the most efficient for plywood. Adhesively bonded wood connections rely on *mechanical adhesion*, rather than *specific adhesion*, which relies on chemical bonds. Mechanical adhesion of wood is effected by the adhesive penetrating the wood fibers and creating a connection between the two surfaces with 'fingered prongs' into each piece connected by a thin adhesive layer in the interface between the two pieces.[221] (**Fig. 56(b)**) Thin adhesively bonded joints are typically preferable because thinner joints generally have fewer internal flaws like air bubbles where cracks can easily begin and propagate.

The surface of plywood manufactured in a hot press is generally poorly prepared and unreliable for structural adhesive bonding. When wood is cut, the fibers are not perfectly parallel to the surface but break the surface at a low angle. The fibers are like tubes and when adhesive is applied the adhesive normally enters these tubes. To create a strong bond the adhesive should penetrate the tubes far enough to enter a portion of the fiber that was not damaged by the process used to make the veneer. When the pile of veneer is pressed to manufacture the final sheet of plywood the press can bend over the openings of the tubes as shown in **Figure 56(a)** and prevent the adhesive from penetrating. To overcome this problem, all plywood that is to be used structurally should be thoroughly sanded before applying the adhesive.[222]

[221] Perry[1], p53-54.
[222] Gordon[2], p170-171.

Butt-jointing two sheets of plywood can be efficiently done using adhesively bonded scarf joints. **(Fig. 57)** Butt jointing can be reliably done because the strength an adhesively bonded joint is far more dependent on its length, not the width. (**Fig. 54**)

4.9.9 Application

Modern plywood was specifically designed to eliminate the deficient properties characteristic of solid wood. In this sense, plywood is one of the earliest materials made by man that could have variable structural properties as the application demanded. While plywood supplanted solid wood, this transition cannot be classified as a purely substitutional application because the applications were generally adapted to the new material, not the old. Before plywood, solid wood was used in ill suited applications for lack of better alternatives. Nonetheless, plywood was not used extensively for structural applications before the introduction of casein in the first decade of the twentieth century. The only structural applications I have found prior to 1909 is in cabinet and chair seat manufacture around 1870[223], which is not very enlightening about the antecedents to the construction of the Schütte-Lanz airships that were over 200 m (656 ft) long. Plywood enjoyed rapid growth in the airplane and boat industry during the two World Wars, but did not really infiltrate the general construction market until after World War II. As Perry explains,

> Plywood is a wood product whose cost is higher than normal timber because of the labor of cutting the thin layers, rejoining the layers with glue, and the cost of the glue itself. To justify plywood's relatively higher cost, plywood must either perform more advantageously than solid wood for a given application, or less wood must be used in the form of plywood than if solid wood was used for the same application.[224]

This is much the situation today with fiber reinforced polymers.

4.9.10 Plywood Used in the Schütte-Lanz Airships

Schütte-Lanz chose beech and ash to make the plywood used in its airships. These woods were chosen for their combination of lightness and strength. Both woods have long fibers, are tough, and easy to work with. The plywood consists of three or more glued veneers. Each veneer is 0.5 to 2 mm thick and the grain is oriented perpendicularly and skewly. The plywood is equally stiff in all directions in the plane of the piece. The orientation of the grain prevents cracking and shearing. Schütte-Lanz used casein adhesive, which could be applied at ambient temperature. The plywood is suitable for producing a wide variety of forms and profiles. It was shaped into plates with double curvature, angles, and **U**-profiles. The plywood is protected from water absorption by a special impregnation and lacquer coating. When the quality of these materials began to diminish towards the end of World War I, Schütte-Lanz developed new glues and treatments.[225]

[223] Wood, p3.
[224] Perry[1], p37.
[225] Schütte, p4 and 19.

4.10 Why Schütte-Lanz is Germane to this Project

Luftschiffbau Schütte-Lanz was founded in 1908 by Johann Schütte, a naval engineer, and Heinrich Lanz, an industrialist who essentially funded the venture, to compete directly with Luftschiffbau Zeppelin, then the only rigid airship constructor in the world. Working with an engineer named Carl Huber, Schütte conceived of a rigid airship that would address all of the perceived problems with the Zeppelin airships. *SL*-1, the first Schütte-Lanz airship was constructed in 1909.

What distinguishes the Schütte-Lanz airships is that their airframes were principally constructed of plywood, as opposed to aluminum characteristic of the Zeppelin ships. That is not to say that Schütte-Lanz did not consider aluminum, among other materials, but they decided at the time that plywood was a better alternative to the aluminum alloys then available.

When Schütte-Lanz entered competition with Luftschiffbau Zeppelin, the Zeppelin concern had entered a period of stagnant innovation. Schütte-Lanz perceived this and thought, as any good entrepreneurs do, that it could build a better ship than Zeppelin. Its decision to use plywood is interesting because the material, though developed in its modern form around the 1830s, had not been used extensively in structural applications, certainly not in any structures of similar scale and lightness as the Schütte-Lanz airships.

The Schütte-Lanz case study, in concert with the Zeppelin case study, affords the opportunity to examine the development of different two materials – plywood and aluminum, in the same application – airships, in the context that both materials were new to the field of structural engineering when first applied to airship construction. For these reasons the Schütte-Lanz case study is germane to the project.

4.11 SL-1, Conception and Construction

4.11.1 Conception of the First Schütte-Lanz Airship

Johann Schütte's interest in airship design stemmed from the crash of LZ-4 in 1908. Schütte thought about the Zeppelin design and identified a number of correctable faults. He tried approaching Luftschiffbau Zeppelin with his criticisms, but was rebuffed. Schütte therefore set out to build a better airship.[226]

The principal design parameters Schütte defined were:

- The shape of the ship should be determined by aerodynamic concerns only, not constructive.
- Engine power-to-weight ratios need to be greatly improved. He saw this as one of the most challenging technical difficulties.
- Using spherical, minimal surface area shapes could minimize the weight of the gasbags.

[226] Schütte, p1.

- The stresses exerted on the airframe by the gasbags could be minimized if the traversal stiffening cables of Zeppelin ships were eliminated.

Schütte and Carl Huber thus determined that an airframe that formed a rigid framework around the gasbags would best limit the stresses caused by the expansion and contraction of the gasbags.[227] A.D. Rettig constructed a model lattice shell in 1909. (**Fig. 58**) There is a dispute about who originated the idea of this spatial, lattice shell. Rettig made no further contribution to the construction of the first Schütte-Lanz airship based on this model.[228] Huber was the principal design engineer and is largely responsible for developing the plywood design and Component forms of the airframe.

Fig. 58: Wood frame model of lattice shell structure, A.D. Rettig, 1909. (Meyer[2])

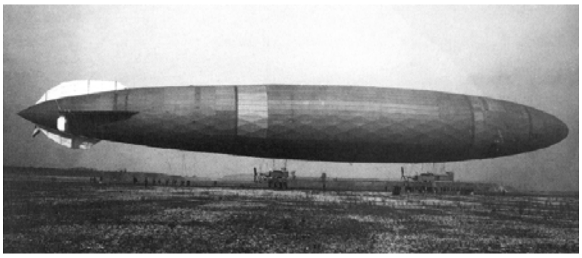

Fig. 59: General view of aerodynamic shape of SL-1. (Schütte)

4.11.2 Materials of the Schütte-Lanz Airship

In 1908, Carl Huber and Johann Schütte considered building the airframe from aluminum, wood, electron metal, a magnesium alloy, and steel. Duraluminum only became available in 1909. Plywood was chosen because it could be used to produce all the possible profiles necessary and wood could be easily procured. Schütte points out that material can never be regarded as a decisive factor of the system, or its construction. He and Huber set out to choose the most appropriate material to construct the structural form represented by Rettig's model. Duralminum was considered when it became available but at the time it age hardened rather quickly, became brittle, and was susceptible to stress rupture.[229]

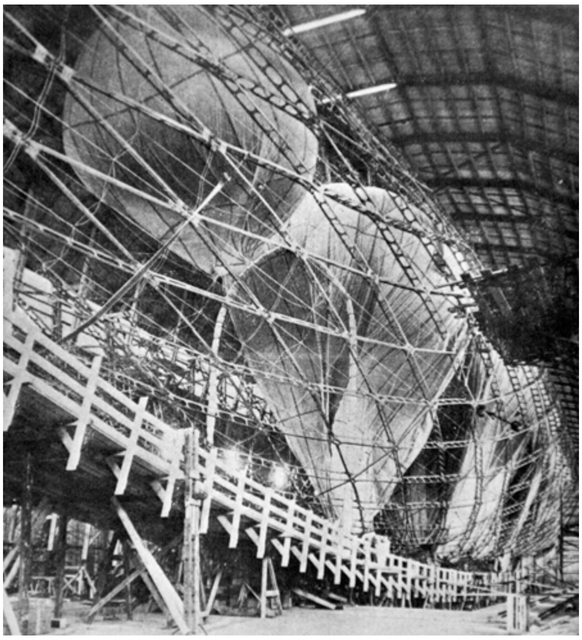

Fig. 60: Plywood airframe of SL-1, under construction. (Meyer[2])

The Schütte-Lanz airframe was finally built with three principle materials: plywood, steel and duralumin. The plywood was suitable for

[227] Schütte, p4.
[228] Meyer[2], p101-103.
[229] Schütte, p4 and 19.

making the girders and basic plates but was not suitable for diagonals subject to axial tension. The tension diagonals were made from high-strength steel wire. The connections between the plywood girders were reinforced with duraluminum plates. Connections between the plywood and the steel cables were made by anchoring the cable through a hollow rivet in the plywood. The hollow rivets were mostly made from brass. Other materials, such as steel or electron metal were used depending on the materials being connected.[230]

A condition of lightweight construction is to use building materials of the highest quality. Schütte-Lanz took advantage of the fact that the properties wood, commercial steel and duraluminum can be improved. The deficiencies of natural wood were obviated my making plywood. Schütte-Lanz varied the fiber orientation and wood types used in the veneers to optimize the design of each Component to work in the most efficient way. The wood was impregnated with a special product and lacquered to protect it from water absorption. Both steel and duraluminum were strained hardened to improve their tensile strength.[231]

4.11.3 The Structure

Construction on SL-1 began in 1909 and ended in 1911. One of the reasons it took so long to build was that the plywood structure was without precedent in scale and application. Many technical problems had to be solved. The airframe was globally shaped to be as aerodynamic as possible. The largest diameter of the hull lies in the front third of its length and from there the diameters the diameter is a constant curvature to the bow and the diameters decrease to the aft.[232] (**Fig. 59**)

The airframe is a spatial framework. (**Fig. 60**) H. Müller-Breslau likened the form of the original design to the form to the Schwedler Küppel. Müller-Breslau evidently suggested this type of form in 1894, around the time that Zeppelin was having problems with Mülller-Breslau and the Prussian Airship Battalion to support the development of his airship design.[233]

The side members are designed as double-tee beams with a depth of 400 mm at the center and reducing to 200 mm at the ends. Each girder is orientated perpendicular to the radius of the hull centerline and spans three nodes. The girders are interconnected at each node. They girders are not planar, rather they have a wavy form as shown in **Figure 61**.[234]

It was discovered in the course of construction that the computed stresses in the connection were too low. This meant that additional diagonal bracing had to be installed, and the connections where two or more girders met had to be reinforced. The connections were reinforced by inserting pressed duraluminum plates hollow riveted to the wood. (**Fig. 55**) To maintain the round cross-sectional form of the hull it was necessary to insert stiffening rings made of triangular girders. The rings were transversally connected by a system of cables arranged in a novel way to account for the spherical form of the gasbags.[235] All of this reinforcement led Schütte to conclude that the originally assumed advantages of the lattice

[230] Schütte, p75.
[231] Schütte, p75.
[232] Schütte, p10.
[233] Schütte, p11.
[234] Meyer[2], p164.
[235] Meyer[2], p164.

shell airframe were lost and that it made sense to switch to the more conventional transversal ring and longitudinal girder System characteristic of Zeppelin airships.[236]

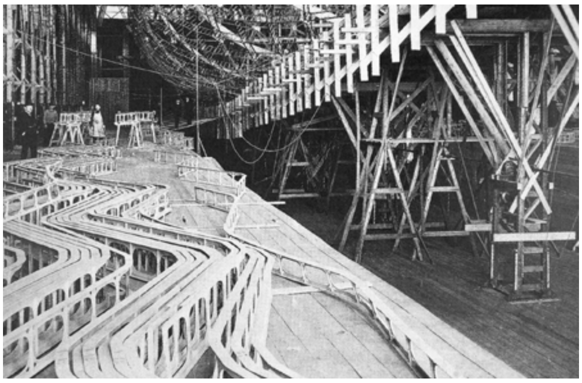

Fig. 61: Form of plywood girder used in the construction of the lattice-shell airframe of SL-1. (Meyer²)

4.11.4 SL-1 to SL-2, the standard War Ship Design

SL-1 flew for the first time 17 October 1911. It was 131 m long and its largest diameter was 18.4 m. Its useful lift was 5,000 kg with an unloaded weight of 21,190 kg. In 1912, SL-1 made fifty-three flights for a total of about 120 hours in the air before it was delivered to the Prussian army. A defective gas-line forced the ship to make an emergency landing in the summer of 1913. A storm tore the ship from its mooring and the ship was destroyed.[237]

SL-1 did not live up to the expectations of its designers. The measures taken to strengthen the ship offset any weight advantages that were expected from using the spatial lattice structure. During flight tests it was also determined that the rhombic dimples in the aerodynamic envelope created by the underlying airframe caused eddies in the airflow, thus increasing drag. It was further determined that the engine technology of the time was not good enough to significantly improve the performance of the airship over a comparable Zeppelin ship designed with a less then ideal streamlined form.[238]

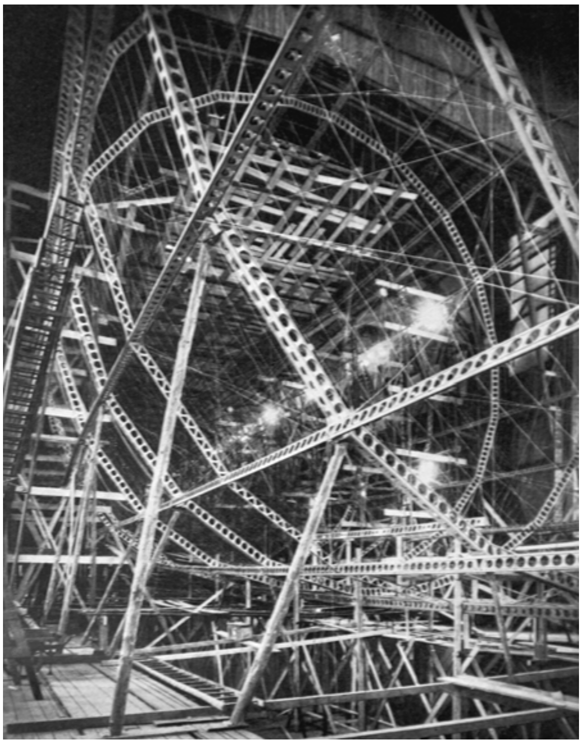

Fig. 62: Airframe of SL-2. (Schütte)

As a consequence, Schütte decided to change to a longitudinal frame system with transversal rings. He adapted the double-**T** and triangular girders developed for SL-1 to the new airframe. (**Fig. 62**) Resistance to buckling was an important parameter in the design of these Components. The advantage of the pure longitudinal frame system was its great strength and that it was simpler to construct. Furthermore, the form caused less air resistance because the lines of structure run in the longitudinal direction.[239]

[236] Schütte, p5.
[237] Meyer², p164.
[238] Schütte, p8.
[239] Schütte, p5 and 8.

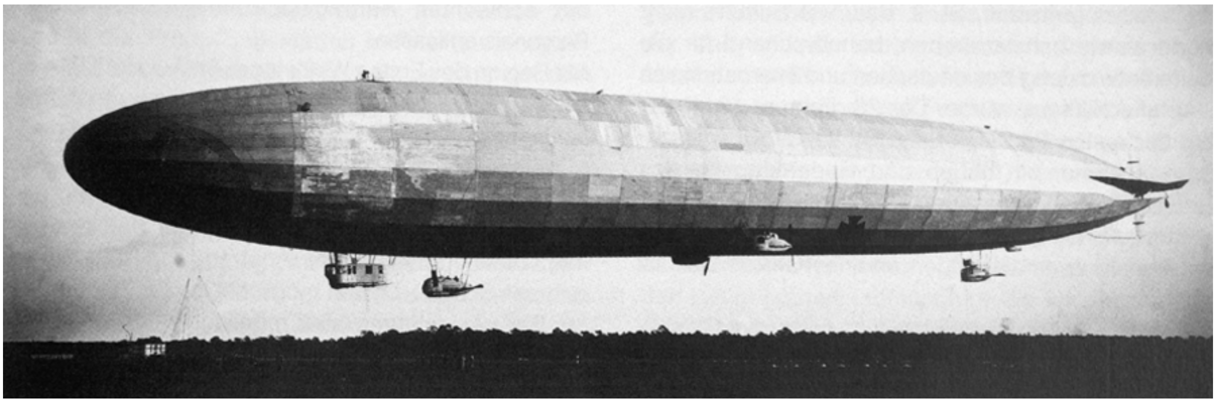

Fig. 63: General view of completed SL-2 series airship. (Schütte)

	1913	1914	1915
Buche	12 m³	28m³	74 m³
Tanne	17 m³	178 m³	400 m³
Esche	28 m³	-	47 m³
Espe	146 m³	230 m³	1115 m³

Table 5: Table showing quantity of wood used to build eight Schütte-Lanz SL-2 type airships from 1913 to 1915. The total quantity of wood can fit inside a cube with a side length of only 11.8 m. Each SL-2 type airship was 153 m long. Its largest diameter was 19.8 m, and it had a volume of 32,500 m^3. (Meyer[2])

The SL-2 series airship was 144 m long with a maximum diameter of 18.2 m. (**Fig. 63**) It had a gas volume of 24,500 m^3 and was capable of 76 km/h. SL-2 was conceived of as a war ship from the beginning. The construction of SL-2 was started in June 1913. It was considered to be the most modern and progressive airship built to date. It was stable and had superior handling characteristics to the Zeppelin airships. Three airships of the SL-2 series were constructed in 1914 and five in 1915. All together, the total wood consumption for these airships could fit in a cube with an edge length of 11.8 m.[240] (**Table 5**)

4.11.5 Adhesives, Scale Limits and the Next Generation Design using Duraluminum

The quality of lacquer and adhesives worsened in the course World War I. Schütte-Lanz began to develop its own adhesive research and development program to develop a highly water-resistant adhesive. No water-resistant adhesive or method of making adhesively bonds existed that could be relied on under prolonged wet conditions. Schütte-Lanz tried polymer based synthetic resins but the resin based adhesives of the time had to be heated and could only be processed in a liquid state. This was not compatible with the production methods of Schütte-Lanz. They tried to coat the edges of Casein bonded joints with a formaldehyde coat, but this too was unsuccessful. In 1917, Schütte-Lanz invented an impregnation technique using a pressurized vessel, perhaps for the first time, to treat the plywood. This technique successfully reduced the woods moisture absorption rate.[241]

[240] Meyer[2], p167.
[241] Meyer[2], p172-173.

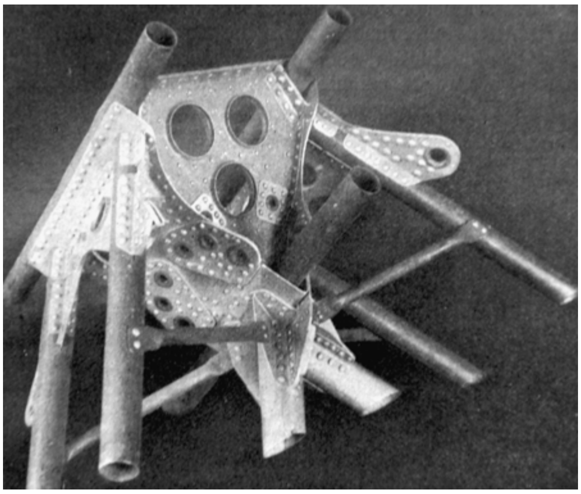
Fig. 64: Duraluminum airframe component under development by Schütte-Lanz at the end of World War I. (Schütte)

Near the end of the war, Schütte-Lanz airships had nearly doubled in size. SL-20, 21, and 22 each had a volume off 56,000 m^3. Schütte personally believed that they had reached the absolute safe maximum size using plywood construction. Schütte-Lanz engineers therefore began development of a duraluminum airframe. The quality of duraluminum had become reliable by this time and Duerener metal works started producing smooth, duraluminum pipe in 1917. Schütte-Lanz focused its research on using these tubes as the base Component of the new airframe.[242] (**Fig. 64**)

Schütte-Lanz developed a workable duraluminum structural system in a relatively short time. They calculated that the duraluminum airframe could take the same load as a plywood airframe of the same weight. The biggest challenge for the Schütte-Lanz engineers was to develop functional connections for the tubular structure.[243] However, no Schütte-Lanz airship made of aluminum was built because the war ended and the company was disbanded as part of the post-war disarmament.

4.12 Conclusions

The Schütte-Lanz development demonstrates the role competition plays in technological development, exposing the limitations of technological thought within one design organization such as Luftschiffbau Zeppelin. By 1914, it was clear that Luftschiffbau Zeppelin had institutionalized certain design principles into its work that stagnated innovation and hindered performance improvements. Schütte-Lanz' competitive challenge forced Luftschiffbau Zeppelin to react and question their premises, adjusting their design culture to accommodate new ideas and conventions thrust upon them from an external force.

More so than Luftschiffbau Zeppelin, Luftschiffbau Schütte-Lanz appears to have been much more open to the use of widely varying materials and employing them where they would perform best. Zeppelin limited its construction to aluminum alloys for most of the structure and connections. The only exception was high-strength steel used for cables.

Structural systems are not necessarily material dependent. Plywood was used successfully in a structure, the airframe, previously associated with aluminum. Plywood's strength-to-weight ratio combined with relatively low cost made it competitive with the more expensive aluminum. The airframes of the SL-2 series of airships exemplify how materials primarily change Component and Element Forms, and not System Forms.

242 Schütte, p22.
243 Meyer[2], p173.

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THE FLAT SLABS OF TURNER & MAILLART

A-05

5.1 A New Structural Type

The invention of the reinforced concrete flat slab in the first decade of the twentieth century marks the emergence of a new structural type for building construction. Its invention shifted conceptual thinking about structures from one-dimension to two. The pioneering spatial structures of August Föppl, Johann Wilhelm Schwedler,[1] and Ferdinand von Zeppelin[2] had little practical effect on engineering practice in their time. (**Appendix A-04, p.A.204-205, Figs. 28 and 29; Appendix A-09, p.A.380, Fig. 35**) Only after the flat slab was introduced and widely applied was resistance to using new structural forms, largely due to inadequate theoretical and analytical tools, overcome. A new period of innovation and transformation began that was similar to what had occurred at the beginning of the nineteenth century.[3]

Prior to the introduction of the flat slab, floor design was characterized by a hierarchical system of one-dimensional load-bearing elements. Robert Maillart, an influential pioneer in flat slab design, wrote,

> For large spans only rolled steel sections and wood were available, and neither of these could be used in any other form than that of the bar, because the restriction of the one-dimensional element was dictated by the rolling process in steel and the growth of wood. With these one-dimensional elements: joists, columns and beams, the engineer was accustomed to calculate and build his structures, in such a manner that they were everything in the world for him.… The slab form was only employed for quite small spans: covering drains, projecting as balconies and as infilling between joists, in the available materials, namely natural stone and plain concrete. This was the situation when reinforced concrete emerged, but at first nothing was changed: it was laid as if it were steel or wood, girders spanned from wall to wall and from column to column. At right angles to these main girders came secondary beams and the space between would be filled in with a slab without however its being comprehended as a special construction element. On the contrary they made haste to divide it up into strips, those strips could then be calculated as beams in the normal manner.[4]

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[1] Schwedler and Föppl where pioneers in the development of spatial structures. They both developed iron lattice structures. Föppl was instrumental in developing methods to analyze three-dimensional structures. Ref. Schwedler (1877) and Föppl (1892).

[2] Zeppelin founded the company that built enormous airships in the first third of the twentieth century. Zeppelin airships had aluminum airframes. The Hindenberg, the largest Zeppelin airship to be built, was 245 m long, 41 m in diameter and had a gas volume of over 200,000 m^3. The airframes were complex, three-dimensional structures.

[3] **Ref. Appendix A-02 and Appendix A-03**.

[4] Bill, p165. From “The development of the beamless floor-slab in Switzerland and in USA” (translation), by Robert Maillart. Originally published under the title “Zur Entwicklung der unterzuglosen Decke in der Schweiz und in Amerika”, in the *Schweizerische Bauzeitung*, vol. 87, 1926.

In the following chapter I will review the development of the flat slab, and I will examine the ramifications its introduction had on the engineering profession in general. I will begin with a brief history of reinforced concrete to explain the origins of the material and to show the development of reinforced concrete floor systems prior to the invention of the flat slab.

5.2 Brief History of Reinforced Concrete

5.2.1 Concrete

The first mortars and weak concretes were developed in ancient Egypt and earlier civilizations, but it was the Romans who first exploited concrete's economy, versatility, and plasticity. The Romans made concrete with a naturally occurring hydraulic cement of volcanic origin called *pozzolana*. The principle source for pozzolana is at Puteoli (now Pozzuoli), near Mount Vesuvius.[5] Pozzolana contains silica and alumina inclusions that allow it to react to lime and harden even under water.[6] Vitruvius wrote described pozzolana as an ideal material for bridge foundations.[7]

The Romans started to use pozzolana concrete in the second century BC. Pozzolana concrete was extensively used in the first century AD during the reconstruction of Rome. The Pantheon, designed by Hadrian and completed in 128 AD, was built with pozzolana concrete. (**Fig. 1**) Its dome is 43.3 m (142 ft) in diameter. The dome of the Pantheon was three times larger than any other dome built until that date.[8]

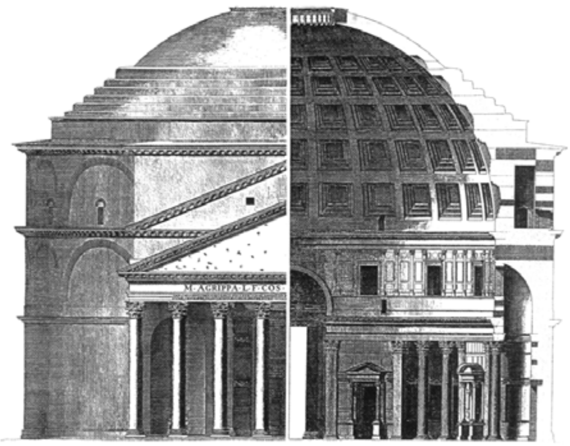

Fig. 1: Pantheon, concrete dome with diameter of 43.3 m, Hadrian, 128 AD. (Mainstone[1])

During the Middle Ages, concrete was used in Christian and Arab construction, though it was considered inappropriate for representative architecture.[9] Concrete was used in the foundations of such important edifices as the Salisbury Cathedral and one of the Strasbourg towers. Concrete was also used extensively in the construction of castles and fortifications in Britain, Germany and Spain.[10]

Cement and concrete use greatly increased after John Smeaton's experiments on the nature of hydraulic cements in the mid-1750s. Smeaton discovered that limestone cements were 'hydraulic' only if they contained silica. Smeaton was searching for a mortar that would set under water to reconstruct the Eddystone Lighthouse. Smeaton completed the lighthouse in

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[5] Peters[1], p58.
[6] Sutherland, p12.
[7] Vitruvius, p196.
[8] Sutherland, p12.
[9] Peters[1], p58.
[10] Peters[1], p377 n73. Specific examples of concrete use during the middle ages are: Kendal Castle, in Britain; the gates of the Alhambra; the watchtower above the Geeraliffe Gardens; and the fortifications of Carmona.

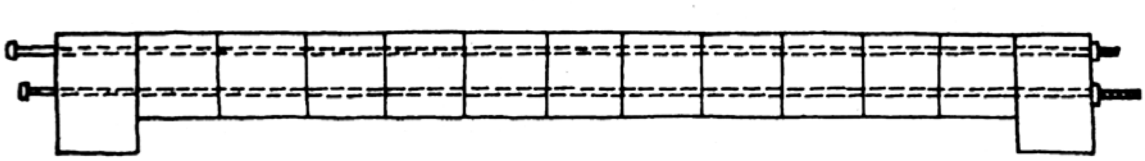

Fig. 2: Prestressed, reinforced masonry beam, Pope, 1811. (Pope)

Fig. 3: First reinforced concrete beam patent, Wilkinson, 1854. Wilkinson used wire rope positioned to resist the principal tensions in the floor. (Mainstone[1])

1756, but the results of his cement experiments were not published until 1791.[11] Smeaton's work led to further experiments by Charles William Pasley[12], Louis-Joseph Vicat and others. Vicat's work, published from 1818 to the 1850s, was particularly influential.[13] Vicat researched the corrosion-protecting qualities of cement on iron wire, and applied this knowledge to the construction of suspension bridge anchorages.[14] Marc Brunel and James Barrett respectively discovered that iron bonded with cement and that these two materials behaved monolithically.[15]

5.2.2 The Precursors of Reinforced Concrete

When reinforced concrete began to emerge in the nineteenth century the idea of combining a material with another to improve its structural behavior was not a new one. Reeds were used to control cracking in sun-baked mud blocks for millennia. Peters writes, "From the late Roman period to the baroque, masons… embedded horizontal and vertical logs in walls for tensile and shear strength. Sometimes they fixed iron bars to the logs and anchored them through to the outside of the building to tie perpendicular walls firmly together. Several logs could be cross-tied together or braced to make elaborate mortised frames that stiffened domes and arches or whole buildings against seismic forces."[16]

The reinforced masonry of the Eglise de Sainte-Geneviève in Paris and cast-iron beams reinforced with wrought iron are both direct precursors to the development of reinforced concrete. (**Appendix A-03, p.A.95, Figure 20 and p.A.103, Fig. 33**) In 1811, Pope published the design for a reinforced masonry beam comprised of masonry blocks tied together by post-tensioned wrought-iron rods located in both the top and bottom of the

[11] Smeaton (1791).
[12] Pasley (1847).
[13] Vicat (1818, 1828, 1846).
[14] Peters[2], p57 and 153; Vicat (1830).
[15] Peters[1], p35.
[16] Peters[1], p65-66.

section.[17] (**Fig. 2**) M. Brunel and Pasley tested a brick cantilever and brick beams reinforced with wrought iron in the 1830s. (**Appendix A-03, p.A.112, Figures 45 and 46**)

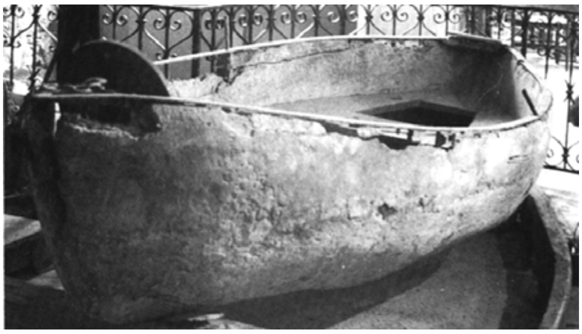

Fig. 4: Ferro-cement boat, Lambot, 1848. (Prade)

5.2.3 The Emergence of Reinforced Concrete in Europe

William Boutland Wilkinson, 1854

In 1854, William Boutland Wilkinson, a plasterer by trade, received the first patent for reinforced concrete with iron bars and cables. (**Fig. 3**) Wilkinson's design shows a marked departure from the composite iron beam – concrete floors built to that date. (**Fig. 21**) Wilkinson use of cables, incapable of resisting compression, shows that he intended the iron to only take tension.[18]

Wilkinson, using the flexibility of the cable, placed the iron to follow the path of the principal tensions under the expected loading. The plasticity of concrete, which is poured *around* the reinforcement, gives the designer the freedom to place the reinforcement most advantageously. Wilkinson splayed the ends of the cables and formed loops to better 'grip' the concrete.[19]

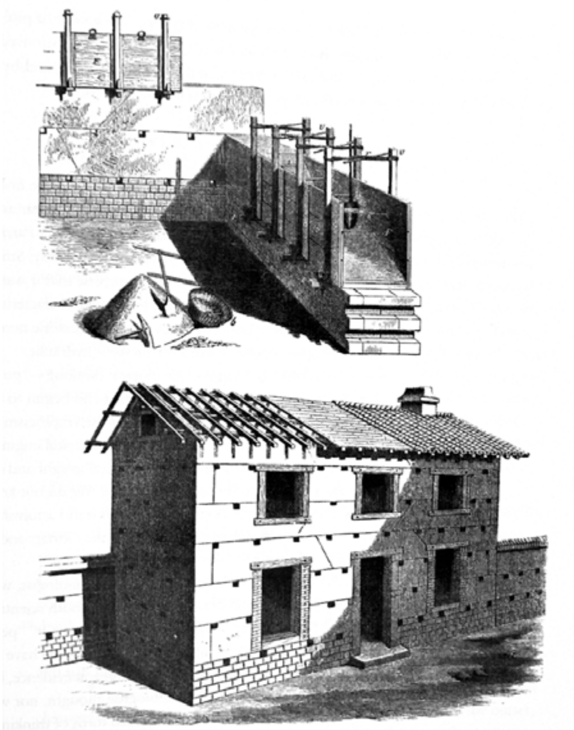

Fig. 5: Illustration of traditional *pisé* construction adapted by F. Coignet to build a massive concrete house, 1852. (Peters[1])

Joseph-Louis Lambot, 1848-1855

Joseph-Louis Lambot built a boat made from a mesh of iron rods encased in a thin layer of hydraulic cement in 1848. (**Fig. 4**) He exhibited the boat at the Paris Exposition of 1855. In the same year he applied for a patent in Brussels for an 'iron cement as a substitute for timber' to be used in structures having to withstand moisture, such as ships, water tanks and tubs for orange trees. The patent included a method for producing a beam or plank. Lambot's system was not commercially successful but his 'invention' did serve the

17 Mainstone[1], p153.
18 Newby, p.xvi.
19 Mainstone[1], p153-154.

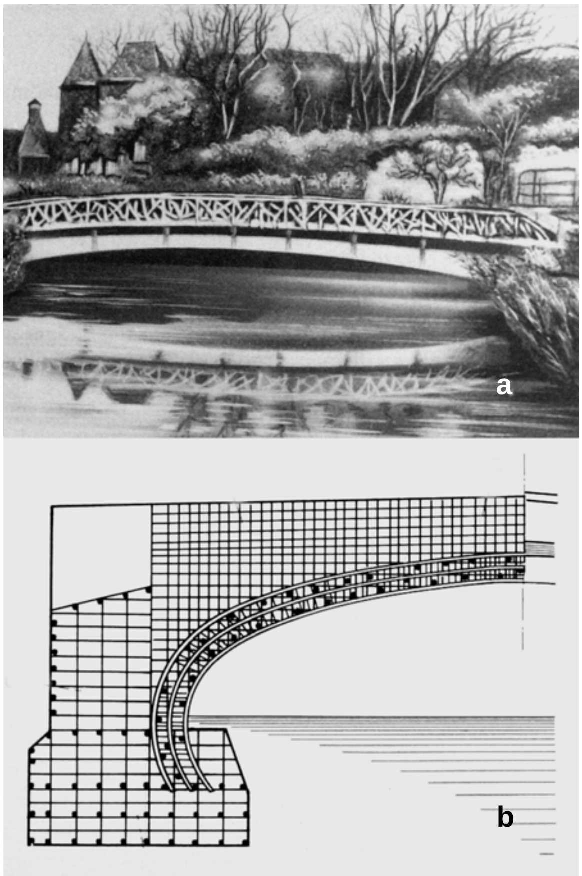

Fig. 6: Reinforced concrete bridge design, Monier. (a) First reinforced concrete bridge in the world, Monier, 1875. (b) Patent design, 1873. (Wittfoht)

useful purpose of raising awareness of the existence of this new material, reinforced concrete.[20] Lambot's construction method for his boat was about three-quarters of a century too early, serving as an obscure precedent to thin-shell reinforced construction pioneered by Eduardo Torroja of Spain, Anton Tedesko, a German émigré to the United States, and Robert Maillart.[21]

François Coignet, 1852-1861

In 1852, François Coignet attempted to build a factory with exposed concrete walls constructed in an ancient manner called *pisé*, whereby earth is compacted between temporary timber panels. (**Fig. 5**) Coignet experimented with different mixes and water content. In his 1861 book, *Béton Aggloméré*, 1861, Coignet suggested the use of concrete with an armature of iron.

Joseph Monier, 1849-1884

Joseph Monier's system of reinforced concrete construction was the first to become commercially successful. Monier's first patent was for flowerpots made of a mesh of iron rods embedded in mortar. He received that patent in 1867, though he made his first pots in 1849. In 1868, Monier extended his patent to include water tanks. He built his first water tank in 1872. Monier next patented a system for constructing bridges and footbridges using reinforced concrete in 1873. (**Figs. 6(b)**) The footbridge shown in Figure 6 was the first known reinforced concrete bridge in the world. Monier built it for the Marquis Tilière in the castle park at Chazelet in 1875. It was 16.5 m long and 4 m wide.[22] (**Fig. 6a**) Finally, Monier patented a design for reinforced concrete beams in 1877.[23] (**Fig. 7**)

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[20] Newby, p.xvii.
[21] Melarangno, p144-149.
[22] Deinhard, p19.
[23] Newby, p.xvii.

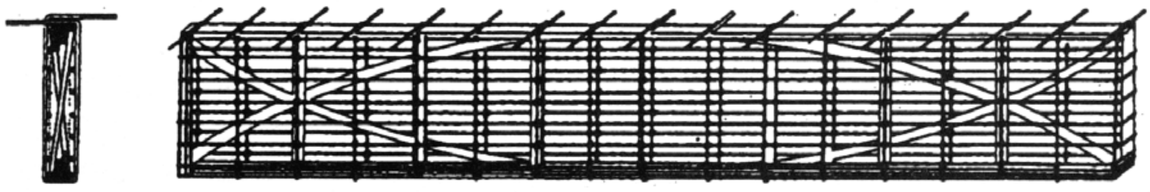

Fig. 7: Reinforced concrete beam patent design, Monier, 1877. (Mörsch, 1902)

Conrad Freytag and G.A. Wayss, Monier System, 1884-1902

The commercial success of the Monier System is principally due to its use by several German firms that purchased the patent rights. Conrad Freytag bought the patent rights for South Germany in 1884, securing right of refusal for the rest of Germany at the same time. The first major structures to be built by Freytag's firm, Freytag & Heidschuch, were six bleaching tanks for a chemical works in Württemberg, which established the suitability of the material for industrial use.[24]

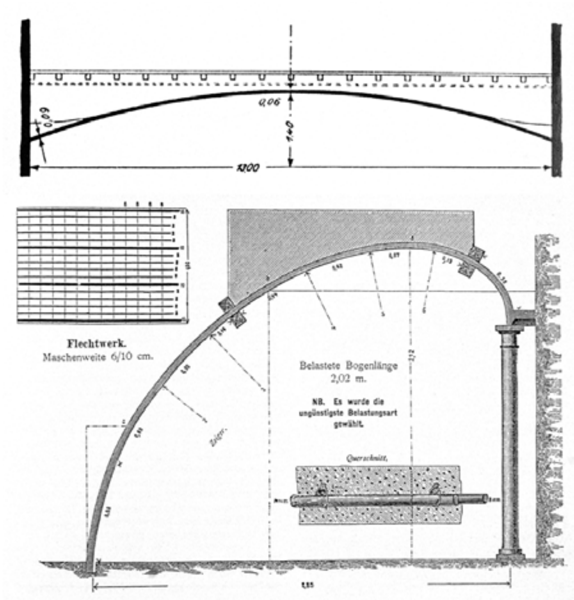

Fig. 8: Thin arch designs by G.A. Wayss using Monier System, 1887. (Wayss)

G.A. Wayss, a graduate of the Technische Hochshule in Stuttgart, bought the North German rights to the Monier patent from Freytag & Heidschuch in 1885. Except for two small areas, these two firms controlled the patent rights for all of Germany.

Wayss and Freytag & Heidschuch jointly carried out fire tests, reinforcement pullout tests, and load tests in Berlin to prove the reliability of the system to the authorities and potential clients. In 1887, Wayss published his work with the results of the tests performed in Berlin in *Das System Monier*. Also included in this book was a structural theory produced by the Prussian State architect Matthias Koenen for the performance of Monier floors and vaults. In 1893, Wayss and Freytag formed a new company by the same name after Freytag's partner, Heidschuch, died in 1891.[25]

The publications of Wayss and Freytag gave credibility to reinforced concrete construction and had a significant impact on future development. Maillart specifically mentions Monier's system as being very influential to the origins of reinforced concrete.[26] The designs for a slab and asymmetrical vault published in Wayss's 1887 book may have influenced Maillart to search for less massive solutions to reinforced concrete structures. (**Fig. 8**) In 1890, Wayss

[24] Newby, p.xviii.
[25] Newby, p.xix.
[26] Bill, p17-18. From "Design in reinforced concrete" (translation), by Robert Maillart, first published in the *Schweizerische Bauzeitung*, 1 January 1938.

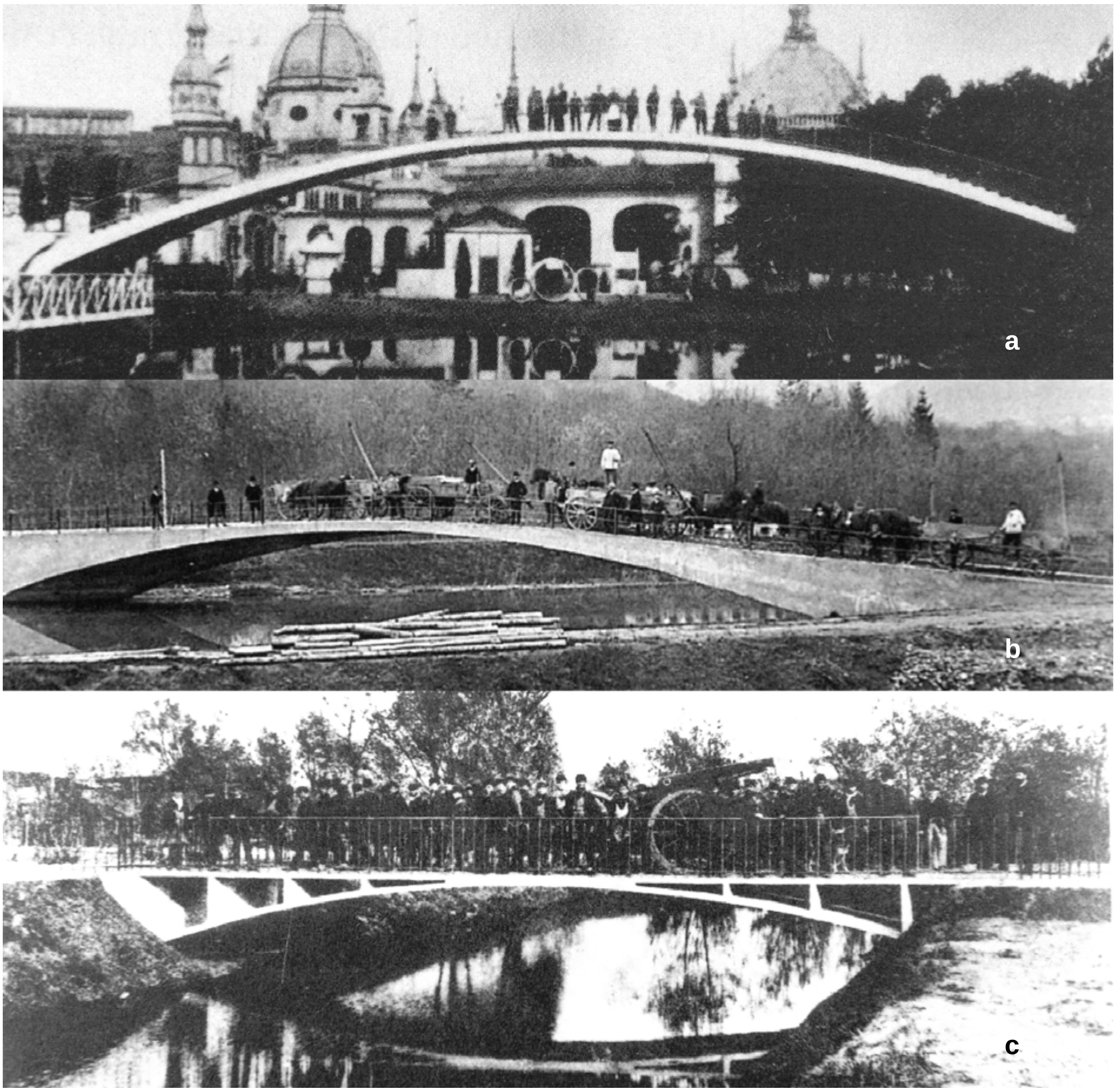

Fig. 9: Three reinforced concrete bridges with thin arches, built by Wayss & Freytag with adapted Monier system. (a) Bremen, Germany: 40 m span, 25 cm thick at center; with Matthias Koenen, 1890. (b) Wildegg, Switzerland: 37 m span, 3.5 m rise, 45° skew, 20 cm thick at crown; 1890. (c) Location unknown: 37.2 m; 1890. (Wittfoht, (a); Bosc, (b); Sutherland, (c))

built an arch bridge with a span of 37.2 m. The arch was only 25 cm thick.[27] (**Figs. 9(a), 9(b) and 9(c)**). **Figure 9(c)** shows a bridge that exhibits an elegance characteristic of Maillart's future bridges.

In the 1890s, the firm learned of the Hennebique system and exploited a loophole they found in the method of calculation to essentially build Hennebique-type structures without paying any royalties or becoming a concessionaire.[28] In 1902, Wayss & Freytag published Emil Mörsch's book *Der Betoneisenbau, seine Anwendung und Theorie*, which publicized his

[27] Sutherland, p15; Deinhard, p20.
[28] Newby, p.xx.

structural theory of reinforced concrete and became the standard text in the German-speaking world. Mörsch was then head of the technical department at Wayss & Freytag.[29]

Edmond Coignet, 1850-1900

Edmond Coignet, François Coignet's son, patented various reinforced concrete products, including: pipes, aqueducts, beams and piles. In 1892, Coignet was awarded the contract to build the main drainage system of Paris using reinforced concrete, thereby giving reinforced concrete a significant vote of confidence by the public sector of the construction industry. At the 1900 Paris Exhibition, Coignet constructed the Chateau d'Eau, which was 45.7 m (150 ft) high.[30]

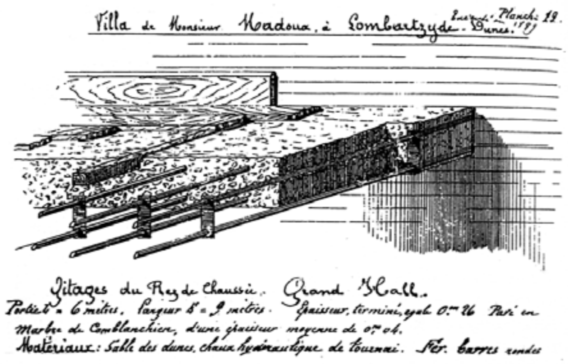

Fig. 10: Early reinforced concrete floor by Hennebique for the Madoux house, Lombartzyde-Dune, 1889. (Bosc)

François Hennebique, 1879-1915

François Hennebique is arguably the most recognizable name from this early period in the reinforced concrete industry. Hennebique, a mason by trade, started his own business when he was twenty-five years of age. He started to use reinforced concrete around 1879. From this date, Hennebique seems to have researched the design of reinforced concrete beams in collaboration with the chief engineer for Belgian roads.[31] **Figure 10** shows an early reinforced concrete floor Hennebique constructed for the Madoux residence in Lombartzyde-Dune in 1889.[32]

In 1892 Hennebique patented his now famous monolithic reinforced concrete system in Belgium and France. (**Fig. 11**) As the success of his system grew, Hennebique ceased to be a contractor and organized a whole network of agencies throughout Europe staffed with engineers and architects and associated contractors specially trained by him. In 1896 alone, over 800 buildings and civil engineering projects were constructed through Hennebique's agencies in Europe. At the 1900 Paris exposition, a pavilion represented Hennebique's system of construction. Hennebique was himself responsible for the construction of two spiral staircases and the floors of the Grand Palais and the Petit Palais.[33]

Ironically, the success of Hennebique's system on display with other proprietary systems at the 1900 exposition marked the beginning of the end of the proprietary systems then dominating the market of reinforced concrete structures.[34] By 1915, competition from an ever more open field of designers and contractors led to the eventual dissolution of the monopolistic type power the Hennebique organization and a few other proprietary systems[35]

[29] Newby, p.xxii.
[30] Huberti, p53-55.
[31] Newby, p.xix.
[32] Bosc.
[33] Newby, p.xx.
[34] For a good list of the many proprietary reinforced concrete systems around 1900, see Marsh and Dunn.
[35] Monier, Considère, Coignet, Ransome…

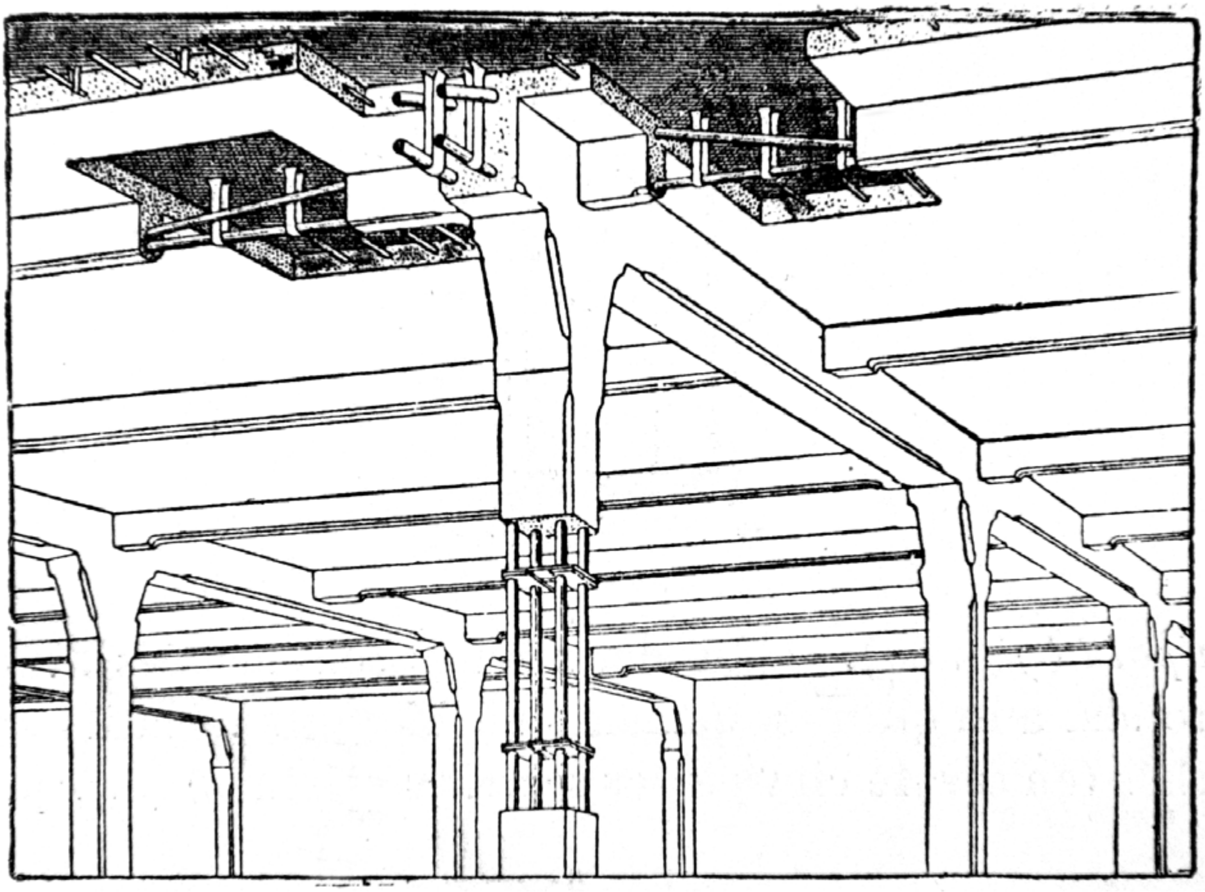

Fig. 11: Axonometric view of the Hennebique System of reinforced concrete building construction, patented 1892. (Bosc)

seemed to have. Hennebique licensees continued to design and build structures, though with significantly smaller market share. The 1901 collapse of the Hotel zum Goldenen Bären in Basel, a Hennebique structure, provoked the establishment of codes that hastened the decentralization of reinforced concrete design.[36] Hennebique's lasting contribution to the field of reinforced concrete was to increase the credibility and reliability of this new material because of the exacting quality control standards he demanded from his engineers and licensed contractors, and by disseminating knowledge through his widely distributed in-house journal, *Le Béton Armé*.[37]

The 1900 Paris Exposition

The 1900 Paris Exposition was widely influential in the development of the reinforced concrete industry. As we have seen, proprietary construction systems from Edmond Coignet, Hennebique, and others were on display for all to see and examine. Hennebique's spiral staircases and Coignet's Chateau d'Eau must have made a great impression on both architects and engineers, such as the Swiss engineer Robert Maillart, who had visited the exposition.[38]

[36] Peters[1], p78.
[37] Newby, p.xxi, 148 and 215.
[38] Newby, p.xx-xxi.

The success of Monier, Coignet, and Hennebique led many others to patent reinforced concrete designs, primarily differentiated by the geometry of the reinforcement. By 1902 there were over 100 different systems patented.[39] In this early period, specialists, such as those representing Hennebique or Wayss & Freytag, performed design and material specification. The design and use of this new material was beyond the normal capabilities and experience of architects, engineers, or builders. Slowly, information kept 'secret' began to be made public, beginning with Charles Rabut's lectures on reinforced concrete in 1897, at the Ecole des Ponts et Chausées.[40]

Fig. 12: First reinforced concrete building in the USA, Ward and Mock, 1876. (Collins)

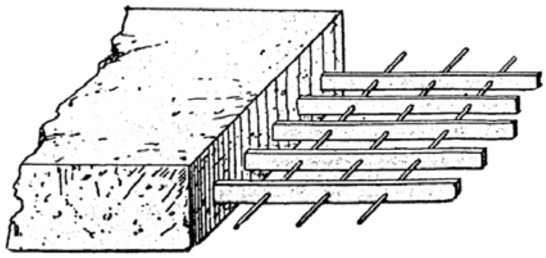

Fig. 13: 'Fireproof' reinforced concrete floor design, Hyatt, c.1875. (Bosc)

One significant outcome of the 1900 Paris Exposition was professional dissatisfaction with the reinforced concrete market being held hostage by a few proprietary systems that hindered innovation and investment in this new material. As the uses of reinforced concrete expanded from buildings to heavy civil applications such as large foundations and maritime constructions, Hennebique's influence, which was focused on building construction, waned and information on how to design reinforced concrete structures was increasingly available in the public domain.[41]

5.2.4 The Emergence of Reinforced Concrete in America

Cement and Concrete in America

The use and development of cement and concrete in America closely paralleled that in Europe, with cement used on a large scale in canal construction such as the Erie Canal around 1817. Cement was largely imported until the end of the nineteenth century when domestic production of Portland cement became sufficient to meet demand at a lower cost than imported cement.[42]

William E. Ward, 1876

William E. Ward was a pioneer in reinforced concrete construction in the United States, building a 'fireproof' house in 1876 in Port Chester with the architect, Robert Mock. (**Fig. 12**)

[39] Marsh (1904); Marsh and Dunn (1906).
[40] Newby, p.xxi.
[41] i.e. Christophe in France (1899); Morel in France (1902); Mörsch in Germany (1902); Emperger in Germany (1901-1942); Buel and Hill in the US (1904).
[42] Newby, p.xxii.

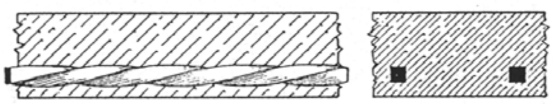

Fig. 14: Twisted square-bar iron reinforcement patented by Ransome, 1884. (Pauser)

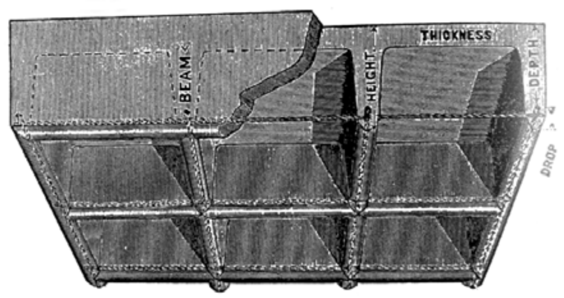

Fig. 15: Ribbed floor construction system, commonly known today as the waffle slab, Ransome, 1889. (Sutherland)

The building was test loaded and Ward presented his work to the American Society of Mechanical Engineers in 1883.[43]

Thaddeus Hyatt, 1875-1878

In 1875, Thaddeus Hyatt, who had developed an iron and glass block sidewalk construction system, invented a fireproof floor system consisting of upright wrought-iron plates with holes through which lateral round bars were placed. This framework was enclosed within a thin concrete slab. (**Fig. 13**) This so-called 'slab ceiling' spanned between the bottom flanges of **I**-section wrought-iron beams, which had a separate cover of concrete. Hyatt modified his system to a composite iron – concrete floor in which he used an inverted **T**-section of iron to take tension and used the concrete slab to resist compression. The inverted **T**-section was subsequently replaced by round bars to make a reinforced concrete beam.[44]

Hyatt performed tests that showed that iron, steel and concrete have compatible coefficients of expansion. While Hyatt's interest was to determine that the reinforced concrete would not be damaged by fire due to internal stresses caused by different thermal expansion coefficients, this property is a fortuitous coincident of nature that makes reinforced concrete possible. Hyatt published a book in 1877[45] in which he included test results carried out by David Kirkaldy in London on 1.5 m (5 ft) span beams with different forms of reinforcement.[46] Hyatt retired from his work with reinforced concrete in 1878 after patenting his system to construct reinforced concrete domes up to 61 m (200 ft) in diameter that incorporated glass block prisms.[47]

Ernest Ransome, 1874-1899

Ernest Ransome, an immigrant to America from England, started his own building firm in 1874. He was involved in the construction of some of the earliest reinforced concrete buildings in America. In 1884 Ransome patented the twisted square reinforcing bar. (**Fig. 14**) In 1889 he introduced the economic ribbed floor construction, commonly known as the waffle-slab, still in use today. (**Fig. 15**) Ransome designed the first reinforced concrete bridges in America. He built several in California in 1889. These bridges were used as models for many other small-span bridges. Ransome was the most prominent reinforced concrete contractor in America until the late 1890s when the Monier, Melan and Hennebique systems became established there.[48]

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[43] Newby, p.xxii.
[44] Newby, p.xxii-xxiii.
[45] Hyatt.
[46] Newby, p.xxiii.
[47] Newby, p.xxiii.
[48] Newby, p.xxiii.

The Ferro-Concrete Company, 1902

In 1902 the Ferro-Concrete Company designed and constructed the 16-story Ingalls Building in Cincinnati. (**Fig. 16**) The design was made up of a combination of the systems of Ransome, Monier, Hennebique and Considère. Considère was a system that was patented in 1904.[49] At the time, the Ingalls Building was the tallest reinforced concrete building in the world.[50]

Fig. 16: Ingalls Building, 16 story reinforced concrete structure designed and built by the Ferro-Concrete Company, Cincinnati, 1902. (Collins)

The American Concrete Institute, 1905-Present

The American Concrete Institute (ACI) was founded in 1905 with the purpose of regulating the design of reinforced concrete structures in America. Carl Condit writes,

> The primary document in the history of concrete construction during the twentieth century is the *Report* of the Joint Committee of the American Concrete Institute and the American Society of Civil Engineers (1909)[51] on the fundamental standards of reinforced concrete design. This report became the basis of all American metropolitan building codes and of much subsequent development in the theory of elasticity of reinforced concrete and the stress analysis of concrete structures.[52]

The influence of the ACI code has gone far beyond America, being the basis for reinforced concrete design regulations in numerous countries around the world.[53]

5.2.5 The Development of Concrete Floor Construction Until 1905

Until the end of the eighteenth century, floors typically were either shallow vaults of concrete or masonry, or comprised of a hierarchical

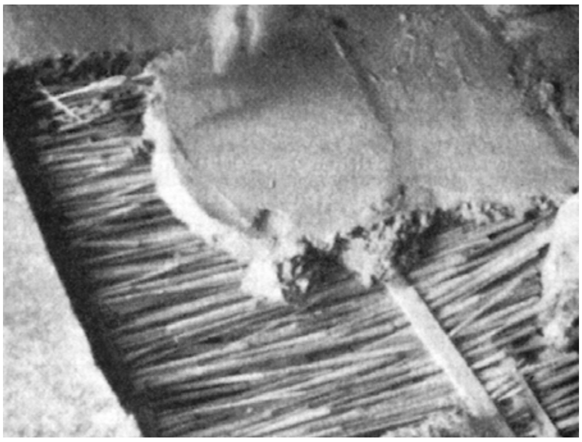

Fig. 17: Straw or reed reinforced plaster floor, used in the East Midlands of England from Elizabethan times up to the 19th century. (Newby)

[49] Newby, p.xxiv.
[50] Sutherland, p17.
[51] These may have been based on the Prussian and Swiss codes written between 1901-1906. i.e. *Provisorische Normen für Projektierung, Ausführung und Kontrolle von Bauten in armiertem Beton*, SIA (1903) and *Provisorische Vorschriften über Bauten in armiertem Beton auf den schweizerishen Eisenbahnen*, Eidg. Post- und Eisenbahndepartement (1906).
[52] Condit, p346.
[53] Faulkes, p15.

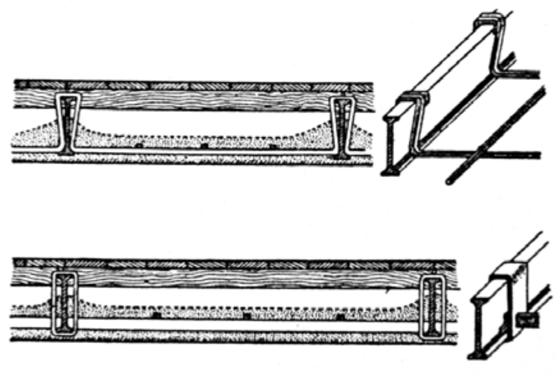

Fig. 18: French method of constructing iron floors, 1840s. (Hamilton)

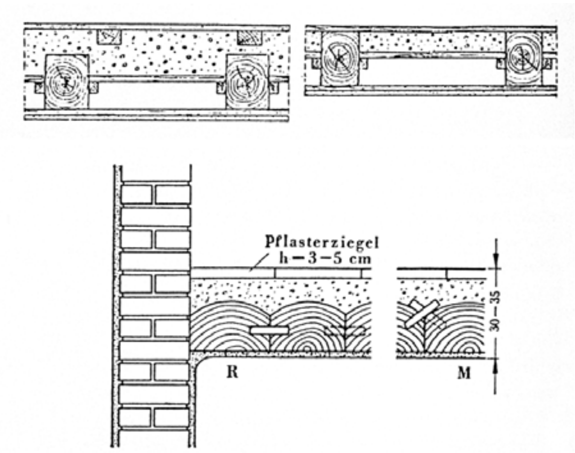

Fig. 19: Composite timber-concrete floor systems. (Pauser)

Fig. 20: Cast-iron beam floor bearing system with shallow masonry or concrete vault supported by bottom flange of beam. Shown: Dennett's arch floor. (Sutherland)

system of beams and columns that supported some kind of wearing surface. However, the use of plaster reinforced with straw or reeds in the East Midlands of England from Elizabethan times up to the 19th century represents one of the earliest precursors to reinforced concrete floor slabs.[54] (**Fig. 17**) Similarly, fire-resisting ceilings were constructed in Paris from the 1840s by supporting plaster of Paris by means of wrought-iron bars and joists, between and around which it was poured as shown in **Figure 18**.[55]

Probably the first reinforced concrete floors were composite structural systems made by pouring concrete onto closely spaced timber beams. (**Fig. 19**) Such a system would only work compositely if some measure were taken to form a liaison between the timber and the concrete, such as nails that would protrude from the timber into the concrete.

The modern development of reinforced concrete floors in the nineteenth century is linked to the early fireproof floors that were constructed in England at the end of the eighteenth century and the first decade of the nineteenth with shallow brick vaults spanning between the lower flanges of cast-iron beams.[56] The masonry in this system was subsequently replaced by concrete. (**Fig. 20**)

The next step was to use the concrete compositely with the iron, such that the concrete resisted compression while the iron primarily resisted tension. (**Fig. 21**) This composite construction emerged about the same time that composite wrought- and cast-iron beams were being developed in England in the 1840s.[57]

As we have seen above, Wilkinson took the next significant step in reinforced concrete floor design by replacing the iron girders with

[54] Sutherland, p49.
[55] Hamilton, p450.
[56] See also **Appendix A-03, p A.96, Fig. 23**.
[57] **Ref. Appendix A-03, p A.102-104**.

cables that were intended to resist only the tensile forces of the floor. (**Fig. 3**) Wilkinson's design is a true reinforced concrete structure because it is constructed to act as a composite material that behaves monolithically. This is different from a composite structural system in which the principle load bearing material does not a secondary component of different material to be structurally stable.

François Coignet patented a floor system similar to the ceiling system shown in **Figure 18**, but he made the system more robust by embedding his framework of **I**-beams with transversal bars completely in the concrete. (**Fig. 22**) This system is an early product in the evolution of the two-way spanning system of the flat slab.

The Monier system is characterized by thin arches and vaults and composite iron-concrete floors, but Maillart claims that Monier's system also included a floor comprised of reinforced concrete beams with a slab that spanned a significantly larger distance between support beams than in the Hennebique system.[58] (**Figs. 23 and 24**)

Hyatt and Ransome made notable developments in reinforced concrete floor construction. Their designs were clearly intended to transfer load in two directions, though Hyatt's iron grid reinforcement still shows a hierarchical bias. (**Fig. 13**) Unfortunately, the work of Hyatt and Ransome had little effect in the reinforced concrete construction market as a whole, because as of 1900, the Hennebique System had a virtual monopoly on the market.

In the first decade of the twentieth century, when the flat slab was finally realized both in America and Europe, reinforced concrete floor construction was dominated by the Hennebique's system of a slab laid on joists

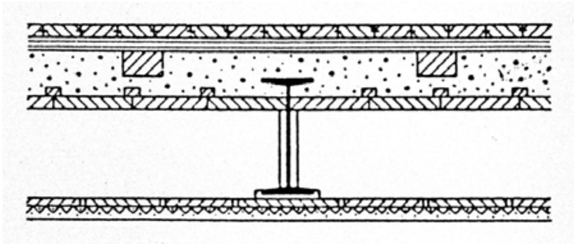

Fig. 21: Composite iron-concrete floor structure using **I**-beams. (Pauser)

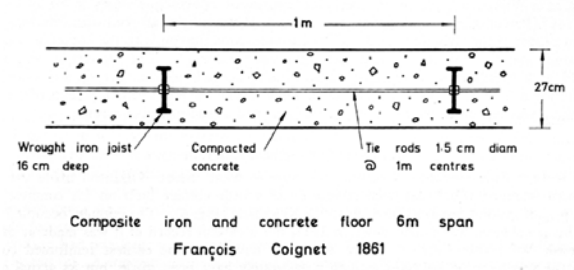

Fig. 22: French system of **I**-beams with transversal members, embedded in concrete, Coignet, 1861. (Pauser, after Skempton)

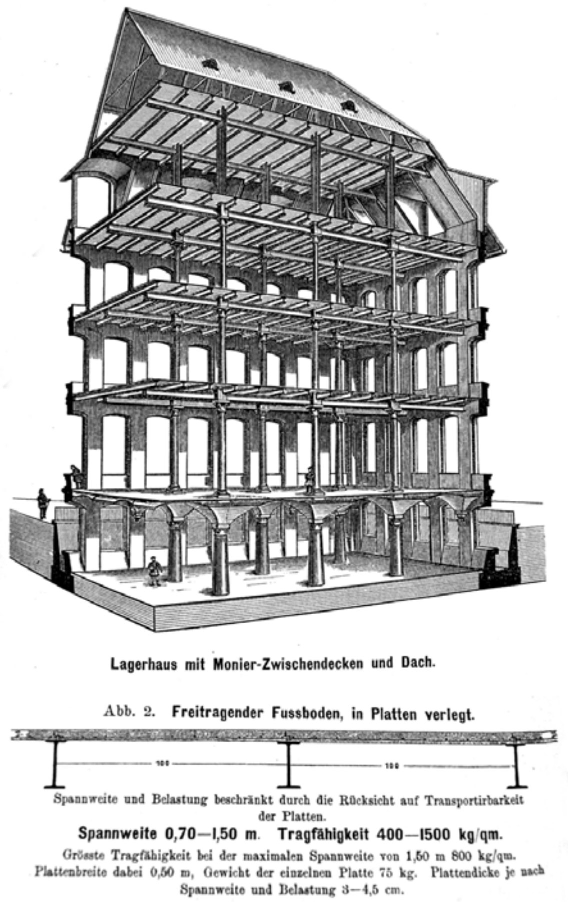

Fig. 23: Monier Composite System (Wayss)

58 Bill, p17-18. From "Design in reinforced concrete" (translation), by Robert Maillart, first published in the *Schweizerische Bauzeitung*, 1 January 1938.

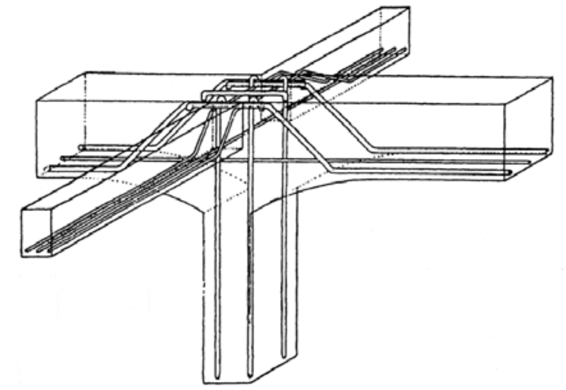

Fig. 24: Monier floor system, probably constructed by Wayss & Freytag. (Newby)

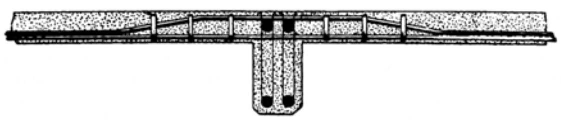

Fig. 25: Reinforcement detail of **T**-beam design incorporated by Hennebique. (Mörsch)

that transferred the load to larger beams. (**Fig. 11**) One notable improvement in Hennebique's system was to detail the reinforcement so that a part of the slab acted monolithically with the support beam such that the structural section was a **T**-beam. (**Fig. 25**) Nonetheless, the load transfer system was clearly unidirectional.

In this early period of reinforced concrete construction, floor design was apparently designed as a substitute for typical iron, steel, and timber structures. Sozen and Siess observed,

> Just as the first motorcars were built to look like horse-drawn carriages, the first reinforced concrete systems were conceived in the image of traditional types. In a timber structure, the planks carried the load to the joists, the joists to the girders, and the girders to the columns; so must they in a reinforced concrete structure. Hence, the flat slab had to be invented rather than developed as one of the obvious applications of reinforced concrete.[59]

The properties of reinforced concrete were not fundamentally understood in this early period of development. This lack extended to both the concrete itself and its behavior when combined with iron to form a new, composite material. The building culture itself hindered development, building massive masonry structures and light iron or wood structures composed of linear elements because of education and construction practice. Early experimentation of the reinforced concrete's particular qualities, notably Monier's thin shell forms, gave way to the more traditional forms of the Hennebique system. This was the situation when C.A.P. Turner and Robert Maillart invented the flat slab.

[59] Faulkes, p1-2, quoting from Sozen and Siess, "Investigation of Multiple-Panel Reinforced Concrete Floor Slabs", from *Journal of the American Concrete Institute* (Proceedings Vol. 60, No. 8) pp. 999-1027, August 1963.

5.3 Antecedents

5.3.1 Early Forms

The earliest slab-type structures were probably large, flat stones placed on top of some type of support, such as in the footbridge shown in **Figure 26**. However, like a bridge composed of logs placed side by side, this is not a true slab, but rather a simple beam. The large stones that can still be seen today covering megalithic tombs may represent a better approximation of the slab, though it is doubtful in any of these cases that the first intention of the designer was to find a way to distribute load in more than one direction. The main limitations of stone and wood are their limited strength and the maximum size in which either can be procured. Wood is also anisotropic and of limited width because of its growth pattern, making it unsuitable for slab like structures unless engineered to act as a plate such as in the form of plywood.

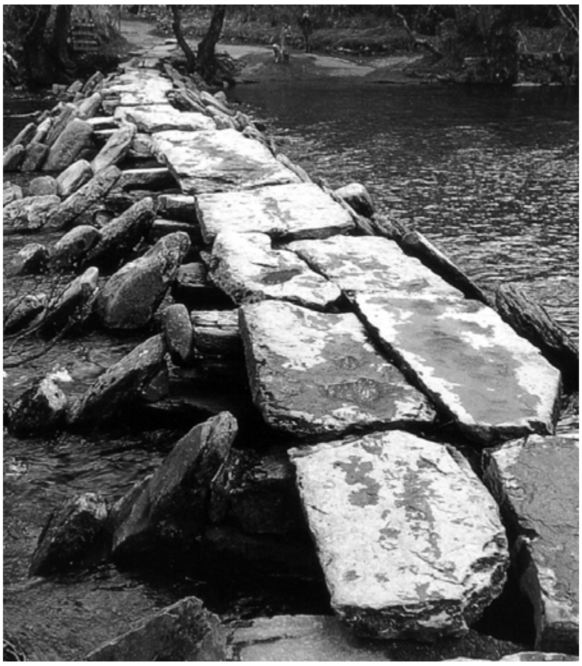

Fig. 26: Primitive bridge using flat stones that are simply supported. (Brown)

5.3.2 Unreinforced Domes

Domes are often cited as early antecedents to shell structures and three-dimensional structures in general. In such a view, these domes would transmit compressive and tensile forces continuously over the whole surface as shown in **Figure 27**. If there is enough transversal resistance, then point loads can be distributed over a wider area of the dome, reducing the necessary thickness of the dome and the need for heavy buttressing to contain the lateral thrust of the dome. The problem is that masonry and unreinforced concrete, such as used in the Pantheon in Rome, cannot support the tensile ring stresses shown. This tension results in a radial cracking pattern characteristic of masonry domes, as shown in **Figure 28**. Such cracking results in a structure that is not a dome in a structural sense, but rather an arch array. This behavior of the masonry dome is what makes it possible to put windows at the base of a dome, such as

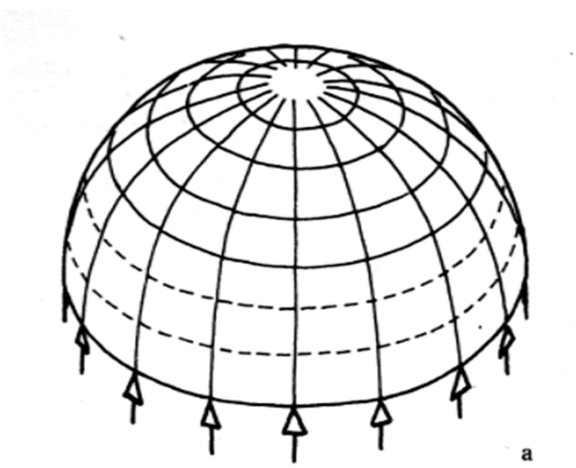

Fig. 27: Internal static equilibrium of domical shells with principal compressions shown by full lines and principal tensions by broken lines. (Mainstone[1])

Fig. 28: (a) Cracking around the base of a concrete semi-dome, Baths of Trajan, Rome. (Mainstone[1])

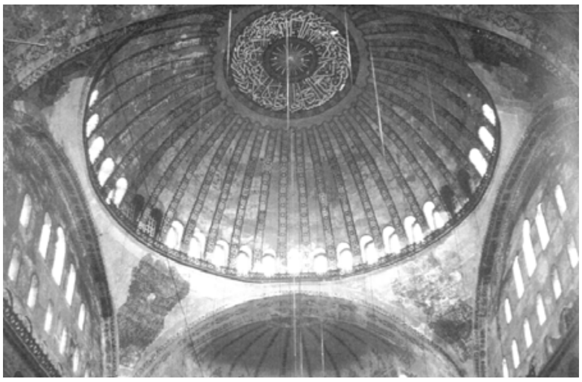

Fig. 29: Dome of Hagia Sophia, illustrative of arch array principle. (Mainstone[1])

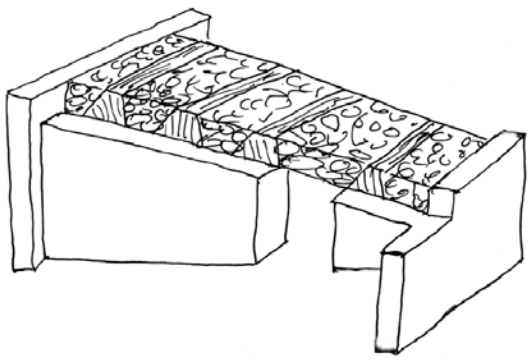

Fig. 30: Two-way timber-masonry vault construction typical of the Italian alpine province of Sondria and the adjacent Swiss Poschiavo Valley. (Dooley)

in the Hagia Sophia, where the highest tensile ring stresses would normally be if it were a true dome structure. (**Fig. 29**)

5.3.3 Timber Supported Flat Vaults

In Medieval Europe, flat vaults were built with timber members. Tom F. Peters describes one form, called *antuada* in the Italian alpine province of Sondrio and *volta plana* in the adjacent Swiss Poschiavo Valley. "This structure is made by wedging stones tightly between parallel, closely spaced, hardwood joists of inverted voussoir-shaped cross section. The wedging transforms most of the bending stress along the beams into thrust on the walls parallel to them, making a composite structure that transfers loads two-dimensionally."[60] (**Fig. 30**)

The *antuada* system is unlike the later development of the fireproof construction in English textile mills at the turn of the nineteenth century that supported shallow masonry vaults on the bottom flanges of cast-iron beams. (**Fig. 20**) The arch action in this cast-iron construction is not intended to transfer a great amount of load transversally to the beams, but to simply transfer all loads to the beams to be carried longitudinally in the normal manner. The only lateral thrust exerted on the perimeter walls comes from floor vaults adjacent and parallel to the edges of the building. Adjacent vaults counteract the thrust of all vaults resting on interior beams.

5.3.4 Reciprocal Frame Structures

A reciprocal frame structure is made of timber beams shorter than the overall span whereby timber beams mutually support one another. Examples of such structures are shown in **Figures 31(a) to 31(d)**. These examples show the variety and potential complexity of such structures. As John Chilton explains, "Before the development of large-scale

[60] Peters[1], p66 and Peters[3].

production methods for iron and steel, the primary strong, lightweight construction material was timber, which was used extensively for relatively long-span structures."[61] To span distances longer than the timber available, carpenters built-up timber members from smaller members, employed trussed construction, and created these reciprocal frames.

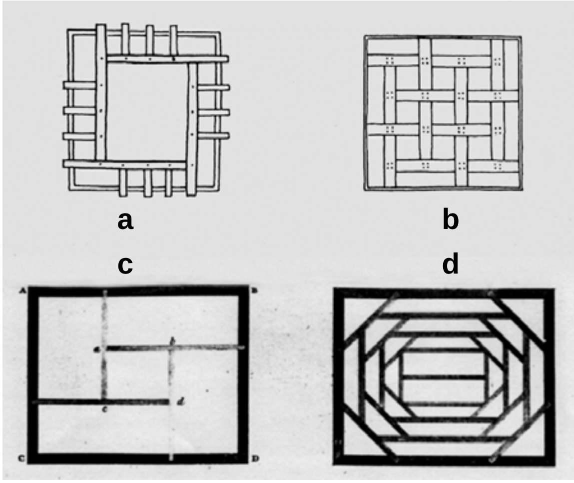

Fig. 31: Examples of reciprocal timber frame structures. (a) 13th century design by Vuillard de Honnecourt. (b) 16th century design b Serlio. (c) and (d) Two designs published by Thomas Tredgold in 1853. (Mainstone (a) and (b), Tredgold (c) and (d))

Vuillard de Honnecourt drew **Figure 31(a)** in his sketchbook under the title 'How to work on a house or tower if the timbers are too short.' This method of construction incorporates the principles of two-way load distribution. Mainstone writes that reciprocal floors "would serve the purpose provided that the loads were not too great; but only at the expense of greater bending-moments in the individual beams than if they had spanned the whole way from wall to wall, and of a tendency for the beams to pull apart where one rested on another. The resulting floor would be appreciably more flexible than one with continuous beams from wall to wall. It nevertheless recurs with variations in numerous later texts up to the late-nineteenth century, and was used, for instance, by Christopher Wren in the tower of the Divinity Schools at Oxford and for framing the first floor and ceilings of Independence Hall in Philadelphia."[62]

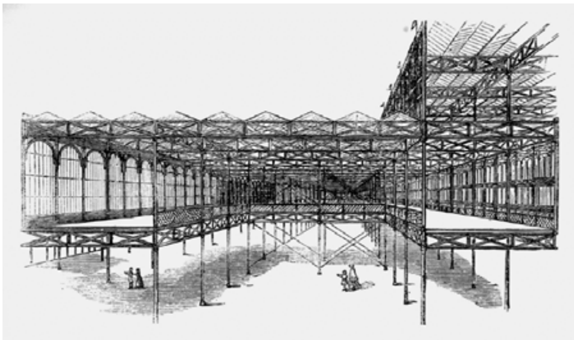

Fig. 32: General view of modular structural system realized by Charles Fox, 1851. (Peters[1])

5.3.5 Two-Way Load Distribution System Employed in the Crystal Palace, 1851

Charles Fox, the contractor who built the Crystal Palace, employed an ingenious system to distribute loads evenly to the girders on all four sides of the modular grid of support columns. (**Fig. 32**) Peters describes how Fox used 'three-dimensional technological thinking' to build the Crystal Palace speedily and economically. Peters writes,

[61] Chilton, p163.
[62] Mainstone[1], p146.

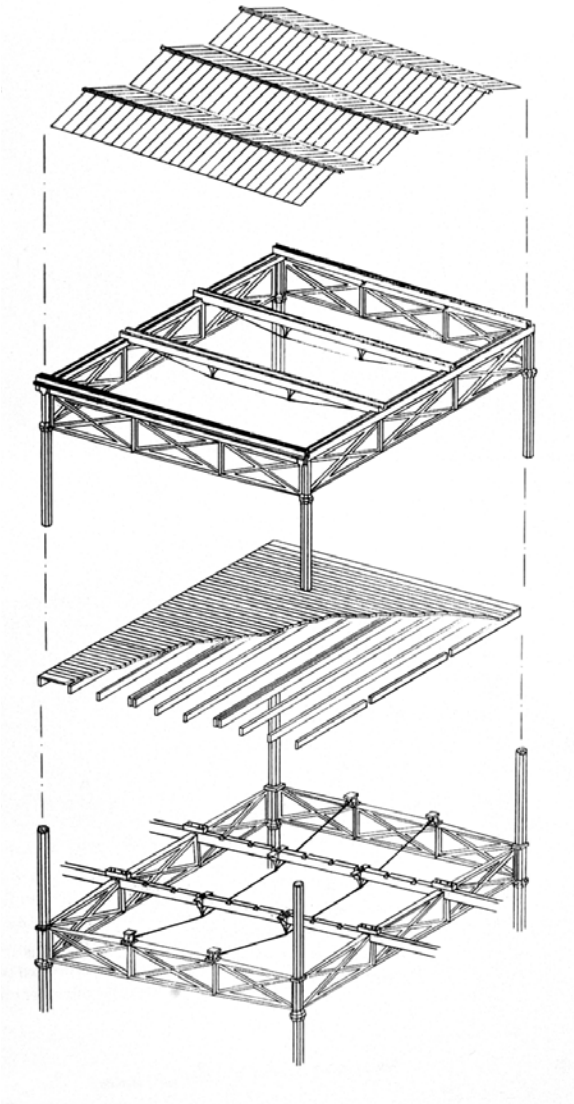

Fig. 33: Exploded axonometric drawing of Fox's inventive two-way load carrying structure for the Crystal Palace. (Peters[1])

Most structural designers think primarily in two dimensions, even today. They design a building in plan and cross-section and create frames two-dimensionally, then stack them one behind the other to form a three-dimensional building… [Joseph] Paxton's sketch for the Crystal Palace is … a cross-section, and the design he developed from it was an extrusion of that cross-section. But Fox designed his structure differently, as a gridded module that was the same in both directions. A cross-section of his module east-west is identical to its north-south cross section. He designed the module to be the same in both the *x*- and the *y*-axes so that it could be added to equally in both directions. This meant that it was structurally non-directional, and the trusses on all four sides of the module were the same.

However, he had to carry the roof and a wooden floor on those trusses, and both of these subsystems were directional because the gutters and joists spanned in one direction only. [**Fig. 33**] In the case of the roof, Fox had one set of trusses carry spread loads, while the other carried the underspanned gutters, which lay 2.4 meters apart, as point loads. The floor was more of a problem: the joists had to be more closely spaced than the roof gutters because boards can only span 40 to 60 centimeters. The joists that lay over the trusses could transmit very little load to the one set of trusses, while the other trusses would have to carry far more. Either Fox had to raise the whole floor higher and put the joists on beams, which would have been wasteful, or he had to devise another system to spread the load equally over the trusses spanning in both directions. To solve this problem he used the same technique he had developed to post-tension the gutter, but rotated the wrought-iron tension rods at right angles. The rods held up two beams that could be weaker because they

> were supported at their third points. He notched the joists into the beams and leveled their tops with the girder, attaching them to the girders by means of primitive straphangers. the tension rods ended at the third points of the other pair of cast-iron girders, which enabled him to distribute the floor loading equally to all four edges of each module and made the girder pair that ran parallel to the floor beams carry their full share of the load. The point is not whether or not this was a good solution, but that the rotation was a simple modification of an underspanning technology that required a shift in geometry and a complex ability to think three-dimensionally.[63]

The floor system devised by Fox is an illustrative example of the benefits of the two-way flat slab in reinforced concrete. By distributing the load of the floor beams in two directions Fox could reduce the depth of the beam he needed to support the floor. The economy of such a system is evident in the figure shown.

5.3.6 Plate Structures and the State of Two-Way Floor Systems, 1905

By the turn of the 20th century, the only field that considered plates or slabs, as a structural element was in mechanical engineering for the construction of boilers.[64] The only suitable plate theory that existed at the time was by the German engineer F. Grashof in the second edition of his book *Theorie der Elasticität und Festigkiet*.[65] In structural engineering, plate structures were little used. Cast-iron plates were used in the construction of viaduct troughs in England at the end of the 18th century,[66] but further development was limited due to the enormous weight of iron plates. Research conducted during the design of the Britannia Bridge led to the development of the plate girder, though this was treated simply as a large beam subject to certain local buckling phenomena. The global form was still a one-dimensional spanning system and therefore did not take full advantage of the anisotropic properties of iron and steel on a macro scale.

In 1905, when C.A.P. Turner published the first commercially successful design for a flat slab in the United States, the predominant flooring system in reinforced concrete construction was Hennebique's system base on steel and timber construction. As we saw, Hyatt and Ransome made early developments of true two-way spanning systems. (**Figs. 13 and 15**) Ransome's waffle slab is a close antecedent to the flat slab. If he had simply reduced the depth of the beam grid to be the same depth as the slab then he would have produced a system similar to the system Robert Maillart patented in 1908 in Switzerland.

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[63] Peters[1], p248.
[64] Bill, p165. From "The development of the beamless floor-slab in Switzerland and in USA" (translation), by Robert Maillart. Originally published under the title "Zur Entwicklung der unterzuglosen Decke in der Schweiz und in Amerika", in the *Schweizerische Bauzeitung*, vol. 87, 1926.
[65] Grasfhof; Timoshenko, p409.
[66] e.g. Telford. **Ref. Appenidix A-02, p.A.45-A.46**.

5.4 The Flat Slab in the USA: C.A.P. Turner, 1905

5.4.1 The First Flat Slab and Mushroom Column

In 1905, the *Engineering News* published C.A.P. Turner's article in which he described his design for a 'mushroom slab.' (**Fig. 34**) Turner is generally given credit for invention of the flat slab, in large part because his was the first commercially successful flat slab design.[67] However, there is one recorded flat slab design that precedes Turner's. In 1902, Orland W. Norcross, an engineer from Boston, patented a flat slab system of reinforcement radiating from the columns so as to eliminate connecting beams between columns for economy. He successfully built a structure on this principle but it seems that his design and name have become a mere footnote in the history of the flat slab. I have only found his patent mentioned in one source.[68] The question arises of whether Turner was aware of Norcross' design. I do not know, though it is curious that Turner waited until 1908 to patent his own design, which seems to bear some resemblance to the description of Norcross' design.

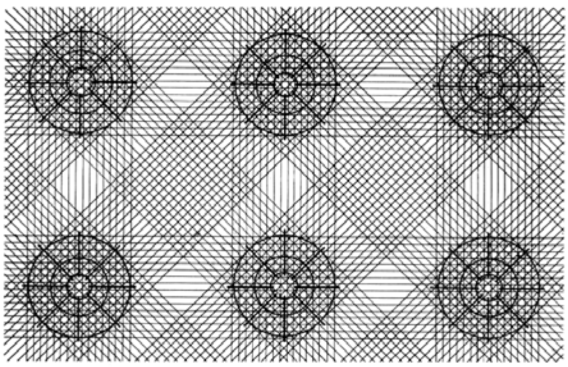

Fig. 34: Turner's flat slab design showing reinforcing iron arranged in four layers radiating from the center of the column in 45° intervals, 1905. (Bill)

Writing in the "Engineering News" in 1905, Turner presented what was probably the first published design of a flat slab. He wrote,

> There is ample room for all to improve radically on present methods of design and computation, which are… following after the manner of structural iron work. Thus far experimental investigation has been confined almost exclusively to simple beams and slabs reinforced in one direction only…. That concrete lends itself readily to reinforcement in all directions should lead the practical constructor, as far as may be, to so reinforce his work that the deformation from a strain in one direction may be offset in part by a force in another direction. Enclosed herewith a study along this line, the idea being to avoid the expensive forms for beams to secure a neat and unbroken ceiling line together with a considerable economy of material without sacrifice of strength.[69]

C.A.P. Turner's design was composed of mushroom columns with the reinforcing bars in the slab arranged orthogonally and diagonally between columns, giving four distinct layers of reinforcement that resist the principle bending moments acting radially in all directions in the vicinity of the column heads. The disadvantage of Turner's arrangement was that by using four layers of bars it was not possible to have adequate reinforcement in the top of the slab section over the columns to counteract negative moment because the thickness of the bar

[67] Faulkes, p2.
[68] Sutherland, p18.
[69] Faulkes, p2. From "Discussion of Reinforced Concrete Warehouse for North-West Knitting Co. Minneapolis, Minnesota", by C.A.P. Turner. First published in *Engineering News*, Vol. 54, No. 15, October 12, 1905. pp 383-384.

layers meant that bars in one direction were closer to being in the middle of the slab than near the top.[70] Turner used a mushroom-type capital for his columns to counter the high level of vertical shear between the interface of the slab and the column. Turner's first mushroom head design, which has a sharp interface between the mushroom head and slab, was later modified to include a supplementary drop-panel to further distribute the load of the slab to the column head. (**Figs. 35(a) and 35(b)**) The problem with Turner's first design is that the sharp connection between the two structural elements creates an area of stress concentration. Turner probably encountered shear-punching failures and modified his design to include the drop-panel. The drop-panel is a wasteful and inelegant remedy since it increases material usage and still includes a sharp interface between slab and column head. As Maillart pointed out in his critique of Turner's design, it would have been better if Turner had simply made a smooth transition between mushroom head and slab as shown by the dashed line in **Figure 35(b)**.[71]

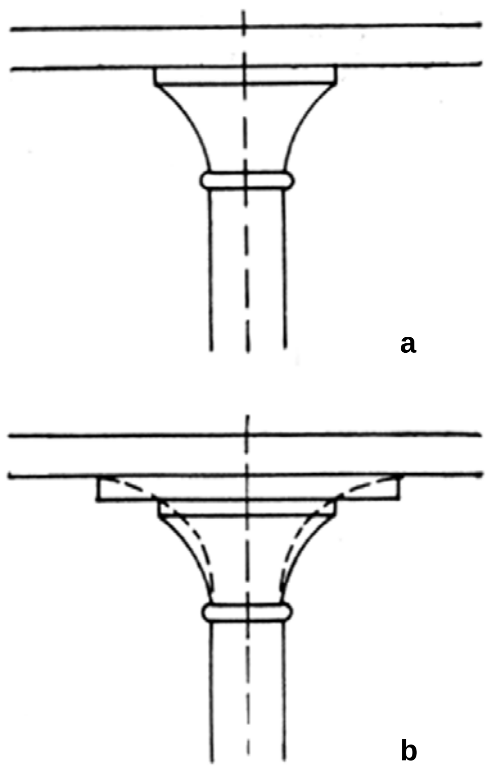

Fig. 35: (a) Turner's original 'Doric style' mushroom column head design. (b) Turner's modified design incorporating a drop-panel. Dashed line shows improved design proposed by Maillart. (Bill)

5.4.2 Acceptance

The flat-slab system was rapidly accepted in the USA. Between 1906 and 1913 the flat slab was used for 80% of all buildings designed for loads of 4.8 kN/m^2 (100 psf) or more.[72] The majority of these buildings were industrial and agricultural facilities. Turner claims that his firm had "introduced the mushroom system in about $200,000,000 worth of buildings and bridges, of spans from 3.7 m to 15.2 m (12 ft to 50 ft), in seven years."[73] Most of this business was for private clients, who were willing risk using a new structural system if it would save construction costs and increase the rentable space. However, this acceptance was not unqualified. Turner wrote in 1914: "The conservative businessman who advances the money, as the writer has found by experience, would usually like a bond, which may amount to anywhere from $5000 to $100,000, to assure him that the structure when completed will come up to guaranty." The "guaranty" involved tests of load carrying capacity and maximum deflection.[74]

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[70] Bill, p165-166. From "The development of the beamless floor-slab in Switzerland and in USA" (translation), by Robert Maillart. Originally published under the title "Zur Entwicklung der unterzuglosen Decke in der Schweiz und in Amerika", in the *Schweizerische Bauzeitung*, vol. 87, 1926.
[71] Bill, p165-166.
[72] Faulkes, p3.
[73] Faulkes, p6.
[74] Faulkes, p3.

5.4.3 The Controversy in Early Theory and Design Methodology

One significant reason for the owner's request for load tests and a guarantee was because the engineering profession was unable to agree on a method for flat slab design. The cost of a flat slab varied markedly depending on which engineer designed it. Under such conditions, "the business man," as Faulkes writes, "was undoubtedly justified in his conservatism."[75] Sozen and Siess comment,

> For the structural engineer, plate action was an entirely new concept. The 'crossing beam analogy' thinking of the slab as two perpendicular beams each carrying a certain proportion of the load in relation to their stiffnesses, helped only to foster the still existing illusion that only part of the load need be carried in a given direction. Grashof's work had already been used by the mechanical engineers in boilerplate problems. However, this work was represented in American engineering literature either simply as formulas without any derivations or as a basis for arriving at questionable conclusions.[76]

The flat-slab debate went on inconclusively and was, at times, confused. One professor of engineering, H.T. Eddy of the University of Minnesota, even argued that flat slabs defied Newton's laws. Turner, who claimed to base his calculations on Grashof's theory, designed his flat slab panels for a negative moment over interior columns equal to WL/50, where W is the total load on the panel (stress/unit area x area) and L is the span length.[77] Maillart later dismissed Turner's claims as being more original than convincing.[78] The American engineer Angus B. McMillan, concluded that the correct figure for this moment was WL/25 also using Grashof's theory. Proponents of the so-called cantilever method doubled the figure again, and used steel quantities four times greater than Turner's.[79]

The practical effects of the controversy over how to design a flat slab were illustrated by McMillan in 1910 when he tabulated the amount of reinforcement needed for a 6 m x 6 m (20 ft x 20 ft) interior panel of a flat slab carrying 9.6 kN/m^2 (200 psf) live load using 6 different methods then in use for the design of flat slabs. (**Table 1**) Methods 1 and 2 used the cantilever approach, and methods 3, 4, 5 and 6 were deduced from Grashof's work.[80]

The table shows that the quantity of reinforcement used could vary by as much as 400% depending on which design method was used. Turner's method gave the smallest quantity of all; small wonder that his clients demanded bonds dependant on satisfactory performance of their buildings in load tests…. Yet Turner's slabs stood, and tests indicated that steel stresses at working loads were well within safe limits. Perhaps the only point of agreement among disputants was that flat slab analysis was complex and [as Maillart stated,] "hardly solvable by calculation."[81]

[75] Faulkes, p3.
[76] Faulkes, p4.
[77] Faulkes, p7.
[78] Bill, p166. From "The development of the beamless floor-slab in Switzerland and in USA" (translation), by Robert Maillart. Originally published under the title "Zur Entwicklung der unterzuglosen Decke in der Schweiz und in Amerika", in the *Schweizerische Bauzeitung*, vol. 87, 1926.
[79] Faulkes, p7.
[80] Faulkes, p5.
[81] Faulkes, p6-7.

Table 1: Comparison of Five Flat Slab Design Methods c.1910

Design Method	Slab Thickness		Steel Stress		Amount of Reinforcement per Panel	
	in	mm	psi	MPa	lbs	kg
1. Cantilever	8	200	16,000	110	2,189	993
2. Turneaure & Maurer	12	305	16,000	110	1,931	876
3. Grashof	8	200	16,000	110	784	356
4. Mensch	8	200	16,000	110	2,120	962
5. Turner* (a)	8	200	16,000	110	549	249
(b)	8	200	13,000	90	718	326
6. McMillan	8	200	16,000	110	1,084	492

* Turner used and recommended a steel stress of 13,000 psi.
(a) was included for purposes of comparison.
Source: Faulkes

Turner's methods were supported by the evidence of his accomplishments. Besides the total worth of all the mushroom slabs Turner erected, his system had been repeatedly tested and guaranteed for strength. As Turner rightly observes, "The record of achievement must have behind it something more than mistaken ideas."[82] Turner's flat slab design was widely used for a time in Europe as well as America, before being supplanted by Maillart's design.

5.5 The Flat Slab in Switzerland: Robert Maillart, 1908

5.5.1 Brief Biography of Robert Maillart

Robert Maillart studied at the Swiss Federal Institute of Technology, in Zurich, under Professor Wilhelm Ritter, who made important contributions to the development of graphic statics. Maillart graduated in 1894. After gaining experience with a small engineering firm, Maillart moved to the Zurich Cantonal Department of Public Works, where he designed and built a mass concrete arched bridge, which the city architect clad in stone. Next, Maillart went to work for a contractor who was building Hennebique-type structures.[83]

In 1901, Maillart had the opportunity to design his first reinforced concrete bridge, a three-hinged arched structure spanning 38 m at Zuoz. (**Fig. 36**) Maillart's design for the Zuoz Bridge was comprised of slabs and walls joined together to form a monolithic box construction, the first known structure of its type in reinforced concrete. This bridge shows Maillart's appreciation of the unique properties of that material.[84]

[82] Faulkes, p6.
[83] Billington, p49-50.
[84] Billington, p13-18.

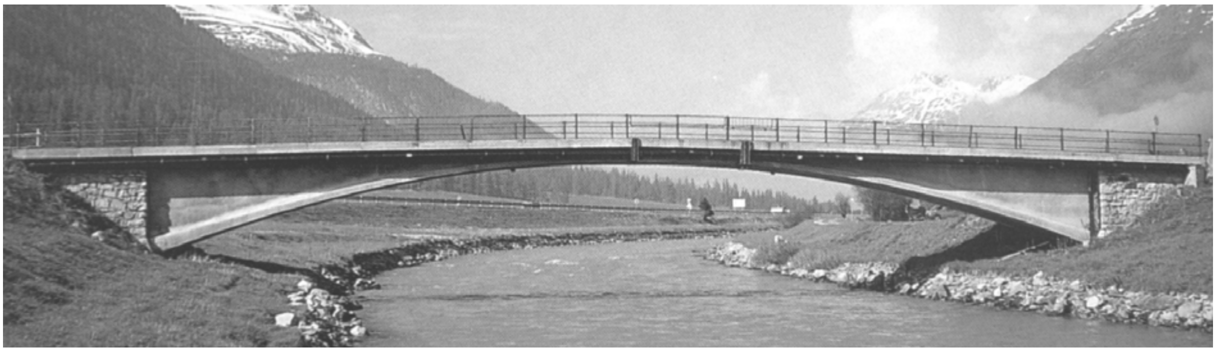

Fig. 36: Zuoz Bridge, first reinforced concrete box section bridge with plain spandrels, Robert Maillart, 1901. (Brown)

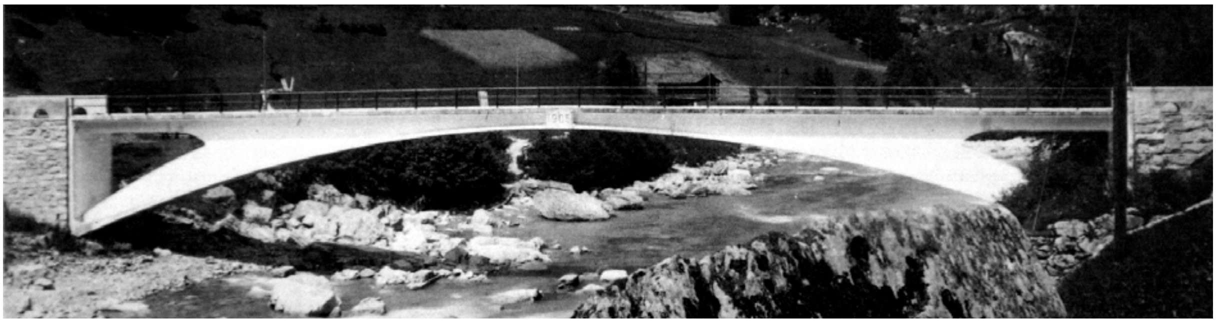

Fig. 37: Tavansa Bridge, 1905. Maillart's improved box section design with material removed from areas of spandrels where cracking was observed in the Zuoz and Bilwil bridges. (Billington[1])

In 1902, at the age of 30, Maillart founded his own design and contracting firm and successfully tendered for a bridge at Bilwil. Maillart's design for this bridge was similar to the one at Zuoz. Like at Zuoz, Maillart observed that small cracks occurred in the solid spandrels, which he attributed to temperature and humidity. In his next bridge design, for the 51 m span Tavansa Bridge built in 1905, Maillart removed material from the spandrels, resulting in the distinctive form shown in **Figure 37**.[85]

Maillart's experience and knowledge gained from the development of the concrete box and its component plate structures must have influenced his thinking about slab structures. The cracking Maillart observed at Zuoz and Bilwil would have raised questions about the two-dimensional stress-distribution characteristics of such plate structures used in his box section bridges.

Maillart turned his attention to the nature and economics of floor construction around 1905. In that year Maillart built the Pfenninger factory in Wädenswil. For the floor design, Maillart eliminated the joists that are characteristic of the Hennebique system, and spaced the columns farther apart than was typical in Hennebique construction, resulting in larger girders and longer slab spans. (**Fig. 38**) Maillart continued to build slabs in this manner for two years before he planned and executed a series of research tests in his construction yard to develop a reliable design for a completely beamless floor.[86]

[85] Billington[1], p26, 36 and 38.
[86] Billington[1], p50.

5.5.2 Conception, Design Development

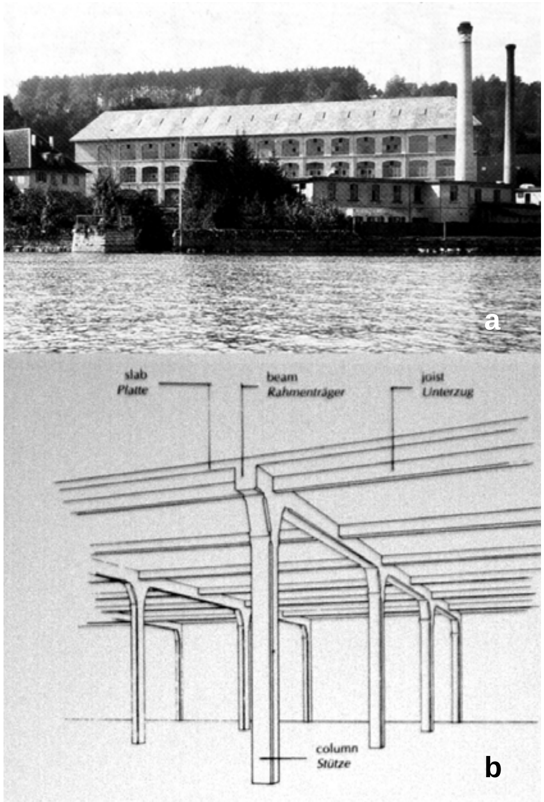

Fig. 38: (a) View of the Pfenninger factory in Wädenswil, Maillart, 1905. (b) Maillart's floor design for the Pfenninger factory eliminates the joists and lengthens the span of the slab in comparison to Hennebique's design. (Billington[1])

Maillart realized, unlike Turner, that it was unnecessary to align the reinforcement with all the changing directions of the principal moments. If it ran in any two directions at right angles, it would also be capable of resisting tension at its level in any intermediate direction. Having only two layers of reinforcement also made it easier than Turner's system to place the reinforcement closer to the correct depths to resist bending.[87] Describing his conception of the flat slab, Robert Maillart wrote,

> The possibilities of reinforcing the concrete slab in a crosswise manner, however, introduced a new type of building element, capable of taking bending stresses in any direction, not only in the two directions of reinforcement, and this meant that it obeys different laws to those upon which the beam theory was used. The procedure of dividing a slab up into strips can never again be applied, even as a rough approximation only. For a suitable theoretical method for solving this problem the Grashof formulas were available, but they were not adequate for the structural engineer, because they are applicable only for a simple case and provide no solution for loadings that are not uniform, or for varying degrees of fixture and a varying section modulus. In order to first obtain a foothold concerning the constructive possibilities of the reinforced concrete slab a few small experimental buildings were set up in the work yard of Maillart & Cie. in Zurich and were loaded in a primitive way with sacks. It resulted that what corresponded to freely turning point supports was unusable in practice on account of the great deflection occurring and the fact that it broke early in the region of the support. Another structure, however, consisting of nine fields with slanting junctions between columns and slab, proved of such stiffness, even with point loads in single fields, that the practical value of the system was proved. This building is of historical importance as it is certainly the first beamless floor of the 'two-way' system. The arrangement of the reinforcing is illustrated in **Figure 39**.[88]

[87] Mainstone, p157.
[88] Bill, p165-166. From "The development of the beamless floor-slab in Switzerland and in USA" (translation), by Robert Maillart. Originally published under the title "Zur Entwicklung der unterzuglosen Decke in der Schweiz und in Amerika", in the *Schweizerische Bauzeitung*, vol. 87, 1926.

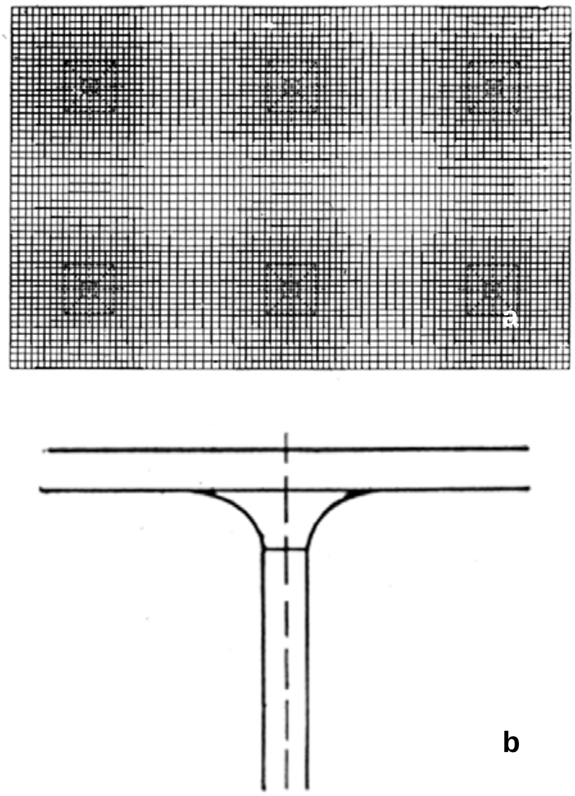

Fig. 39: Maillart's flat slab with (a) two-way arrangement of reinforcing steel and (b) mushroom column head with smooth transition between the slab and column, 1907. (Bill)

After Maillart had proven his system to be practical, which he recognized as behaving differently from any of the traditional structural elements, he had to create a method for design. Maillart continues,

> The problem now was how could it be constructed and dimensioned. [sic] The purely theoretical way appeared to be inaccessible and although theoretical solutions have been found today even for ununiformly distributed loading and the slab is given its long deserved consideration as a special structural element, this valuable theoretical knowledge has little to offer for practical use, even until today [1936]. Because the elastic resistance of the column and stiffening above it play such a decisive part, as these earliest experiments demonstrated, and because of the difficulty of taking these conditions fully into account even in the beam, one can hardly hope that a calculation method for use in practice will be evolved in the near future.
>
> In order to achieve an experimental basis for the problem, and to acquire a sufficient basis for practice, the same firm erected a large structure with nine fields each of four meters length. The slab was only 8 cm thick to assure the greatest possible elasticity. At the same time, a number of beams resting on freely turning supports were built of the same thickness and with the same reinforcement. Beneath the slab at distances of 25 cm were measuring points. With the aid of a mobile crane a single load of 1000 kg could be loaded at any desirable position; a meter grid gave 144 loading points. The influence of the single loads could now be judged since the deflection curve of the slab could be compared with the deflection of beams under the corresponding known bending moments. The shear stresses might have damaging effects especially in the concrete but not on a serious enough scale for consideration in building practice. Through the addition of the ordinates in the single curves, the curve of deflection is obtained for different loadings, and if all the curves are brought together, that are of equal sign for the curvature at any given point, the most dangerous loading for this position is found. Since some of the edges of the outer fields are built-in and some are free to turn or rest on beams supported by columns, the effects in the end fields could also be found. It is this dealing with the end fields that is the cause of the greatest theoretical trouble. [89]

[89] Bill, p165-166.

Maillart patented a flat slab design on January 20, 1909. He emphasized the great load-carrying capacity of monolithic concrete, the low cost, and the aesthetic appearance of the flat slabs with smoothly curved capitals over the columns.[90]

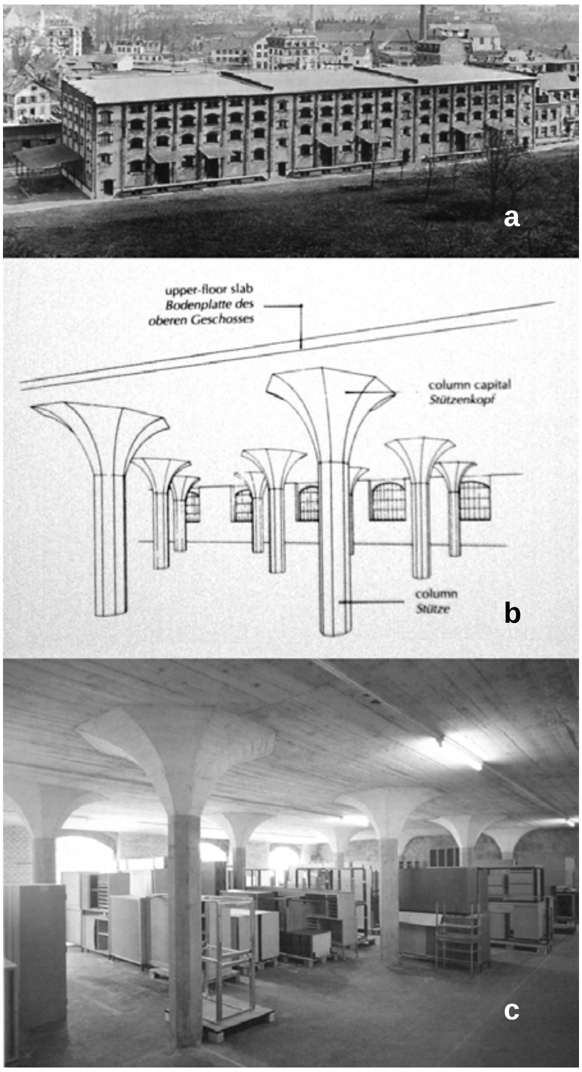

Fig. 40: Lagerhaus Gesellschaft Building, 1910. (Marti, (a); Billington, (b) and (c))

5.5.3 Influence of Turner

David Billington, who has had extensive access to the public and private documents of Robert Maillart, claims that Maillart appears to have independently developed his design for a flat slab with mushroom columns, dismissing the need for any further exploration of the topic. While I do not necessarily doubt this conclusion, I find the emphasis on an independent flash of brilliance to be an unnecessary qualifier to Maillart's achievement. It is difficult for me to believe that Maillart was completely ignorant of Turner's invention after his design was published in a widely distributed American journal of engineering and his design was used in the construction of perhaps hundreds of structures by 1908. Definitive proof is evidently not available, but the significance of Maillart's achievement is not at all reduced in any case.

Independently of knowledge about Turner's design or not, Maillart rationalized the design of the mushroom column supported flat slab in a way that clearly exhibits a superior command and understanding of the structural principles and behavior of such a structure. Maillart's arrangement of reinforcement and his design of the mushroom capital are more simple, economic, and efficient than Turner's.

5.5.4 Application

The first commercial construction using Maillart's flat slab was the Lagerhaus-Gesellschaft Building, constructed in Zurich in 1910. (**Fig. 40**) The slab was designed for a loading of 2000 kg/m^2 (410 psf) Billington writes,

> Maillart's patent of 1909 shows an uninspired triangular capital connecting the columns to the slab, a solution that was dramatically improved [in the Lagerhaus-Gesellschaft Building]. Maillart achieved this continuous curvature with a concrete form that uses only straight formwork. He was the building contractor as well as the designer, and he therefore had to be economically

[90] Billington, p50.

> competitive. But he wished to make an elegant form that would express clearly the flow of forces between the horizontal floor structure and vertical column. It was again Maillart's design view that led him to this new form in spite of there being no mathematical theory to guide him. Whereas the applied scientists struggled for formulas, Maillart built numerous buildings with no difficulties and substantial profits.[91]

Maillart designed the column heads with a hyperbolic form so that the resistance to shear stresses would be constant. In later designs, Maillart formed the mushroom head flatter to equalize the resistance to negative bending over the columns as well. In Maillart's words, "In both cases the column fuses into the floor slab corresponding to the play of forces."[92]

Maillart's mushroom column supported flat slab design quickly became the standard in Switzerland while the rest of Europe used Turner's system if using a flat slab at all. Maillart's system spread to France, Spain, and Russia due to his personal business ventures in those countries. Interestingly, Maillart specifically notes that development in Germany lagged behind other countries in the development and application of the flat slab "for want of official sanction for this type of construction."[93] The mushroom-headed column was later eliminated, after Maillart, to further maximize the use of the space under the flat slab, shear stresses being resisted by a steel structure incorporated into the thickness of the slab over the column. Finally, the economic and structural advantages of Maillart's system superceded Turner's design. Maillart's two-way arrangement of reinforcement is now standard throughout the world today.

5.6 A New Material, Implementation, Theory vs. Practice

5.6.1 Innovation and Regression

While concrete and iron had been known for hundreds of years, their combination constituted a new material when they were made to work compositely. In reinforced concrete structures, the concrete is designed to take compressive stresses and works compositely with the steel transfer shear stress. The steel resists tension. Reinforced concrete emerged progressively out of the development of iron floor systems with masonry or unreinforced concrete infill. Despite early pioneering efforts, reinforced concrete only stood out as a new, unique material when Lambot exhibited his boat, flowerpots and water tanks, that is, in wholly original forms for what was until then considered a masonry type material, commonly referred to in England as 'artificial stone.' Monier similarly demonstrated these new applications for this material. What is striking is that both Lambot and Monier took a material most people would have associated with structures primarily in compression and used it in applications that are clearly subjected to significant tensile stresses.

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[91] Billington, p50-51.
[92] Bill, p165-166. From "The development of the beamless floor-slab in Switzerland and in USA" (translation), by Robert Maillart. Originally published under the title "Zur Entwicklung der unterzuglosen Decke in der Schweiz und in Amerika", in the *Schweizerische Bauzeitung*, vol. 87, 1926.
[93] Bill, p165-166. From "The development of the beamless floor-slab in Switzerland and in USA" (translation), by Robert Maillart. Originally published under the title "Zur Entwicklung der unterzuglosen Decke in der Schweiz und in Amerika", in the *Schweizerische Bauzeitung*, vol. 87, 1926.

Interestingly, when Monier expanded the applications for his system of reinforced concrete to building construction, he largely reverted to established means of floor construction, encasing iron beams in concrete. However, Monier's system, as exploited by the German firm Wayss & Freytag, was also used to construct exceptionally thin arches.

The demonstrated success of Monier's patent led others to follow suit. The next thirty years were ones of intense experimentation, principally with respect to the shape and arrangement of the reinforcing iron. The problem lay in the fact that most of these developments were protected by patents that were not "conducive to a parallel development in understanding the fundamental properties and behaviour of this form of construction."[94] Yeomans continues,

> While considerable experimental work had been carried out by the patentees, this was naturally directed towards proving the performance of their own particular method. What was needed was the intervention of professional and academic research which would be able to examine more general aspects of performance. The pattern of reinforcing which was in the control of the designer clearly affected the behaviour of the structure, but how was not certain. Performance also depended upon site workmanship but again it was not clear how the behaviour of the completed structure was related to this.[95]

Nonetheless, towards the end of the nineteenth century, more was understood about the importance of aggregate quality and controlling the proportions of cement, aggregate and water. Ironically, this knowledge came with a marked turn towards conservative design, notably represented by the Hennebique System.

5.6.2 System Hennebique: A Case of Substitution

Through a combination of good design, quality control, organizational management, marketing, and fortuitous timing, the Hennebique System became the dominant reinforced-concrete structural system in the world at the turn of the twentieth century. However, as concrete went from a niche to a mainstream product, the structural system utilized by Hennebique clearly appealed to that segment of the population that was uncomfortable using a still-new material with many unknowns about its structural behavior. Hennebique's system of joists, beams, and columns made it easy to understand how the material was working. By using a familiar form there was a performance benchmark by which to judge the material against established materials.

From this study, we see a potentially important development pattern in modern materials. That is, the material first undergoes a period of innovation, bringing attention to it and creating a body of knowledge about its properties and behavior. Then, for reasons of economy and marketability, the material is substituted for an established material. This subject is specifically address in the body of this thesis.

Yeomans writes,

> As reinforced concrete became more frequently designed by consulting engineers rather than by the system owners, the professions became interested in developing a greater understanding of its behaviour. The design and construction of buildings in reinforced concrete clearly needed more

[94] Yeomans, p114.
[95] Yeomans, p114.

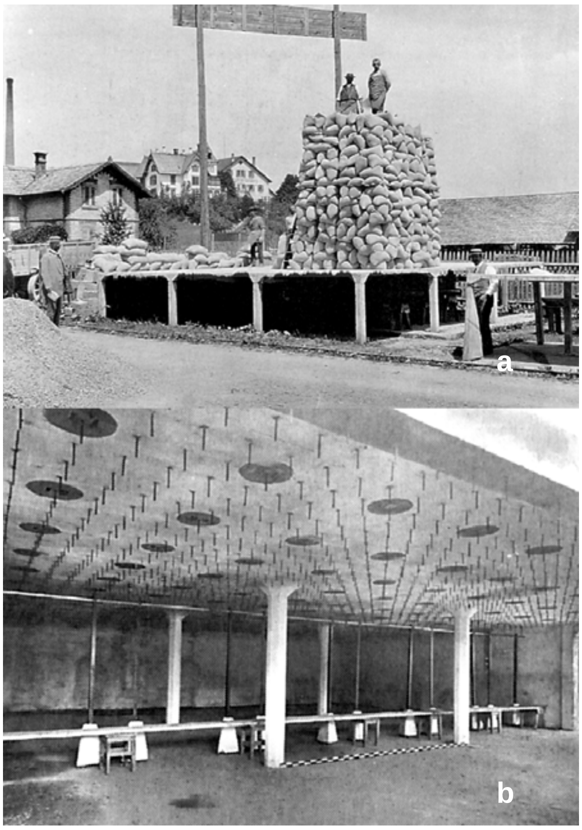

Fig. 41: Maillart's (a) loading and (b) deflection tests used to establish an empirical method of flat slab design. (Marti)

understanding and involvement by the engineer than did steel construction. In addition, while steel was limited to simple beams and columns, the plasticity of reinforced concrete offered the possibility of building forms which had not been at first envisaged and these needed to be understood.[96]

5.6.3 Theory and Practice

The flat slab originates from the desire to make the most efficient use of a new material and construction technique. Its invention at the turn of the twentieth century corresponds with the coinciding development of structural theory, and reinforced concrete as a common structural material. There was no real practical material available to engineers before then to make a plate floor structure. Iron and steel could be used but until the thickness of plates made from those materials can be made larger for stiffness reasons without adding significant weight as a solid plate does, these materials are not reasonable choices.

It was not until 1864, when Barré de Saint Venant, building upon the work of C.M.L.H. Navier, published formulae for shear and bending moment that took into account the non-linear stress distribution of a beam in bending made from a material that does not have similar tensile and compressive strength. Navier's linear-elastic model was only good for materials such as wrought iron or steel, which have almost equal strength in tension and compression.[97] Grashof's plate theory, published in 1878, provided a basis from which to begin defining behavioral characteristics in formulaic terms, though it was not complete. Maillart's full-scale model experiments augmented the knowledge lacking in contemporary flat slab structural theory and analytical methods. (**Fig. 41**)

The flat slab was highly criticized, particularly in the academic community. There were some that felt that such structures were a danger because their behavior could not be mathematically explained. The influence of the French engineering education system had pervaded most European and American academic institutions. This created a culture of 'mathematical' dependency, limiting innovation.

Turner and Maillart saw the economic potential and structural superiority of this system and were practical enough to accept that even if they could not mathematically explain how the flat slab functions, it was good enough for them to know that it works.

[96] Yeomans, p114-115.
[97] Heyman, p29-34.

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PRESTRESSED CONCRETE BRIDGES

A-06

6.1 Prestressed Concrete and Bridges

6.1.1 Prestressed Concrete

Ordinary reinforced concrete will crack if subjected to any appreciable tensile stress. The reinforcement prevents this cracking compromising the security of the structure, but if crack widths are not controlled the steel reinforcement can corrode and possibly lead to failure. It is therefore necessary to keep stresses in the reinforcement low to limit crack width, meaning that the full capacity of high strength steel cannot be utilized. Similarly, the concrete is not being used to its full advantage because it is subjected to tension over some portion of its section rather then compression.

Prestressing overcomes these drawbacks by introducing a compressive force into the concrete before it is subjected to dead and live load to ensure that the concrete is either always in compression or subjected to only a limited amount of tensile stress that will ensure cracks do not exceed certain widths. Prestressing is applied by either tensioning the reinforcement against the concrete or by exerting external forces on the concrete. Jacking two halves of an arch apart at the crown against the abutments is an example of the latter case. In the case of the reinforcement being tensioned against the concrete, this procedure can be done in two ways. The reinforcement, or tendon, can be stressed before the concrete is placed and released after the concrete has cured. (**Fig. 1**) This is called ‘pre-tensioning’ and the prestressed reinforcement is usually bonded directly to the concrete. If the tendon is placed in a duct that protects it from being bonded to the concrete while the concrete is placed, it can be stretched and anchored after the concrete has cured, which is called ‘post-tensioning’. (**Fig. 2**) The advantage of the latter method is that the tendon can be placed in more complex geometries allowing the stresses in the concrete to be optimally controlled.

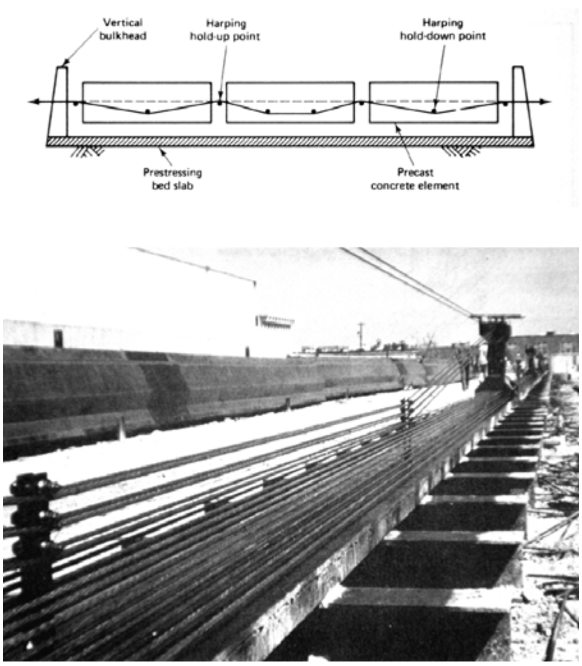

Fig. 1: Precasting bed with pretensioning. (a) Schematic of pretensioning bed. (b) Harping of tendons in a prestressing bed system. (Nawy)

The principle of prestressing is shown in **Figure 3**, which also corresponds to the pre-tensioning method of applying prestress. The stages of prestressing are as follows:

1. The force in the prestressing tendon is zero.
2. The tendon is stressed.
3. Concrete is poured around the prestressed tendon; the stress in the concrete is zero.
4. The concrete curing process is at an end. The tendon is released. Due to its elasticity, the tendon tries to contract but is hindered from doing so either by the bond between it and the concrete or by the anchorages. It therefore compresses the concrete, i.e. the concrete is prestressed an is placed in compression.
5. The concrete attempts to escape the load, becoming shorter due to creep. A further shortening results from shrinkage during the curing process. This shortening of the prestressed concrete member in turn results in a loss of prestress in the tendon.
6. The initial prestress force in the tendon must be so large that, despite the shortening of the member, sufficient prestress remains to prevent tensile stresses in the concrete or to limit these stresses to permissible levels once the member is loaded.[1]

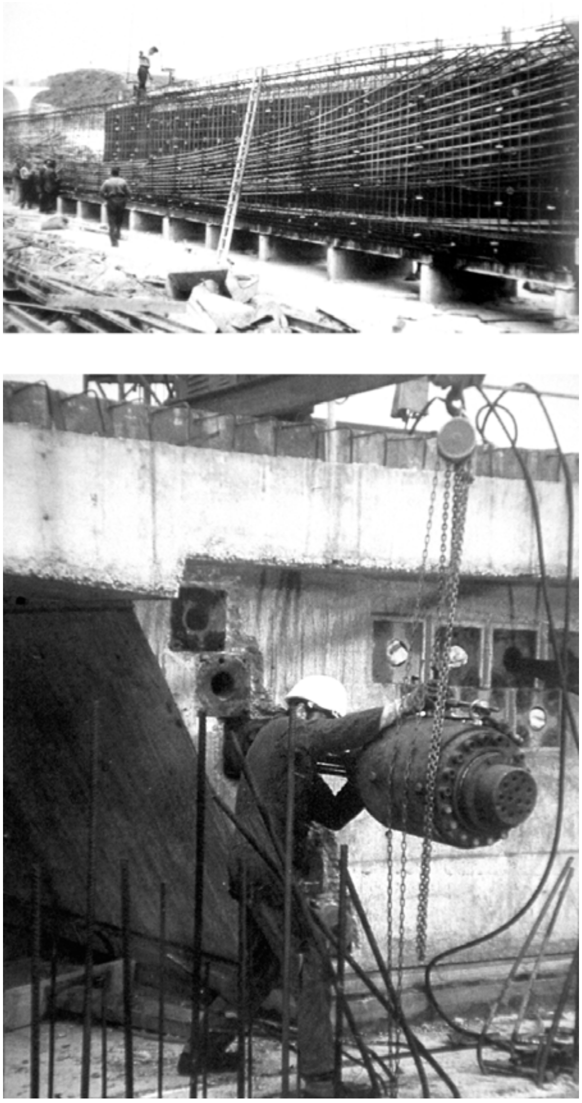

Fig. 2: Post-tensioned concrete. (a) Duct of a post-tensioned tendon. (Mainstone) (b) Post-tensioning showing jack used to stress the prestressing tendons. (Marrey)

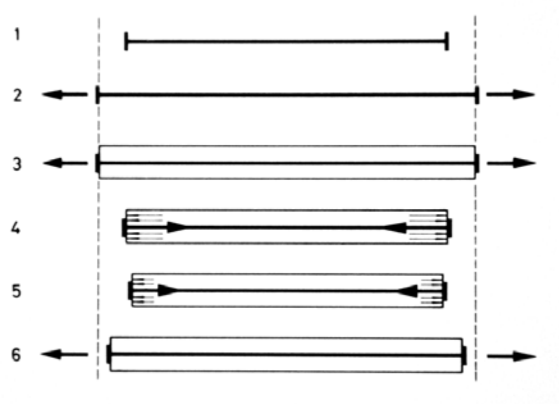

Fig. 3: Schematic showing principle of prestress transfer to the concrete. (Möll)

[1] Möll, p8.

Fig. 4: One of the first recorded concrete bridges, Fowler, 1869, England. (The Railway Gazette)

Fig. 5: Langwies Viaduct, reinforced concrete arch with 100 m span, Züblin, 1916. (Wittfoht)

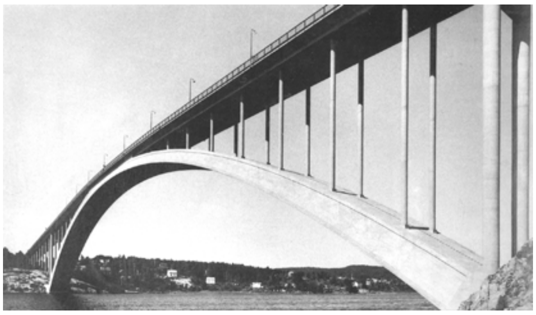

Fig. 6: The Sandö Bridge over the Ängermandälv, Sweden with a 269 m span, Skanska Cementgjuteriet AB, 1942. (Wittfoht)

6.1.2 Concrete to Reinforced Concrete Bridges

Cement was first used as mortar in bridge piers as early as the Romans.[2] The French began using cement in the 19th century after the Englishman John Smeaton[3] and Frenchman Joseph-Louis Vicat[4] made experiments with the material. One of the first post-Roman concrete bridges to be built was an arch bridge that traversed the railroad tracks of the Metropolitan and District Railways in Kensington, UK. This massive, un-reinforced concrete bridge was constructed between 1866 and 1869. Fowler[5] designed the bridge in conjunction with the contractors, as a trial application of that material. The bridge has a 22.9 m (75 ft) clear span, a rise of 2.3 m (7.5 ft), is 1.1 m (3.5 ft) thick at the crown and 1.7 m (5.5 ft) at the haunches, and is 3.7 m (12 ft) wide. The illustration shown in **Figure 4** was taken at the time of its construction. The bridge was demolished in September 1873 for unidentified reasons.[6]

Joseph Monier received a patent for a reinforced concrete bridge design on 13 August 1873. In 1875, Monier constructed the first known reinforced concrete bridge in the world in the castle park of the Marquis Tilière de Chazelet. (**Appendix A-05, p.A.239, Fig. 6**) The patent drawing indicates that the bridge would have behaved as a beam though it had a shallow, longitudinal curve. It was 16.5 m long and 4 m wide. The railings, which were designed to resemble tree branches, were also made of reinforced concrete.[7]

[2] Vitruvius, p196.
[3] Smeaton (1791).
[4] Vicat (1830).
[5] Presumably John Fowler, who was president of the British Institution of Civil Engineers (ICE) in 1865. The technical details published in *The Railway Gazette*, are reprinted from an article on the bridge first published in *Engineering*, 20 December 1867. *Engineering* was the publication of ICE. The article refers to the designer only as Mr. Fowler.
[6] *The Railway Gazette*, p159.
[7] Deinhard, p19.

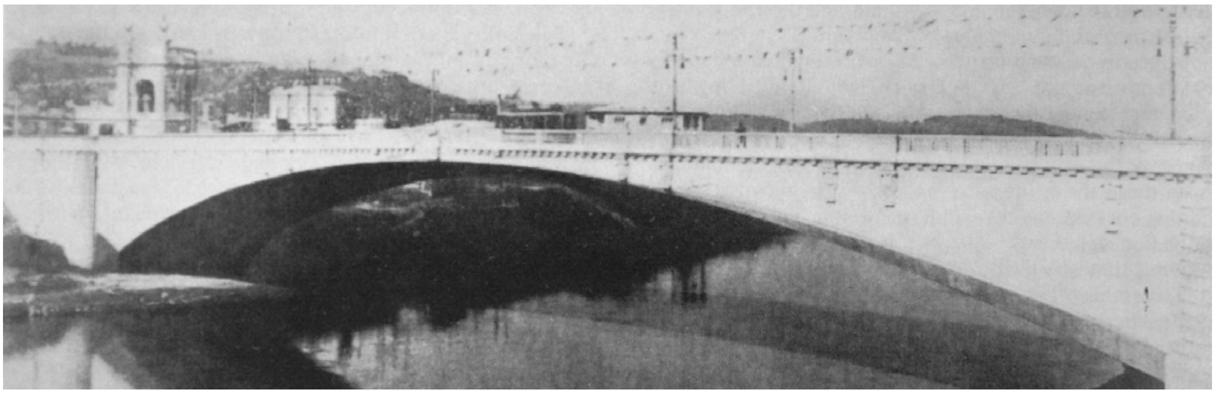

Fig. 7: Risorgimento Bridge over the Tiber River in Rome, first deck stiffened arch, Società Porcheddu, 1911. (Hennebique Archiv)

6.1.3 The Reinforced Concrete Arch

Reinforced concrete arches were built with spans of up to 40 m until the turn of the 20th century. Emil Mörsch led the development towards longer spans with his 70 m (230 ft) long bridge over the Isar in Munich completed in 1904. This 3-hinged arch with spandrel columns supporting the deck set a record for a "Monier" type bridge.[8]

The limits of reinforced concrete arch construction were further expanded in 1914 by the completion of Eduard Züblin's Langwies Viaduct for the Chur-Arosa Railway RhB in Graubünden, Switzerland. (**Fig. 5**) This fixed-end arch spans 100 m (328 ft) and has a rise of 42 m (138 ft).[9]

By the beginning of the Second World War, reinforced-concrete arch bridges were being constructed with spans well over 200 m (656 ft). The Sandö Bridge over the Ängermandälv in Sweden has a 269 m (883 ft) span and was built between 1938 and 1942.[10] (**Fig. 6**)

6.1.4 Robert Maillart invents the Concrete Box Section

The Swiss engineer Robert Maillart was a pioneer in reinforced concrete design and his work established new aesthetic qualities that reflected the particularities of the material. Maillart's Tavanasa Bridge, built in 1905, represents a radical shift in the design and aesthetics of reinforced concrete bridges. (**Appendix A-05, p A.259, Fig. 37**) In that bridge he removed material from the spandrels of the bridge that had been a zone of cracking in previous bridges at Zuoz and Bilwil that had closed spandrels.[11] (**Appendix A-05, p A.259, Fig. 36**)

Maillart is credited with introducing the box section to reinforced concrete bridge design, which he employed in the Zuoz Bridge built in 1901. Maillart also led the development of the deck-stiffened arch. The Risorgimento Bridge, which had a box section, was the first bridge to demonstrate the benefits of using the deck to stiffen a slender arch. It was built by the Società Porcheddu, the Italian licensee of the Hennebique system, in 1911 over the Tiber River in Rome.[12] (**Fig. 7**)

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[8] Mörsch (1906), pp201-202.
[9] Peters[1], 285-286.
[10] Wittfoht, p138.
[11] Billington[2], p26, 36 and 38.
[12] Deinhard, p41. Marcus, H. "Die Risorgimento-Brücke über den Tiber in Rom", *Armierter Beton*, 1912, p294.

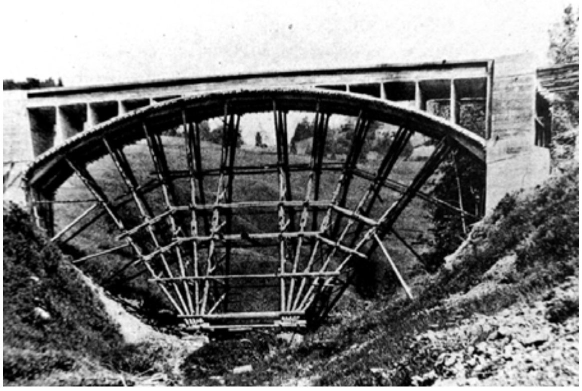

Fig. 8: Flienglibach Bridge, first deck stiffened arch built by Robert Maillart, 1923. (Billington[1])

Maillart's first deck stiffened arch design was the Flienglibach Bridge, built in 1923. (**Fig. 8**) Maillart realized the potential benefit of using the deck to stiffen the arch after he had observed cracking in the shallow deck beams of the Aarburg Bridge, which he designed in 1912. The Aarburg Bridge had a non-structural parapet. Maillart saw that the arch and deck interacted. Maillart's thinking was possibly influenced by examples of stiffened arches, such as the American truss-stiffened arch bridges built of timber, presented in lectures by Wilhelm Ritter while he was a student at the ETH-Zurich.[13] (**Appendix A-03, p A.88, Fig. 13**)

The effective cross-section of the Flienglibach Bridge is formed by the bridge deck and the concrete parapets along both sides. These in turn are supported by transverse spandrel walls. The arch is thereby loaded with normal forces only while the deck and parapet resist all bending induced stresses. Wittfoht writes,

> This was for all practical purposes the birth of the deck-stiffened arch, a structural element that was to become important in the years to come. After having attempted to emulate the truss and framework structures of timber and steel, reinforced concrete's strength was found to lie not in trusswork but rather in plates and shells. The labour costs involved in the simpler shuttering of large surfaces were lower, and less steel was required because of the better load-carrying capacity of such elements.[14]

6.1.5 Beam Bridges and the Introduction of the Steel Cable to Concrete

In comparison with advances in span length made by reinforced concrete arch bridges, reinforced concrete beam bridges advanced only modestly. In an arch the material is primarily in compression, whereas in a beam it is subjected to bending, meaning that a part of the cross-section is in tension and requires reinforcement. The span had to be kept small in order to limit the unavoidable cracking and attendant corrosion that occurred when the tensile strength of the concrete was exceeded by the elongation of the reinforcement. For this reason, the bridge over the Seine at Villeneuve-St.George near Paris, built in 1939, remained the longest reinforced concrete beam bridge in the world with a span of 78 m (256 ft).[15] (**Fig. 9**)

The main hindrance to building large spans with reinforced concrete is its high self-weight. Engineers created truss forms in order to reduce the dead load and increase the span of reinforced concrete beam bridges. The Rue Lafayette Bridge over the railway lines at the Paris-East Railway Station, designed by Albert Caquot and built from 1927 to 1928, is an impressive example of this the trussed reinforced concrete bridge. It was 10.40 m wide and

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[13] Billington[2], p114.
[14] Wittfoht, p122.
[15] Wittfoht, p143-144.

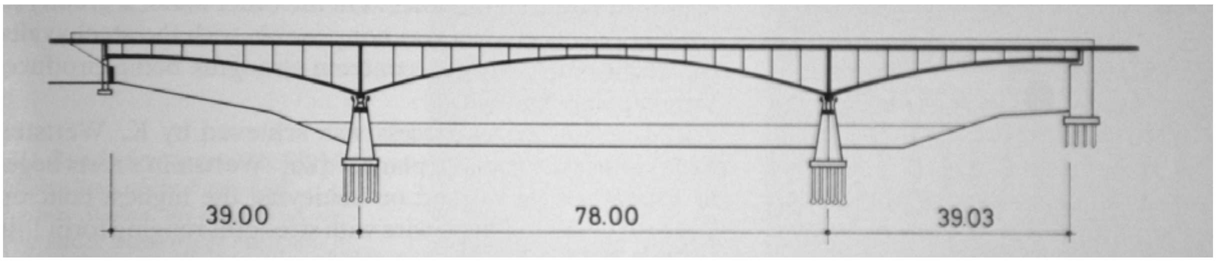

Fig. 9: The Villeneuve-St. George over the Seine near Paris, the longest reinforced concrete beam bridge in the world with a span of 78 m, 1939. (Wittfoht)

two spans of 71.9 and 76.9 m respectively.[16] (**Fig. 10**) However, trussing introduces its own problems, particularly with the need for higher quality control to address the increased complexity of construction.

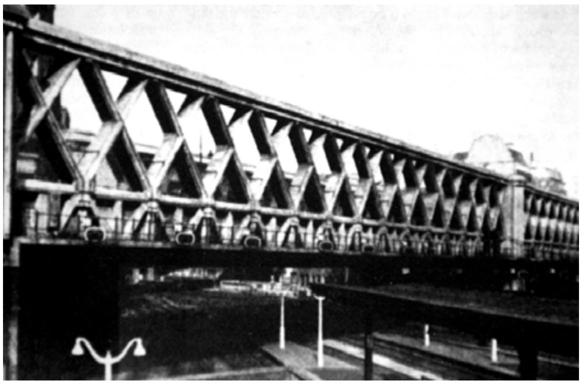

Fig. 10: Trussed reinforced-concrete bridge, the Rue Lafayette Bridge over the railway lines at the Paris-East Railway Station with a span of 71.87 m and a depth of 10.40 m, Caquot, 1928. (Deinhard)

The problem of dead load was overcome by replacing mild steel with prestressed, high-strength steel. Prestressing allowed designers to conceive of theretofore-unimaginable spans using concrete. In turn, this new structural type has led to the evolution of distinct structural forms that maximally exploit the high compressive strength of concrete and the high tensile strength of steel. This will review the development of prestressed concrete, its use in bridges, its influence on construction process, the influence of government regulations on its development, and the use of external prestressing starting in the 1970s.

6.2 Antecedents

6.2.1 The Compound Bow

The bow is perhaps the earliest human technology to utilize prestressing. A bow stores the energy of human muscles and, when released, propels an arrow or other projectile. A particular type of bow was developed in the hot climates of the Mediterranean called the ‘composite’ bow. The composite bow had a core of wood sandwiched by dried tendon on the tension surface and horn on the compression face. The wood is only lightly stressed since it is near the middle of the thickness of the bow. The horn and tendon store energy and perform more reliably in hot weather then the Spanish yew wood used to make the long bow characteristic of the armies in more cool climates of England, France and Germany during the Middle Ages. Composite bows of the kind described above were used in Turkey and elsewhere from at least the time of Homer, whose fabled character, Ulysses, used such a bow.

[16] Deinhard, p94.

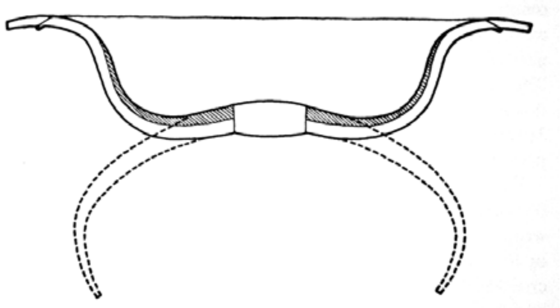

Fig. 11: The compound bow. Its unstrung form is shown by the dashed lines. (Gordon)

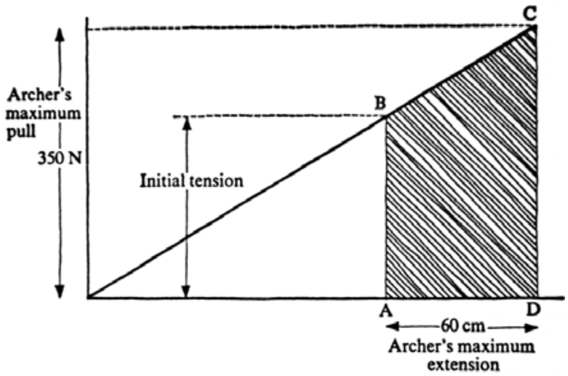

Fig. 12: Graph showing the amount of energy that can be stored in the bow in relation to the archer's maximum pull distance if the bow is prestressed when strung. (Gordon)

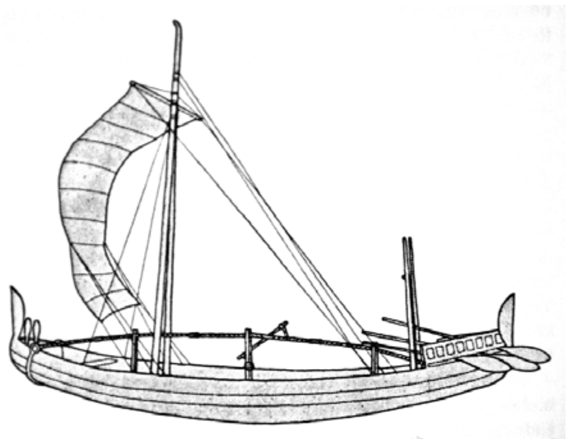

Fig. 13: Egyptian sea-going vessel made of wood modeled after Nile boats made of bundled reeds. The wooden ship retains the vertical ornaments at stem and stern characteristic of the reed-built boats. The wooden planks would have been short and poorly fastened therefore this ship retains the traditional Egyptian hogging-truss, c.2,500 BC. (Gordon)

The bow is prestressed by stringing it. **Figure 11** shows the shape of the composite bow before and after stringing. This prestressing increases the potential energy that can be stored in the bow when drawn by an archer. Humans can draw a bow with an approximate maximum force of 350 N (80 lb), but the maximum length of pull, which is about 60 cm (24 in), limits this amount. **Figure 12** shows the potential energy that can be stored in a prestressed bow so the archer's maximum force can be used when drawing the bow.[17]

6.2.2 Egyptian Boats: the Hogging-Truss

Egyptian Nile boats constructed of bundled reeds are another ancient example of prestressing technology. The ends of the bundles were tied to curve upwards, providing a vertical decoration at the bow and stern, a form that survives today in the high stem-posts of Mediterranean rowing boats such as the Venetian gondola and the Maltese *dghaisa*. These boats were often heavily loaded in the ends of the ship, which caused the ship to 'hog', despite the fact that the maximum buoyancy is in the center of the boat. Hogging is when the two ends tend to droop and the middle of the hull rises. This condition is precisely the opposite of the tendency of roof trusses and bridges to sag in the middle.[18] As early as 3000 BC, the Egyptians installed what is now called a 'hogging-truss' to correct this problem.

J.E. Gordon writes,

> [The hogging-truss] consisted of a stout rope which was passed over the tops of a series of vertical struts, its two ends being looped under and round the ends of the ship, so as to prevent them from drooping. (**Fig. 13**) This rope could be tightened by some form of 'Spanish windlass'. The latter device is a skein of cords which can be twisted – and so

[17] Gordon, p78-83.
[18] Gordon, p223.

shortened – by means of a long stick or lever thrust through its middle. Thus the big reed hull could be strained to any degree of straightness or vertical curvature which the skipper happened to fancy. As the art of shipbuilding progressed, the Egyptians came to construct their hulls from timber, rather than from bundles of reeds. But, since most of the planks were very short and nearly all of the fastenings might be described as wobbly, the need for the hogging-truss remained.[19]

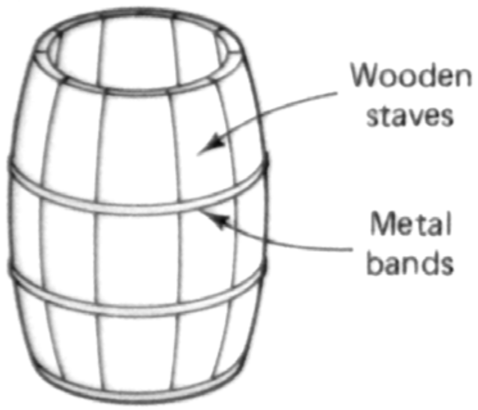

Fig. 14: Wooden barrel showing arrangement of wood staves and stressed iron hoops. (Nawy)

6.2.3 *Wooden Barrels and Cartwheels*

Wooden barrels and cartwheels were manufactured using the basic principles of prestress. Coopers wound rope or metal bands around wooden staves to form barrels. (**Fig. 14**) When the bands were tightened the staves were compressed together, allowing them to resist the hoop tensions applied by the pressure of liquids stored in the barrel. Likewise, the rim of the wooden cartwheel was prestressed by shrinking a metal band around its circumference, which also acted as a wearing surface.[20]

Fig. 15: Liesberg Bridge, three-span continuous reinforced-concrete beam bridge, Maillart, 1935. (Billington)

6.2.4 *Reinforced Masonry Beam, Pope, 1811*

A direct antecedent to the prestress concrete beam is Thomas Pope's reinforced masonry beam patent of 1811.[21] (**Appendix A-05, p A.237, Fig. 2**) For this beam to work, the nut on the threaded iron rods would have to be tightened, compressing the individual pieces of masonry together such that the friction between them would be sufficient for them to monolithically resist the compressive stress in bending while the iron rod resists the tension, thus this beam is prestressed.

6.2.5 *Making the Tubes of Britannia Bridge Continuous, Stephenson and Clark, 1850*

The tubes of the Britannia Bridge were connected to one another in such a way as to make the four simple spans behave as a continuous beam. (**Appendix 03, p A.138, Fig. 72**) To make the beams continuous segments were joined with a slight angular difference so that when the raised end of the connecting beam was lowered a prestress would be introduced over the supports that would in turn cause the beam to deflect upward at mid-span.[22] The increased stress over the piers was accounted for with more material in the sections of the

[19] Gordon, p224-225.
[20] Podolny and Muller, p4.
[21] Mainstone, p152.
[22] Clark, p704.

tubes over the piers where increased weight did not matter. The precamber of the beam reduced the amount of material needed at mid-span since the prestress reduced the imposed live load stress there.[23] This resulted in a lower dead load at the most critical sections of the 461.2 m (1,513 ft) long continuous tubes.

6.2.6 Introducing Precamber in Liesberg Bridge, Robert Maillart, 1935

Similar to the Britannia Bridge, Robert Maillart introduced a precamber into the mid-span of a three-span continuous beam bridge he built at Liesberg in 1935. (**Fig. 15**) The side spans were designed to deflect downwards after the formwork was removed, thus introducing the precamber into the center span. This form of prestressing allowed Maillart to keep the depth of the beam relatively small despite being designed for the load of a locomotive.[24]

6.3 Early Developments

6.3.1 William Boutland Wilkinson, 1854

In 1854, William Boutland Wilkinson, an Englishman, received the first patent for reinforced concrete. One variant of his patent specified the use of cables for the reinforcement. Wilkinson arranged these cables to resist the tensile stress in continuous beams. (**Appendix A-05, p A.237, Fig. 3**) Wilkinson did not prestress the cable however.

6.3.2 Peter H. Jackson, 1886

On 27 October 1886, P.H. Jackson, an American, applied for a patent in which steel bars with threaded ends are placed in footpaths, roofs, floors, etc. Anchor nuts allow the bars to properly transfer the tensile stress from the ends of the bars to the concrete. He proposed that the bars be placed in ducts or wrapped with paper, loam, or a similar material in order to prevent the steel from bonding with the concrete and to enable the bars to elongate when the anchor nuts were tightened.[25]

6.3.3 C.F.W. Doehring, 1888

C.F.W. Doehring, of Berlin, Germany, was the first to closely investigate the problem of cracking in concrete. He proposed a solution in 1888 after performing several tests in which he applied tensile stress to both concrete and steel. Doehring determined that a greater force was necessary to load to failure both the steel and the concrete simultaneously than if they were loaded to failure one after the other. This meant that the concrete member produced using his method had a greater strength.[26]

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[23] Clark, p557 and 559.
[24] Billington[1], p197.
[25] Möll, p3. Jackson, Peter H. San Francisco (USA), USA-Pat. 375 999, 27.10.1886 / 3.1.1888.
[26] Möll, p3-4. Döehring, C.F.W. Berlin, DRP 249 007, 17.1.1912 / 9.7.1912.

6.3.4 C.R. Steiner, 1908

C.R. Steiner received a patent in the United States in 1908 in which he wrote that the reinforcement must be so highly stressed that when the member is loaded, the prestress-induced compressive forces in the concrete are first neutralized before tensile stresses arise and cracking occurs in the concrete. Steiner explained that the purpose of this was to avoid tensile stresses entirely in the concrete or at least reduce them to a minimum.[27] Similar proposals and findings for the avoidance of cracking in concrete can be found in other patents of the time.[28]

Wittfoht explains that it would have been impossible for these patents to have been successful because the prestress forces would have been too small. Creep and shrinkage, which were not well understood at the time, would have largely neutralized the force. Also, the steel and concrete strengths available at the time were not high enough to greatly increased the prestress force.[29]

6.3.5 Karl Wettstein, 1919 and 1924

Karl Wettstein, a Czechoslovakian, achieved the first practical success with his “elastic concrete planks.” Wettstein had begun experiments to achieve the highest concrete strength possible in 1919. In 1924, he patented his “concrete planks,” which were 6 to 50 mm (0.24 to 2 in) thick and 2 to 6 m (6.6 to 19.7 ft) long with a width of 50 cm (19.7 in). These planks were reinforced with prestressed piano wire placed in the concrete near the surface. Wettstein claimed one of his planks could be bent 90° without cracking, attesting to their ductility.[30]

6.3.6 R.E. Dill, 1925

In 1925, R.E. Dill of Nebraska, USA, tensioned high-tensile steel wires after the concrete had hardened. The wires were coated with some material to prevent bonding between the steel and concrete. Dill explicitly explained the advantage of using high-strength steel with a high elastic limit in lieu of ordinary reinforcing bars.[31]

While all of these early developments are significant to the general history of prestressed concrete, they had limited contemporary influence. The adoption and success of prestressing technology in the construction industry was primarily due to the pioneering efforts of Eugène Freyssinet, of France, and Franz Dischinger, of Germany.

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[27] Möll, p4-5. Steiner, C.R. Gridley (USA), USA-Pat. 903 909, 10.2.1908 / 17.11.1908.
[28] Möll, p5. Crisenberry in USA (1915); Sacrez in France (1907); and Wilson in Britain (1917).
[29] Wittfoht, p144.
[30] Möll, p5-6. Wettstein, K. Brüx (Tschechosl. Republik), öst. Pat. 95 934, 30.11.1921 / 11.2.1924, Priorität d. Anm. I. Dtschl. V. 26.1.1921; Wettstein-Bretter. *Neue Bauwelt*, 3 (1948) 38, p605; Wettstein, K. “Entwicklung der Wettsteinbetonbretter”, from *Betonsteinzeitung* (1948) 3, p41-45.
[31] Podolny and Muller, p4.

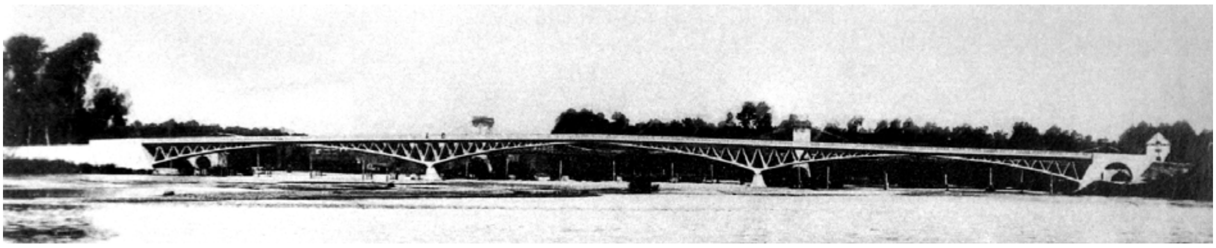


Fig. 16: Bridge at Le Veurdre, France, 3-hinged arch with triangulated spandrels with three spans of 72.5 m each, Freyssinet, 1907. (Wittfoht)

6.4 Eugène Freyssinet, 'Father of Prestressing'

6.4.1 The Influence of Charles Rabut

Eugène Freyssinet became fascinated by the idea of prestressing while he was a student at the Ecole des Ponts et Chaussées because of the lectures given by one of his instructors, Charles Rabut. Freyssinet used Rabut's method for connecting the two abutments of an arch together for a 50 m (164 ft) long scale model of a reinforced concrete arch that was founded on poor soil. Freyssinet joined the two abutments with a concrete tie that was prestresssed with a hundred cold-drawn wires of 8 mm (0.33 in) diameter. These wires were stressed close to their elastic limit and were anchored in pairs by wedges in pierced steel plates.[32]

6.4.2 The Bridge at Le Veurdre, 1907-1911

In 1907, Freyssinet, then working for the department of Ponts et Chaussées in Moulins, was tasked to design replacements for a three suspension bridges over the Allier River. One of these bridges, at Le Veurdre, is composed of three 72.5 m (238 ft) spans. Each span is a three-hinged arch that supports the deck by triangular spandrels. (**Fig. 16**) Freyssinet used the 50 m (164 ft) model arch mentioned above to gain an insight into the behavior of the bridge. Freyssinet wanted to use precast concrete but was unable to because of technical shortcomings at the time.[33]

The decentering operation was carried out with horizontal hydraulic jacks placed in the crown of the arches, the first time this method was employed. These jacks enabled the arches to be lifted off of the formwork so that the centering could be removed. The bridge was opened in 1910.[34]

By the spring of 1911 the crown of the middle arch had sunk by 13 cm (5.1 in) for inexplicable reasons. Freyssinet conjectured that the modulus of elasticity of concrete had decreased in the course of the year, but at the time he could not definitively explain or understand the problem. The deflection of the arch resulted in a substantial increase in stresses because of the change in the line of thrust. To remedy the problem, Freyssinet

[32] Freyssinet[1], p28.
[33] Freyssinet[1], p27.
[34] Freyssinet[1], p29.

replaced the jacks in the niches used for striking the centering and jacked the arch up to its previous position and closed the hinge at the crown with concrete.[35]

Wittfoht explains,

> This hinge at the crown was a disastrous concession to the regulation of the day. Freyssinet had originally designed a 2-hinged arch. This dramatic sinking of the crown was actually due to a phenomenon that has often been observed but even at that day has not been completely explained, namely creep. While shrinkage is understood to be the contraction of concrete during curing, creep is a change in shape which occurs when concrete is loaded over a period of time. Freyssinet with his variable modulus of elasticity was on the right track.[36]

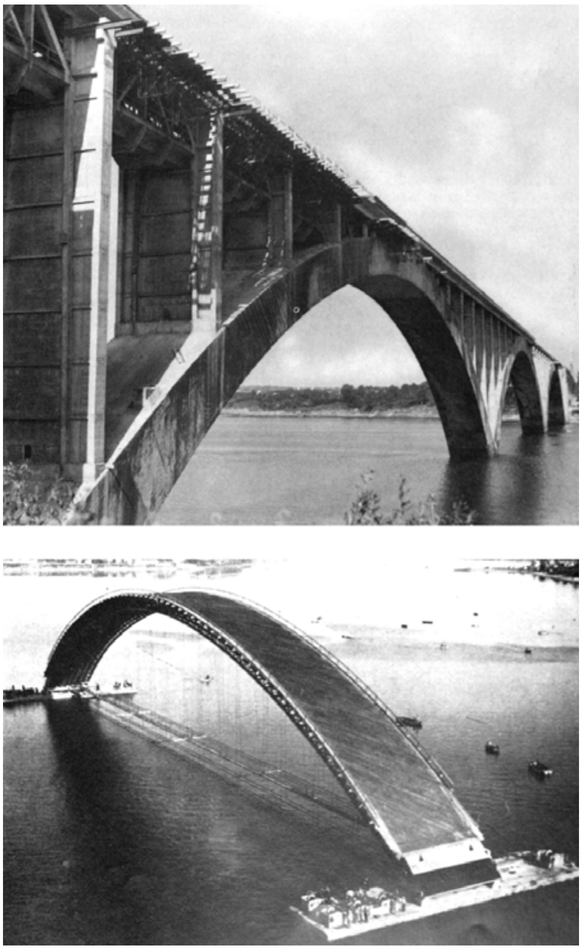


Fig. 17: (a) Elorn Bridge with three spans of 186.4 m each, Freyssinet, 1930. (b) Self-supporting centering used to construct the Elorn Bridge being floated into place. (Wittfoht)

6.4.3 Creep and Prestressed Concrete, 1911-1928

Freyssinet is generally credited for being the first to investigate and explain the process of shrinkage and creep in concrete. Freyssinet began his investigations in 1911 and devoted himself to the study of the time-dependent behavior of high-strength concrete under load.

Freyssinet used the construction of the Elorn Bridge in 1930 to intensively study the mechanisms of creep and shrinkage in concrete. The Elorn Bridge has three spans of 186.4 m (611.5 ft) each. (**Fig. 17**) These were the longest reinforced concrete arches in the world at the time. Freyssinet presented the results of his investigations in a paper for an international conference on bridge and structural engineering in 1928 and in his book *Une Revolution dans la Technique du Beton*. His explanation of creep was finally accepted at the Second International Conference for Bridge and Building Construction, held Luttich in 1930.[37]

As a result of his tests, observations, and studies, Freyssinet was able to work out the amount of prestress loss in the steel for a given concrete strength. Freyssinet recognized the significance of axial shortening of the concrete in applying prestress and concluded that concrete creeps less the higher its compressive strength and the greater its degree of compaction, though this is not the same for shrinkage. Freyssinet concluded that the

[35] Freyssinet[1], p30.
[36] Wittfoht, p125. Freyssinet[1], p27.
[37] Freyssinet[1], p30; Freyssinet[3], p5-34; Freyssinet, E. “Le Pont de Plougastel”, presented to the Second International Conference for Bridge and Building Construction, Vienna, 24-28 September 1928. Proceedings published Vienna, 1929.

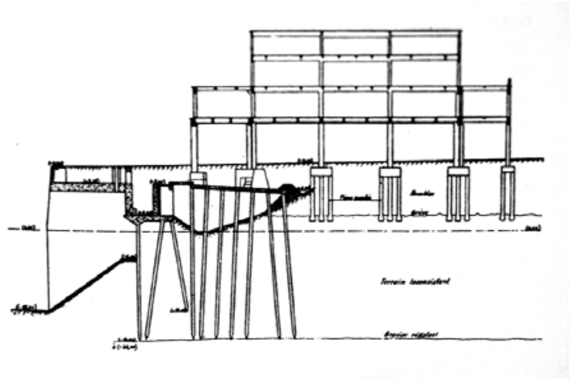


Fig. 18: Cross-section through the Gare Transatlantique at Le Havre. (Grote and Marrey)

significance of a particular loss of prestressing force due to creep is less the larger the prestressing force that remains after the losses have occurred. Therefore, the steel should be prestressed to such a high degree that the prestress transferred to the concrete will be sufficient despite any losses, which requires a high strength steel.[38]

Freyssinet patented his ideas concerning the detailing and shape of prestressed concrete in France and numerous other countries. He received his French patent in October 1928.[39]

6.4.4 Powerline Pylons, Freyssinet's First Applications of Prestressing, 1929-1933

In 1928, Freyssinet, working for himself now, contracted with the company Forclum to design prestressed concrete power-line pylons, as Otto Glösser had probably done a year earlier in Czechoslovakia. After four years Freyssinet was able to produce pylons that weighed 40% less, used one-third the steel reinforcement and was far more resistant to repeated loading than conventional pylons. Unfortunately, the economic depression in the United States and Europe resulted in the scrapping of the pylon production line after only a few thousand pylons had been sold.[40]

6.4.5 The Gare Transatlantique at Le Havre, 1934

The Gare Transatlantique at Le Havre was being built to dock the largest passenger-ship of its time, the SS *Normandie*. (**Fig. 18**) During construction the landing station began to sink into a layer of mud at a rate of up to 25 mm (1 in) per month. In February 1934, Freyssinet was asked to address the problem. He proposed to consolidate the structure with a novel and untried prestressing method, in addition to a number of other unproven techniques.[41]

Grote and Marrey, describing Freyssinet's intervention, write,

> [Freyssinet] joined the existing columns by three *in situ* concrete beams which were prestressed by wires passing along the outer surfaces of the beams and looped around semicircular and movable concrete saddle pieces at their ends, which were thrust outwards by jacks, thus tensioning the wires. (**Fig. 19**) The joints were then filled with concrete.
>
> Concrete piles were cast in two-meter sections above holes in these beams and driven 30 m downward by jacking to avoid the vibrations caused by impact…. Finally the whole structure was jacked against the piles to the intended level.

[38] Freyssinet[3], p64-66.
[39] French Patent 680 547, 2.10.1928 / 1.5.1930; German DRP Patent 622 746 5.4.1929 / 5.12.1935 and öst. Pat. 134 523, 27.9.1929 / 25.8.1933.
[40] Grote and Marrey, p16 and 18. Glöser, Otto, DRP 557 829, Berman patent:L Vorrichtung zm Herstellen von Eisenbetonbauteilen Apliance for the production of reinforced concrete elements, CSR, 29.4.1927 / 18.5.1933. Eugène Freyssinet. *Un amour sans limite*. Paris, Éditions du Linteau, 1993, p50.
[41] Grote and Marrey, p18. Eugène Freyssinet. *Un amour sans limite*. Paris, Éditions du Linteau, 1993, p54.

> Eight months after the decision to prepare the equipment and four months after beginning work, settlement in the worst affected zones stopped.[42]

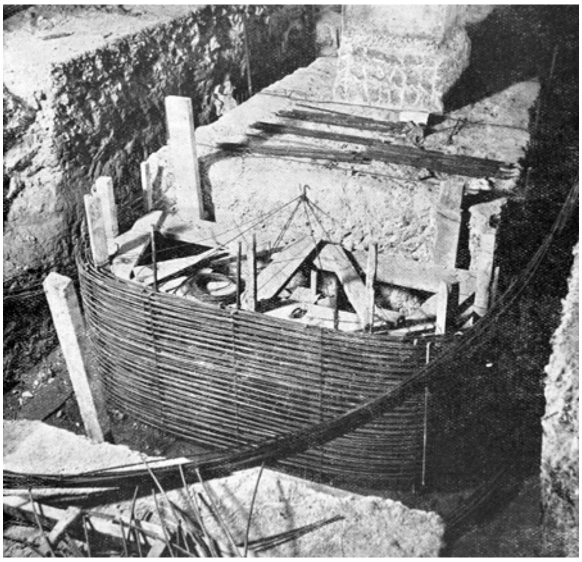


Fig. 19: Prestressing system used by Freyssinet to jack the tendons of the tie beams in the Gare Transatlantique at Le Havre, 1934. (Freyssinet)

Freyssinet's success brought him and his invention much attention. Many engineers visited the site, including Edme Campenon, whose company worked with Freyssinet throughout the rest of Freyssinet's career. In 1943, Fritz Leonhardt, at that time a young officer in Hitler's construction service *Organization Todt*, traveled to Paris for a private lecture on the "*révolution dans l'art de bâtir*" by Freyssinet. Leonhardt would go on to become an influential figure in the development of prestressed concrete technology.[43]

6.5 Franz Dischinger

6.5.1 The Saale Bridge, 1928

In 1928, Franz Dischinger finished building an arch bridge with a span of 68 m (223 ft) and a suspended deck. The arch contained a steel tension ribbon that could be stressed to counteract the effect of the forces on the arch to maintain a 'true-form' arch that would remain free of bending stress. The installation of the tensioning ribbon was designed such that it could be re-tensioned after any shortening of the arch due to shrinkage and creep. This function required that the tendon remain free to glide inside the concrete cover, which also required corrosion protection measures to be taken. The ribbon had to be stressed more often and to a higher stress than Dischinger had expected in order to keep the bending moments in the arch to tolerable levels.[44]

6.5.2 The Mathematical Calculation of Creep and Shrinkage, 1937 and 1938

Dischinger's experience with the Saale Bridge induced him, like Freyssinet, to study the shrinkage and creep of concrete. He was successful in describing both phenomena in mathematical form such that he could calculate the size of the creep and shrinkage relative to time.[45] His two scientific works on the subject, published in 1937 and 1939, provide the basis for all later work on the subject.[46]

[42] Grote and Marrey, p18 and 20.
[43] Grote and Marrey, p20 and 22. Leonhardt, Fritz. "Interview zum 85.Geburtstag", from *Beton und Stahlbetonbau* 89 (7/1994), p183.
[44] Buschmann, p405 and 407; Dischinger, *Bauingenieur* 1937, p487-621.
[45] Dischinger, *Bauingenieur* 1937, p487-621.
[46] Dischinger, *Bauingenieur* (1937); Dischinger (1939).

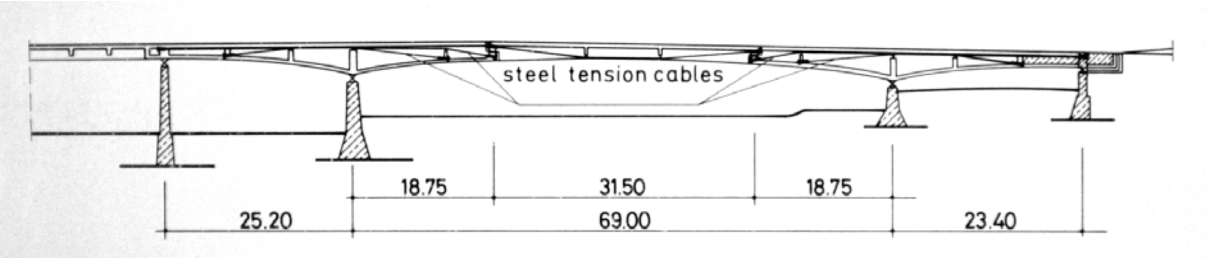
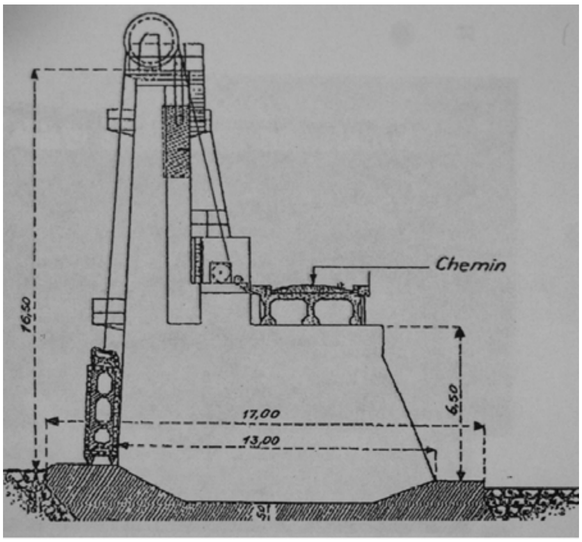


Fig. 20: The Aue Bridge, the first prestressed beam bridge and the first with external prestressing in the world, Dischinger, 1936. Section showing arrangement of prestressing tendons. (Wittfoht)



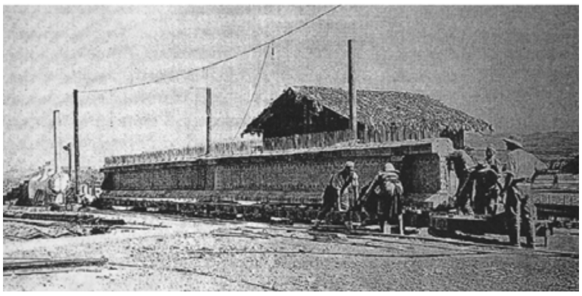


Fig. 21: Prefabricated prestressed I-girder bridges for a service footway on the weir of the Wadi Fodda River, Freyssinet, 1937-1939. (a) General section of weir and footbridge. (b) Pre-cast girder before installation showing ends of the stirrups that will monolithically connect the deck slab to the girder. (Grote and Marrey)

6.6 The First Prestressed Concrete Bridges

6.6.1 Aue Bridge, Franz Dishinger, 1936

Franz Dischinger became head of the faculty for concrete structures at the Technische Hochschule Berlin-Charlottenburg in 1932. While there he began to apply his ideas concerning true-form, moment-free structures to beam bridges.[47] In 1936, Dischinger designed the first prestressed concrete beam bridge in the world, the Aue Bridge in Saxony, Germany.[48] (**Fig. 20**) The Aue Bridge is also the first bridge with external prestressing; a mode of prestressing that only became important at the end of 1970s.

Dischinger modeled the design of the Aue Bridge on underspanned beams made of steel or timber. In Dischinger's design, the steel prestressing-tendon was deflected such that the moments produced by the self-weight of the bridge counteract the moments produced by the prestressing force. The tendons were placed without bond to the concrete such that they were free to glide when stressed. The tendons could be prestressed later to preserve the 'true form' prestressing profile if significant losses occurred due to shrinkage and creep.[49]

[47] Wittfoht, p146. Deinhard, p109-110. Dishinger, F. "Entwicklung und Fortschritte im Eisenbeton". *Die Bautechnik*, 1937, p539.
[48] The Aue Bridge was originally called the Adolf Hitler Bridge. Ref. Grote and Marrey, p28 and Vogel[1].
[49] Wittfoht, p146. Deinhard, p109-110. Dishinger, F. "Entwicklung und Fortschritte im Eisenbeton". *Die Bautechnik*, 1937, p539.

Later developments in prestressing accepted that it is more practical to accept a certain value of bending moment to account for losses rather than to require prestressing of the tendons.[50]

Dyckerhoff & Widmann built the Aue Bridge as part of a 300 m (984 ft) long reinforced concrete bridge project. Construction began 8 December 1935 and all of the steel tendons were stressed to 215.7 N/mm^2 by 22 December 1936. In May 1937, the tendons had to be re-stressed to correct for the losses due to shrinkage and creep. The largest deflection at mid-span of the 69 m (226 ft) long mid-section was 3 cm (1.2 in). In contrast, deflections as large as 4 cm (1.6 in) were measured in the 40 m (131 ft) span approach spans constructed of normal mild-steel reinforced concrete.[51]

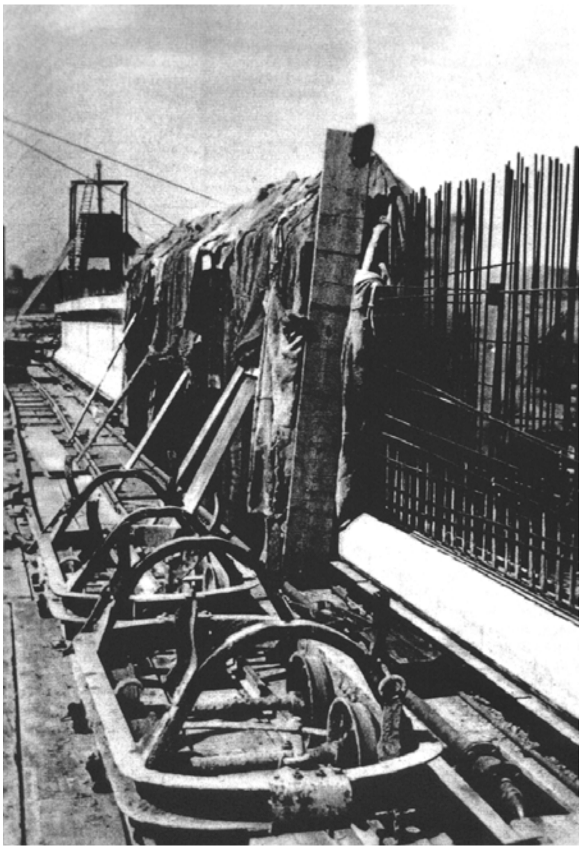


Fig. 22: Fabrication of the pre-cast prestressed concrete girders for the Öulde Bridge. The rear section of the girder is complete; the central section is hooded for steam heating; and on the right, only the lower flange has been cast. (Grote and Marrey)

6.6.2 Pre-Cast, Pre-Tensioned Beams at Wadi Fodda, Eugène Freyssinet, 1937-1939

While Dischinger was developing a post-tensioned, externally reinforced prestressing system, Freyssinet was concentrating on a pre-tensioned system in which the tendon is bonded with the concrete. Freyssinet began his development of prestressed concrete beams with bond as early as 1933 when he and Wayss & Freytag, who had licensed Freyssinet's patent in Germany, designed and tested a 20 m (65.6 ft) long experimental beam.[52] Perhaps Freyssinet used results from this experiment in his solution for the foundation settlement problem of the passenger terminal at Le Havre in 1934.

Freyssinet built his first prestressed concrete bridge for a service footway on the weir of the Wadi Fodda River between 1937 and 1939. (**Fig. 21(a)**) Three prestressed **I**-girders of 20 m (65.6 ft) span traverse each of the four 17.3 m (56.8 ft) sluice gates. From a similar design for a bridge at Öulde built somewhat later, we can assume that these **I**-girders were fully prestressed, so there is no tension in the concrete, and that the shear stress is transferred by means of pre-tensioned stirrups that also linked the beams to the cast-in-place deck-slab. (**Fig. 21(b)**) For this project, Freyssinet also designed the sluice gates and 50 km (31 miles) of concrete pipes with prestressing. These girders and sluice gates are the first prefabricated prestressed concrete beams.[53]

[50] Wittfoht, p146.
[51] Deinhard, p109-110.
[52] Wittfoht, p147. Freyssinet, Eugéne. Überblick über die Entwicklung des Gedankens der Vorspannung. Deutscher Sonderdruck aus dem Bericht der Jubiläumstagung des DBV 1949. Wiesbaden 1950.
[53] Grote and Marrey, p22.

6.6.3 Öulde Bridge, Wayss & Freytag, 1938

In 1938, Wayss & Freytag built the Öulde Bridge in Westphalia, the first road bridge constructed with girders prestressed according to Freyssinet's patents. Freyssinet constructed a prototype at the yard of Campenon-Bernard near Paris to teach the director of Wayss & Freytag and two engineers how to make the girder. Freyssinet introduced them to the processing bed, the cast-iron mould elements that can be adjusted to varying dimensions, the jacks, prestressing and anchoring of the wires, prestressing of the stirrups against the newly cast concrete in order to avoid diagonal tensile stresses, pouring by sections, vibrating, and steam curing.[54] (**Fig. 22**)

The Öulde Bridge had four, 33 m (108.3 ft) long prestressed concrete **I**-beams and a reinforced concrete deck. (**Fig. 23**) The beams had a depth of 1.60 m (5.2 ft), giving it a span to depth ratio of 20:1, a slenderness that had not been achieved for a beam bridge before.[55]

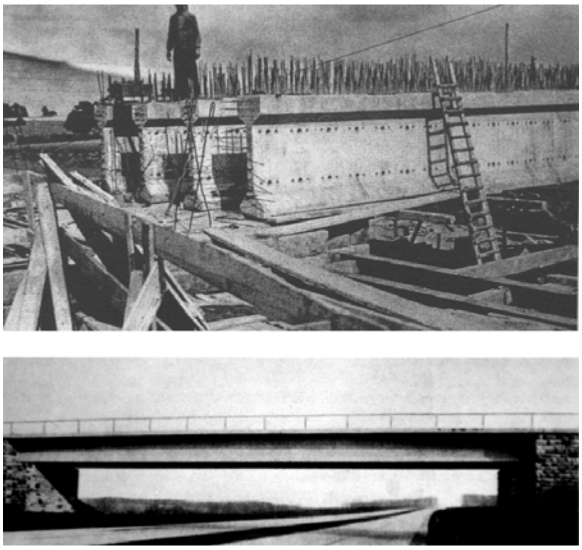


Fig. 23: The Öulde Bridge comprised of four 33 m long prestressed concrete I-beams and a reinforced deck, Wayss & Freytag, 1938. (a) Under construction. (b) The finished bridge. (Grote and Marrey)

6.6.4 Progress Delayed, Freyssinet, 1941

In 1941, Freyssinet started construction on the Luzancy Bridge overt the Marne River that was to be built using a new prestressing method that would have a great influence on the future of prestressed concrete bridge construction. Much of the technology Freyssinet was going to introduce was new: the fabrication of the members, construction method, the type of prestressing and the manner in which the prestressing force was transferred to the concrete, and finally its appearance. Construction was stopped because of World War II and recommenced in 1945.[56]

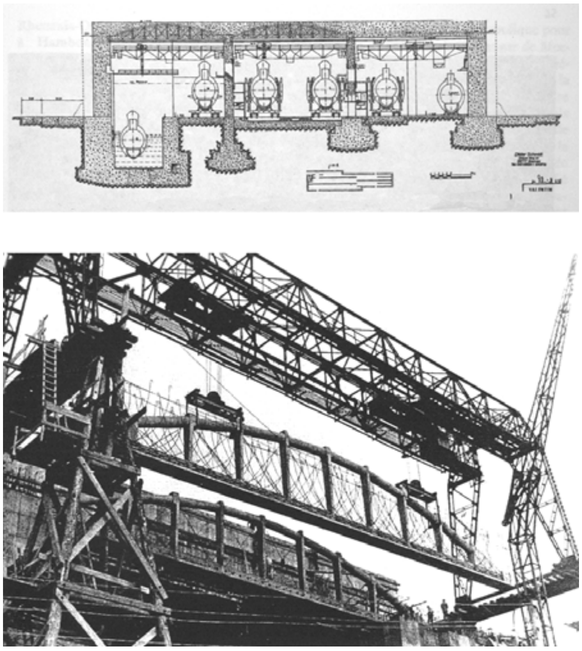


Fig. 24: Prestressed concrete roof girder with 30 m spans used to construct German submarine pen, 1943. (a) Cross-section of submarine pen. (b) Roof girder being lifted into place during construction. (Grote and Marrey)

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54 Grote and Marrey, p32.
55 Deinhard, p112.
56 Wittfoht, p148.

6.7 The War Years

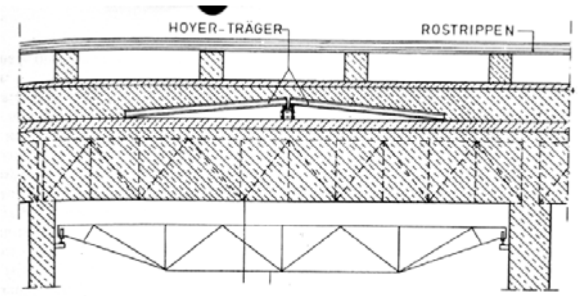

Fig. 25: Cross-section through roof of an early German submarine pen built during World War II using Melan girders embedded in the concrete that doubles as support for the formwork and the reinforcement of the concrete structure. Relieving Hoyer beams and a grating on top provide additional bomb protection. (Grote and Marrey)

During the Second World War little development occurred in the area of bridge structures, including prestressed concrete, due to war-related shortages of steel. Reinforced concrete was greatly used, however, since a structure could be built with a minimal quantity of steel. Perhaps this period can be considered the general transition period during which Europe changed from a predominantly stone and steel building culture to one that predominantly used reinforced concrete. The post-war reconstruction period further encouraged the transition since reinforced concrete offered the most readily available and cost-effective way to rebuild.

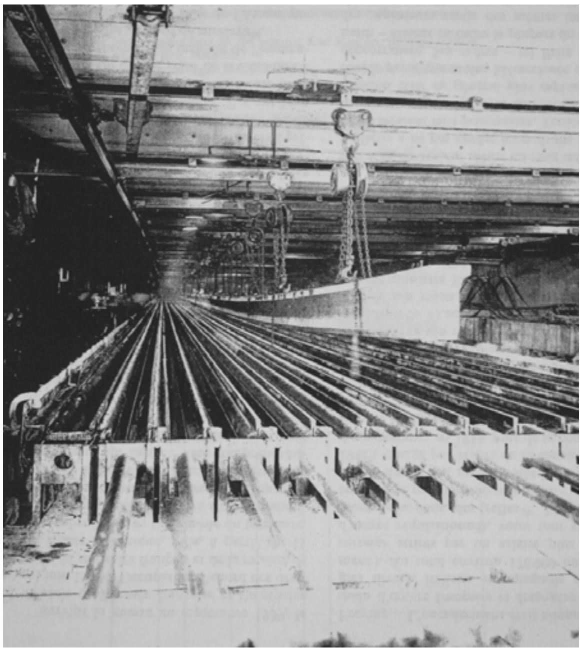
Fig. 26: Production line for Ewald Hoyer's divisible prestressed concrete girders precast in 100 m lengths. (Grote and Marrey)

Since the Germans occupied France, the Germans largely controlled construction activities in the two countries where prestressed concrete was being most intensively developed. The Germans built a number of prestressed concrete I-girder bridges using the casting moulds used for the Öulde Bridge. Prestressed concrete was also used to construct the roof structures of fortified buildings such as pillboxes on the coast and long span roofs for submarine pens. (**Fig. 24**). Earlier submarine pen roofs were built using Melan girders, which are structural steel girders intended to support the formwork for casting reinforced concrete beams. (**Fig. 25**) The girders were encased in the concrete such that they also served as the reinforcement of the finished beam. The Melan system was first introduced for the construction of reinforced concrete arches. The Melan-type roof proved to be too weak to resist Allied bombing and prestressed girders were introduced to make a stronger roof without making it unduly thick.[57]

Ewald Hoyer, an Austrian, introduced mass-produced pre-tensioned construction methods to the production of prestressed concrete beams. While Freyssinet, Dischinger and Ulrich Finsterwalder preferred special anchorage pieces, Hoyer employed small-diameter wires that

[57] Grote and Marrey, p47 and 77.

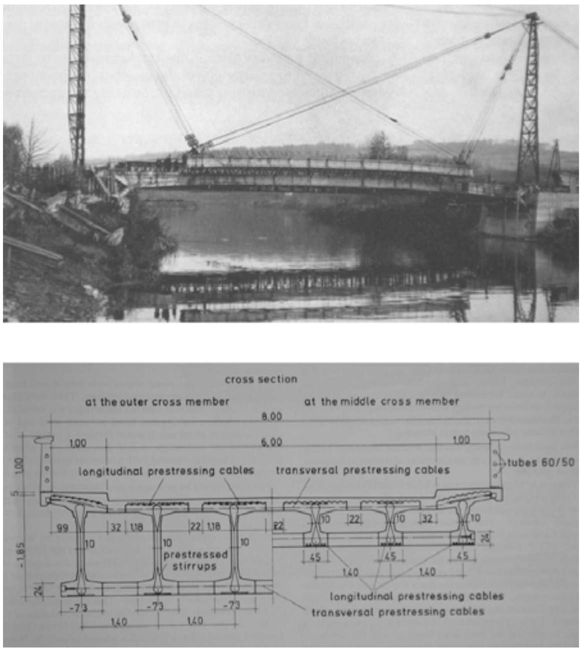

Fig. 27: (a)Cable crane erection method for constructing the Luzancy Bridge, 55 m span, Freyssinet, 1945. (b) Cross section of the Marne Bridge at Esbly, typical of Freyssinet's six Marne River bridges. Section shows arrangement of longitudinal and transversal prestressing cables and the prestressed stirrups. (Wittfoht)

were bonded directly with the concrete to keep them in place. Hoyer's method allowed him to cast 100 m (328 ft) long prestressed beams that could be cut to smaller lengths. (**Fig. 26**) Freyssinet had used pretensioned tendons with bond earlier in his explorations with prestressed concrete, but he had completely abandoned the system after he started using his anchorage-cone system in September 1939.[58] (**Fig. 30**) Freyssinet may have independently created the cone-type anchorage, but the first known person to invent it was the Swiss suspension bridge builder Guillaume Henri Dufour. Dufour published his idea for a cone-head anchorage in 1831.[59]

Generally, few significant technological developments were made in prestressed concrete construction during the Second World War. Further developments had to wait until the end of the war when Freyssinet completed his bridge at Luzancy.

6.8 The Marne River Bridges: Freyssinet, 1945

After the war ended, Freyssinet recommenced construction on Luzancy Bridge that he had begun in 1941 but left unfinished after only the abutments had been built because of the war. Completed in 1945, the 55 m (180.4 ft) span Luzancy Bridge was the first prestressed concrete bridge built after the war. The bridge design was widely published and so had a great influence on the development of prestressed concrete in the post-war period.[60]

The Luzancy Bridge is a segmental box-girder bridge 8 m (26.2 ft) wide and comprised of three girders with twenty-two segments each, the depth varying from 1.75 m (5.7 ft) at the abutments to 1.22 m (4 ft) in the center. The three girders were assembled in three sections; two consisting of three segments each at the abutments, and the central part some 40 m (131.2 ft) long consisting of sixteen pre-assembled pieces each. The three central segments, which weighed 90 metric tons (99 tons) each, were placed with cable cranes, a construction method that Freyssinet preferred because it eliminated the need for falsework.[61] (**Fig. 27**)

When all the segments were in place, the whole structure was post-tensioned in three directions using longitudinal and lateral cables in addition to prestressed stirrups. The

[58] Grote and Marrey, p46 and 48. Eugène Freyssinet. *Un amour sans limite*. Paris, Éditions du Linteau, 1993.
[59] Peters, p133. Dufour, Guillaume Henri. *Quelques notes sur les ponts suspendus*. Bibliotheque Universelle vol. 48, Sciences et Arts. 1831. pp254-291.
[60] Grote and Marrey, p98.
[61] Grote and Marrey, p98 and 100.

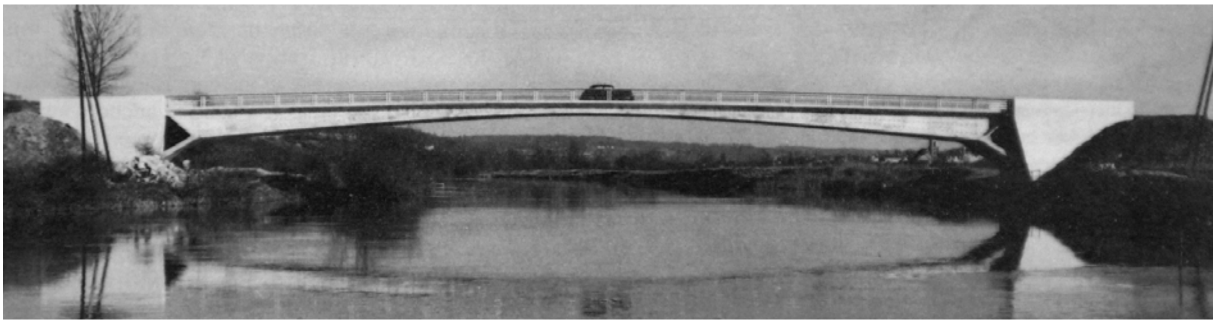

Fig. 28: Marne Bridge at Ussy, Freyssinet. (Wittfoht)

advantage of post-tensioning is that the tendons can be jacked against the concrete, which reduces the size and weight of the jacking equipment required to stress the tendons. After the tendon is stressed, grout is injected along its length to make a bond between the tendon and the concrete.

Freyssinet first used this method when constructing the Elbeuf-sur-Andelle Bridge, a short 10.5 m span bridge located east of Rouen, completed in 1942.[62] The success of the Luzancy Bridge firmly established this method of prestressing, which has become the leading method employed in the construction of prestressed concrete bridges today.[63]

Freyssinet fully recognized the progress he had achieved in the state-of-the-art prestressing technology and remarked upon the opening of the bridge that the bridge at Luzancy was the first prestressed concrete bridge "to belong without a doubt to a new era."[64] The success of Freyssinet's bridge led to contracts to build five more across the Marne at Esbly, Annetz, Trilbordou, Ussy (**Fig. 28**), and Chargai-St. Jean. The spans were increased to a distance ranging from 76 to 80 m (250 to 262 ft), and other improvements were made on the basis of the experience gained from the construction of the first bridge.[65]

6.9 Reconstruction and the Age of Prestressed Concrete

The completion of Freyssinet's Marne Bridge at Luzancy in 1945 marks the beginning of a new era of bridge construction using prestressed concrete technology. Prestressed concrete was extensively used in Post-World War II Europe to reconstruct badly damaged infrastructure, particularly in Germany. In 1950, Dyckerhoff & Widmann began construction on the Gänstor Bridge over the Danube at Ulm, Germany, which was a two-hinged frame with an 82.4 m (270.3 ft) span.[66] (**Figs. 29**) From 1950 onwards, some 108,000 prestressed concrete bridges were built worldwide.[67] The longest spans of these bridges grown to 300 m, the 301 m Stolmasundet Bridge in Norway was completed in 1998.[68]

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[62] Grote and Marrey, p58 and 98.
[63] Wittfoht, p150.
[64] '*appartienne sans reserve à cet âge nouveau*,' from Freyssinet's article "Le Pont de Luzancy" in *Travaux*, May 1946, p161.
[65] Grote and Marrey, p100.
[66] Wittfoht, p156.
[67] Bonstedt, slide 11.
[68] Feani, Juhani Virola. World's Longest Bridge Spans. http://www.hut.fi/Units/Departments/R/Bridge/longspan.html.

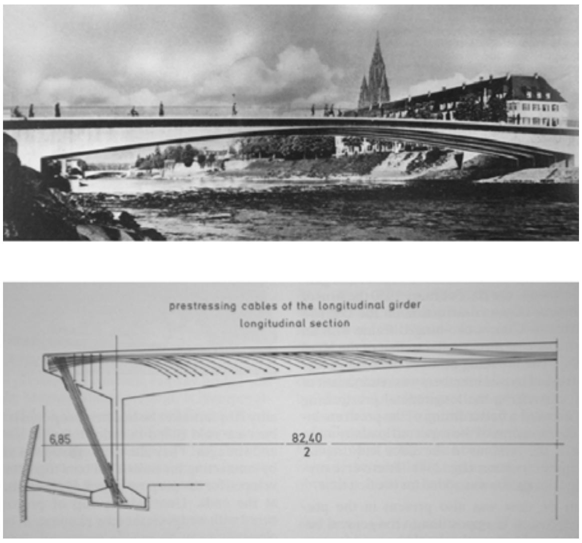

Fig. 29: (a) Gänstor Bridge at Ulm, Germany, with 82.4 m span, Dyckerhoff & Widmann, 1950. (b) Arrangement of the prestressing tendons in the main girder of the Gänstor Bridge. (Wittfoht)

The application of prestressed concrete generally followed separate paths in France and Germany from 1950 onwards. In France, engineers continued to use the simply supported beams similar to those at Wadi Fodda and Öulde, and they continued to develop prefabricated systems comprised of a number of elements set together to form a whole such as Freyssinet's Marne River Bridges.. In Germany, the construction of cast-in-place prestressed concrete bridges developed to a much higher level. Fritz Leonhardt, Ulrich Finsterwalder and the firm of Polensky & Zöllner made significant developments in the art of constructing cast-in-place prestressed concrete bridges. As Wittfoht writes, "Both approaches have their indisputable advantages and unavoidable disadvantages. They have coexisted side by side up to the present day, providing one another with the impetus for further advancement, and to a certain extent, combining."

Technological developments in prestressed concrete occurred in three principle areas: anchorage design, construction process, and the application of external versus internal prestressing tendons. I will treat each separately in the following sections.

6.10 Anchorage Design

6.10.1 Pre-Tensioned Tendons with Bond to the Concrete

Pre-tensioned tendons with bond to the concrete were the first widely used means of anchoring prestressing tendons. This process is illustrated in **Figure 1**. Freyssinet and Wayss & Freytag used this system for their first bridges, though they used devices at the ends of the tendons to ensure that the axial forces would be predictably transferred. During World War II, the Austrian engineer Ewald Hoyer simplified the procedure by simply cutting the tendon and allowing the bond between the tendon and the concrete to keep the tendon from slipping. This method introduced mass production to the design of prefabricated prestressed concrete girders because with his method he could pretension a long tendon and cast a long beam that could be cut up to any desired lengths. This method is widely used today, though the beams are cast individually so that only the tendon has to be cut and not the concrete itself.[69]

[69] Grote and Marrey, p46 and 48.

6.10.2 Post-Tensioned Tendon with Conical Head

In September 1939, Freyssinet introduced an improved system of prestressing anchor that allowed the structure to be prestressed after the concrete has cured, or post-tensioned. Post-tensioning allows the cable to be placed in more complex geometries to better reflect the distribution of stress than is practical using pre-tensioning. As mentioned above, jacking equipment for post-tensioning is lighter than for pre-tensioning because the tendon can be jacked directly against the hardened concrete.[70]

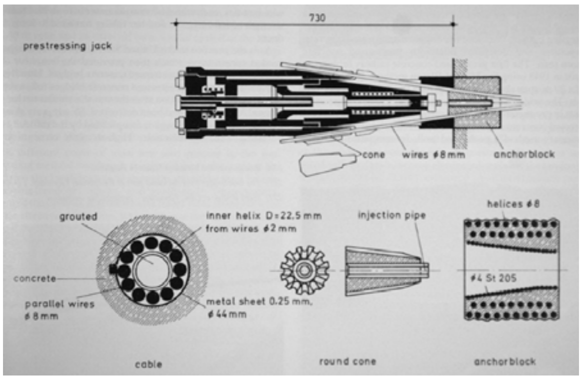

Fig. 30: Freyssinet's prestressing tendon anchorage system with conical wedge. (Wittfoht)

Freyssinet anchored the tendon with a conical wedge after the jack had stretched one end of the tendon to the required elongation. (**Fig. 30**) The anchorage was embedded flush with the surface of the concrete and the jacking equipment could be mounted to stress individual tendons of a typical 12-tendon bundle one at time. This allowed a gradual transfer of the prestress force to the concrete.[71] One disadvantage of the wedge system is that there is some amount of slip that occurs when the tendon is released from the jack and the conical wedge sets in place. This slip causes some amount of prestress loss that has to be taken into account by the design engineer in addition to the effects of shrinkage and creep when calculating the initial level of prestress.

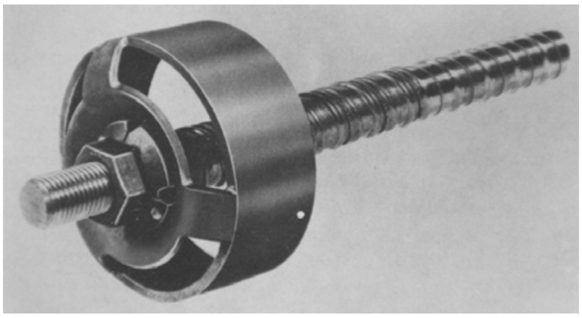

Fig. 31: Dywidag prestressing system with bell anchorage. (Wittfoht)

6.10.3 Threaded Rod with Tensioning Nut

In 1927, Richard Färber, of Breslau, Germany,[72] patented an anchorage system in which threaded bars were placed in a type of jacket to prevent bond with the concrete.[73] Tightening a nut against the cured concrete tensioned the bars. Färber wrote, "As the tendon can be jacked against the hardened concrete, the jacking equipment can be lightweight, and each bar can be stressed separately."[74] This system is similar to the design of Thomas Pope's reinforced masonry beam, published in 1811. (**Appendix A-05, p A.253, Fig. 2**) Färber's anchor design was not successful at the time.[75]

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[70] Grote and Marrey, p58.
[71] Wittfoht, p150.
[72] now Wrocław, Poland.
[73] Grote and Marrey, p58 and 98. Färber, Richard. German Patent DRP 557 331, 1927 / 1932.
[74] Wittfoht, p150.
[75] Grote and Marrey, p60.

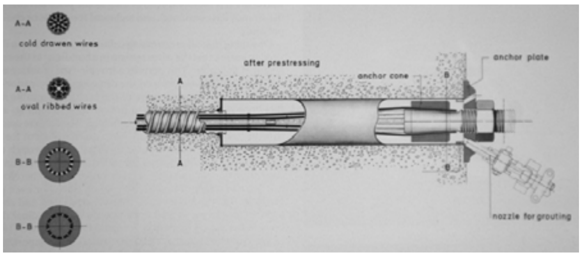

Fig. 32: PZ prestressing system combining conical wedge and anchor bolt. (Wittfoht)

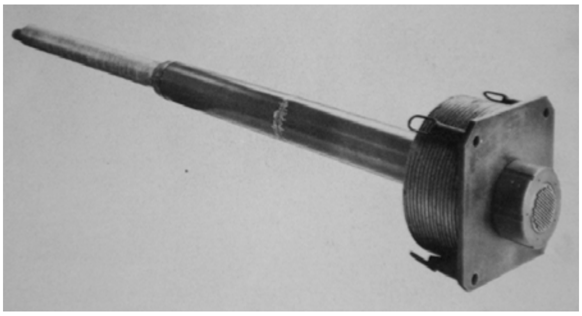
Fig. 33: BBRV prestressing system with button head anchorage. (Wittfoht)

Some twenty years later, the German firm of Dyckerhoff & Widmann, with its chief engineer Ulrich Finsterwalder, devised a widely successful threaded rod system that is marketed as the Dywidag System. (**Fig. 31**) Dyckerhoff & Widmann used this anchorage system for the first time in the construction of the Gänstor Bridge in Ulm, Germany in 1950. The threaded bar tendons are primarily 26 mm (1 in) diameter 60/90 steel with a yield strength of 600 N/mm^2 (87 ksi) and a tensile strength equal to 900 N/mm^2 (130.5 ksi). Later bars of 80/105 steel were used. The advantages of this system over Freyssinet's conical wedge anchorage was that there are no losses due to the unavoidable slip of the conical wedges used, and the tendons can be precisely tensioned because the elongation can be exactly measured. However, the applied prestress forces are lower because the bar stock used to make the threaded rod cannot be fabricated with the same high strengths as strands or cables. Finsterwalder believed that it was not necessary to employ such high strengths, preferring to accommodate some tension in the structure when subjected to extreme load cases.[76]

6.10.4 Other Advances in Anchorage Technology

The PZ system, invented by Polensky & Zöllner combines the advantages of high strength cables with those of bar anchorages such as the Dywidag System. Polensky & Zöllner used high-strength steel cables and anchored them with a conical wedge that is itself anchored by the primary anchoring system that is a short threaded rod with nut. (**Fig. 32**) While this system is more complex than either the Freyssinet or Dywidag systems, it does solve the problem of slip in Freyssinet's system while giving the precise control over the prestressing force that is possible by tensioning the tendon with the nut.[77]

The BBRV (Brandestini, Birkenmaier, Ros, Vogt) System, a Swiss design highly regarded for its dependability, achieves the same goals of the PZ system, but eliminates the conical wedge and introduces an intermediate step of anchoring each wire in a tendon by forming a button at the end. (**Fig. 33**) This system requires great precision when cutting the cables to length but also gives superior precision and control during stressing operation.[78]

In 1951, a railway bridge across the Neckar Canal in Heilbronn, Germany was built using the Baur-Leonhardt prestressing system. (**Fig. 34**) In this system, the tendons are wrapped around semi-circular anchor blocks that are jacked to prestress all of the tendons at the

[76] Wittfoht, p153. Finsterwalder. "Dywidag-Apannbeton". , 1962, pp141-158.
[77] Wittfoht, p153. Leonhardt, F. *Spannbeton für die Praxis* (2nd ed.) Wilhelm Ernst & Sohn : Berlin, 1962.
[78] Wittfoht, p153.

same time. Freyssinet first used a similar system similar stabilize the foundations of the Gare Transatlantique at Le Havre in 1934.[79] The variable geometry of the prestressing tendons between the supports and over the intermediate piers of the Neckar Bridge is clearly visible in **Figure 34**.

Fig. 34: Baur-Leonhardt prestressing system used in the first post-tensioned prestressed concrete railway bridge over the Neckar at Heilbronn, Germany, 1951. (Wittfoht)

It took 25 years before the wires in a tendon were replaced with strands. Introduction of strand versus cable tendons led to further developments. Over time, the prestressing force concentrated at an anchorage was steadily increased from 220 N/mm^2 (32 ksi) used in the Dischinger's Aue Bridge to 400 metric tons (441 tons) or more for special applications.[80]

6.11 Construction Process and Method

6.11.1 The Problem of the Centering

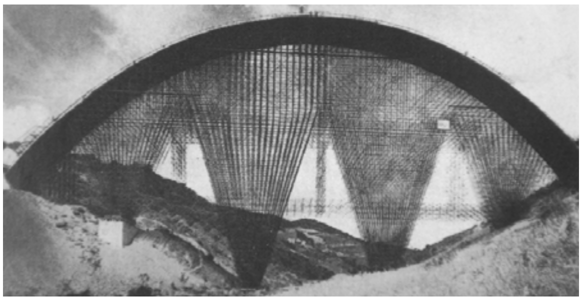

Fig. 35: Centering for the Aglio Viaduct for the motorway between Milan and Naples, Italy. 164 m span, 44 meters high. (Wittfoht)

A major disadvantage of concrete construction is the necessity to erect formwork and, for long span bridges, temporary structures on which to support it. The cost of the formwork, which includes an enormous investment of time, labor, and material, can a significant coast in a project. Christian Menn, a Swiss engineer, has estimated that the formwork is an average 20% of the total cost of a bridge.[81] In comparison, the cost of the super-structure is 34.5% of the total, the rest being comprised of the substructure and accessories.[82] The magnitude of this problem is amply demonstrated in **Figures 35 and 36**, showing the centering for the Aglio Viaduct and the Po Bridge, both located in Italy. The reliability of such a centering can only be determined by laboriously checking every connection, which can easily number in the tens of thousands. As spans grow larger the safety of such centering methods becomes questionable. During construction of the 269 m (882.5 ft) arch of the Sandö Bridge in Sweden in 1939, eighteen people were killed when the centering collapsed. As a result of such drawbacks, significant efforts have been expended to reduce the cost of formwork for long span bridges and improve safety during their construction.[83]

[79] Grote and Marrey, p18 and 20.
[80] Wittfoht, p155.
[81] Based on the average costs of nineteen Swiss bridges built between 1958 and 1985.
[82] Billington[3], p167.
[83] Wittfoht, p136-137.

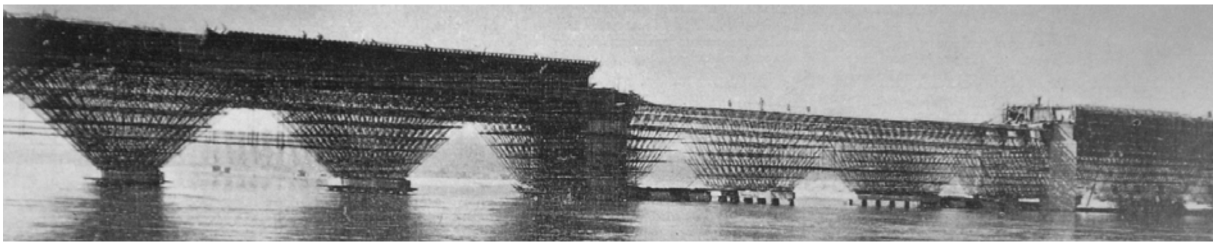

Fig. 36: Falsework built for the entire length of the bridge to construct the Po Bridge for the Autostrade del Sol, Italy. (Wittfoht)

The use of cast-in-place, fixed formwork for the construction of prestressed concrete beam bridges has its place however, as it is generally suitable for bridges of short to moderate spans. Early prestressed concrete bridges were either built *in situ*, such as the railway bridge over the Neckar River at Heilbronn, Germany, or with precast beam elements characteristic of Freyssinet's early work on the Wadi Fodda Bridge.[84] (**Figs. 34 and 21**) The advantage of cast-in-place construction is that the span of the bridge is not limited, unlike the pre-cast segment that has a maximum practical span 50 m (164 ft). Situations where the terrain is rather flat and the bridge is not very high are particularly favorable to the use of fixed formwork.

For longer span bridges, the methods of concrete construction must be rationalized if concrete is to be competitive with steel construction. To do this, precast construction was first developed. The span limits of precast construction do not adequately address the requirements of long span bridge building. In response, material-adapted construction methods have been developed that take advantage of the possibilities of assembly-line production methods, which have resulted in the most successful advances in the art of prestressed concrete construction since the 1960s.[85]

6.11.2 Early Developments in Concrete Construction Process

The Melan Method

One early invention to reduce the cost of formwork for reinforced concrete construction was to eliminate the falsework necessary to support the formwork. Joseph Melan, a professor of bridge construction, was issued a patent in 1892 for what became known as the Melan construction method. This method, first developed for concrete arch construction[86], eliminated the intermediate supporting structure of the formwork by using a steel frame from which the formwork was suspended to cast the structure. The frame itself was then concreted into the structure as its reinforcement.[87] (**Fig. 37**) This method was later applied to beam structures as well, as in the construction of the German submarine pen shown in **Figure 25**. The greatest drawback of the Melan method is that the amount of steel required exceeds that necessary to function simply as reinforcement. The method was only employed in those cases where the cost of the additional steel was lower than using conventional centering.

[84] The bridge over the Neckar River is the first post-tensioned prestressed concrete railway bridge to be built. The bridge was prestressed using the Baur-Leonhardt system.
[85] Wittfoht, p215.
[86] The first bridge built using the Melan method was the "Swimming School" Bridge in 1898 in Steyr, Austria.
[87] Deinhard, p31-33.

The Melan method was used to construct a number of large bridges in Germany and particularly in the United States during the 1920's and 30's despite the relatively high cost of steel and the excessive quantity of that material required by the method. According to Wittfoht, the Melan method was of decisive importance for reinforced concrete bridge construction in the United States. Up until 1894 there were no noteworthy reinforced concrete bridges in the United States because of the poor quality of cement available and a shortage of qualified labor.

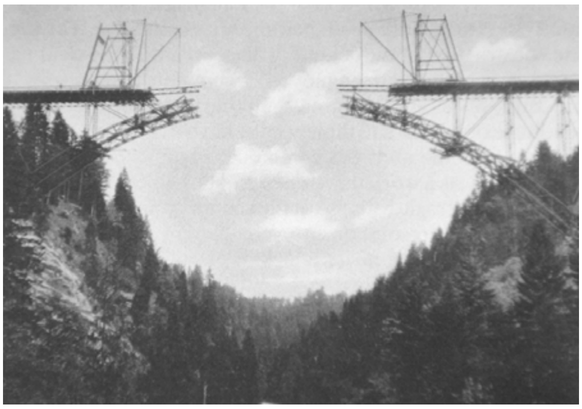

Fig. 37: Erection of the steel lattice truss to construct the Ammer Bridge at Echelsbach, Germany, using the Melan Method, 1929. (Wittfoht)

The success of the Melan method can be attributed partly to the marketing skills of Fritz von Emperger, who introduced the technology in America, but the primarily reason was the situation of the North American construction industry at the time. An engineer of the Concrete Steel Engineer Company noted, "Labour in the United States is relatively expensive, from the labourer to the engineer. As a result, simpler designs… are preferred. The Monier construction method, for example, has still not gained foothold… The Melan construction method immediately met with increasing interest primarily because the reinforcement was placed in the form of easy-to-assemble rigid arches. This assured that a good bond between both materials could be expected even under the most unfavourable conditions…."[88]

Self-Supporting Centering

Eugéne Freyssinet introduced self-supporting centering in 1930 to build the Elorn Bridge at Plougastel, France. This arch bridge had three spans of 186.4 m (611.5 ft) each; the longest reinforced concrete arches in the world when completed. (**Fig. 17(a)**) Freyssinet designed a floating, self-supporting centering to build the bridge. (**Fig. 17(b)**). The centering was founded on two pontoons tied together by cables. The centering was floated into position with the tide and a method was devised to lift the centering onto previously concreted springings. This construction used *only* 200 m^3 (262 cu yd) of timber. The economic savings of Freyssinet's centering design helped Freyssinet win the design competition for the bridge. The bridge was concreted in three stages: lower chord, box-section walls, and upper chord. A total of 25,000 m^3 (32,700 cu yd) of concrete and all of the shuttering and reinforcement were brought into place by two travelers, each with a lifting capacity of 20 kN (4,500 lb), operating on cables strung 860 m (2,822 ft) across the river.[89]

Another type of self-supporting centering was used to construct a series of arch bridges in Austria by the firm, Konrad Beyer & Co., using a stayed-cantilever method first proposed by Thomas Telford to construct a cast-iron arch over the Menaï Strait between England and Whales in 1811.[90] Telford did not ever use this method, but it was developed by an Italian

[88] Wittfoht, p140-141. Mueser. *Die Erfolge der Eiseneinlage in den Eisenbetonbauten Nordamerikas*. Deutscher Beton-Verein, Vorträge 1904, p115.
[89] Deinhard, p71-72. ref. "Mitteilungen der Société Anonyme des Entreprises Limousin: Die Brücke von Plougastel". *Beton und Eisen*, 1929, p293.
[90] Telford, p543. **Ref. Appendix A-02, p A.49, Fig. 23**.

Fig. 38: Erection of the timber lattice truss centering with a 180 m span for the First Nösslach Bridge, Austria. (Wittfoht)

engineer named Cruciani[91], an Italian. The centering was constructed of timber lattice trusses erected using cantilever construction in which the arch sections were anchored back from both sides with cables and the center section was placed by a cable crane. (**Fig. 38**)

Both of these methods introduced improvements over fixed formwork that were used as a basis for improvements in the way long-span, prestressed concrete beam bridges were constructed.

Fig. 39: 150 metric ton precast, Prestressed concrete girder being floated into position by enormous cranes for the bridge over the Rhine at Vallendar, Germany, 1959. (Wittfoht)

Prefabrication

Prestressed concrete beams can be prefabricated as single span units or in segments. *Segmental* construction is described in more detail below. This section will briefly focus on pre-cast, single-span prestressed concrete beams.

Eugène Freyssinet introduced pre-cast pre-stressed beams in the Waddi Fodda footway constructed at the end of the 1930s. (**Fig. 21**) Such concrete **I**-sections are standardized today and used mainly for short highway spans of 30 to 50 m (100 to 164 ft). These girders are particularly economical if a large number of approximately equal spans are to be built at a low height above existing terrain so that the beam can be transported adjacent to its final position and simply hoisted into place.[92]

The 50 m (164 ft) span limit of precast is primarily due to the length and weight of a beam that can be safely and economically transported from its place of fabrication to the construction site. Larger precast spans can be made, as was done in the construction of the bridge shown in **Figure 39**. The beam shown weighs 150 metric ton (165 ton) and required

[91] Aigner, F. "Stahlbetonbogenbrücken auf der Österr. Brenner-Autobahn". *Der Bauingenieur*, 1968, pp91-95.
[92] Wittfoht, p215.

the use of extraordinarily heavy equipment to place it. Such equipment can only be used in a limited number of situations.

In some pre-cast girder bridges, "platforms" are built above the piers to increase the span. The 1,000 m (3,280 ft) long Kosi Bridge in India was built this way. The pre-cast girders are 50 m (164 ft) long and the piers are spaced 65 m (213.3 ft) on center. (**Fig. 41**)

Speed or erection is one significant advantage of precasting. This is useful in situations it is critical to limit interruption of to existing traffic on highways, railroads or over waterways. **Figure 40** shows how precast girders were used to construct a field access bridge over a railway line in Germany.

6.11.3 Modern Developments in Concrete Construction

Cantilever Construction

The single most influential construction method developed for long-span, prestressed-concrete bridges is the cantilevered construction method. The first reinforced concrete beam bridge successfully built using cantilever construction was a road bridge over the Rio De Peixe at Herval, Brazil in 1930. (**Fig. 42**) Emilio Baumgart, the engineer, decided against using falsework because of the difficult currents of the river. The construction of the normally reinforced concrete bridge was carried out in 1.5 m (4.9 ft) segments by extending the timber shuttering. The cantilever construction method for a reinforced concrete beam bridge was only again used in 1937 in England because of the constructive difficulties this method introduces, particularly with respect to deflection.[93]

Fig. 40: Erection of precast prestressed concrete girders over railway tracks in Germany to construct a field access bridge. (Wittfoht)

Prestressed concrete is well suited to this method of construction however. The first prestressed concrete beam bridge to be built using the cantilever method was for the Lahn Bridge at Balduinstein in 1950-51. The bridge, with a span of 62 m (203.4 ft), was built under the direction of Ulrich Finsterwalder while he was chief engineer for Dyckerhoff & Widmann. Steel bars, which could be coupled together end to end, were used for the prestressing tendons.[94]

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[93] Deinhard, p95; Wittfoht, p196; "Bemerkenswerte Eisenbetonbalkenbrücken in Brasilien", from *Beton u. Eisen*, 1931. p204.

[94] Wittfoht, p196. Finsterwalder, U.; Schambeck, H: Von der Lahnbrücke Balduinstein bis zur Rheinbrücke Bendorf. Der Bauingenieur. 1965, pp85/91. Finsterwalder, U. Technishe Entwicklung des freien Vorbaus bis zum Bau der Rheinbrücke Bendorf. Vorträge Betontag 1965, pp. 322/377, DBV Wiesbaden.

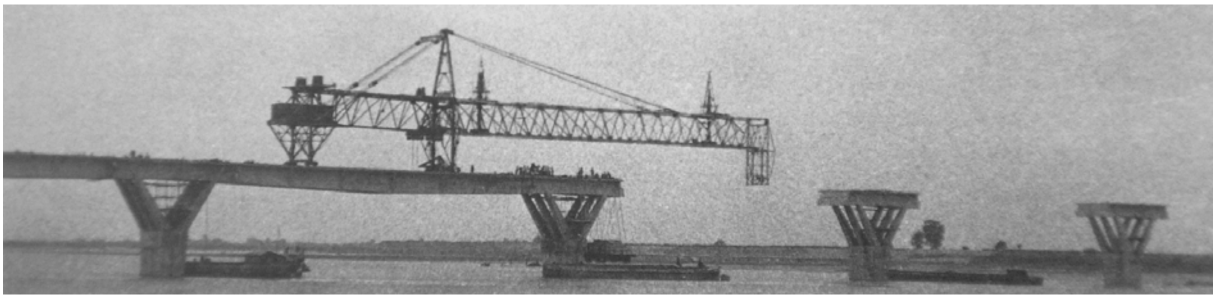

Fig. 41: Kosi Bridge near Kursela, India, 1000 m long bridge built with 50 m long pre-cast girders and piers with "platforms" 15 m long to extend distance between piers. (Wittfoht)

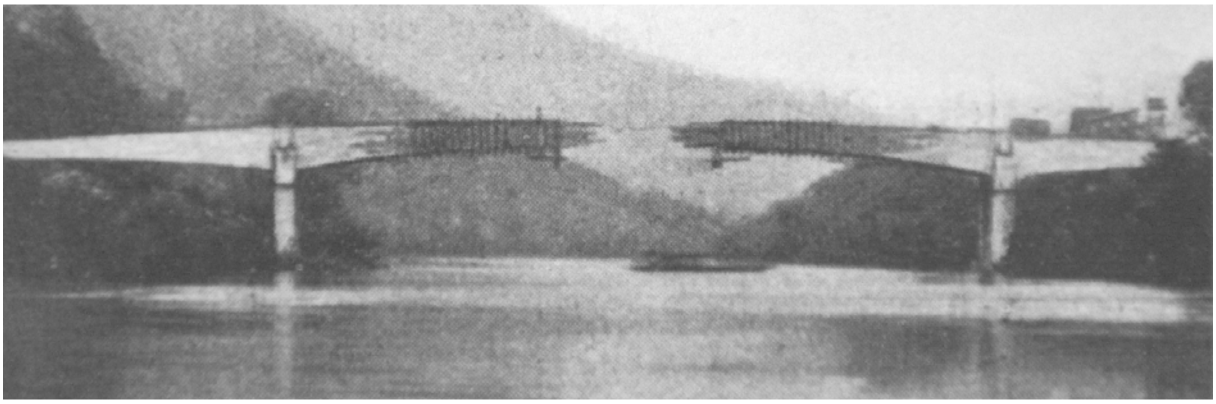

Fig. 42: First reinforced concrete bridge built using the cantilever construction method over the Rio de Peixe in Brazil, Emilio Baumgart, 1930. (Deinhard)

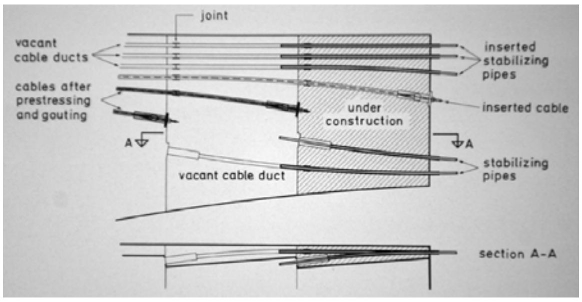

Fig. 43: Diagram of Polensky & Zöllner's system to maintain the geometry of tendon ducts while concrete was being placed using shaped bars. This system is used in cantilever construction to eliminate couplers. (Wittfoht):

Early efforts to substitute cables for iron rods in order to take advantage of higher prestressing forces that would reduce the number of tendons needed in the section failed because of unsolved problems with coupling the tendons together. This problem was overcome in 1954 by Polensky & Zöllner with a simple system in which bars formed to the desired cable geometry were inserted into ducts that retained their exact position and shape while the concrete was placed. (**Fig. 43**) After the concrete cures the bars are removed and cables threaded through.[95]

Cantilever construction eliminates large falsework construction, leaves traffic lanes under the bridge open during construction, and rationalizes the construction process with constantly repeating segmental construction

[95] Wittfoht, p198-200. Wittfoht, H. Die "Neue Autobahnbrücke über den Main bei Bettingen", from *Betontag*, 1961, p273.

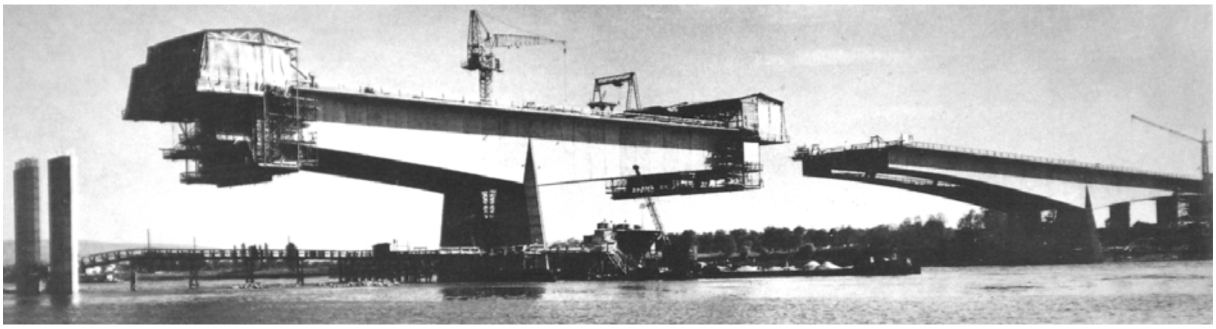

Fig. 44: Example of form travelers used in cast-in-place, cantilever construction method. Image shows the Rhine Bridge at Bendorf, built in 1965 with a 208 m center span. A temporary access bridge can be seen that is used to transport workers to the pier from which work on the bridge is advancing. (Wittfoht)

Fig. 45: Example of cantilever construction method using auxiliary cable stays. Image shows construction of the Lahntal Bridge at Limburg, Germany. The bridge was commenced at both abutments and advanced toward the center. (Wittfoht)

cycles. Cantilever construction is particularly advantageous for long spans that cross deep valleys and waterways, and terrain with poor foundation conditions, where constructing a traditional centering is impractical.[96]

By the 1960s, prestressed concrete beams with spans over 250 m (820 ft) were being constructed using the cantilever method, such as the 256 m (840 ft) span of the Zoo Bridge across the Rhine in Cologne in 1961. Specially designed travelers allow segment lengths of 3 to 4 m (9.8 to 13.1 ft) to be achieved, greatly increasing the speed and efficiency of construction.[97] (**Fig. 44**)

Cantilever Construction with Auxiliary Cable Stays

A disadvantage of normal cantilever construction is the need to transport labor and material to the pier sites, often located in water or on difficult terrain, because the bridge is usually started at the piers with work progressing simultaneously on either side such that one side balances the other. (**Fig. 44**) After work is completed at one pier site, the formwork must be transferred to the next. This disadvantage can be avoided by starting construction at the abutments, which requires that auxiliary cable stays be used to support the relatively large cantilever before the bridge reaches the next pier. Once the first pier has been reached, the pylons of the cable stays are moved over the pier to support the next segment. Construction

[96] Wittfoht, p196.
[97] Wittfoht, p196 and 204.

Fig. 46: Example of traveling girders that support formwork from below the bridge deck. Note the openings provided for at the top of the piers to allow the traveling girder to pass. Image shows the construction of the 1.5 km Pfeddersheim Viaduct for the A 61 between Ludwigshafen and Krefeld, Germany. (Wittfoht)

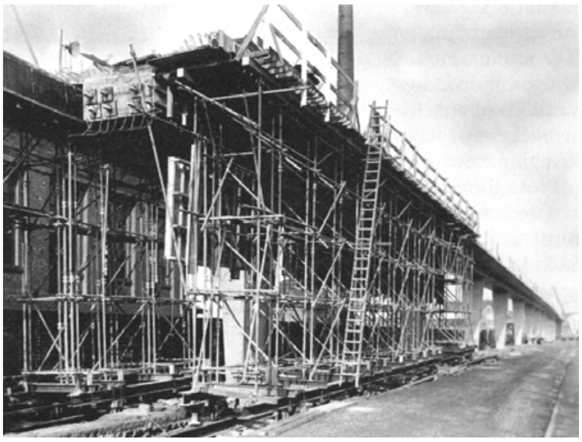

Fig. 47: Rail mounted formwork used for a pedestrian and bicycle bridge at the Casellastrasse in Frankfurt/Main, Germany. (Wittfoht)

can proceed from both sides of the bridge, though this increases costs due to the necessity to supply two construction sites.[98] (**Fig. 45**)

Traveling Formwork

The use of reusable and movable formwork can be a viable alternative to fixed formwork when the bridge is greater than 300 m (984.3 ft) in length and the overall cross-section is constant. The spans of the bridge should be kept constant where possible because unequal spans disrupt the regular cycle of progress and increase costs. This general rule contradicts traditional rules of bridge design that related the span to the pier height. (**Fig. 46**) Openings in the piers must be provided for to let the girders pass from one pier to the next. Leaving them open saves costs and allows access to the bearings for maintenance.[99]

If the structure has a low height and crosses level terrain, traveling formwork that moves on the ground can be used. The simplest form of this type is to mount the falsework on rails. (**Fig. 47**) If the bridge traverses difficult terrain, such as a deep valley, or water, it is necessary to use self-supporting formwork that can span the required distances. Two types of self-supporting formwork can be used: a traveling girder, which passes beneath the bridge, and a traveling gantry, which passes above the bridge. Such travelers are either supported by two piers or by one pier and the previously completed portion of the structure. The girders must be two spans long to obviate the need for intermediate supports. The

[98] Wittfoht, p210-213.
[99] Wittfoht, p215, 222 and 224.

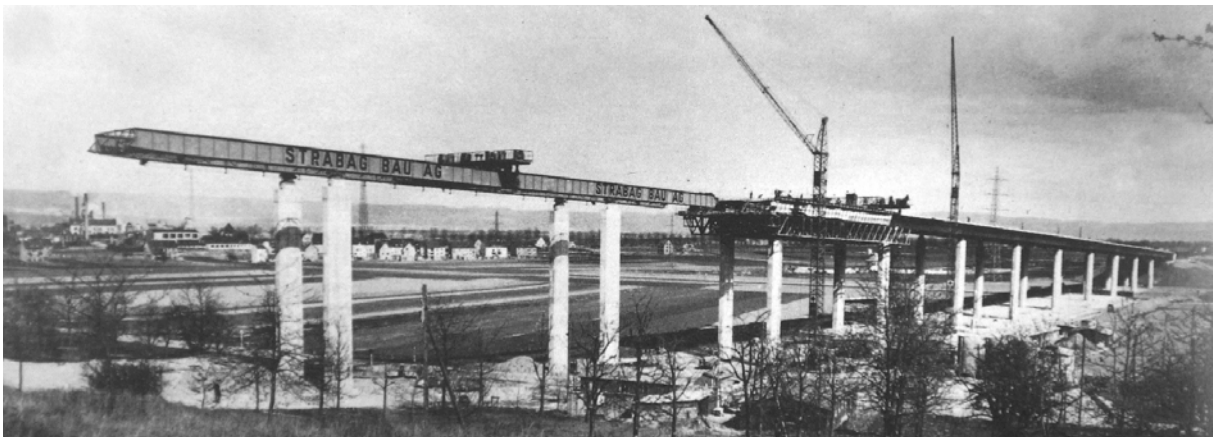

Fig. 48: Bridge on the Kettiger Hang for the B 9 between Bonn and Koblenz, Germany, first prestressed concrete bridge built using self-supporting traveling formwork, 1959. (Wittfoht)

formwork advances on two crane trucks, on traveling at the front of the structure and the other resting on the completed sections of the bridge.[100]

The first prestressed concrete bridge constructed using self-supporting traveling formwork was the Kettinger Hang Bridge located between Bonn and Koblenz, Germany, in 1959. (**Fig. 48**) The builders of this bridge learned that the economy of using traveling formwork is limited to relatively long spans unless the equipment can be used on another project. The Kettinger Hang Bridge was not long enough to justify the expense of the traveling formwork structure.[101]

Fig. 49: Krahnenberg Bridge, first prestressed concrete bridge built using span-by-span cantilever construction method with self-supporting traveling formwork. (Wittfoht)

The Krahnenberg Bridge at Andernach, Germany, built from 1961 to 1964, was the first prestressed concrete bridge to be built with span-by-span cantilever construction using self-supporting traveling formwork. (**Fig. 49**) The bridge's total length was 1,100 m (3,609 ft) and was located on a hillside with unstable slope conditions. The advantage of building span-by-span is that the concrete dead load is transferred to the superstructure after the prestressing of the segment has been completed, thereby eliminating the otherwise unavoidable fluctuations in stress due to the various loading conditions of a girder that is a free cantilever until it reaches the next pier. Since the effect of the dead load of each newly cast segment does not affect the stress in the main superstructure it is possible to increase the segment lengths to 7 m.[102]

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[100] Wittfoht, p218.
[101] Wittfoht, p218.
[102] Wittfoht, p218, 200 and 202.

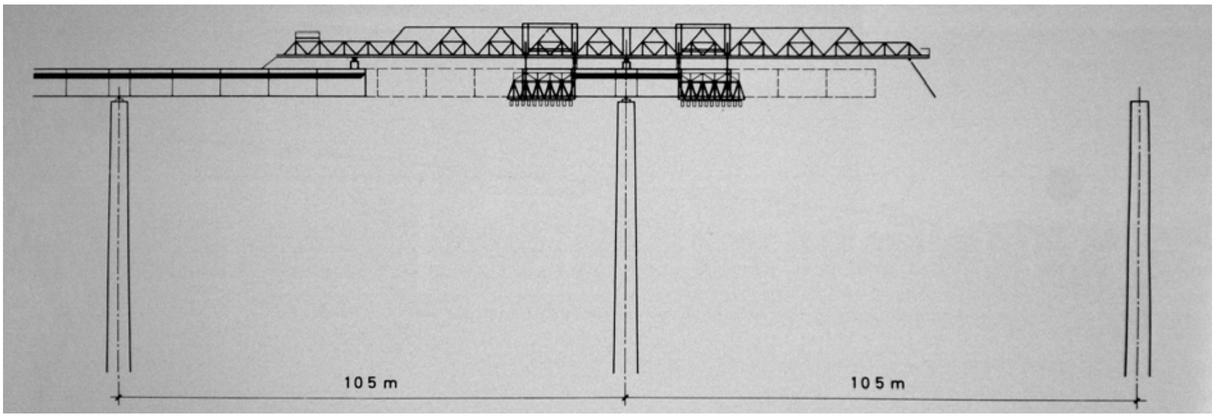

Fig. 50: Diagram showing concreting sequence for the construction of the Siegtal Bridge at Eiserfeld, Germany using span-by-span launching gantry designed for long spans. (Wittfoht)

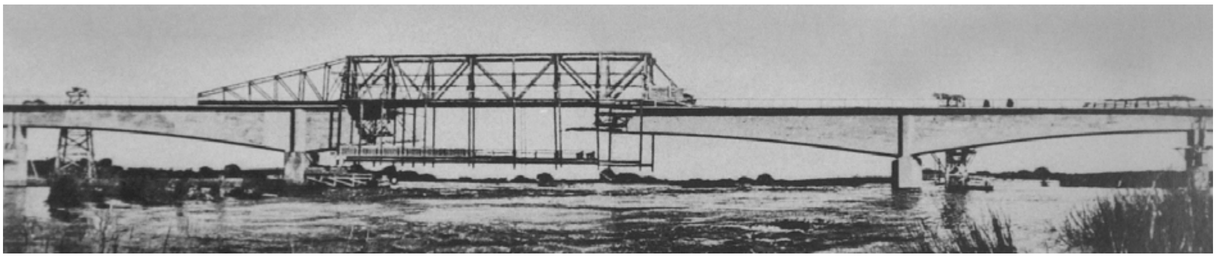

Fig. 51: Railroad bridge across the Lualaba River at Kongola in the former Belgian Congo, first reinforced concrete bridge built using a launching gantry, 1938. (Wittfoht)

Traveling Formwork for Large Spans

A segmental span-by-span launching gantry illustrated in **Figure 50** can be used for a continuous series of large or very large spans. The gantry is supported on the end of a half completed span and the next pier. Construction then continues symmetrically from both sides until the already completed portion is reached, after which the gantry is moved to the next section. Segments can either be precast or cast-in-place. Which of the two variations is chosen is dependent on site and construction conditions as well as the construction schedule.[103]

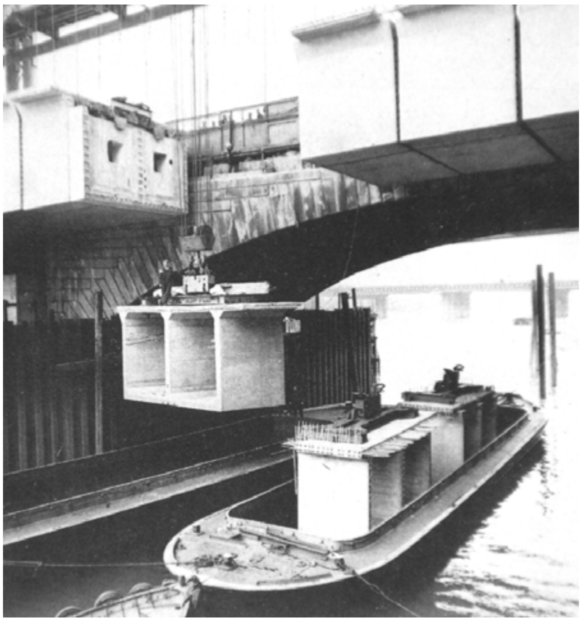

Fig. 52: New London Bridge being assembled using precast box-section segments. (Wittfoht)

Using a launching gantry was first tried in 1938 for the construction of a reinforced concrete railroad bridge across the Lualaba River at Kongola in what was then the Belgian Congo. A steel bridge was fitted with launching noses to be used as the gantry. (**Fig. 51**) The steel

[103] Wittfoht, p236.

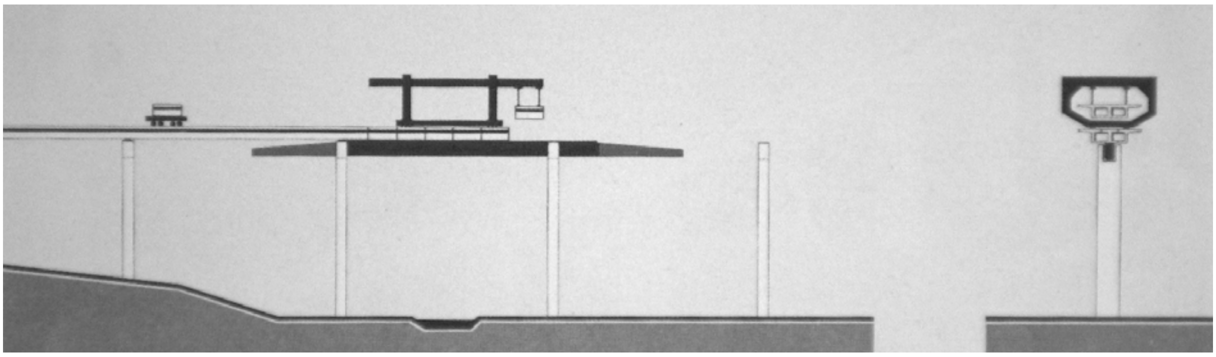

Fig. 53: Illustration of span-by-span construction of precast segments. (Wittfoht)

bridge was in fact used as a bridge after the Kongolo Bridge was finished.[104]

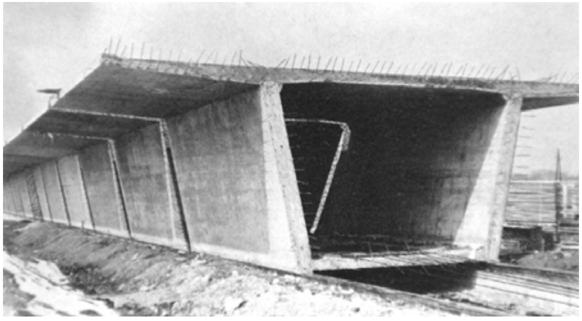

Fig. 54: Precast segments atop the guide-way used to construct the Ager Bridge in Austria, 1959-1962. Exposed rebar used to make mechanical connection with normal reinforcement between segments and to 50 cm gap left between segments used to facilitate tendon placement, after which the gaps were filled with concrete. (Wittfoht)

The second bridge to be constructed using a launching gantry was the La Voulte railway bridge in 1953. The gantry for this bridge was made from Bailey bridges from which the formwork could be suspended and moved along the entire length of the gantry. The new London Bridge, with spans of 80 – 103 - 80 m (262.5 - 337.9 - 262.5 ft), was constructed by suspending precast segments from a temporary steel bridge before the tendons were threaded through them and the joints were closed. The segments were transported to the site by barge.[105] (**Fig. 52**)

Span-by-span construction with a launching gantry was demonstrated during the building of several bridges with 30 to 50 m (98.4 to 164 ft) spans for the French highway system. The segments were either suspended from a launching gantry or supported from below by a launching girder over an entire span until the point of zero moment in the next span. (**Fig. 53**) After the prestressing cables were threaded through and stressed, the gantry could be moved to the next span.[106]

As early as 1958, Polensky & Zöllner performed a study to determine what launching equipment was best suited for certain conditions. Their study of formwork girders above and below the deck and through the box girder cross-section itself concluded the following:

- The use of a launching girder was found to be most suitable for span-by-span casting of the superstructure and was therefore to be recommended for smaller spans up to approximately 50 m (164 ft).
- Larger spans could be more efficiently built using span-by-span segmental construction with a launching gantry.

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[104] Wittfoht, p236.
[105] Wittfoht, p236-237.
[106] Wittfoht, p237.

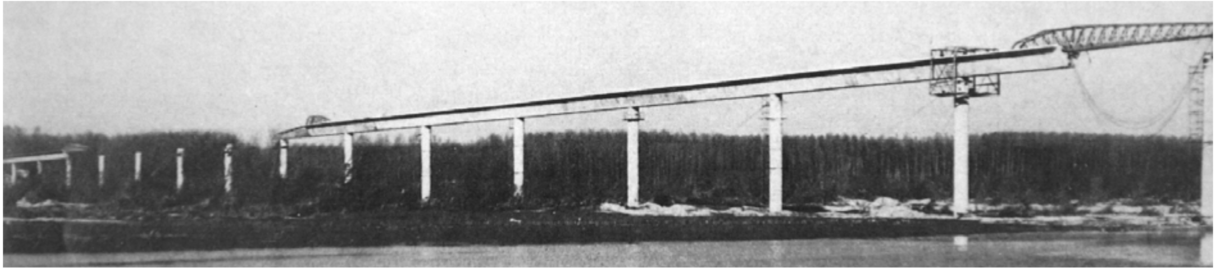

Fig. 55: Complete bridge unit being incrementally launched into place during construction of a pipeline bridge over the Po River near Pavia, Italy. (Wittfoht)

- The formwork girder that passed through the box-girder cross-section itself was only to be recommended if the external conditions of the project eliminated the use of the other two systems.[107]

Incremental Launching

Launching can be used for bridges of moderate length that are either straight or have a constant curvature. With incremental launching the bridge is 'extruded' from a fixed forming station on one abutment of the bridge and jacked into position as segments are incrementally completed.[108]

A precursor to this method was the Ager Bridge in Austria, built from 1959 to 1962. To construct this bridge 8.5 m (27.9 ft), 180 metric ton (198 ton) segments precast at one end of the bridge were pushed into their final position over a guide-way fixed on falsework. A 50 cm (19.7 in) joint was left between each segment to facilitate placement of prestressing cables that were continuous over the whole length of the bridge. (**Fig. 54**) After the tendons were in place the joints were concreted. The cables were deflected at the joints to give them a draped form and cables were encased in concrete to protect them against corrosion after they were stressed.[109]

The first bridge to be incrementally launched with present methods was a 450 m long bridge over the Inn River in Kufstein, Austria. Each segment was cast directly onto the previous one in stationary formwork directly behind one of the abutments. After the curing of the last segment, the entire completed section was jacked forward to vacate the formwork for the next segments. Intermediate supports were used to decrease the stresses in the superstructure while the bridge was being launched.[110]

One impressive example of bridge constructed using the launching method is an oil pipeline bridge over the Po at Pavia, Italy. This bridge has a total length of 1370 m with spans of 36 m. The bridge is comprised of girders continuous over six to seven spans. Each continuous girder was launched from the bank one after the other. (**Fig. 55**) Steel launching noses on both ends reduced the large variable cantilever moments incurred in the girders while launching.[111]

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[107] Wittfoht, p237.
[108] Wittfoht, p232.
[109] Wittfoht, p232. Leonhardt, F.; Baur, W. "Die Agerbrücke, eine aus Gross-Fertigteilen zusammengesetzte Spannbeton-brücke". *Bautechnik*, 1963, pp241-245.
[110] Wittfoht, p232. Leonhardt, F. Baur, W. "Erfahrungen mit dem Taktschiebeverfahren…". *Beton- und Stahlbetonbau*, 1971, 161-167.
[111] Wittfoht, p235. Giuliani, G. "Il nuovo ponte per l'oleoletto", *Cemento*, 1970, p163-174.

6.11.4 Segmental Construction

Why Segments?

Segmental construction emerged in reaction to deficiencies in other prestressed concrete construction systems, such as the practical span limits of pre-cast, single-span girders, and the size, complexity and cost of fixed formwork for cast-in-place structures. Cast-in-place and pre-cast segmental construction methods have been developed and both are currently used. The choice between which one to use depends on local conditions, the size of the project, time allowed for construction, restrictions on access and environment, and the equipment available to the contractor.[112]

Characteristics of Cast-in-Place Segments

Podolny and Muller, two designers of prestressed concrete segmental bridges, write,

> In cast-in-place construction, segments are cast one after another in their final location in the structure. Special equipment is used for this purpose, such as travelers (for cantilever construction) or formwork units moved along a supporting gantry (for span-by-span construction). Each segment is reinforced with conventional un-tensioned steel and sometimes by transverse or vertical prestressing or both, while the assembly of segments is achieved by longitudinal post-tensioning.
>
> Because the segments are cast end-to-end, it is not difficult to place longitudinal reinforcing steel across the joints between segments if the design calls for continuous reinforcement. Joints may be treated as required for safe transfer of all bending and shear stresses and for water tightness in aggressive climates. Connection between individual lengths of longitudinal post-tensioning ducts may be made easily at each joint and for each tendon.

Fig. 56: Match-cast box girder segments in precasting yard for the construction of the Choisy-le-Roi Bridge, France. (Podolny and Muller)

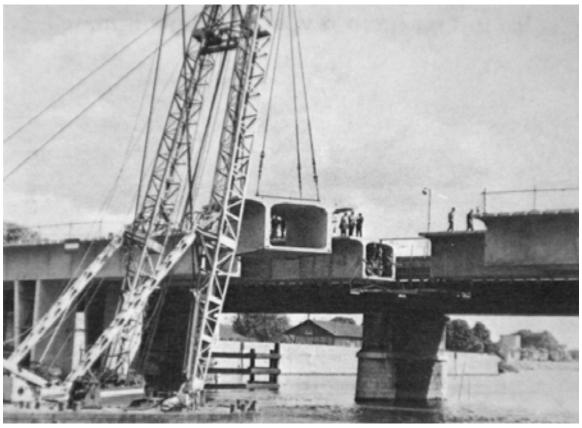

Fig. 57: Segment of the Choisy-le-Roi bridge being placed with a floating crane. (Podolny and Muller)

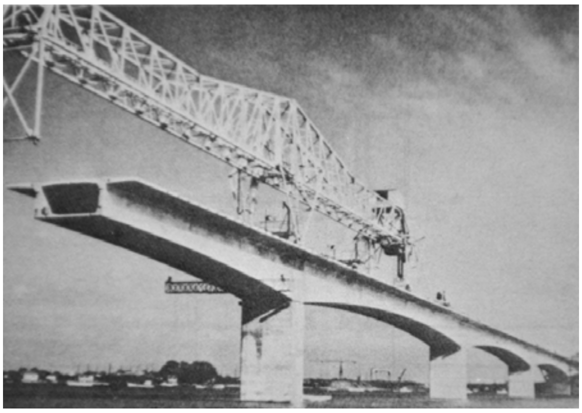

Fig. 58: Oleron Viaduct, construction view showing cantilever span. (Podolny and Muller, after the American Concrete Institute)

[112] Podolny and Muller, p17-18.

Fig. 59: Long Key Bridge, Florida, USA, construction view. 3.7 km total length with 36 m spans, Muller and Figg, 1976. (AFPC et al.)

The method's essential limitation is that the strength of the concrete is always on the critical path of construction and it also influences greatly the structure's deformability, particularly during construction. Deflections of a typical cast-in-place cantilever are often two or three times those of the same cantilever made of precast segments.

The local effects of concentrated forces behind the anchors of prestress tendons in a young concrete (two or four days old) are always a potential source of concern and difficulties.[113]

Characteristics of Precast Segments

Podolny and Muller continue,

> In precast segmental construction, segments are manufactured in a plant or near the job site, then transported to their final position for assembly. Initially, joints between segments were of conventional type: either concrete poured wet joints or dry mortar packed joints. Modern segmental construction calls for the match-casting technique, as used for the Choisy-le-Roi Bridge and further developed and refined, whereby the segments are precast against each other, preferably in the same relative order they will have in the final structure. No adjustment is therefore necessary between segments before assembly. (**Figs. 56 and 57**) The joints are either left dry (in areas where climate permits) or made of a very thin film of epoxy resin or mineral complex, which does not alter the match-casting properties. There is no need for any waiting period for joint cure, and final assembly of segments by prestressing may proceed as fast as practicable.
>
> Because the joints are of negligible thickness, there is usually no mechanical connection between the individual lengths of tendon ducts at the joint. Usually no attempt is made to obtain continuity of the longitudinal conventional steel through the joints, although several methods are available and have been applied successfully (as in the Paco Kennewick cable-stayed bridge, for example). Segments may be precast long enough in advance of their assembly in the structure to reach sufficient strength and maturity and to minimize both the deflections during construction and the effects of concrete shrinkage4 and creep in the final structure. If erection of precast segments is to proceed smoothly, a high degree of geometry control is required during match casting to ensure accuracy.[114]

[113] Podolny and Muller, p17.
[114] Podolny and Muller, p17.

Choice Between Cast-in-Place and Precast Construction

Cast-in-place methods and precast methods essentially produce the same final structure and both methods have been successfully used. The following list, compiled by Podolny and Muller, examines some of the criteria that might determine which method is used. Each method clearly has its advantages and faults.

1. *Speed of Construction* Basically, cast-in-place cantilever construction proceeds at the rate of one pair of segments 10 to 20 ft (3 to 6 m) long every four to seven days. On the average, one pair of travelers permits the completion of 150 ft (46 m) of bridge deck per month, excluding the transfer from pier to pier and fabrication of the pier table. On the other hand, precast segmental construction allows a considerably faster erection schedule.

- For the Oleron Viaduct, the average speed of completion of the deck was 750 ft (228 m) per month for more than a year. (**Fig. 58**)

- For both the B-3 Viaducts in Paris and the Long Key Bridge in Florida, a typical 100 to 150 ft (30 to 45 m) span was erected in two working days, representing a construction of 1300 ft (400 m) of finished bridge per month. (**Fig. 59**)

- Saint Cloud Bridge near Paris, despite the exceptional difficulty of its geometry and design scheme, was constructed in exactly one year, its total area amounting to 250,000 sq ft (23,600 sq m). (**Fig. 60**)

- It is evident, then, that cast-in-place cantilever construction is basically a slow process, while precast segmental with matching joints is among the fastest.

Fig. 60: Saint Cloud Bridge, on Paris Ring Road over the Seine River, France, 1975. General view showing difficult geometry and the use of three-dimensional pre-stressing to counter torsion. (Podolny and Muller)

Fig. 61: Chillon Viaduct, Montreux, Switzerland. Construction view. 1970. (Wittfoht)

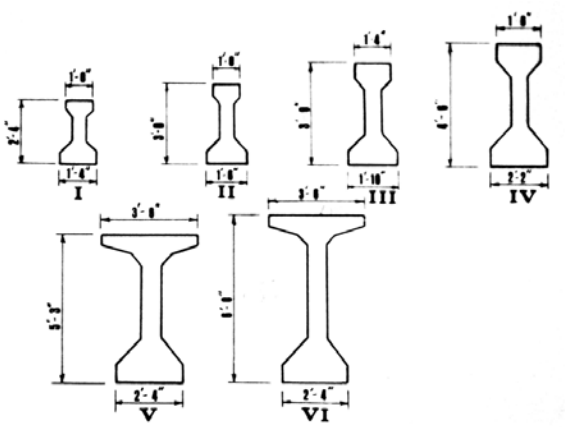

Fig. 62: Standard precast prestressed **I**-section, AASHTO (Podolny and Muller)

2. *Investment in Special Equipment* Here the situation is usually reversed. Cast-in-place requires usually a lower investment, which makes it competitive on short structures with long spans (for example, a typical three-span structure with a center span in excess of approximately 350 ft (100 m)).

In long, repetitive structures precast segmental may be more economical than cast-in-place. For the Chillon Viaducts with twin structures 7000 ft (2134 m) long in a difficult environment, a detailed comparative estimate showed the cast-in-place method to be 10% more expensive than the pre-cast. (**Fig. 61**)

3. *Size and Weight of Segments* Precast segmental is limited by the capacity of transportation and placing equipment. Segments exceeding 250 tons are seldom economical. Cast-in-place construction does not have the same limitation, although the weight and cost of the travelers are directly proportional to the weight of the heaviest segment.

4. *Environment Restrictions* Both precast and cast-in-place segmental permit all work to be performed from the top. Precast, however, adjusts more easily to restrictions such as allowing work to proceed over traffic or allowing access of workmen and materials to the various piers.[115]

6.12 The Precast I-Section and the Concrete Box Girder

6.12.1 The I-Section

The precast prestressed concrete **I**-girder, as introduced by Eugène Freyssinet at Waddi Foda between 1937 and 1939, is still standard today. (**Fig. 62**) The form of the concrete **I**-section is attributable to somewhat different reasons than the steel **I**- or wide-flanged beam it resembles. Steel's equal resistance to both compression and tension determine the symmetrical section of steel **I** -sections. The constant depth of a steel structural section is primarily a function of the economic rolling process used to fabricate it. In the prestressed concrete **I**-girder, the section of the top flange primarily resists compressive loads, though it must often resist low tensile stresses when initially prestressed and, if the beam is continuous, adequate steel reinforcement must be included to resist the negative moment-induced tensile forces over the supports.

The tapered sides of the flanges and the reduced section of the web are both related to the use of reusable forms in the fabrication of the girders. Pre-casting operations are highly industrialized, so reusing the forms offsets the additional cost of more complicated formwork. Additionally, optimizing the section reduces material costs and, importantly, reduces dead load. By minimizing the dead load, it is possible to either construct larger girders or make transportation and placement easier. The tapered flanges make it easier to release formwork from the newly cast girder and creates a smoother transition of forces between flange and web, which reduces stress concentrations at the corners where cracking can occur, something steel can better resist because of its higher toughness and ductility.

The bottom flange is typically larger than the upper flange. This is because of both constructive and structural reasons. Constructively, the section is sized to accommodate the

[115] Podolny and Muller, p17-18.

prestressing tendons and, if post-tensioned, the ducts, in addition to normal reinforcement and cover. Structurally, prestressing initially compresses the bottom flange to a high stress. The girder is designed such that the bottom flange will either have zero stress or very low tensile stresses under normal live load conditions. If tensile stress is allowed, it is minimized to prevent cracking or limit crack widths such that water cannot penetrate to the steel.

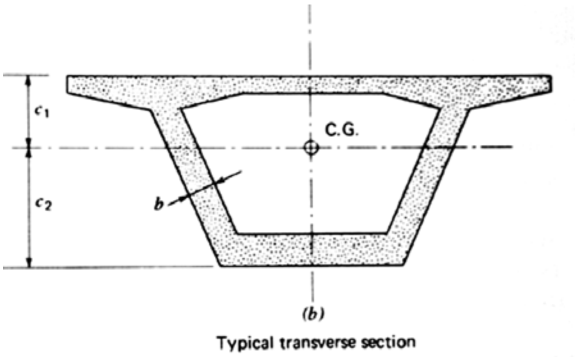

Fig. 63: General section of box girder. (Podolny and Muller)

6.12.2 The Box Section

Robert Stephenson introduced the structural box section in the construction of the Britannia Bridge, which was constructed of wrought iron.[116] Robert Maillart introduced the first concrete box section with the construction of the reinforced concrete arch bridge at Zuoz in 1901. (**Appendix A-05, p A.259, Fig. 38**) Eugène Freyssinet's Marne River Bridges were made into cellular box sections by connecting the individual precast girders to one another with transversal prestressing. (**Fig. 27(b)**)

Box girders are the most efficient and economical design for a bridge in the majority of cases.[117] (**Fig. 63**) Box girders are particularly suited to complex traffic requirements and curved road geometries because of the torsional stiffness of the closed section. The box cross-section's large elastic section modulus makes it a suitable form for span-by-span cantilever bridge construction. It is superior to the pi-section because a smaller range of stresses is induced in its cross-section by the constant variations in loading that occur during construction. This allows deflections to be better controlled.[118]

One limitation in the use of the box section is that a minimum section depth is necessary to provide adequate clearance between the top and bottom flanges during construction and for maintenance purposes. A minimum internal clearance is about 1.60 m, which means that the minimum total section depth is approximately 2.40 m.[119] With a slenderness of L/20[120], the minimum span must be 45 - 50 m, which is conveniently the maximum practical limit of precast girders.

Box girder decks were initially made integral with the piers when the bridge was constructed using the balanced cantilever method. Special expansion joints were provided at the center of each, or every other, span to allow for volume changes and to control differential deflections between individual cantilever arms. Now decks are made continuous over several spans with pier bearings designed to allow for expansion.[121]

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116 **Appendix A-03, p A.75, Fig. 1**.
117 Podolny and Muller, p12.
118 Wittfoht, p215.
119 Vogel[1].
120 L = span length
121 Podolny and Muller, p12-13.

Fig. 64: Villeneuve Saint-Georges Bridge, France, Lossier. (AFPC et al., after J. Châtelain)

The top flange of the box section is generally an integral part of the deck. Oftentimes, the deck is cantilevered on either side of the box section. The variable thickness of the cantilevers is structurally and materially efficient. The webs of the box section can be thickened from the inside so that they can be varied as structurally necessary and still retain clean lines on the exterior for aesthetic purposes. The number of webs depends upon the width of the bridge and the depth of the superstructure.[122] The principle prestressing tendons can either be located in the webs or externally. I will review both cases in more detail below. Internal tendons typically mean that the webs must be thicker than if external tendons are employed. Thinner webs can be constructed using external tendons but any savings in dead load is partially offset by increased weight due to the anchorage points that must be built inside the box section for the external tendons. The bottom flanges of box girders can be minimized since the prestressing tendons are in the webs of the beam.

6.13 External versus Internal Prestressing

6.13.1 Early Use of External Prestressing

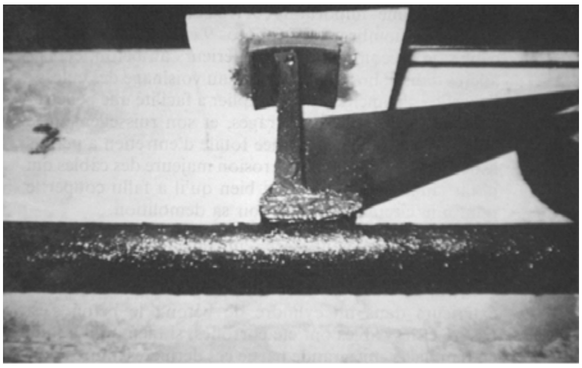
The first prestressed concrete bridge, Franz Dischinger's Aue Bridge built in 1936, was externally prestressed. With few exceptions, all subsequent developments occurred with the tendons embedded in the concrete until the 1970s because of the success and international circulation of Eugène Freyssinet's ideas. Several early constructions where external prestressing was used are useful to mention to illustrate the problems encountered with external prestressing.

The first externally prestressed bridge to be built after the Aue Bridge was the Sclayn Bridge in 1950. The Sclayn Bridge was built in Belgium under the supervision of Gustave Magnel. A whole series of overpasses over highways in Belgium were subsequently built between 1960 and 1970.[123] Others built some bridges in France and Britain as well.

These first trials of external prestressing did not perform well because severe corrosion problems occurred in several of these bridges. The fumes of steam locomotives corroded the cables of the Aue Bridge but maintenance prevented their further degradation. However, the mild-strength steel bars used were incapable of being stressed to a high enough degree to function properly after relaxation had occurred. The bars had to be re-stressed twice, in 1962 and 1980.[124]

[122] Wittfoht, p190 and 215.
[123] SETRA, p3-4.
[124] AFPC, p6.

The Villeneuve-Sainte-Georges Bridge, designed by Henri Lossier and built in France, was very costly. (**Figs. 64**) Lossier designed the bridge such that the cables could be replaced but his system of concrete and steel balancers to assure proper deviation of the cables was prohibitively expensive, which dissuaded other engineers from pursuing development of this system.[125] (**Fig. 65**)



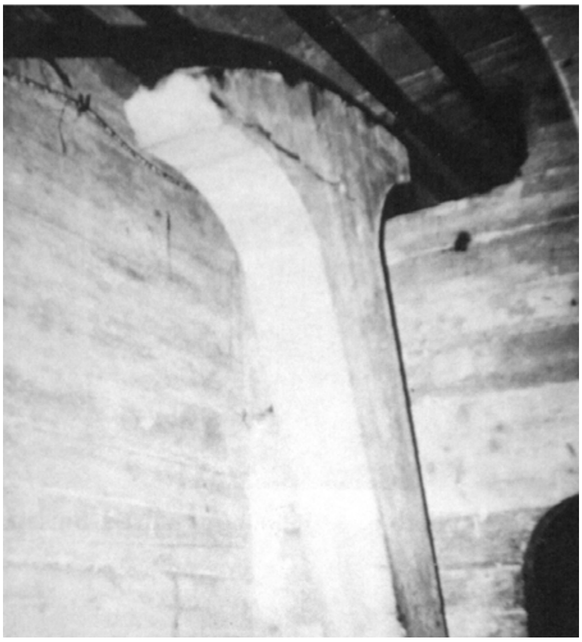


Fig. 65: Cables of the Saint-Georges Bridge were designed to be easily replaced. Lossier designed balanced cable deviators to assure the geometry of the cables. (a) Steel balanced deviator. (b) Reinforced-concrete balanced deviator. (AFPC et al., after J. Châtelain)

The cables of several other French bridges were attacked by corrosion. The design of the Can Bia Bridge was detailed poorly and water ran along the tendons, causing corrosion. Fifteen to twenty-five per cent of the cables ultimately ruptured because of negligent maintenance and the bridge had to be demolished.[126] (**Fig. 66**)

In Britain, the tendons on the Bournemouth Bridge had to be replaced soon after the bridge was completed. The large diameter mono-strand tendons used were placed under a fragile PVC sheath protected by a product that may possibly have encouraged the corrosion. A similar problem occurred with the overpasses built in Belgium. The tendons were protected by a cylinder of concrete installed *in situ* after the tendons were tensioned. The concrete cylinder cracked because of shrinkage, which in turn lead to corrosion of the steel.[127]

These experiences resulted in a poor opinion of external prestressing, but the main problems were of corrosion, not mechanical behavior, meaning that better design and technology could overcome the corrosion problem.[128]

6.13.2 The Return of External Prestressing

French engineers reintroduced external prestressing in the 1970s to strengthen existing prestressed concrete structures. Many bridges with internal prestressing were insufficiently prestressed at the time of their construction and required strengthening by adding tendons, necessarily external to the concrete.[129] The reintroduction of external prestressing was made

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[125] AFPC, p4.
[126] SETRA, p4.
[127] SETRA, p4.
[128] SETRA, p4.
[129] SETRA, p4-5.

Fig. 66: Corroded and ruptured cables of the Can Bia Bridge. (AFPC et al., after J. Châtelain)

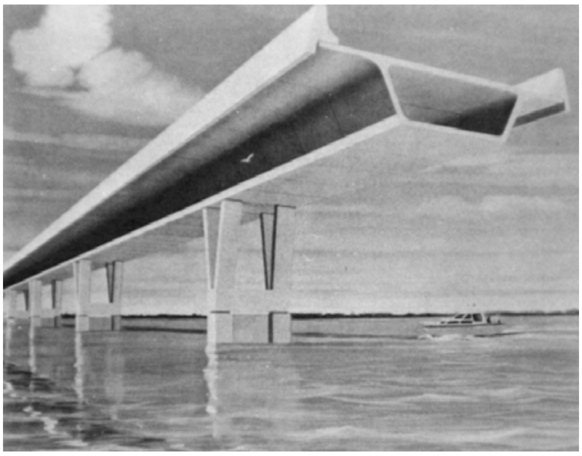

Fig. 67: General view of Long Key Bridge, Florida, USA. 3.7 km total length with 36 m spans. Muller and Figg, 1976. (Podolny and Muller)

possible by technological advances. The development of high strength steel cables permitted the number of external cables to be reduced, which facilitated the conception and construction of these bridges. The experience of reinforcing these prestressed concrete bridges between 1973 and 1978 led to the development of viable corrosion protection systems and permitted designers to develop an appreciation for the advantages of external prestressing, which led them to consider using it for new construction. The principle advantages of external prestressing are the considerable simplification of the geometry of the cables and the important reduction in prestressing losses due to friction. Engineers were further interested in the possibilities of replacing the cables sometime in the future, thus reinventing the forgotten ideas of Henry Lossier.[130]

6.13.3 External Prestressing Construction in the USA

External prestressing was developed largely in the United States from 1978-1979 by Jean Muller and Eugene Figg, and in France from 1980 to 1981, on the initiative of the Service d'etudes techniques des routes et autoroutes (SETRA).[131] The first externally prestressed bridges to be built in this new period are the Long Key and Seven Mile Bridges in Florida designed by Muller and Figg. (**Fig. 67**) The Long Key Bridge is 3.7 km (2.3 mile) in length with 36 m (118 ft) spans and the Seven Mile Bridge is 1.23 km (0.8 mile) in length with 41 m (134.5 ft) spans. For the Long Key Bridge, a span-by-span, precast segmental construction process was used. (**Fig. 59**) Segments for a whole span were laid on a traveling girder and then the cables were threaded through and jacked. A production rate of approximately two spans per week was achieved.[132]

In the USA, the complicated construction process of conventional prestressed concrete box girder bridges in combination with high labor and construction costs, and aggressive time schedules led to the adoption of precast segmental construction.[133] The principal objective in the use of external prestressing in the USA is to reduce the cost of bridge construction.

[130] AFPC, p7.
[131] SETRA, p5.
[132] ATPC, p120.
[133] Vogel[1].

External prestressing was developed to reduce the dead load of the structure, principally by reducing the thickness of the webs. The technology chosen was the most simple and economic possible and does not permit easy replacement of the cables. These economies were necessary to make the design of precast, segmental box-girders competitive against the precast girder system standardized by AASHTO.[134] Asia and elsewhere followed the US example.[135]

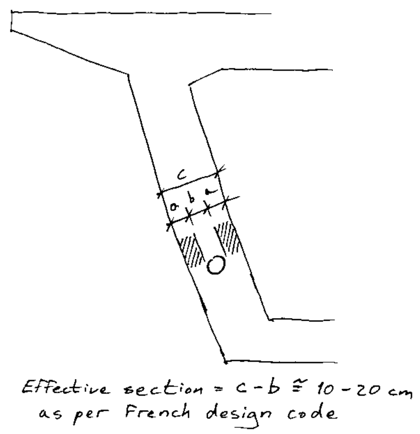

Fig. 68: Sketch showing French design code that excludes the thickness of the web that includes the prestressing duct from the calculation of the effective section. (Dooley, after Vogel[1])

6.13.4 External Prestressing Construction in France

External prestressing was developed in France by SETRA in collaboration with private enterprises.[136] One motivation for adopting external prestressing in France was the French design codes for internally prestressed box girders. French codes required that the effective section of the web could not include the plane where the prestressing ducts are, as indicated in **Figure 68**. Therefore the web was 10 to 20 cm thicker than if that section were included in the calculation, making the web unduly heavy.[137]

The technological objectives in France differed substantially from those in the USA. While the focus in the USA was above all on economy, particularly construction costs, in France the focus was on improving the quality of bridge construction. The simplified geometry of external prestressing was valued for limiting unwanted deviations in the cable that can cause friction losses. The grouting procedure is facilitated using external prestressing, which guarantees a more reliable protection against corrosion. Separating the cables from the webs also reduces the possibility that corrosion will be caused by the infiltration of water and corrosive deicing chemicals in cracks in the web.[138]

Corrosion can generally be better controlled using external prestressing because the exposed or sheathed cables are easier to access and examine than internal tendons. There is no reliable, non-destructive test for corrosion of internal prestressing cables.[139] Finally, external prestressing permits the future replacement of the cable if necessary, but only if the chosen technology used permits such an eventuality. Such a technology costs more than the simpler system adopted in the United States. The difference in cost is 2% to 3% the total cost of the bridge, which is not negligible. The French justify this additional expenditure even if the probability of changing the cable is extremely unlikely because of the quality of the

134 AFPC, p7-8.
135 Vogel[1].
136 AFPC, p8.
137 Vogel[1].
138 AFPC, p8.
139 Vogel[1].

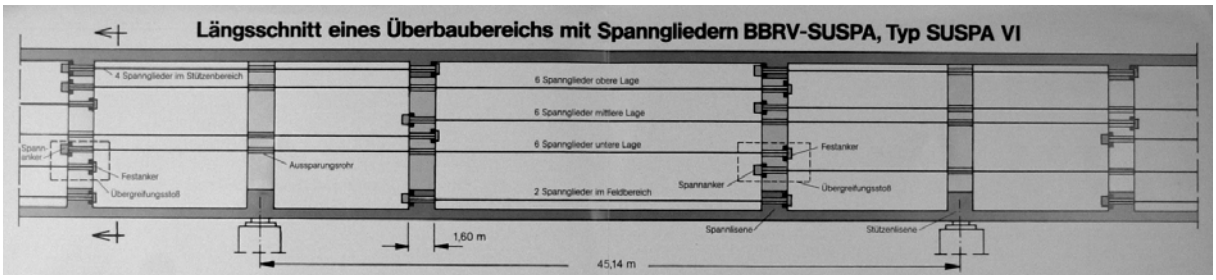

Fig. 69(a): Berbke Bridge, first modern externally prestressed bridge built in Germany, longitudinal section showing arrangement of straight cables. (SUSPA)

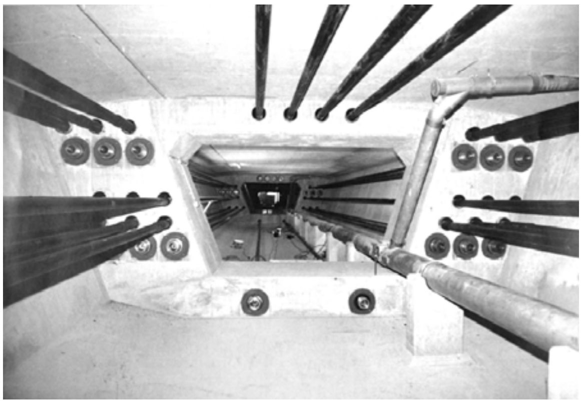

Fig. 69(b): Berbke Bridge, internal view. (SUSPA)

Fig. 70: Pi section, open from below with external prestressing. Saint-Agnant Viaduct. (SETRA, after Freyssinet)

corrosion protection. They value having the *possibility* to replace the cables in the future for reasons we may not be able to foresee today.[140]

6.13.5 External Prestressing Construction in Switzerland and Germany

External prestressing was slow to be adopted in Switzerland and Germany where it only became part of common practice at the end of the 1980s. In Germany, design codes impeded the use of external prestressing. The first modern externally prestressed bridge in Germany was the Berbke Bridge on the A 46 between Arnsberg and Westfalen, built between 1988 and 1989. Design codes made it impossible to deviate the tendons. As a result the engineers had to model the deviation using multiple straight line tendons vertically offset from one another.[141] (**Fig. 69(a) and 69(b)**)

External prestressing was forced upon Germany and Switzerland when it was found that many post-tensioned bridges, where the tendons were always in the webs, had severe corrosion problems because of improper grouting. The situation was compounded by the lack of non-destructive testing procedures for corrosion of conventional prestressing inside the concrete.[142]

German codes now favor external prestressing, even though it costs more than

140 AFPC, p8.
141 SUSPA and Vogel[2].
142 Vogel[1].

internal prestressing in that country. This increased cost is accepted for the security of being able to easily inspect the cables for corrosion. German bidding procedures allow internal prestressing to be proposed but a 5% to 10% penalty is imposed on the bid price.[143]

6.13.6 Types of Externally Prestressed Structures

External prestressing is commonly used with closed section box girders where the cables have further protection from corrosive elements. (**Fig. 63**) External prestressing can also be used on structures that are open from below having a Pi section, closed structures of triangular section with solid webs, and closed box-type structures with lattice webs.[144] (**Figs. 70, 71, and 72**)

6.13.7 Tendon Deviation Geometries

There are three principal geometries in which external prestressing cables can arranged. Staggered deviators can be used, such as those used in the Long Key Bridge. (**Fig. 73**) The disadvantage of this system is that the cables are spaced progressively further from the web, which introduces eccentricities into the structure.[145] Using a vertical arrangement would reduce the effective moment arm of the cables, which is already reduced by external prestressing because the cables cannot pass through the flanges of the box section. (**Fig. 74**)

Multiple deviators can be used to achieve a semi-parabolic layout that models the parabolic geometry possible when using internal prestressing. (**Fig. 75**) Such a layout requires more deviators, reducing the weight savings advantage of removing the cables from the web, and introduces more friction losses than a more simple arrangement.

Fig. 71: Triangular section with closed webs of corrugated steel. Vallon de Maupré Viaduct at Charolles. (a) General view. (b) Inside triangular box girder. (SETRA)

Fig. 72: Box section with lattice-type webs and external prestressing. Glacières Viaduct, France. (Marrey)

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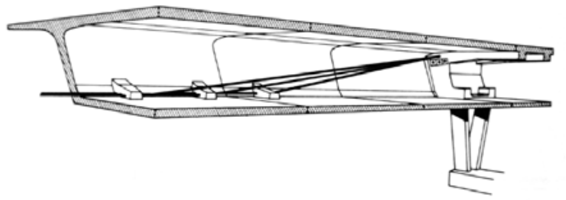
[143] Vogel[1].
[144] SETRA, p36-38.
[145] ATPC, p11.



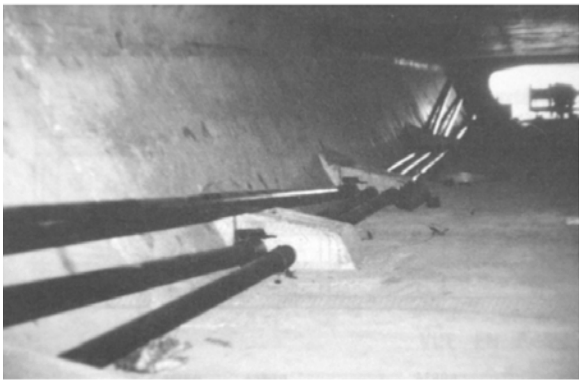


Fig. 73: Design of staggered deviators of Long Key Bridge. (AFPC)

Fig. 74: Viaduct of Sermeaz, showing vertical arrangement of cables similar to geometry of Long Key Bridge without the same extreme eccentricities. (SETRA, after M. Virlogeux)

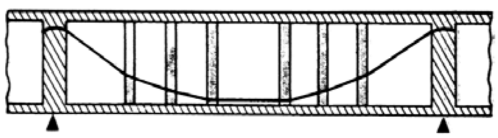


Fig. 75: Multiple deviators with semi-parabolic geometry. (SETRA)

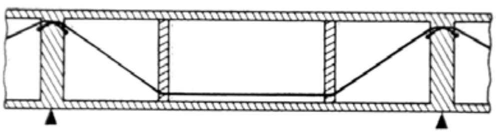


Fig. 76: Simplified polygonal geometry.

The simplified polygonal layout is probably the most common arrangement of external prestressing. (**Fig. 76**) Like any of the preceding geometries, the difference in performance between the parabolic geometry of an internal cable with a uniformly distributed prestress force and the simplified polygonal geometry of an external cable with concentrated deviation forces is negligible enough that it will not be a controlling factor in a decision between which system to use.[146]

6.13.8 Advantages of External Prestressing

Corrosion is always a risk in reinforced concrete bridges where deicing agents are used or in corrosive environments such as near the ocean. While both internal and external prestressing tendons are susceptible to corrosion, external prestressing makes it easy to check for corrosion and it can be easily replaced if designed with that intent. No reliable, non-destructive method exists to test internal prestressing cables for corrosion. Furthermore, the risk of corrosion is disassociated completely from the risk of cracking of the adjacent concrete.

External prestressing simplifies construction by improving the conditions for concreting and installing the prestressing tendons. The anchorage detail is particularly simplified. (**Fig. 77**) Grouting external tendons is more reliable because the geometry is simplified and multiple injection ports can be used. The quality of the grouting can be non-destructively controlled when construction is complete.

External tendons reduce the weight of the structure and improve their strength by reducing the material in the webs and limiting the number of weak points of the section caused by the ducts housing the tendons.

Finally, the possibility of over-tensioning external tendons is reduced since the cable is

[146] Vogel[1].

straight between deviations, which minimize the angular deviations that cause friction that can block movement in internally placed prestressing tendons.[147]

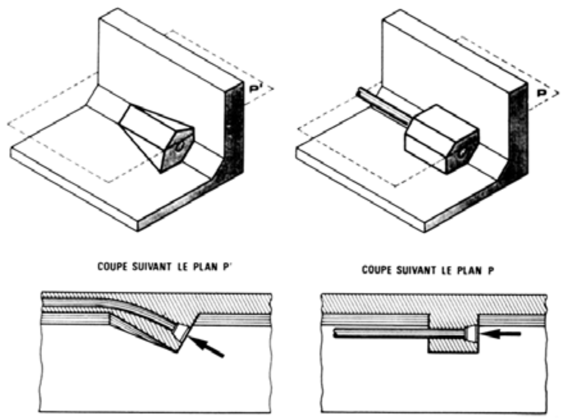


Fig. 77: Comparison of anchorage details for internal and external prestressing. (AFPC)

6.13.9 Disadvantages of External Prestressing

External prestressing is more expensive because the depth of the lever arm is reduced, which means a higher relative prestress force is needed. (**Fig. 78**) Also, more anchorages are needed with external prestressing.[148] Additionally, the full strength of the steel cannot be exploited because there is no bond between the cable and the concrete. According to the Swiss concrete building code SIA 162, the full yield stress of the steel can be used when calculating for internal prestressing, but only approximately 70% of the yield strength can be used for external prestressing.[149]

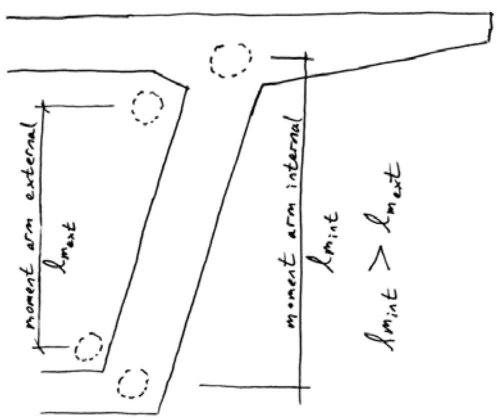


Fig. 78: Illustration of comparative moment arm between internal and external prestressing. There is clearly a mechanical advantage to using internal prestressing. (Dooley, after Vogel[1])

The geometry of external prestressing cannot be adjusted as well as internal prestressing to counter the moment. However, the approximation is sufficiently close not to make this issue a controlling factor.

External prestressing is more vulnerable to damage than internal prestressing. External tendons are particularly sensitive to fire and corrosive agents of animal origin such as the droppings of birds and rodents. Human habitation of bridges, of which evidence has been found, poses a clear danger particularly with respect to the lighting of fires. The location of utilities inside the bridge section also poses a risk if the structure manager does not properly train maintenance technicians.[150]

Constructively, external prestressing has its own particular difficulties, such as being able to adjust the tubes that pass through the concrete. These must be adjusted to the geometry of the cable. Also, tensioning jacks are difficult to handle in the confines of the box section. To reduce the number of tendons in externally prestressed bridges, tendons with very high strength are used, such as 19 T 15 tendons and even stronger tendons for large spans. These cables require heavy, cumbersome jacks to tension them and adequate provision

[147] SETRA, p5-6; ATPC, p9.
[148] Vogel[1].
[149] SIA 162, code 3 24 18.
[150] SETRA, p7.

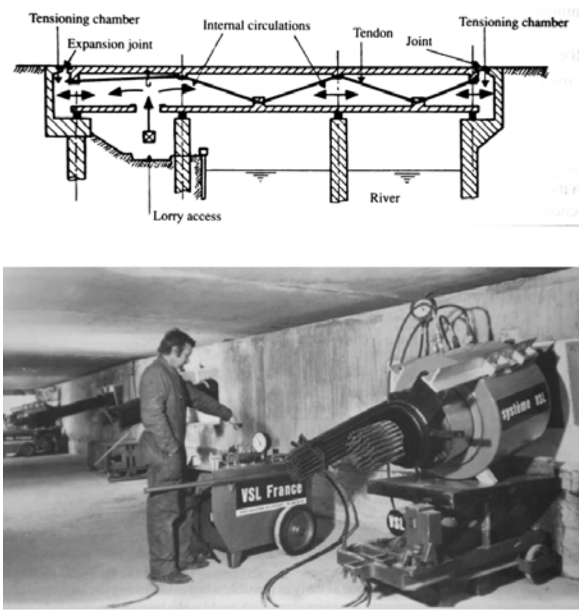


Fig. 79: (a) Diagram showing access hole provided for tensioning jack. (b) View of jacking procedure. (SETRA)

must be made for easy installation and removal of the jacks. Sometimes access holes must be made in the bottom flange of the bridge to do so.[151] (**Figs. 79**)

6.13.10 Economic Considerations

It is not easy to establish an economic comparison between a conventional prestressing solution and one with external tendons. Several contradictory factors exist between the relationships of material quantities and unit prices, such as the fact that weight savings in a web of reduced section using external tendons is offset by the weight of deviators and anchoring bulkhead beams.[152]

For the same effective tension, an external tendon is less efficient than an internal tendon because of its simplified polygonal geometry, the reduced depth of moment arm, and the fact that only 70% of its yield strength can be used in design. Conversely, the external tendon always has more useful strength than an internal tendon of comparable layout because of less friction caused by having less angular deviation points.[153]

Lastly, for the same amount of steel, external prestressing costs more than conventional prestressing because of the specific materials used for items such as the ducts and the arrangement of the anchorages, which have particular requirements depending on the system used. The problems posed by adjusting the tubes passing through the concrete and handling the tension jacks also increases costs.[154]

In terms of initial cost, external prestressing is advantageous only if it considerably lightens the structure and if its construction can be industrialized. When jobsite industrialization is too limited – the construction of the deviation crossbeams poses a serious obstacle in this respect, the immediate economic advantage is practically negligible. Nevertheless, external prestressing gives considerable long term value by increasing constructive quality and durability of the structure, and facilitating monitoring, which reduces operating costs.[155]

[151] SETRA, p7.
[152] SETRA, p9.
[153] SETRA, p9.
[154] SETRA, p9.
[155] SETRA, p9.

6.14 Conclusions

6.14.1 Prestressed Concrete, Origins of a Modern Material

Unlike the historically traditional materials – stone, timber and iron, prestressed concrete had to be invented, not found. Prestressed concrete cannot be described as the logical product of simple advances in normal reinforced concrete because neither Freyssinet nor Dischinger first applied it as such. They both used stressed tendons in an auxiliary capacity.

Freyssinet first used prestressing to tie other structural elements together. In the Elorn Bridge prestressed tendons resisted the horizontal thrust of the self-supporting arched formwork. In the Gare Transatlantique, prestressing was used to tie the tops of the foundation piles together in order to halt their displacement that was causing unacceptable settling of the structure. While the tendons in Gare Transatlantique were encased in a beam, their primary purpose was again to tie together, not create a new type of beam or even keep the concrete from cracking. Later, in the Bridge at Le Verdure, Freyssinet encountered another settling problem that led to his research into the shrinkage and creep behavior of concrete. He fixed this bridge's displacement problem by reinstalling the jacks he used in the crown of the arch to 'push' the arch off its centering. These early experiences led Freyssinet to identify the advantage of prestressing concrete to avoid tension and overcome problems of shrinkage and creep, but the kernel of the idea emerged from problems distinctly removed from is later application to beam bridges and cantilevered utility poles.

Similarly, Dischinger first used prestressing in a reinforced concrete arch in order to ensure the thin arch remained a true arch free of tension induced by moment stresses. This early experiment revealed the problem with prestressing losses due to shrinkage and creep, which led to Dischinger's own exploration of this behavior in concrete and finally to his own identification of the benefits prestressing could bring to the design of reinforced concrete beams.

These developments clearly show how a new material can simply appear out of efforts to overcome problems with existing technologies. In Freyssinet's case, he used well-established principles of ties to keep two different structures from displacing horizontally. When he identified the new structural problem of creep (shrinkage was already known) he made the creative link that led to prestressed concrete. Dischinger was only trying to find ways to minimize tension in the arch due to moment stress by using cables to keep the arch in constant compression. In this respect, Dischinger's initial experiment is much closer in principle to the later invention of the prestressed concrete beam than was Freyssinet's tied structures. Interestingly, it was Freyssinet who went on to have more influence on the development of prestressing as a technology.

6.14.2 Bridges and Prestressed Concrete

Prestressing makes concrete competitive against steel for large span bridges. Before the introduction of prestressing, reinforced concrete was only used for arch structures and relatively short beams. The disadvantages of arch construction are the relatively limited number of sites where an arch is an appropriate solution and the high abutment costs for arches incurred by the need to counter high horizontal thrusts. In beam construction, normal steel reinforcement works at relatively low stresses, which quickly become inefficient because of the high dead load associated with concrete structures. Prestressing takes optimal advantage of concrete's high compressive strength and the tensile strength of high-strength steels by using the steel to keep the whole concrete section in compression, or nearly so. Prestressing the bottom flange of the beam minimizes tensile stresses in the structure under normal service loads, which limits cracking, thereby decreasing the chance of corrosion and increasing the durability of the structure.

6.14.3 The Influence of Construction Process on Form

In this chapter I have recorded a development in construction process and technology that is associated with this material. The benefits of this development however are not necessarily in the refinement of the structural form of prestressed concrete. There are three primary forms used for the construction of prestressed concrete bridges – the **I**-section, the pi-section, and the box section. These forms are largely determined by statics. The **I**-section and box section are both products of the development of iron and steel structures. The pi-section is perhaps an original form characteristic of this material, though I have not thoroughly researched the question of when the first composite steel plate-girder and concrete deck was first utilized.[156] From the information I have researched, I conclude that the structural forms are primarily the result of statics, not construction process.

The most significant structural advance that can be directly linked to the development of different construction processes is in the area of material optimization. This advance is intrinsically linked to the use of external prestressing and segmental construction. External prestressing reduces concrete volume in the webs, though some of this offset by additional cable connection and deviation points. Segmental construction in combination with external prestressing means that less passive-steel reinforcement has to be used since it is not necessary to have continuity between segments with the passive reinforcement; it is sufficient to use the friction of the prestressed elements, in combination with shear keys, to keep the structure together.

6.14.4 The Influence of Government Regulation

The development of concrete in general presents a good case study to examine the role of government and other regulating organizations in the development of materials. In both the last chapter and this chapter we saw that the role of patents can both limit development and drive experimentation motivated by the desire to avoid patent infringements that may either be not allowed at all or be subject to licensing fees. In this chapter, we saw that several designers sought to avoid Freyssinet's near monopoly on internal prestressing technology by trying to use external prestressing. These early experiments in external prestressing revealed a number of problems with the technological capacity to ensure good corrosion protection. The result was a generally unfavorable view of this mode of prestressing, a stigma that was only overcome when corrosion of internal prestressing was identified as a significant problem in the late 1960s. The rekindled interest in external prestressing came down to the fact that engineers now had to deal with the fact that corrosion was a problem no

[156] Thomas Telford used cast-iron plates to construct the decks of some of his cast-iron arch bridges but I do not know if he intended these to work compositely with the arch.

matter which type of prestressing was used, but at least external prestressing could be easily inspected and possibly replaced if necessary.

The history of prestressed concrete centers around France and Germany where engineers pioneered major advancements in the technology. Interestingly, both countries have widely different approaches to regulating the design of such structures. In France, innovation is encouraged and engineers are given wide latitude to try different methods. Conversely, Germany's regulations are very conservative and there is great pressure to follow the prescribed codes without deviation. The fact that so many structural innovations come out of Germany would make for an interesting study, which would be far too involved to go into here. Suffice it is to say, that this area demands further inquiry. I probe further into the dynamics between government and regulatory influences on structural innovation in the main body of the thesis.

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A TYPOLOGY OF STRUCTURAL FORM
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7.1 Typology of Structural Form

The following pages are copied from Heino Engel's book *Structure Systems.* These pages are referenced in **Chapters 03** and **04**. For a more complete explanation of the system, see **Chapter 03, p34-36**.

Please excuse the legibility. I recommend consulting Engel's book directly.

7.2 Bibliography

Engel, Heino. Tragsysteme - Structure Systems (German - English, revised edition). Heino Engel and Verlag Gerd Hatje : Ostfildern-Ruit, Germany. 1997.

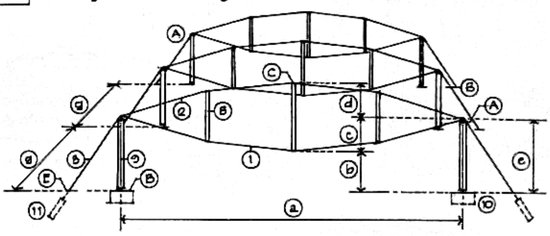
Formaktive
Tragsysteme

1

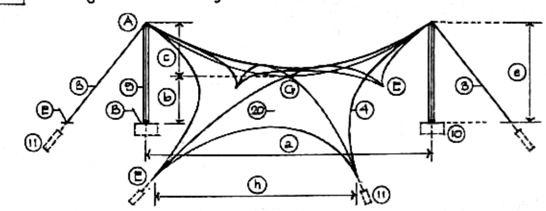
Definition	FORMAKTIVE TRAGSYSTEME sind Tragsysteme aus flexibler, nicht-steifer Materie, in denen die Kraftumlenkung durch geeignete FORMGEBUNG und durch charakteristische FORMSTABILISIERUNG erfolgt	FORM-ACTIVE STRUCTURE SYSTEMS are structure systems of flexible, non-rigid matter, in which the redirection of forces is effected through particular FORM DESIGN and characteristic FORM STABILIZATION
Kräfte / Forces	Die Systemglieder werden dabei primär nur durch gleichartige Normalkräfte belastet, d.h. entweder auf Druck oder auf Zug: SYSTEME IN EINFACHEM SPANNUNGSZUSTAND	Its basic components are primarily subjected to but one kind of normal stresses, i.e. either to compression or to tension: SYSTEMS IN SINGLE STRESS CONDITION
Merkmale / Features	Die typischen Struktur-Merkmale sind: KETTENLINIE (HÄNGELINIE) / STÜTZLINIE / KREIS	The typical structure features are: CATENARY / THRUST LINE (PRESSURE L.) / CIRCLE

Bestandteile und Bezeichnungen / Components and denominations

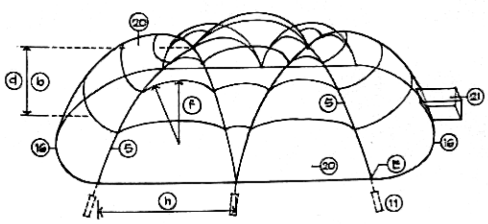
1.1 Seilsysteme / Cable systems



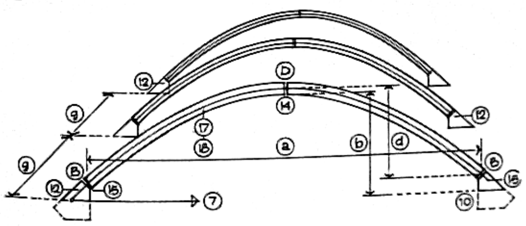
1.2 Zeltsysteme / Tent systems



1.3 Pneusysteme / Pneumatic systems



1.4 Bogensysteme / Arch systems



System-Glieder / System members

1	Tragseil, Lastseil	suspension cable, load cable
2	Stabilisierungsseil, Spannseil	stabilization cable, stress cable
3	Rückhalteseil, Abspannseil, Stag	retaining cable, stay, guy
4	Randseil	edge cable, boundary cable
5	Kehlseil	valley cable
6	Hängeseil	hanger
7	Zugband, Zuganker	tie rod, tieback
8	Druckstab, Spreizstab	compression rod (bar), spreader
9	Stütze, Pylon, Mast	column, pylon, mast, support
10	Fundament, Gründung	foundation, footing
11	Erdanker, Abspannanker	soil anchor, retaining anchor
12	Widerlager	abutment
13	Gelenk	pin joint, hinge
14	Scheitelgelenk	crown hinge, top hinge, key hi.
15	Fußgelenk, Kämpfergelenk	base hinge, impost hinge
16	Ankerring	anchor ring
17	Bogen, Stützbogen	arch, funicular arch
18	Gelenkbogen	pinned arch, hinged arch
19	Strebepfeiler	buttress
20	Tragmembrane	bearing membrane
21	Luftschleuse	air lock
22		
1-5	Funktionsseile	functional cables

Topografische Systempunkte / Topographical system points

A	Aufhängepunkt	suspension point
B	Fußpunkt, Basispunkt	base point
C	Hochpunkt	peak, high point
D	Scheitelpunkt	key, top, crown, vertex, apex
E	Ankerpunkt, Abspannpunkt	anchor point, retaining point
F	Auflagerpunkt	point of support, bearing point
G	Tiefpunkt	low point
H		
I		

Systemabmessungen / System dimensions

a	Stützweite, Spannweite	span
b	Lichte Höhe	clear height, clearance
c	Durchhang, Pfeilhöhe	cable sag
d	Stich (-höhe), Pfeilhöhe	arch (cable) rise
e	Stützenhöhe	column height, support height
f	Krümmungsradius	radius of curvature
g	Binderabstand	spacing, frame distance
h	Ankerpunkt-Abstand	distance of anchor points
i		
j		

Fig. 1: Form-Active Structures. (Engel)

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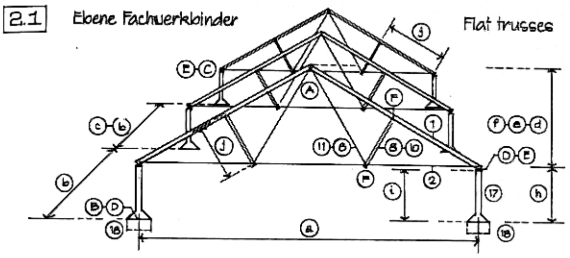
Definition / Übersicht / Spannweiten

Vektoraktive
Tragsysteme **2**

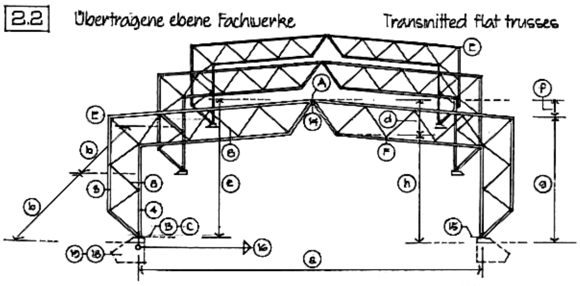
Definition	VEKTORAKTIVE TRAGSYSTEME sind Tragsysteme aus festen, geraden Linienelementen (Stäben), in denen die Kraftumlenkung durch VEKTOR-TEILUNG, d.h. durch MEHRGLIEDRIGES AUFSPALTEN DER KRÄFTE bewirkt wird	VECTOR-ACTIVE STRUCTURE SYSTEMS are structure systems of solid straight line elements (bars, rods), in which the redirection of forces is effected through VECTOR PARTITION, i.e. through MULTI-DIRECTIONAL SPLITTING OF FORCES
Kräfte / Forces	Die Systemglieder (Gurte, Stäbe) werden dabei zu einem Teil auf Druck, zum anderen Teil auf Zug belastet: SYSTEME IM KOOPERATIVEN DRUCK- UND ZUGZUSTAND	The system members (chords, web members) are subjected with one part to compression, with the other part to tension: SYSTEMS IN COACTIVE COMPRESSION AND TENSION
Merkmale / Features	Die typischen Struktur-Merkmale sind: DREIECK-VERBAND und KNOTEN-VERBINDUNG	The typical structure features are: TRIANGULATION and POINT CONNECTION

Bestandteile und Bezeichnungen / Components and denominations

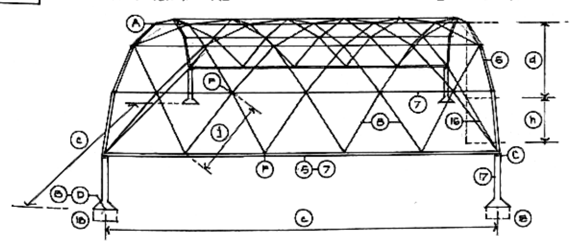
2.1 Ebene Fachwerkbinder — Flat trusses



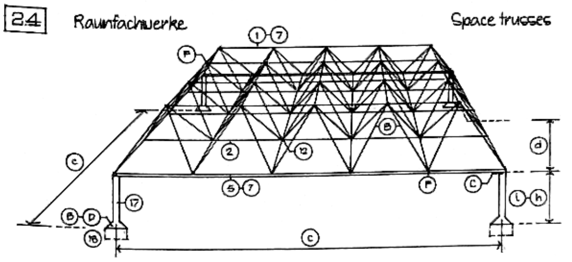
2.2 Übertragene ebene Fachwerke — Transmitted flat trusses



2.3 Gekrümmte Fachwerke — Curved trusses



2.4 Raumfachwerke — Space trusses



System-Glieder / System members

1	Obergurt	Top chord
2	Untergurt	Bottom chord
3	Außengurt	Outside chord
4	Innengurt	Inside chord
5	Randgurt	Edge chord, boundary chord
6	Quergurt	Cross chord
7	Längsgurt	Longitudinal chord
8	Gitterstab, Stab	Web member, bar, rod
9	Diagonalstab, Strebe	Diagonal (member), strut
10	Druckstab, Druckglied	Compression member (rod, bar)
11	Zugstab, Zugglied	Tension member (rod, bar)
12	Knoten, Knotenverbindung	Spot joint, point connection
13	Gelenk	Hinge, pin joint
14	Scheitelgelenk	Crown hinge, top hinge
15	Fußgelenk	Base hinge
16	Zuganker, Zuganker	Tie rod, tieback
17	Stütze	Column, support
18	Fundament, Gründung	Foundation, footing
19	Widerlager	Buttress
20	Auflager	Bearing, support
21	Einspann-Auflager	Fixed-end bearing
22		
23		

Topografische Systempunkte / Topographical system points

A	Scheitelpunkt	Peak, top, crown
B	Fußpunkt, Basispunkt	Base point
C	Auflagerpunkt	Point of support, bearing point
D	Einspannpunkt	Fixed-end point
E	Traufpunkt	Eaves point
F	Knotenpunkt	Connection point
G		
H		

Systemabmessungen / System dimensions

a	Stützweite, Spannweite	Span
b	Binderabstand	Spacing, frame distance
c	Stützenabstand	Column distance, c. spacing
d	Konstruktionshöhe	Depth (of construction)
e	Binderhöhe	Frame height
f	Neigungshöhe	Rise, pitch height
g	Traufhöhe	Eaves height
h	Lichte Höhe	Clear height, clearance
i	Stützenhöhe, Stützenlänge	Support height, column length
j	Stablänge	Rod length, bar length
k		
l		

Fig. 2: Vector-Active Structures. (Engel)

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Definition / Übersicht / Spannweiten

Schnittaktive Tragsysteme 3

Definition — SCHNITTAKTIVE TRAGSYSTEME sind Tragsysteme aus massiven, steifen Linienelementen - einschließlich deren Verdichtung als Platte -, in denen die Kraftumlenkung durch MOBILISIERUNG VON SCHNITTKRÄFTEN bewirkt wird

SECTION-ACTIVE STRUCTURE SYSTEMS are structure systems of solid, rigid linear elements - including their compacted form as slab -, in which the redirection of forces is effected through MOBILIZATION OF SECTIONAL FORCES

Kräfte / Forces — Die Systemglieder werden dabei primär auf Biegung beansprucht, d.h. durch innere Druck-, Zug- und Scherkräfte: SYSTEME IM BIEGEZUSTAND

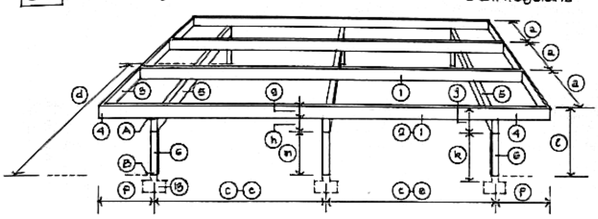
The system members are primarily subjected to bending, i.e. to inner compression, tension and shear: SYSTEMS IN BENDING

Merkmale / Features — Die typischen Strukturmerkmale sind: QUERSCHNITT-PROFIL und KONTINUITÄT DER MASSE

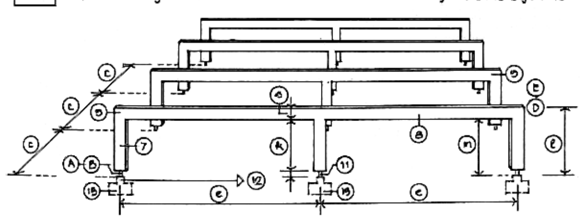
The typical structure features: SECTIONAL PROFILE and BULK CONTINUITY

Bestandteile und Bezeichnungen / Components and denominations

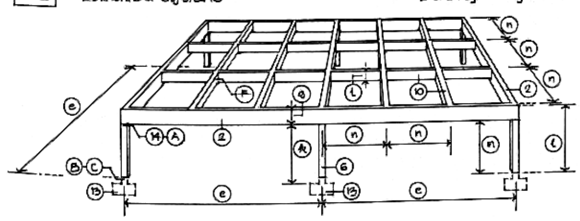
3.1 Balken-Systeme — Beam systems



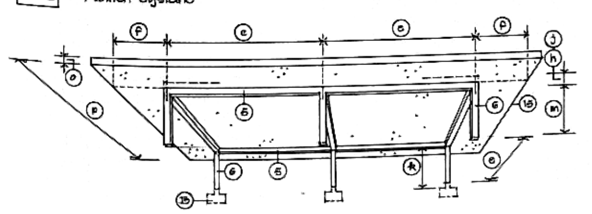
3.2 Rahmen-Systeme — Rigid frame systems



3.3 Balkenrost-Systeme — Beam grid systems



3.4 Platten-Systeme — Slab systems



System-Glieder / System members

1	Balken (Durchlaufbalken)	Beam, joist (continuous beam)
2	Randbalken	Edge beam, boundary beam
3	Stirnbalken, Brüstungsbalken	Spandrel beam
4	Kragarm	Cantilever
5	Unterzug	Girder
6	Stütze	Column, support
7	Rahmenstiel, Rahmenpfosten	Frame column, ~ leg, ~ post
8	Rahmenriegel	Frame girder
9	Rahmenecke, Rahmenwinkel	Frame knee, frame corner
10	Rostbalken	Grid beam, grill beam
11	Fußgelenk	Base hinge
12	Zuganker, Zugband	Tie rod, tieback
13	Fundament, Gründung	Foundation, footing
14	Auflager	Bearing, support
15	Plattenrand	Slab edge
16		
17		

Topografische Systempunkte / Topographical system points

A	Auflagerpunkt	Point of support, bearing point
B	Fußpunkt, Basispunkt	Base point
C	Einspannpunkt	Fixed-end point
D	Rahmen-Eckpunkt	Frame corner point
E	Traufpunkt	Eaves point
F	Kreuzungspunkt	Intersection point
G		
H		

Systemabmessungen / System dimensions

a	Balkenabstand	Beam (joist) spacing
b	Balken-Spannweite, Feldweite	Beam span, bay dimension
c	Binderabstand	Girder spacing
d	Binder-Spannweite, Unterzug-~	Girder span
e	Stützenabstand, Stützweite	Column distance
f	Kraglänge	Cantilever length
g	Balken-Konstruktionshöhe	Beam depth
h	Unterzug-Konstruktionshöhe	Girder depth
i	Rosthöhe	Beam grid depth
j	Gesamt-Konstruktionshöhe	Total depth of construction
k	Stützenhöhe, -länge, Stiel-~	Column height, ~ length
l	Traufhöhe	Eaves height
m	Lichte Höhe	Clear height, clearance
n	Rostabmessung, Rastermaß	Grid dimension, ~ measurement
o	Plattendicke	Slab depth
p	Plattenbreite (Plattenlänge)	Slab width (slab length)
q		

Fig. 3: Section-Active Structures. (Engel)

Flächenaktive Tragsysteme 4

Definition

FLÄCHENAKTIVE TRAGSYSTEME sind Tragsysteme aus biegeweichen, jedoch druck-, zug- und scherfesten Flächen, in denen die Kraftumlenkung durch FLÄCHENWIDERSTAND und durch geeignete FLÄCHENFORMGEBUNG bewirkt wird

SURFACE-ACTIVE STRUCTURE SYSTEMS are systems of flexible, but otherwise compression-, tension-, shear-resistant surfaces, in which the redirection of forces is effected by SURFACE RESISTANCE and particular SURFACE DESIGN

Kräfte / Forces

Die Systemglieder werden dabei primär durch Membrankräfte beansprucht, d.h. durch Kräfte, die parallel zur Fläche wirken: SYSTEME IM MEMBRANSPANNUNGSZUSTAND

The system members are primarily subjected to membrane stresses, i.e. to stresses acting parallel to the surface: SYSTEMS IN MEMBRANE STRESS CONDITION

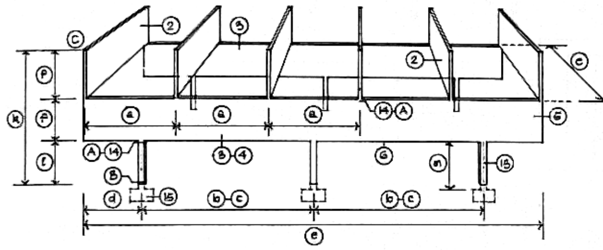
Merkmale / Features

Die typischen Strukturmerkmale sind: TRAGWERK als RAUMABSCHLUSS und FLÄCHENGESTALT

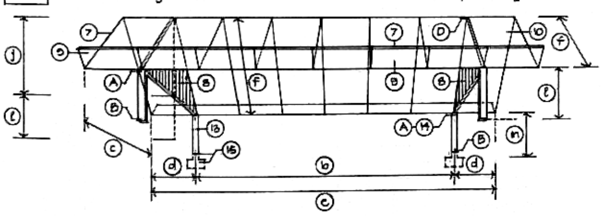
The typical structure features are: STRUCTURE as SPACE ENCLOSURE and SURFACE SHAPE

Bestandteile und Bezeichnungen / Components and denominations

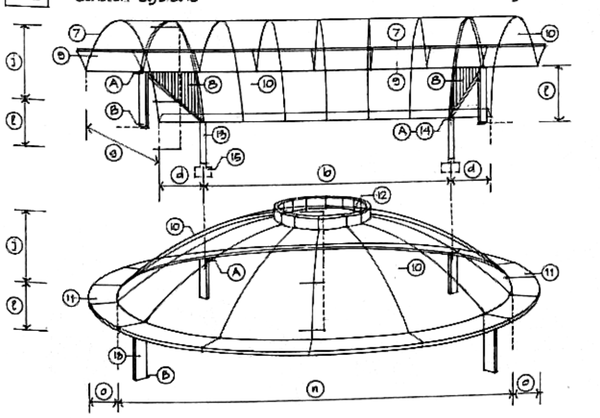
4.1 Scheiben-Systeme / Plate systems



4.2 Faltwerk-Systeme / Folded plate systems



4.3 Schalen-Systeme / Shell systems



System-Glieder / System members

1	(Trag-) Scheibe, Tafel	(struct.) Plate, panel, shear wall
2	Querscheibe, Schotte	Cross plate, transverse plate
3	Längsscheibe	Longitudinal plate
4	Scheibenunterzug, Sch-Träger	Plate girder
5	Kragarm	Cantilever
6	Scheibenrand	Plate edge, plate boundary
7	Schalenrand	Shell edge
8	Queraussteifer, Querscheibe	Diaphragm, cross stiffener
9	Randversteifer,	Edge stiffener, edge beam
10	Schalenmembrane, Tragmembr.	Shell membrane, bearing m.
11	Ringträger, Fußring, Zugring	Hoop girder, base ring, tension ring
12	Druckring	Compression ring
13	Stütze	Column, support
14	Auflager	Bearing, support
15	Fundament; Gründung	Foundation, footing
16		
17		
18		

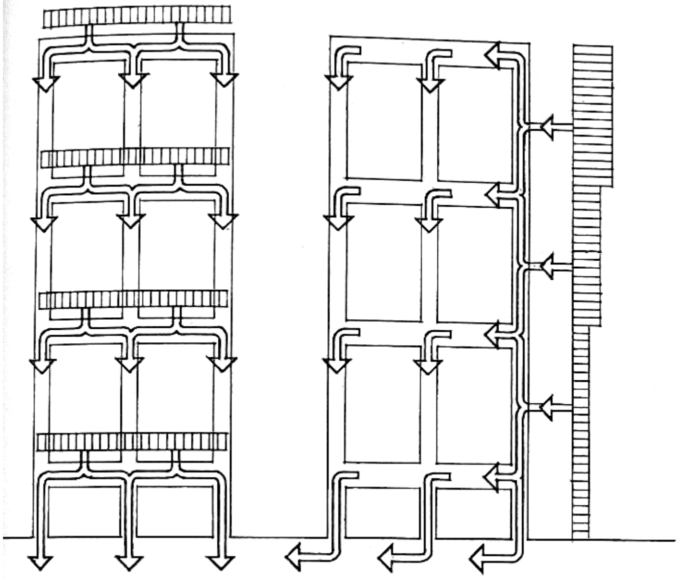
Topografische Systempunkte / Topographical system points

A	Auflagerpunkt	Point of support, bearing point
B	Fußpunkt, Basispunkt	Base point
C	Traufpunkt	Eaves point
D	Scheitelpunkt, Firstpunkt	Top, key, crown, apex, vertex
E	Einspannpunkt	Fixed-end point
F		
G		

Systemabmessungen / System dimensions

a	Scheibenabstand	Plate spacing, pl. distance
b	(Scheiben-, Schalen-) Spannweite	(Plate, shell) span
c	Stützenabstand	Column spacing, c. distance
d	Kraglänge	Cantilever length
e	Scheibenlänge, Tafellänge	Plate length, panel length
f	Scheibenhöhe, Tafelhöhe	Plate height, panel height
g	Schalenbreite	Shell width
h	Schalenlänge	Shell length
i	Konstruktionshöhe	Depth of construction
j	Pfeilhöhe, Stichhöhe	Rise
k	Traufhöhe	Eaves height
l	Lichte Höhe	Clear height, clearance
m	Stützenlänge, Stützenhöhe	Column length,
n	Schalendurchmesser	Shell diameter
o	Ringträger-(Zugring-) Breite	Hoop girder (tension ring) width
p		
q		

Fig. 4: Surface-Active Structures. (Engel)



Funktion der höhenaktiven Tragsysteme

Höhenaktive Tragsysteme sind Strukturen zur Kontrolle der Höhenlasten, d.h. ihrer Aufnahme, ihrer Ableitung zur Erde und ihrer Abgabe (= Lasten-Erdung):

1 die in der Höhe, d.h. über der Erdfläche anfallenden Vertikallasten: Dach- und Geschoßlasten

2 die durch die Höhenentwicklung bewirkten Horizontallasten: Wind- und Schwingungslasten

Höhenlasten fallen in jedem Bauwerk an. Je höher das Bauwerk ist, desto größer ist der Einfluß der Tragstruktur auf die Bauform

Function of height-active structure systems

Height-active structure systems are devices for the control of of height loads, i.e. their reception, their transfer to the ground, and their discharge (= load grounding):

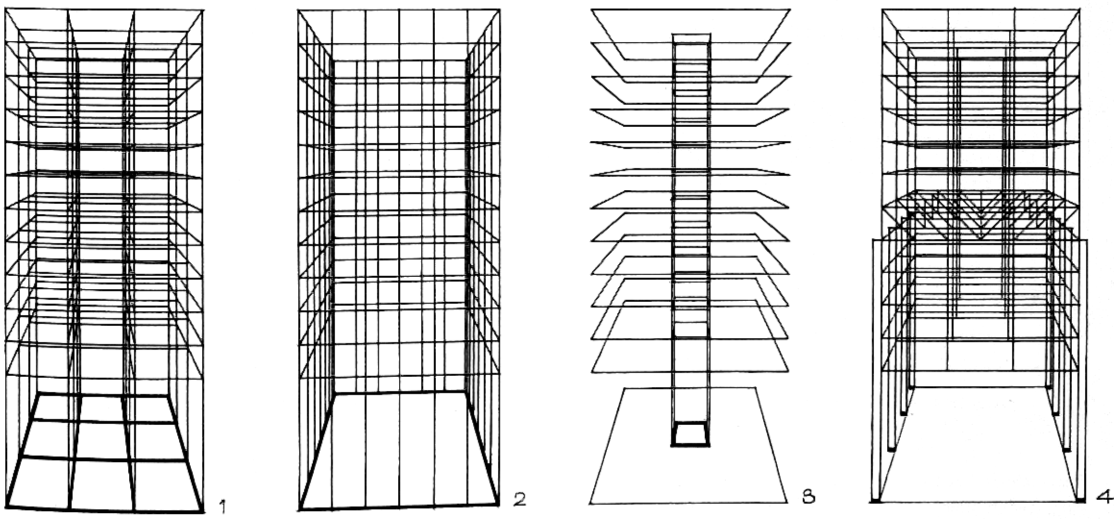
1 the vertical loads incuring in elevation, i.e. above ground surface: roof loads and storey loads

2 the horizontal loads effected by height extension: wind loads and vibration loads

Height loads incur in every building. The taller the building is, the greater is the influence of the bearing device on the building form

Die Eigenständigkeit der Hochwerke beruht nicht auf einem spezifischen MECHANISMUS der Kraftumlenkung wie bei den anderen 4 Tragwerk-'Familien', sondern auf der vorherrschenden FUNKTION der Hochwerke (wie vor). Zur Auslösung dieser Funktion bedienen sich die Hochwerke der Mechanismen der übrigen 4 Tragwerk-'Familien'

The distinction of the height structures does not rest upon a specific MECHANISM of redirecting forces as with the other 4 structure 'families', but on the predominant FUNCTION of height structures (see before). For the performance of this function the height structures make use of the mechanisms of all the other 4 structure 'families'



Einteilung der höhenaktiven Tragsysteme / Prototypen
Classification of height-active structure systems / Prototypes

1 RASTER-Hochwerke — GRID highrises
2 MANTEL-Hochwerke — CASING highrises
3 KERN-Hochwerke — CORE highrises
4 BRÜCKEN-Hochwerke — BRIDGE highrises

Fig. 5: Height-Active Structures. (Engel)

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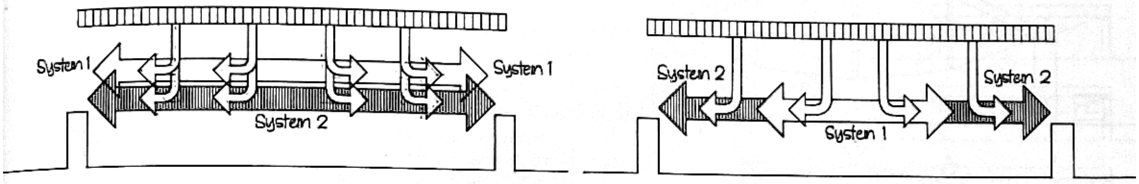
Definition / Potential

Hybride
Tragsysteme
6

HYBRIDE TRAGSYSTEME
sind Systeme, in denen die Kraftumlenkung durch Zussammenwirkung von zwei oder mehreren - in ihrer tragenden Funktion prinzipiell gleichrangigen - Mechanismen aus verschiedenen Tragwerk-'Familien' erfolgt
Die Zusammenwirkung wird bewerkstelligt durch zwei mögliche Formen der Systemverknüpfung: ÜBERLAGERUNG oder KOPPLUNG

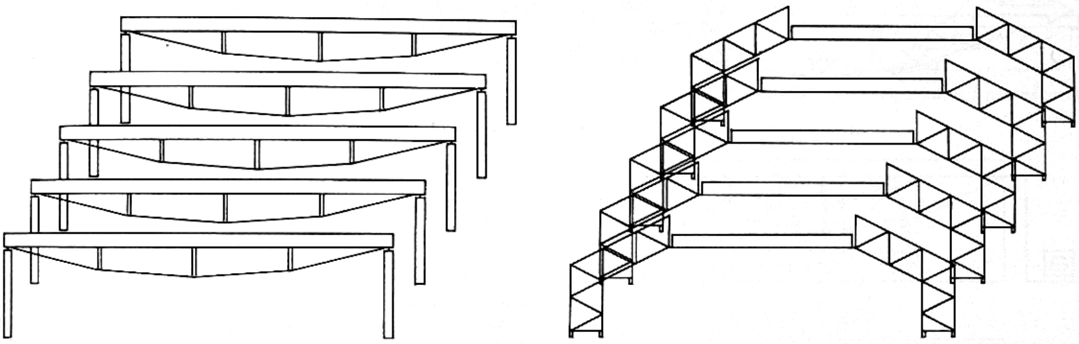
HYBRID STRUCTURE SYSTEMS
are systems, in which the redirection of forces is effected through the coaction of two or several - in their structural function basically equipotent - mechanisms from different structure 'families'
The coaction is being performed by two possible kinds of systems linkage: SUPERPOSITION or COUPLING

Definition 'Hybrid'



System-Überlagerung Systems superposition

System-Kopplung Systems coupling



Terminologische Abgrenzungen

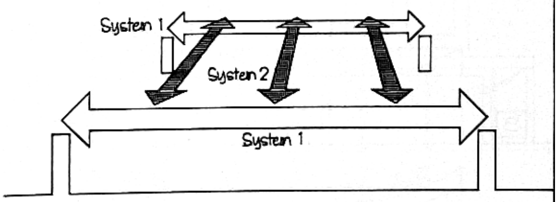
Unrichtige 'Hybrid'-Bezeichnung

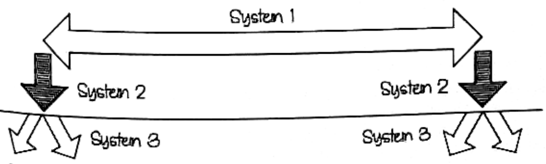
Unter hybriden Tragsystemen sind NICHT diejenigen Systeme zu verstehen, in denen Einzelfunktionen des Tragwerkes (Lastaufnahme, Lastableitung, Lastabgabe, Stabilisierung usw.) von Konstruktionen aus unterschiedlichen Tragwerk-'Familien' wahrgenommen werden

Terminological demarcations

Incorrect 'hybrid' denomination

Hybrid structure systems are NOT to be understood as those systems in which component bearing functions (load reception, load transfer, load discharge, stabilizations, etc.) are performed by constructions each belonging to a different structure 'family'





Fehleinstufung hybrider Tragsysteme

Hybride Tragsysteme können NICHT als abgrenzbare Tragwerk-'FAMILIE' oder strukturell bestimmbarer Tragwerk-'TYP' gelten:
1 Sie haben keinen typischen Mechanismus der Kräfte-Umlenkung
2 Sie entwickeln keinen spezifischen Kräfte- bzw. Spannungszustand
3 Sie verfügen nicht über kennzeichnende Strukturmerkmale

Misinterpretation of hybrid structure systems

Hybrid structure systems do NOT qualify as a unique structure 'FAMILY' or as a structure 'TYPE' definable by specific structural characteristics:
1 They do not possess an inherent mechanism for redirection of forces
2 They do not develop a specific condition of acting forces or stresses
3 They do not command structural features characteristic to them

Fig. 6: Hybrid Structures. (Engel)

Geometrische Einordnung der Tragglieder

Zusammenhang zwischen Geometrie und Tragfunktion der Einzelteile
(hier im Wesentlichen auf Schwerkraftlasten bezogen)

Geometric classification of structure members

Relationship between geometry and structural function of single members
(here essentially referring to gravitational loads)

Tragwerkformen entstehen in Vorstellung, Modell oder Entwurf als Gebilde von Linien und Flächen im Raum. Tragwerkformen setzen sich daher grundsätzlich aus geometrischen Elementarfiguren zusammen

Jeder geometrischen Elementarfigur sind je nach Standort und Stellung im Raum bestimmte Möglichkeiten ihrer statischen Funktion und konstruktiven Verwendung innerhalb des Tragsystems gegeben

Structure forms in conception, model or design originate as configurations of lines and planes in space. Structure forms, therefore, are basically composed of the elementary figures of geometry

To each elementary figure of geometry, depending on location and position in space, definite potentialities of structural function and constructional implementation within the structure system are inherent

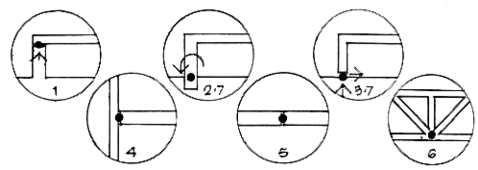
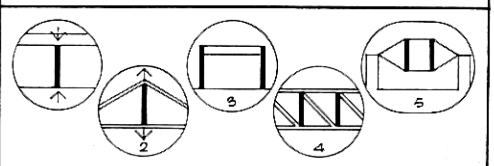
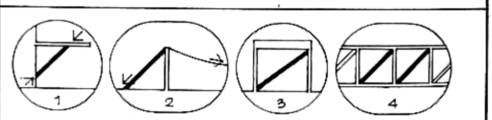
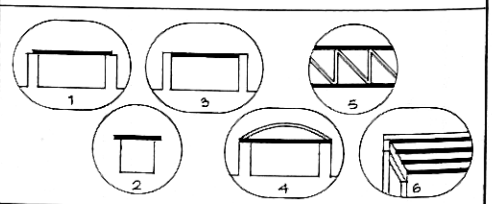
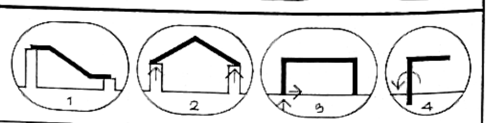
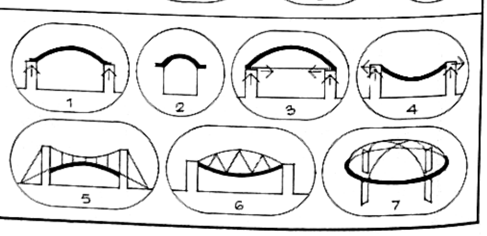
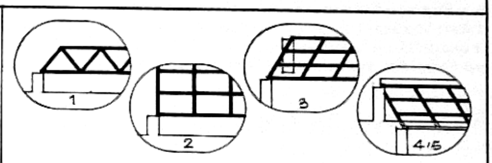
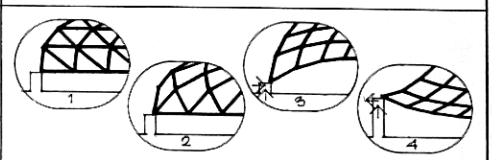
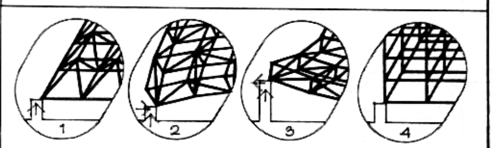
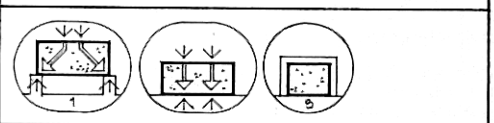
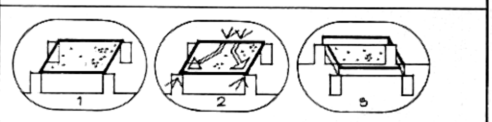
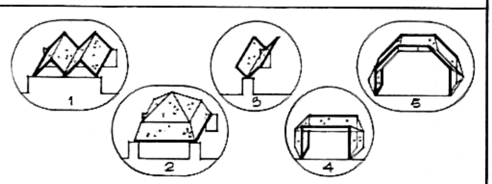
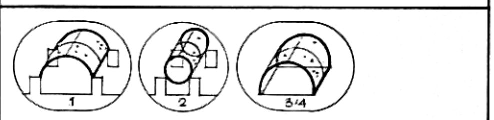
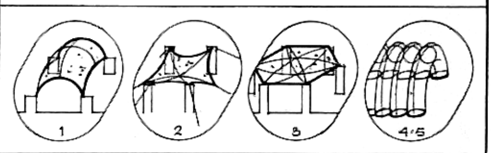
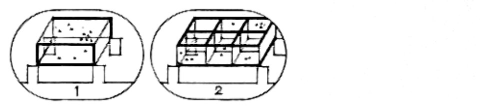
Geometrie / Geometry	Stellung / Figur / Position / figure			Tragkomponenten / Konstruktionsglieder / Structure components / Construction members		
PUNKT ① POINT		•	1 2 3 4 5 6 7	Auflager Einspannung Gelenk Verbindung Stoß Knoten Basis, Fuß	support, bearing fixed-end joint hinge, pin joint connection, joint (butt) joint node point base	
gerade LINIE ② straight LINE	senkrecht vertical	|	1 2 3 4 5	Stütze, Säule Hängeglied (Rahmen-) Stiel Vertikalstab Spreizglied	support, column suspension, hanger (frame) leg vertical rod, ~ bar spreader bar	
	schräg inclined	/	1 2 3 4	Strebe Rückhalteseil Aussteifung Diagonalstab	strut, brace restraining cable bracing diagonal member	
	waagrecht horizontal	—	1 2 3 4 5 6	Balken, Träger Sturz (-balken) Riegel Zuganker Ober-/Untergurt (Parallel-) Rippe	beam, girder lintel, header frame girder tie rod top / bottom chord (parallel) rib	
komplexe LINIE ③ complex LINE	geknickt bent	L	1 2 3 4	Knickbalken Giebelbalken Rahmen Kragstütze	bent beam gabled beam rigid frame, bent cantilevered column	
	gekrümmt curved	⌒	1 2 3 4 5 6 7	gekrümmter Balken Segmentsturz (Stütz-) Bogen Tragseil Stabilisier.-Seil Unter- (Ober)gurt Ringanker	curved beam segmental lintel (funicular) arch load cable stabilization cable bottom (top) chord tie beam, base ring	

Fig. 7: Geometric classification of structural members. This page and opposite. (Engel)

Geometrie Strukturform 7

Geometrie / Geometry	Stellung / Figur / Position / figure			Tragkomponenten / Konstruktionsglieder / Structure components / Construction members		
verknüpfte LINIE (4) jointed LINE	eben flat		1 2 3 4 5	Fachwerk Mehrfeld-Rahmen (Balken-) Rost Kassetten Kreuzrippen	truss multi-panel frame beam grid waffles cross ribs	
	gekrümmt curved		1 2 3 4	Fachwerk Lamellen-Raster (Stütz-) Gitter (Hänge-) Netz	truss lamella grid (thrust) lattice (suspension) mesh	
	räumlich spatial		1 2 3 4	Raumfachwerk Raumgitter Raumnetz 2-achsiger Rahmen /	space truss space lattice space mesh biaxial frame	
ebene FLÄCHE (5) flat PLANE	senkrecht vertical		1 2 3	(Trag-) Scheibe Tragende Wand Aussteifung	structural plate bearing wall bracing panel	
	wagrecht horizontal		1 2 3	(Trag-) Platte Horiz'scheibe Aussteifung	structural slab horiz. plate girder bracing plate	
	gefaltet folded		1 2 3 4 5	Prismat. Faltwerk Pyramid. Faltwerk / Faltträger Faltrahmen Faltwerk-Bogen	prismatic fold. str. pyramid fold. str. folded plate beam folded plate frame folded plate arch	
komplexe FLÄCHE (6) complex PLANE	einfach gekrümmt singly curved		1 2 3 4	Schale Rohr / Luftschlauch / Gewölbe Lufthalle	shell tube / air tube vault air-supported roof	
	doppelt gekrümmt doubly curved		1 2 3 4 5	Schale Zeltmembrane Luftkissen Luftschlauch Röhre	shell tent membrane air cushion air tube shell tube	
	kombiniert combined		1 2	Kastenrahmen Schalenrost	box frame plate grid	

A METHOD OF MATERIAL SELECTION
COPIED FROM ASHBY

A-08

8.1 A Material Selection Tool

The following pages are copies from Michael F. Ashby's *Material's Selection in Mechanical Design*. Ashby is a professor in the Department of Engineering at the University of Cambridge UK. His book describes a procedure for material selection in mechanical design. Ashby has created material selection charts the allow materials to be compared against one another for a large number of property variables that he calls material indices. (**Fig. 3**) Combining these indices with the selection charts allows optimization of the materials selection process. Ashby also presents a system for matching material properties and processing technologies. (**Figs. 4 and 11**) Copied in this appendix are three examples of material selection charts. (**Figs. 6 to 11**) Two materials selection examples are included at the end of this appendix starting on page 349.

8.2 Bibliography

Ashby, Michael F. *Materials Selection in Mechanical Design* (second edition). Butterworth Heinemann : Oxford and Boston. 1999.

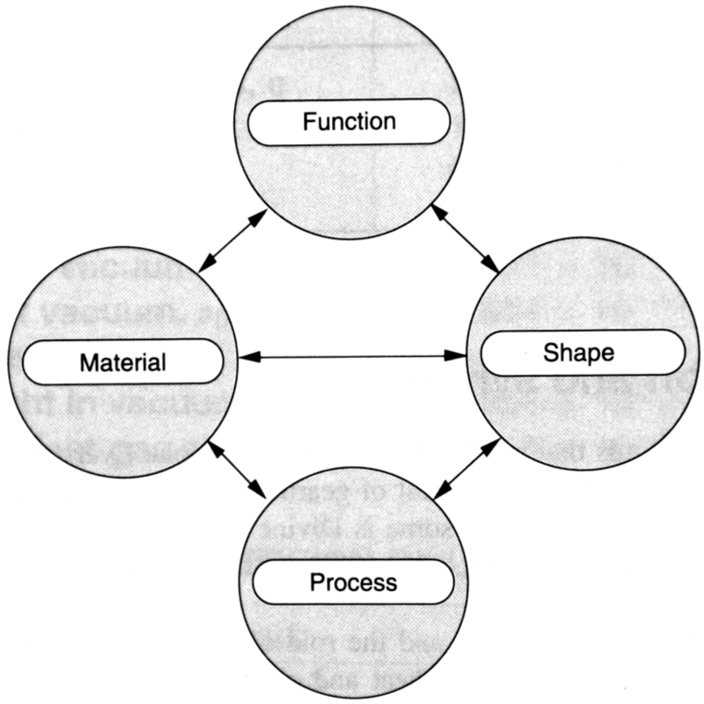


Fig. 1.1 Function, material, process and shape interact. Later chapters deal with each in turn.

Fig. 1: Ashby's design model is based on the premise that function, material, process and shape interact. (Ashby)

Table 1.3 Material properties shown on the charts

Property	*Symbol*	*Units*
Relative cost	C_m	(–)
Density	ρ	(Mg/m^3)
Young's modulus	E	(GPa)
Strength	σ_f	(MPa)
Fracture toughness	K_{Ic}	$(MPam^{1/2})$
Toughness	G_{Ic}	(J/m^2)
Damping coefficient	η	(–)
Thermal conductivity	λ	(W/mK)
Thermal diffusivity	a	(m^2/s)
Volume specific heat	$C_{P\rho}$	(J/m^3K)
Thermal expansion coefficient	a	(1/K)
Thermal shock resistance	ΔT	(K)
Strength at temperature	$\sigma(T)$	(MPa)
Specific wear rate	W/AP	(1/MPa)

Fig. 2: Material properties and their symbols used on the selection charts. (Ashby)

Table 1.4 Examples of material-indices

Function	*Index*
Tie, minimum weight, stiffness prescribed	$\frac{E}{\rho}$
Beam, minimum weight, stiffness prescribed	$\frac{E^{1/2}}{\rho}$
Beam, minimum weight, strength prescribed	$\frac{\sigma_y^{2/3}}{\rho}$
Beam, minimum cost, stiffness prescribed	$\frac{E^{1/2}}{C_m \rho}$
Beam, minimum cost, strength prescribed	$\frac{\sigma_y^{2/3}}{C_m \rho}$
Column, minimum cost, buckling load prescribed	$\frac{E^{1/2}}{C_m \rho}$
Spring, minimum weight for given energy storage	$\frac{\sigma_y^2}{E_\rho}$
Thermal insulation, minimum cost, heat flux prescribed	$\frac{1}{\lambda C_m \rho}$

(ρ = density; E = Young's modulus; σ_y = elastic limit; C_m = cost/kg; λ = thermal conductivity; κ = electrical conductivity; C_p = specific heat)

Fig. 3: Some material index examples. (Ashby)

Table 1.5 Process classes

Casting	(sand, gravity, pressure, die, etc.)
Pressure moulding	(direct, transfer, injection, etc.)
Deformation processes	(rolling, forging, drawing, etc.)
Powder methods	(slip cast, sinter, hot press, hip)
Special methods	(CVD, electroform, lay up, etc.)
Machining	(cut, turn, drill, mill, grind, etc.)
Heat treatment	(quench, temper, solution treat, age, etc.)
Joining	(bolt, rivet, weld, braze, adhesives)
Finish	(polish, plate, anodize, paint)

Fig. 4: Classification of processing technologies. (Ashby)

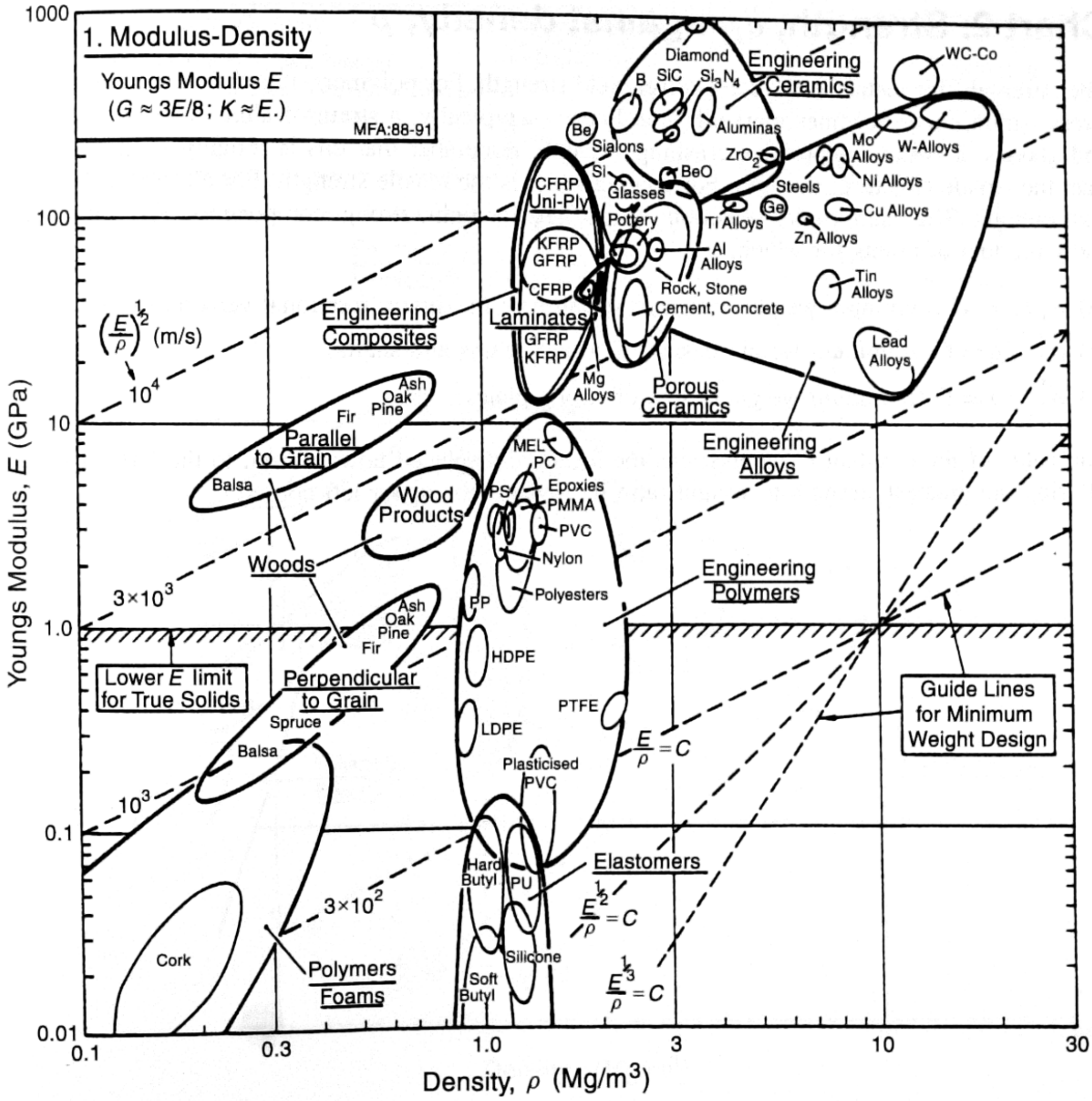


Fig. 5: Material selection chart relating Young’s E Modulus to Density. (Ashby)

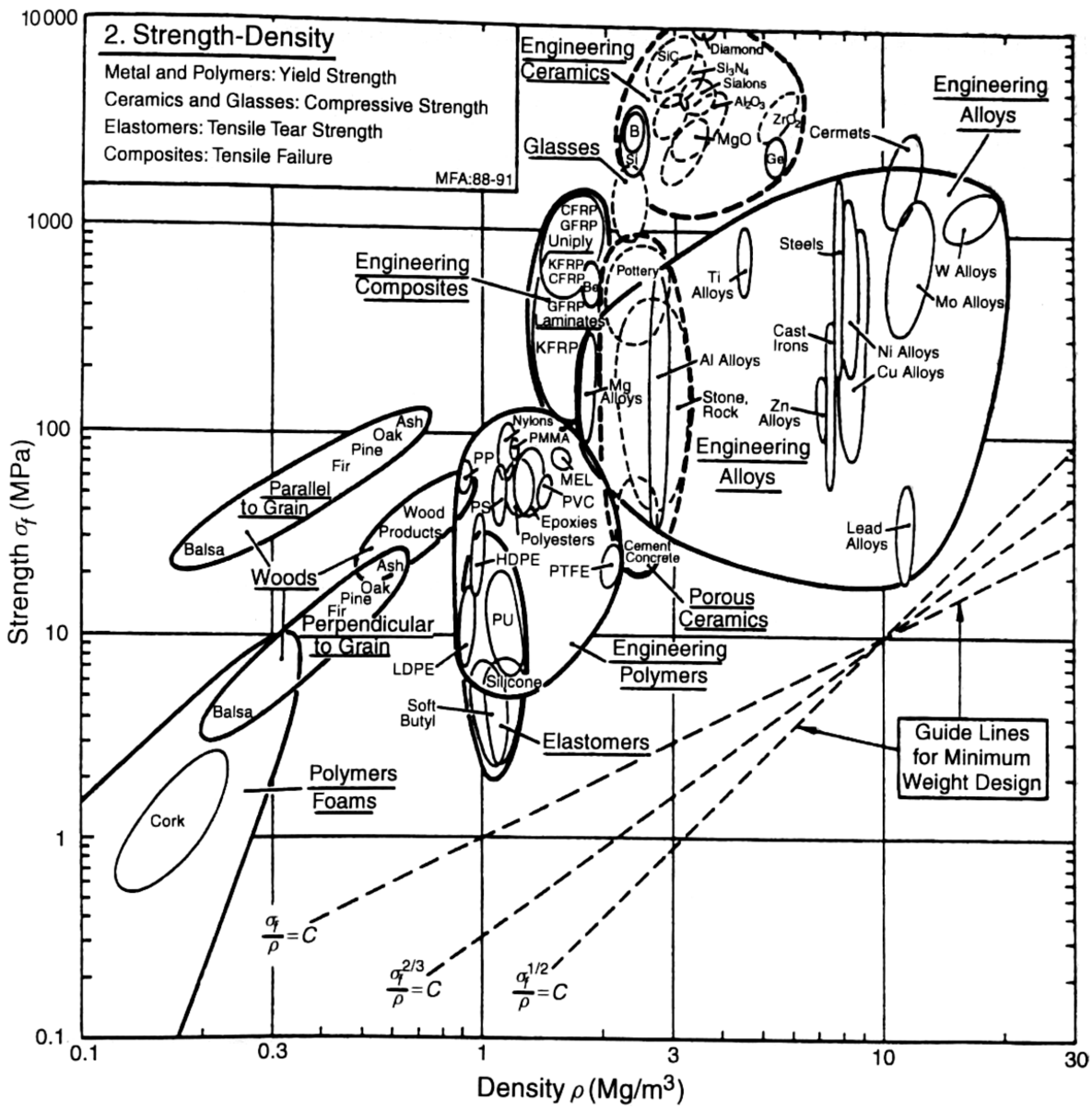


Fig. 6: Material selection chart relating Strength to Density. (Ashby)

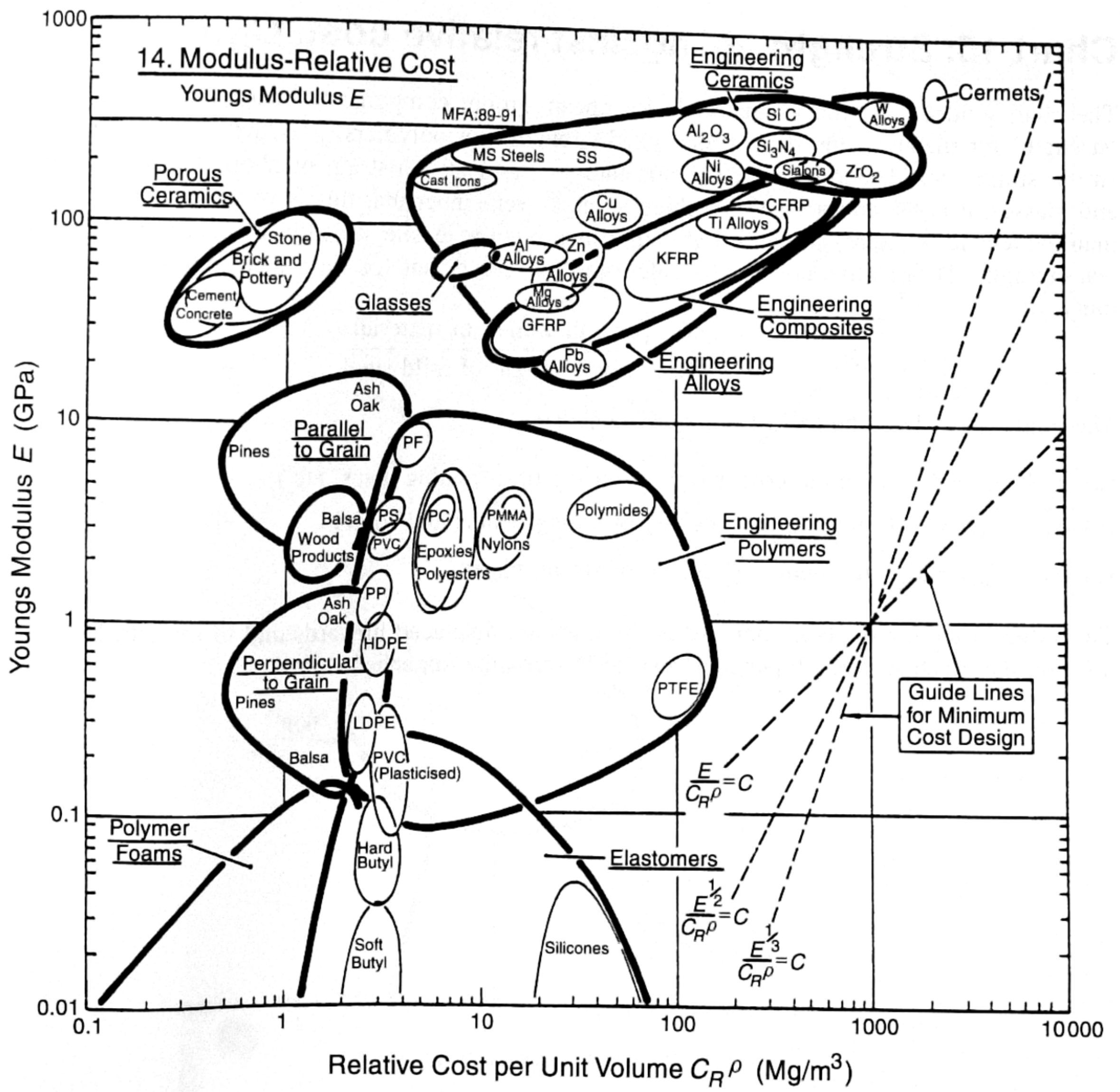


Fig. 7: Material selection chart relating Young's E Modulus to Relative Cost. (Ashby)

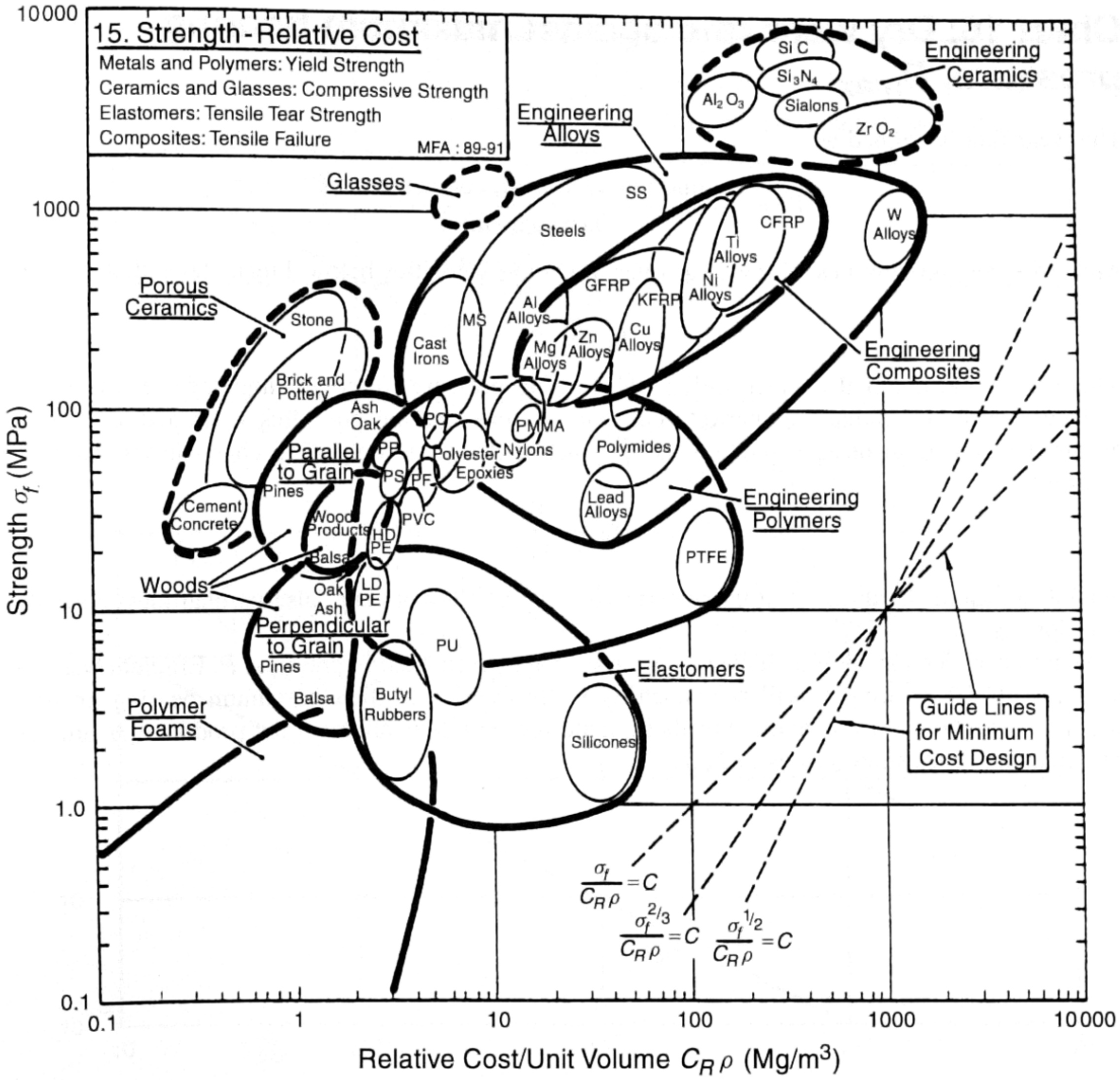


Fig. 8: Material selection chart relating Strength to Relative Cost. (Ashby)

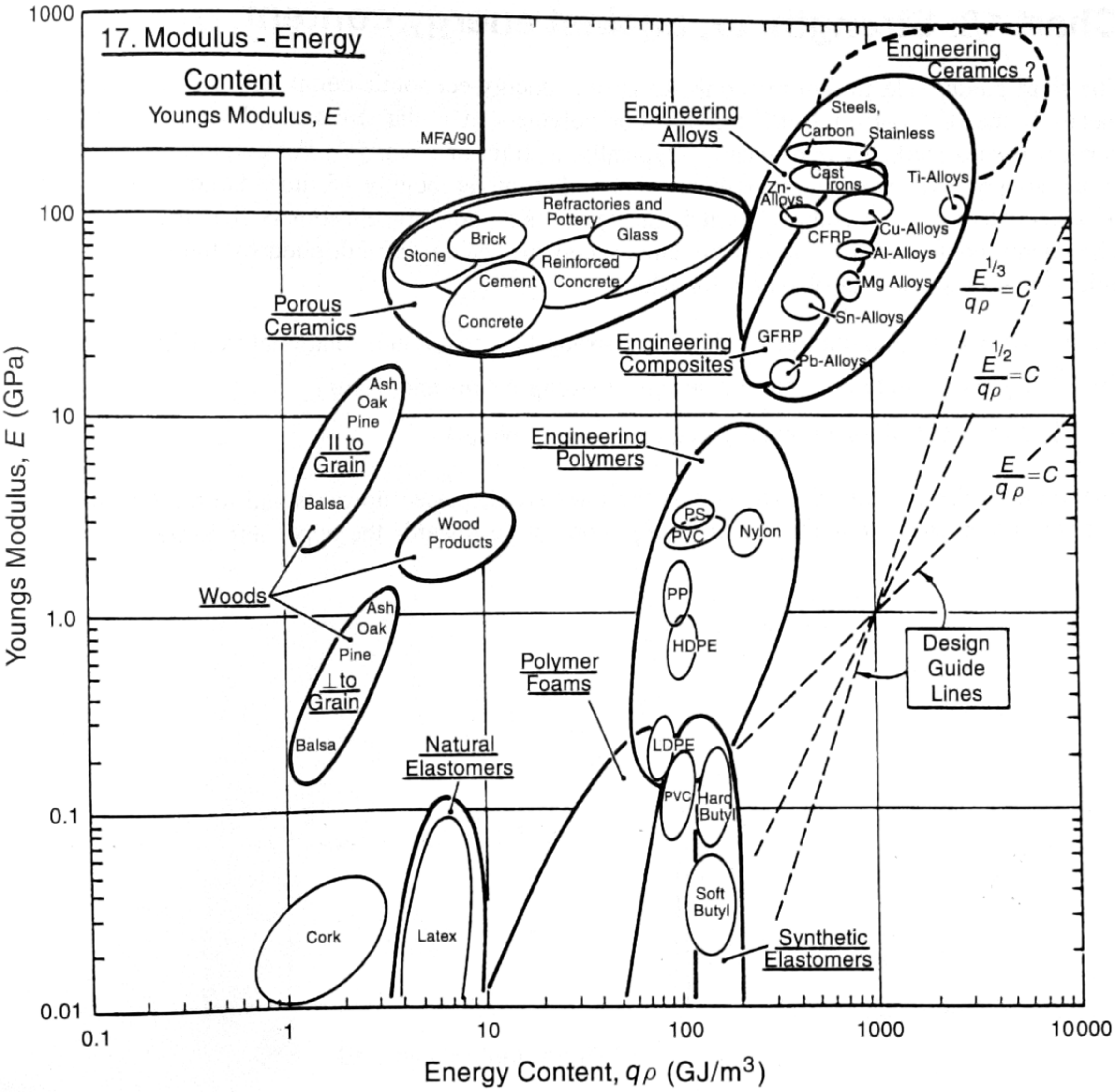


Fig. 9: Material selection chart relating Young’s E Modulus to Energy Content. (Ashby)

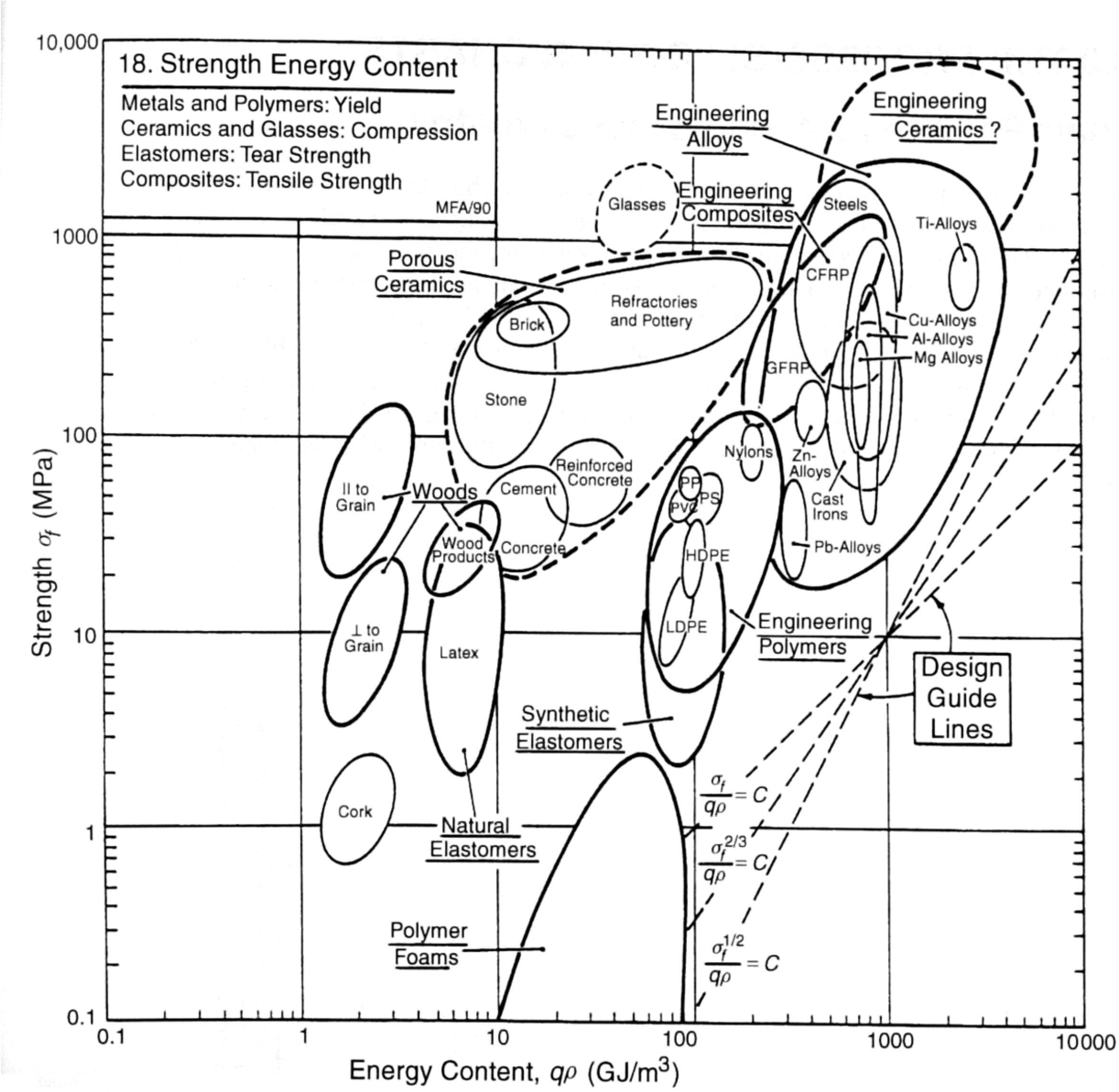


Fig. 10: Material selection chart relating Strength to Energy Content. (Ashby)

Material Class

Process Class		Metals					Ceramics & Glasses				Polymers & Elastomers			Composites		
		Ferrous	Refractory	Precious	Heavy	Light	Cementitous	Vitreous	Fine	Glasses	Thermosets	Thermoplastics	Elastomers	PMCs	MMCs	CMCs
Casting	Gravity	2	1	2	2	2	0	0	0	1	0	0	0	0	0	0
	Low pressure	2	0	2	2	2	0	0	0	2	0	0	0	0	1	0
	High pressure	1	0	2	2	2	0	0	0	1	0	0	0	0	2	0
	Investment	2	2	2	2	2	0	0	0	0	0	0	0	0	0	0
Moulding	Injection	0	0	2	0	0	0	0	0	2	2	2	2	2	0	0
	Compress	0	0	2	0	0	0	0	0	2	2	2	2	2	1	0
	Blow	0	0	0	0	0	0	0	0	2	0	2	0	0	0	0
	Foam	0	0	0	0	0	0	0	0	0	2	2	2	0	0	0
Deformation	Cold	2	0	2	2	2	0	0	0	0	0	0	0	0	0	0
	Warm	2	0	2	2	2	0	0	0	0	0	0	0	0	0	0
	Hot	2	2	2	2	2	0	0	0	2	0	0	0	0	0	0
	Sheet	2	1	2	2	2	0	0	0	0	0	2	0	0	1	0
Machining	Turn	2	2	2	2	2	0	1	0	0	2	2	0	2	2	0
	Mill	2	2	2	2	2	0	1	0	0	2	2	0	2	2	0
	Grind	2	2	1	2	2	0	2	2	2	0	0	0	0	2	2
	Polish	2	2	2	2	2	0	2	2	2	0	0	0	0	1	2
Powder Methods	Sinter/HIP	2	2	2	2	2	0	2	2	1	0	2	0	0	2	2
	Slip cast	0	0	0	0	0	0	2	2	1	0	0	0	0	0	1
	Spray forming	2	2	2	2	2	0	2	2	2	2	2	0	2	0	0
	Hydration	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0
Composite Forming	Lay-up	0	0	0	0	0	0	0	0	0	0	0	0	2	0	2
	Mould	0	0	0	0	0	0	0	0	0	2	2	2	2	0	0
	Squeeze-cast	1	0	0	2	2	0	0	0	0	0	0	0	0	2	0
	Filament wind	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0
Molecular Methods	PVD	0	2	2	2	0	0	0	2	.0	0	0	0	0	1	0
	CVD	0	2	2	2	0	0	0	2	0	0	0	0	0	1	2
	Sputtering	2	2	2	2	2	0	0	0	0	0	0	0	0	0	0
	Electroforming	1	0	2	2	0	0	0	0	0	0	0	0	0	0	0
Special Methods	Electrochemical	2	2	2	2	2	0	0	0	0	0	0	0	0	2	0
	Ultrasonic	1	2	0	0	0	0	2	2	2	0	0	0	0	0	2
	Chemical	2	2	2	2	2	0	2	2	2	0	0	0	0	0	0
	Thermal Beam	2	2	2	2	2	0	2	2	2	2	2	2	2	2	2
Fabrication	Weld/braze	2	2	2	2	2	0	0	0	0	0	2	0	0	0	0
	Adhesive	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	Fasten	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	Microfabrication	2	2	2	2	2	0	2	2	2	2	2	2	2	2	2

Fig. 11: The Material-Process Matrix, indicating what processing technologies are suitable for a given material. 2 indicates that a process is viable; 1 that it could be used under special circumstances; and 0 means the process is not viable. (Ashby)

Radio telescopes do not have to be quite as precisely dimensioned as optical ones because they detect radiation with a longer wavelength. But they are much bigger (60 metres rather than 6) and they suffer from similar distortional problems. Microwaves have wavelengths in the mm band, requiring precision over the mirror face of 0.25 mm. A recent 45 m radio telescope built for the University of Tokyo achieves this, using CFRP. Its parabolic surface is made of 6000 CFRP panels, each servo controlled to compensate for macro-distortion. Recent telescopes have been made from CFRP, for exactly the reasons we deduced. Beryllium appears on our list, but is impractical for large mirrors because of its cost. Small mirrors for space applications must be light for a different reason (to reduce take-off weight) and must, in addition, be as immune as possible to temperature change. Here beryllium comes into its own.

Related case studies

Case Study 6.5: Materials for table legs
Case Study 6.20: Materials to minimize thermal distortion

6.4 Materials for table legs

Luigi Tavolino, furniture designer, conceives of a lightweight table of daring simplicity: a flat sheet of toughened glass supported on slender, unbraced, cylindrical legs (Figure 6.5). The legs must be solid (to make them thin) and as light as possible (to make the table easier to move). They must support the table top and whatever is placed upon it without buckling. What materials could one recommend?

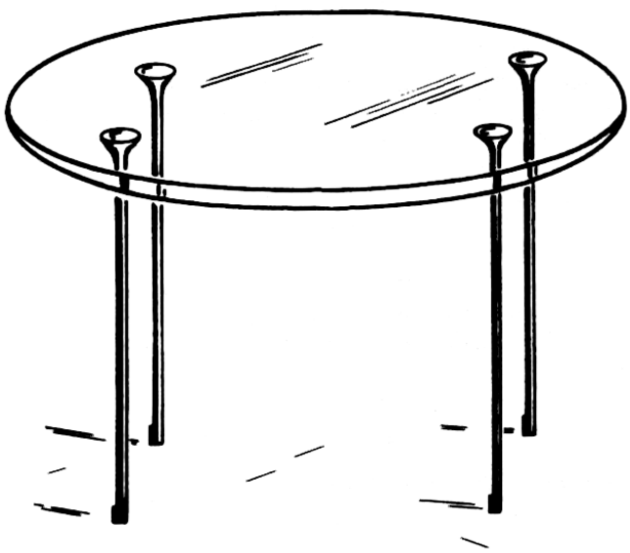


Fig. 6.5 A lightweight table with slender cylindrical legs. Lightness and slenderness are independent design goals, both constrained by the requirement that the legs must not buckle when the table is loaded. The best choice is a material with high values of both $E^{1/2}/\rho$ and E.

Figs. 12-15: Material selection example 1: choosing a material for slender table legs. (Ashby)

Table 6.5 Design requirements for table legs

Function	Column (supporting compressive loads)
Objective	(a) Minimize the mass (b) Maximize slenderness
Constraints	(a) Length ℓ specified (b) Must not buckle under design loads (c) Must not fracture if accidentally struck

The model

This is a problem with two objectives*: weight is to be minimized, and slenderness maximized. There is one constraint: resistance to buckling. Consider minimizing weight first.

The leg is a slender column of material of density ρ and modulus E. Its length, ℓ, and the maximum load, P, it must carry are determined by the design: they are fixed. The radius r of a leg is a free variable. We wish to minimize the mass m of the leg, given by the objective function

$$m = \pi r^2 \ell \rho \tag{6.6}$$

subject to the constraint that it supports a load P without buckling. The elastic load P_{crit} of a column of length ℓ and radius r (see Appendix A, 'Useful Solutions') is

$$P_{\text{crit}} = \frac{\pi^2 EI}{\ell^2} = \frac{\pi^3 E r^4}{4\ell^2} \tag{6.7}$$

using $I = \pi r^4/4$ where I is the second moment of area of the column. The load P must not exceed P_{crit}. Solving for the free variable, r, and substituting it into the equation for m gives

$$m \geq \left(\frac{4P}{\pi}\right)^{1/2} (\ell)^2 \left[\frac{\rho}{E^{1/2}}\right] \tag{6.8}$$

The material properties are grouped together in the last pair of brackets. The weight is minimized by selecting the subset of materials with the greatest value of the material index

$$\boxed{M_1 = \frac{E^{1/2}}{\rho}}$$

(a result we could have taken directly from Appendix B).

Now slenderness. Inverting equation (6.7) with $P = P_{\text{crit}}$ gives an equation for the thinnest leg which will not buckle:

$$r = \left(\frac{4P}{\pi^3}\right)^{1/4} (\ell)^{1/2} \left[\frac{1}{E}\right]^{1/4} \tag{6.9}$$

The thinnest leg is that made of the material with the largest value of the material index

$$\boxed{M_2 = E}$$

* Formal methods for dealing with multiple objectives are developed in Chapter 9.

Figs. 12-15: Material selection example 1: choosing a material for slender table legs. (Ashby)

The selection

We seek the subset of materials which have high values of $E^{1/2}/\rho$ and E. Figure 6.6 shows the appropriate chart: Young's modulus, E, plotted against density, ρ. A guideline of slope 2 is drawn on the diagram; it defines the slope of the grid of lines for values of $E^{1/2}/\rho$. The guideline is displaced upwards (retaining the slope) until a reasonably small subset of materials is isolated above it; it is shown at the position $M_1 = 6\,\text{GPa}^{1/2}/(\text{Mg/m}^3)$. Materials above this line have higher values of

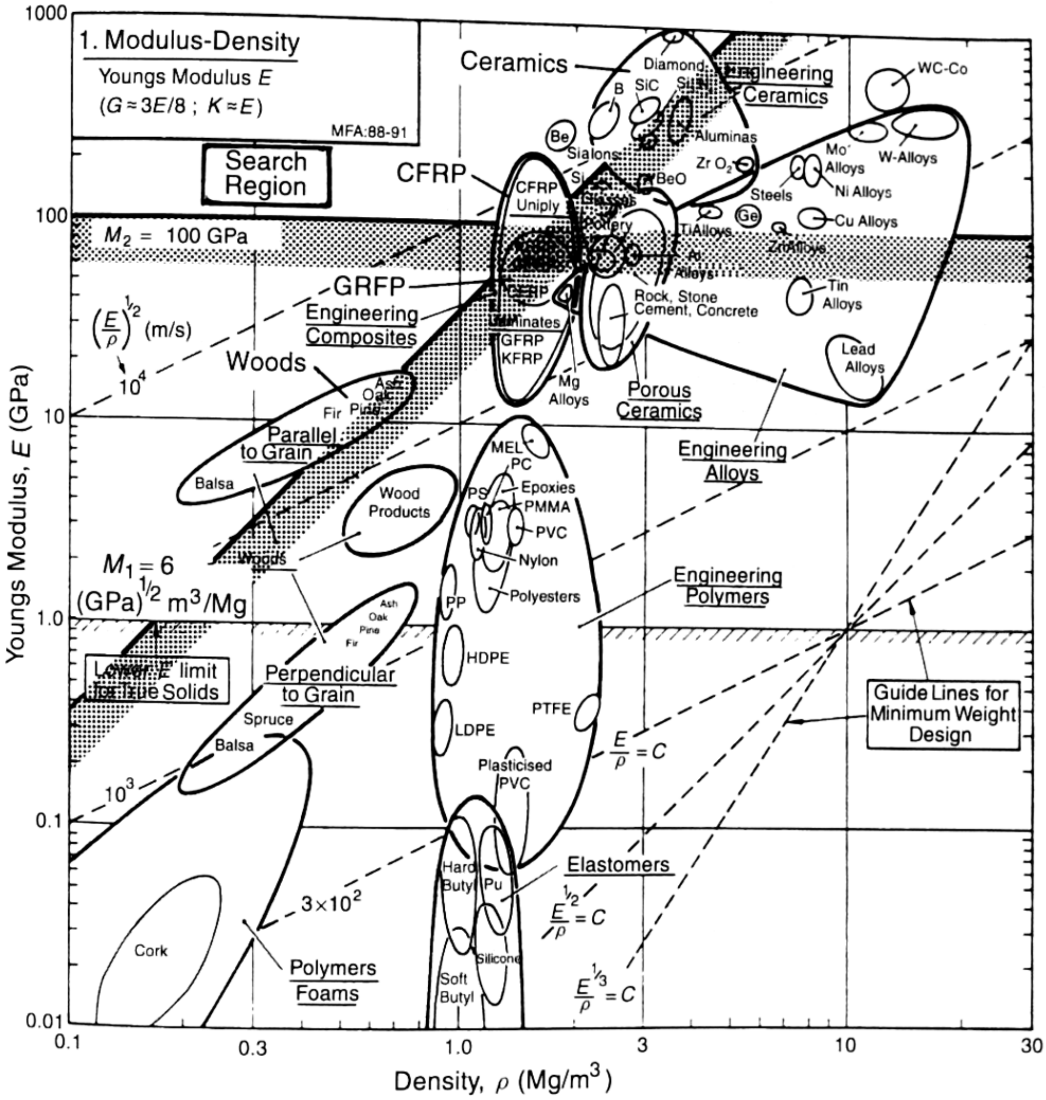


Fig. 6.6 Materials for light, slender legs. Wood is a good choice; so is a composite such as CFRP, which, having a higher modulus than wood, gives a column which is both light and slender. Ceramics meet the stated design goals, but are brittle.

Figs. 12-15: Material selection example 1: choosing a material for slender table legs. (Ashby)

Table 6.6 Materials for table legs

Material	M_1 ($GPa^{1/2}m^3/Mg$)	M_2 (GPa)	*Comment*
Woods	5–8	4–20	Outstanding M_1; poor M_2. Cheap, traditional, reliable.
CFRP	4–8	30–200	Outstanding M_1 and M_2, but expensive.
GFRP	3.5–5.5	20–90	Cheaper than CFRP, but lower M_1 and M_2.
Ceramics	4–8	150–1000	Outstanding M_1 and M_2. Eliminated by brittleness.

M_1. They are identified on the figure: *woods* (the traditional material for table legs), *composites* (particularly CFRP) and certain special *engineering ceramics*. Polymers are out: they are not stiff enough; metals too: they are too heavy (even magnesium alloys, which are the lightest). The choice is further narrowed by the requirement that, for slenderness, E must be large. A horizontal line on the diagram links materials with equal values of E; those above are stiffer. Figure 6.6 shows that placing this line at $M_1 = 100\,\text{GPa}$ eliminates woods and GFRP. If the legs must be really thin, then the shortlist is reduced to CFRP and ceramics: they give legs which weigh the same as the wooden ones but are much thinner. Ceramics, we know, are brittle: they have low values of fracture toughness. Table legs are exposed to abuse — they get knocked and kicked; common sense suggests that an additional constraint is needed, that of adequate toughness. This can be done using Chart 6 (Figure 4.7); it eliminates ceramics, leaving CFRP. The cost of CFRP (Chart 14, Figure 4.15) may cause Snr. Tavolino to reconsider his design, but that is another matter: he did not mention cost in his original specification.

It is a good idea to lay out the results as a table, showing not only the materials which are best, but those which are second-best — they may, when other considerations are involved become the best choice. Table 6.6 shows one way of doing it.

Postscript

Tubular legs, the reader will say, must be lighter than solid ones. True; but they will also be fatter. So it depends on the relative importance Mr Tavolino attaches to his two objectives — lightness and slenderness — and only he can decide that. If he can be persuaded to live with fat legs, tubing can be considered — and the material choice may be different. Materials selection when section-shape is a variable comes in Chapter 7.

Ceramic legs were eliminated because of low toughness. If (improbably) the goal was to design a light, slender-legged table for use at high temperatures, ceramics should be reconsidered. The brittleness problem can be by-passed by protecting the legs from abuse, or by pre-stressing them in compression.

Related case studies

Case Study 6.3: Mirrors for large telescopes
Case Study 8.2: Spars for man-powered planes
Case Study 8.3: Forks for a racing bicycle

Figs. 12-15: Material selection example 1: choosing a material for slender table legs. (Ashby)

6.5 Cost — structural materials for buildings

The most expensive thing that most people buy is the house they live in. Roughly half the cost of a house is the cost of the materials of which it is made, and they are used in large quantities (family house: around 200 tonnes; large apartment block: around 20 000 tonnes). The materials are used in three ways (Figure 6.7): structurally to hold the building up; as cladding, to keep the weather out; and as 'internals', to insulate against heat, sound, and so forth).

Consider the selection of materials for the structure. They must be stiff, strong, and cheap. Stiff, so that the building does not flex too much under wind loads or internal loading. Strong, so that there is no risk of it collapsing. And cheap, because such a lot of material is used. The structural frame of a building is rarely exposed to the environment, and is not, in general, visible. So criteria of corrosion resistance, or appearance , are not important here. The design goal is simple: strength and stiffness at minimum cost. To be more specific: consider the selection of material for floor beams. Table 6.7 summarizes the requirements.

The model

The way of deriving material indices for cheap, stiff and strong beams was developed in Chapter 5. The results we want are listed in Table 5.7. The critical components in building are loaded either

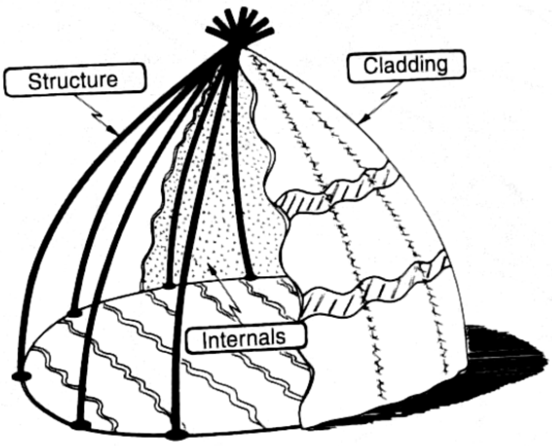


Fig. 6.7 The materials of a building perform three broad roles. The frame gives mechanical support; the cladding excludes the environment; and the internal surfacing controls heat, light and sound.

Table 6.7 Design requirements for floor beams

Function	Floor beams
Objective	Minimize the cost
Constraints	(a) Length L specified
	(b) Stiffness: must not deflect too much under design loads
	(c) Strength: must not fail under design loads

Figs. 16-19: Material selection example 2: choosing a material for floor beams. (Ashby)

in bending (floor joists, for example) or as columns (the vertical members). The two indices that we want to maximize are:

$$\boxed{M_1 = \frac{E^{1/2}}{\rho C_m}}$$

and

$$\boxed{M_2 = \frac{\sigma_f^{2/3}}{\rho C_m}}$$

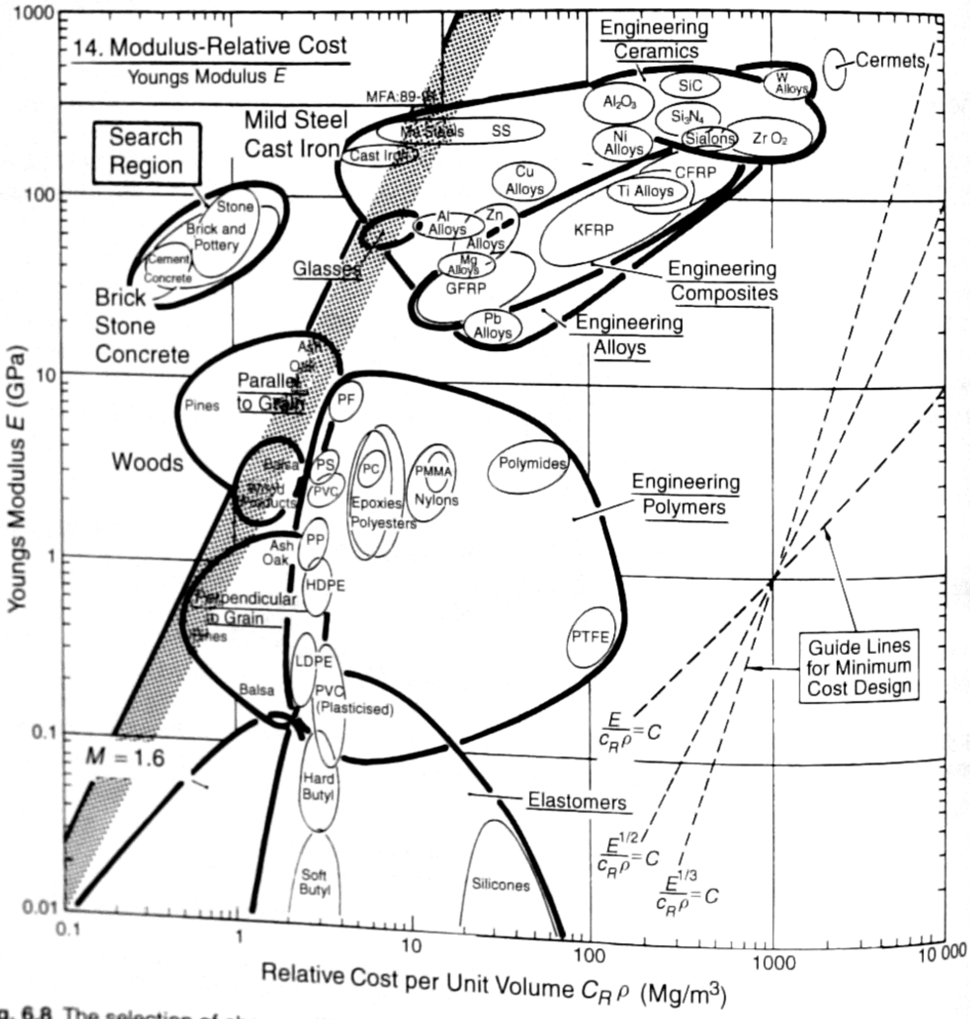


Fig. 6.8 The selection of cheap, stiff materials for the structural frames of buildings.

Figs. 16-19: Material selection example 2: choosing a material for floor beams. (Ashby)

where, as always, E is Young's modulus, σ_f is the failure strength, ρ is the density and C_m material cost.

The selection

Cost appears in two of the charts. Figure 6.8 shows the first of them: modulus against relative cost per unit volume. The shaded band has the appropriate slope; it isolates concrete, stone, brick, softwoods, cast irons and the cheaper steels. The second, strength against relative cost, is shown in Figure 6.9. The shaded band — M_2 this time — gives almost the same selection. They are listed, with values, in the table. They are exactly the materials of which buildings have been, and are, made.

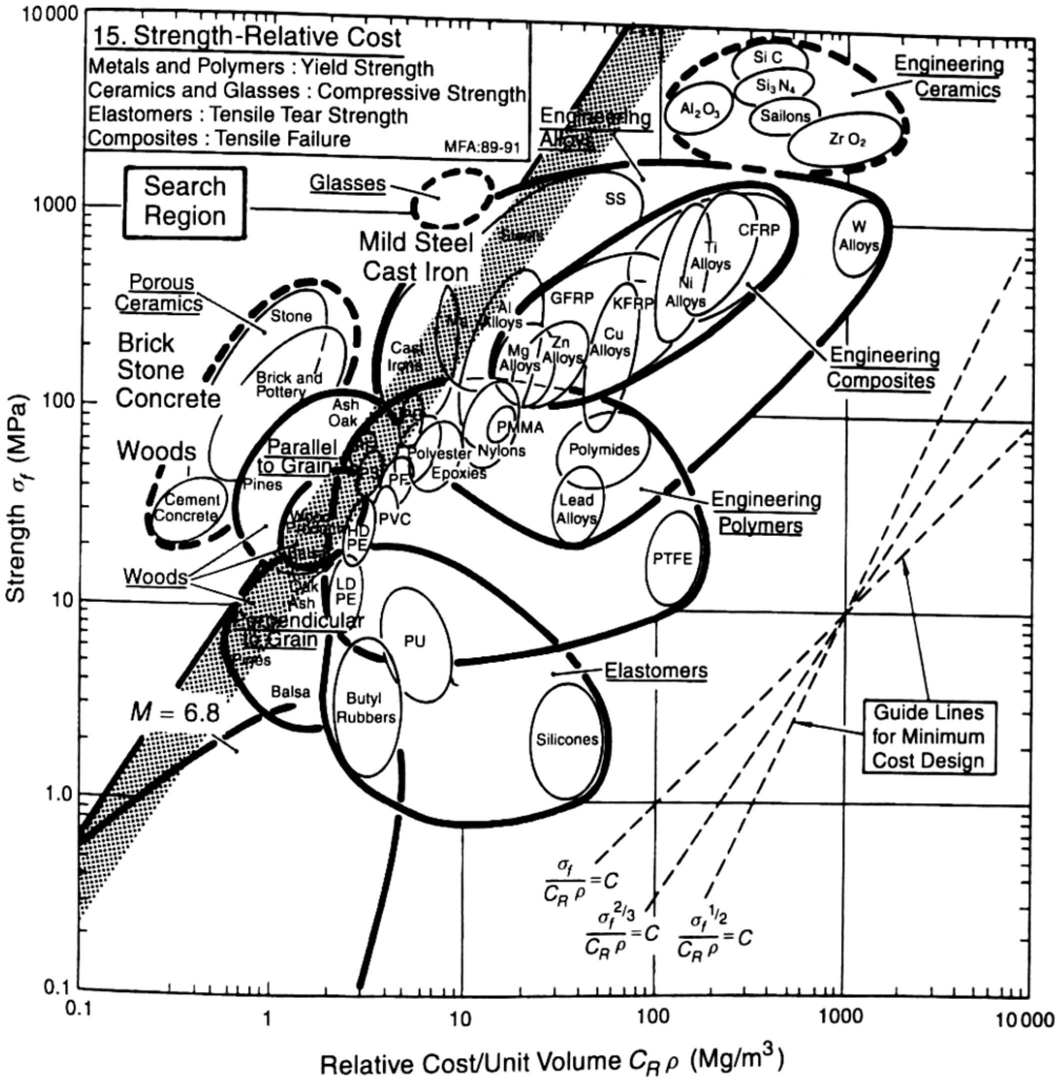


Fig. 6.9 The selection of cheap, strong materials for the structural frames of buildings.

Figs. 16-19: Material selection example 2: choosing a material for floor beams. (Ashby)

Table 6.8 Structural materials for buildings

Material	M_1 $(GPa^{1/2}/(k\$/m^3))$	M_2 $(MPa^{2/3}/(k\$/m^3))$	*Comment*
Concrete	40	80	Use in compression only
Brick	20	45	
Stone	15	45	
Woods	15	80	Tension and compression, with freedom of section shape
Cast iron	5	20	
Steel	3	21	
Reinforced concrete	20	60	

Postscript

It is sometimes suggested that architects live in the past; that in the late 20th century they should be building with fibreglass (GFRP), aluminium alloys and stainless steel. Occasionally they do, but the last two figures give an idea of the penalty involved: the cost of achieving the same stiffness and strength is between 5 and 10 times greater. Civil construction (buildings, bridges, roads and the like) is materials-intensive: the cost of the material dominates the product cost, and the quantity used is enormous. Then only the cheapest of materials qualify, and the design must be adapted to use them. Concrete, stone and brick have strength only in compression; the form of the building must use them in this way (columns, arches). Wood, steel and reinforced concrete have strength both in tension and compression, and steel, additionally, can be given efficient shapes (I-sections, box sections, tubes); the form of the building made from these has much greater freedom.

Further reading

Cowan, H.J. and Smith, P.R. (1988) *The Science and Technology of Building Materials*, Van Nostrand-Reinhold, New York.

Related case studies

Case Study 6.2: Materials for oars
Case Study 6.4: Materials for table legs
Case Study 8.4: Floor joists: wood or steel?

Figs. 16-19: Material selection example 2: choosing a material for floor beams. (Ashby)

CHRONOLOGIES OF MATERIAL EVOLUTION

A-09

9.1 Introduction

This appendix summarizes the chronological evolution of materials. I include: significant advances in processing technologies; the invention or incorporating inter-disciplinary technologies that helped to advance the development of the materials; major socio-political factors; and relevant economic and production data. Progress can be dated more precisely for some materials than others. I give dates where possible, otherwise I indicate the general period.

Any dates given should be accepted with the understanding that precise dates are often only possible to determine from patent documents. As a general rule, any patent date or patentee should be viewed in the context of their being the first to stake a claim to a particular invention but they may not be the original inventor. However, the dates are good enough for my purpose here.

More detailed information can be variously found on certain periods or subjects in the annexed case studies. The history of plywood until the start of World War I can be found in **Appendix A-04, p.A.209-A.224**. **Appendix A-10** is a combination of these general summaries and more detailed information found in the case studies.

Bibliographic references in this chapter are different from the rest of the thesis. Where applicable, the source for each date or development is referenced by a reference code and page number. References and their codes are listed at the beginning of each chronology. A complete bibliography is at the end of the chapter.

The purpose of these chronologies is to give an overall view of material evolution. I analyze these chronologies in **Appendix A-10** for certain trends or characteristics that indicate key mechanisms of development for each material, and development characteristics that can be applied more generally to multiple materials.

9.2 Stone, Pre-History to Present

Dates are mostly approximate.

9.2.1 Beginnings

Found, small construction (walls; stacking).

Simple, short beams (plates to span short distances). (**Fig. 1**)

Found, large construction. Dolmens made by using earth fill to push stones on top of other stones. (**Fig. 2**)

Corbelled arch (development of the construction process; stacking). (**Fig. 3**)

9.2.2 Bronze Age: 4000 to 3000 BC

Bronze age tools.

Dressed, post and lintel construction. (**Fig. 4**)

Use of hydrostatic pressure in wood to split stone in Egypt.

Egyptian pyramids, construction management, mega-project. (**Fig. 5**)

Simple arch, two stones leaned against one another. (**Fig. 6**)

Relieving arch, basic understanding of structural principles. (**Fig. 7**)

9.2.3 Iron Age: Beginning before 1000 BC in Western Asia and Egypt

Iron age tools.

7th and 8th Centuries BC: Greeks begin to transition from a timber to stone as the main structural material for constructing civic and religious buildings.

Dressed, post and lintel construction. Refinement of stone carving in Egyptian and Greek buildings. (**Fig. 8**)

c.515 BC: good evidence that the Greeks were using simple cranes. (Coulton, p144)

9.2.4 Roman Period: 210 BC to 427 AD

Dressed arch, large stones, large abutments, semi-circular form. (**Fig. 9**)

Radial and linear arrays of the arch make domes and vaults. (**Fig. 10**)

Expansion of civil infrastructure; water supply = aquaducts. (**Fig. 11**)

9.2.5 Middle Ages: 500 to 1500

Development of novel structural systems to push limits of stone construction; flying buttress. (**Fig. 12**)

Use of wrought iron to reinforce slender stonework medieval construction. (**Fig. 13**)

Quadripartite and Sexpartite Vaults; Sexpartite vault has lower horizontal thrust in plane of wall during construction than a quadripartite vault. (Mark, p163) (**Fig. 14**)

9.2.6 Renaissance: 14th Century Italy to 17th Century

Hydraulic power was probably harnessed to mechanize cutting during this period.

Limits of dome and arch thrust lines pushed. An iron chain was later installed at the base of the dome of St. Peters in Rome to constrain thrust on the cylinder. (**Fig. 15**)

Iron ties used in arch construction, mainly in Italy. (**Fig. 16**)

Fillipo Brunelleschi invents a way to construct a dome that is self-supporting during construction, eliminating costly formwork. (**Fig. 17**)

17th century: Period of increased knowledge and theory created about behavior and design of the arch.

1675 Robert Hooke publishes an anagram in a book unrelated to the arch that goes: "*ut pendet continuum flexile, sic stabit contguum gigidum inversum*, translating into the knowledge that the catenary is the perfect form to carry the same loads applied to a hanging chain in tension as an arch in compression.

1697 David Gregory publishes statement saying: "When an arch of any other figure is supported, it is because in its thickness some catenaria is included." Therefore, it was known that as long as the thrust line lies within the masonry, then the arch will stand.

French engineers began to develop a theory of arches to replace the empirical methods inherited from the past.

1695 Lahire's *Traité de Méchanique* published the funicular polygon used for arch analysis. Used literally in timber construction. (**Appendix A-03, p.A.86, Fig. 10**)

1712 Lahire publishes *Histoire de L'Académie des Sciences*, in which he records his studies of how to determine the proper dimension of pillars.

1729 and 1730: Couplet wrote two memoirs on arch thrust. In the first he repeated some of the work of La Hire and made calculations on the forces imposed by an arch on its centering during construction. In the second, he states precise assumptions about material behavior, noting that the friction of the voussoirs prevents sliding, but does not prevent separation by rotation. He assumes that ambient stresses are low enough to dismiss the importance of crushing strength.

1748 Poleni publishes his *Memoire istoriche*, giving a comprehensive review of the state of knowledge about arch theory. Poleni had been appointed in 1743 to report on the dome of St. Peter's in Rome. He published an image of a hanging chain as the reciprocal form of an arch, earlier suggested by Hooke. (**Fig. 18**) He determined that the thrust line of the arch was safely within the thickness of the dome, though he still recommended that supplementary ties be installed at the base of the dome to restrain the horizontal thrust.

1770 Eglise Ste.-Genevieve built in Paris. Stone structure is heavily reinforced with wrought iron. (**Fig. 19**)

1773 Coulomb publishes a *mémoire* in which he describes how an arch can fail by rotation at the quarter points and middle. (**Fig. 20**) Therefore arch design needs to consider both the relative sliding of the wedges and the possibility of relative rotation. Coulomb does not give definite rules for designing arches, but determines the thrust limits necessary for stability.

1773 J-R. Perronet publishes his study of arch settlement after the centering is removed. (**Fig. 21**)

1791 Perronet builds Pont de la Concorde in Paris. (**Fig. 22**) The low rise of the arch illustrates the cumulative knowledge created about arch design in theory to that time.

9.2.7 End of Massive Stone Construction, c.1905

Last major stone structures built in the early 20th century. The slenderness of the bridges of the Rhätische Railroad of Graubünden, Switzerland reflect influence of graphic statics and development of quality mortar. (Peters[1], p75) (**Fig. 75**)

9.2.8 Contemporary use of Structural Stone

Stone is typically used in prestressed structures subject to high axial compressive forces. (**Figs. 24, 25 and 26**)

Fig. 1: Stone beam bridge. (Sealy)

Fig. 2: Dolman Lanyon Quoit at Cornwall, England. (Stonepages)

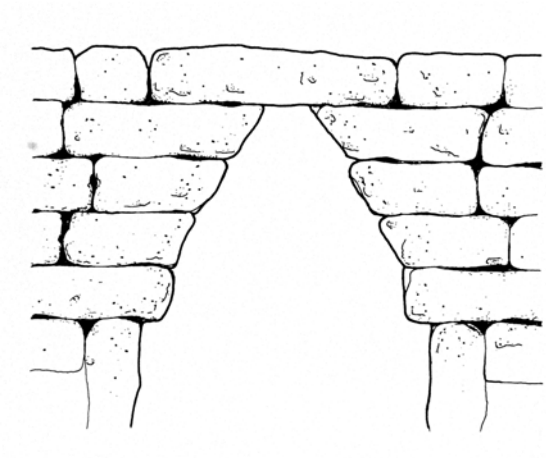


Fig. 3: Corbelled arch. (Mark)

Fig. 4: Stonehenge, 3100 BC to 2100 BC. (Stonepages)

Fig. 5: The Great Pyramid of Cheops, Giza, 2566 bc. (touregypt)

Fig. 6: Simple arch of two stones leaning against one another, north entrance to the Pyramid of Cheops, Giza. (Mainstone)

Fig. 7: Lion’s Gate, Mycenae, c.1300 BC. (Dinsmoor)

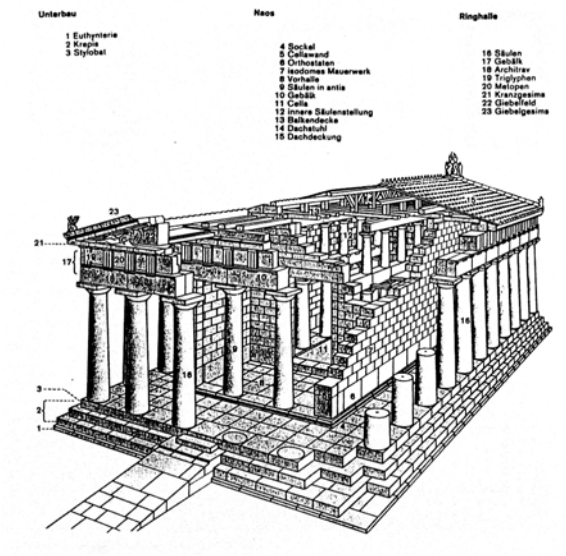


Fig. 8: Greek temple. (Hoffmann)

Fig. 9: Voussoir arch. Porta di Giove, Farlerii Nove. 3rd century bc. (Mainstone)

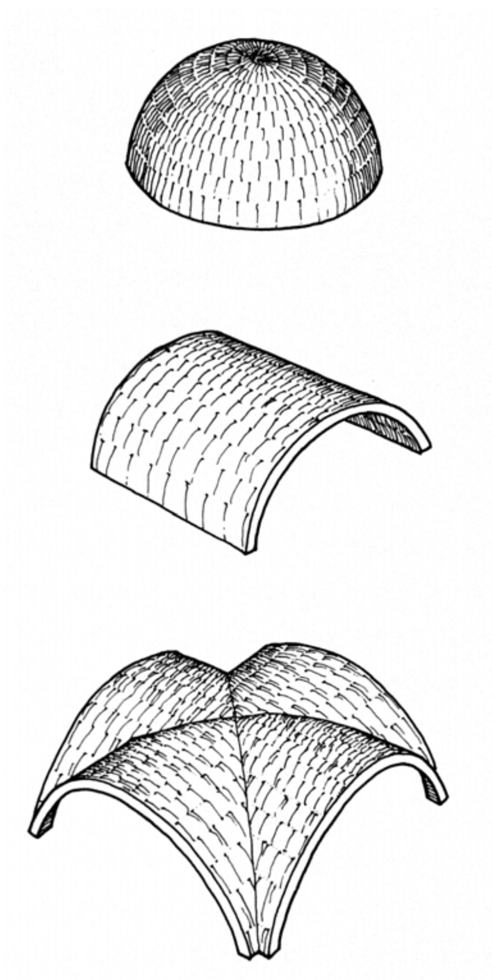


Fig. 10: Domes and vaults. (Mark)

Fig. 11: Pont du Gard, France. Late first century BC.

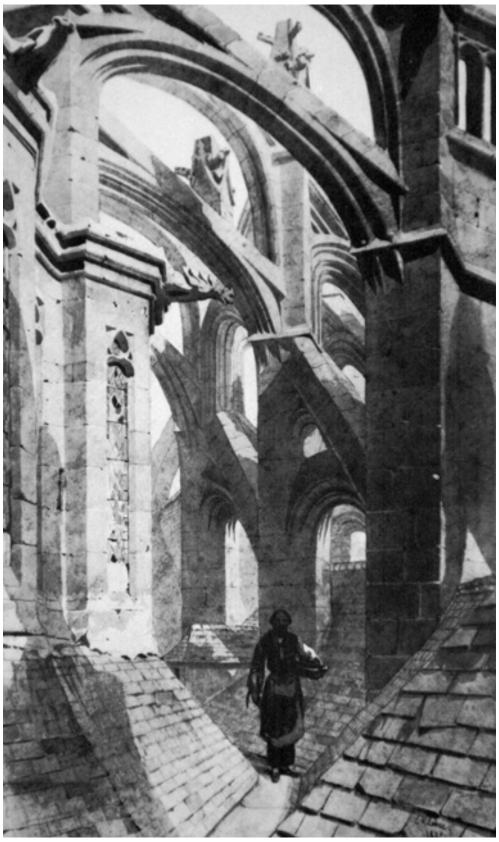


Fig. 12: Flying Buttress. (Recht)

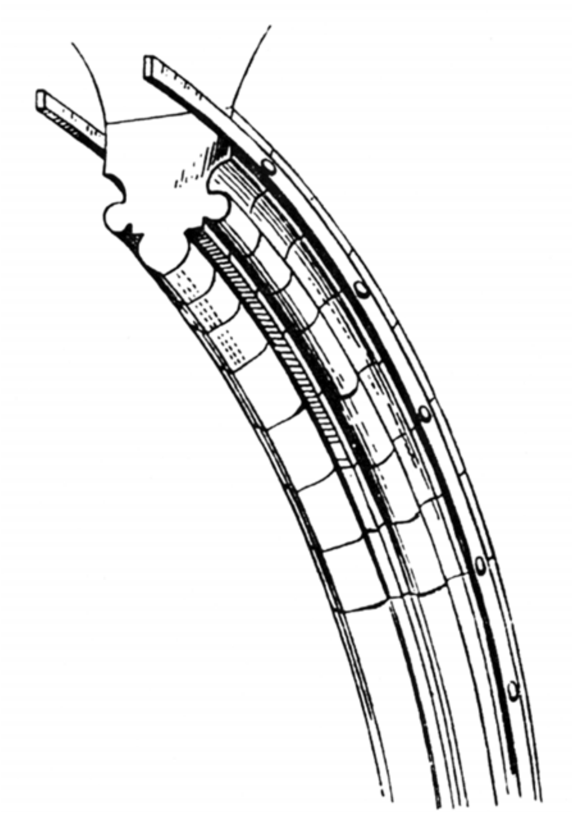


Fig. 13: Iron reinforced arch, Saint Chapelle, Paris, 1247. (Hamilton)

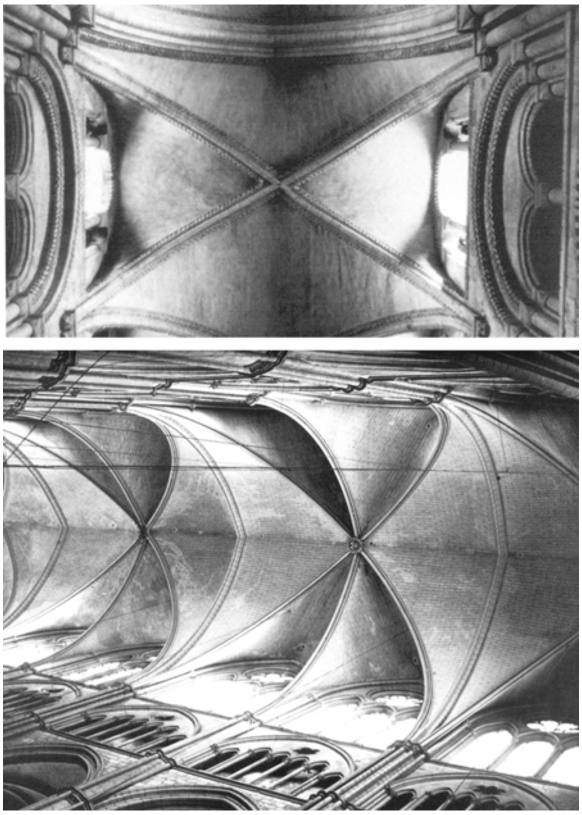


Fig. 14: Quadripartite and Sexpartite Vaults. (Mark)

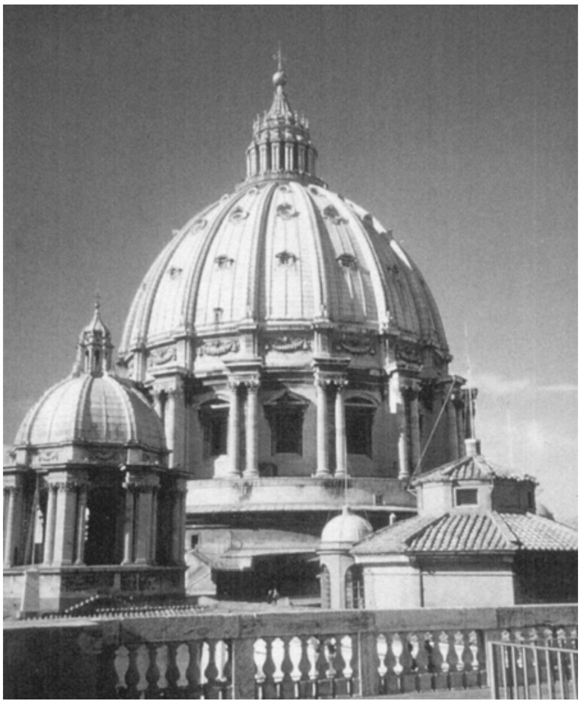


Fig. 15: St. Peters, Rome.

Fig. 16: Wrought-iron tied arch. (Gärtner)

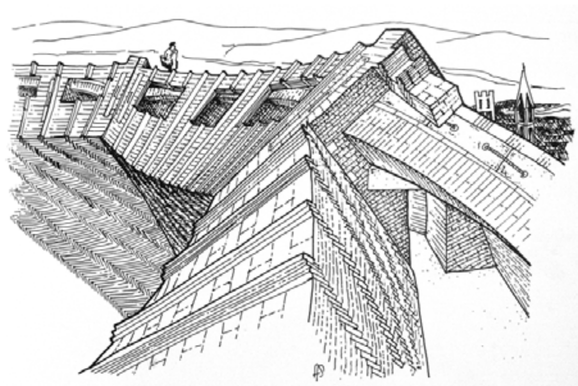


Fig. 17: Duomo, Florence. General view and construction detail. Brunellesxhi. (Gärtner; Mark)

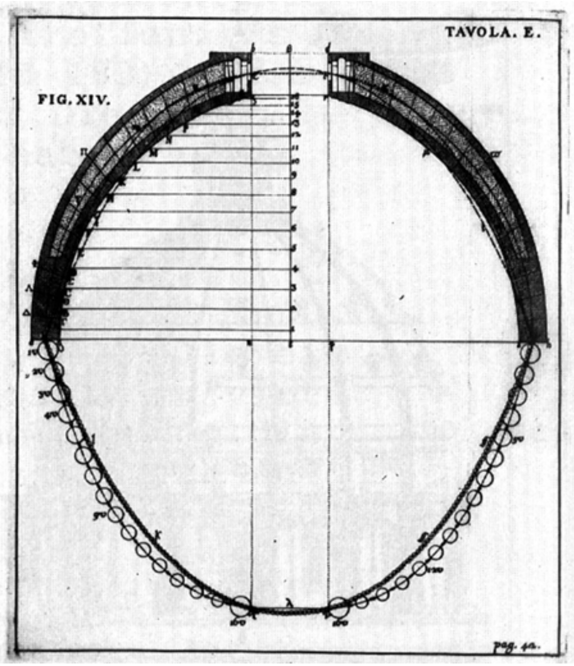


Fig. 18 : Poleni, St. Peters analysis, 1743. (Mislin)

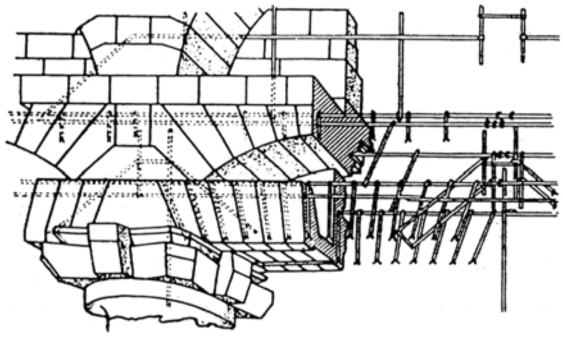


Fig. 19: Iron-reinforced stonework of Ste.-Geneviève, Paris, 1770.

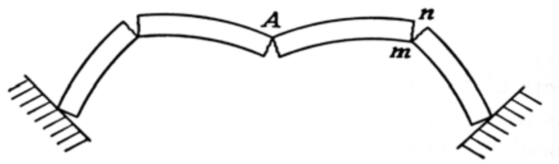


Fig. 20: Rotation failure of arch studied by Coulomb, 1773. (Timoshenko)

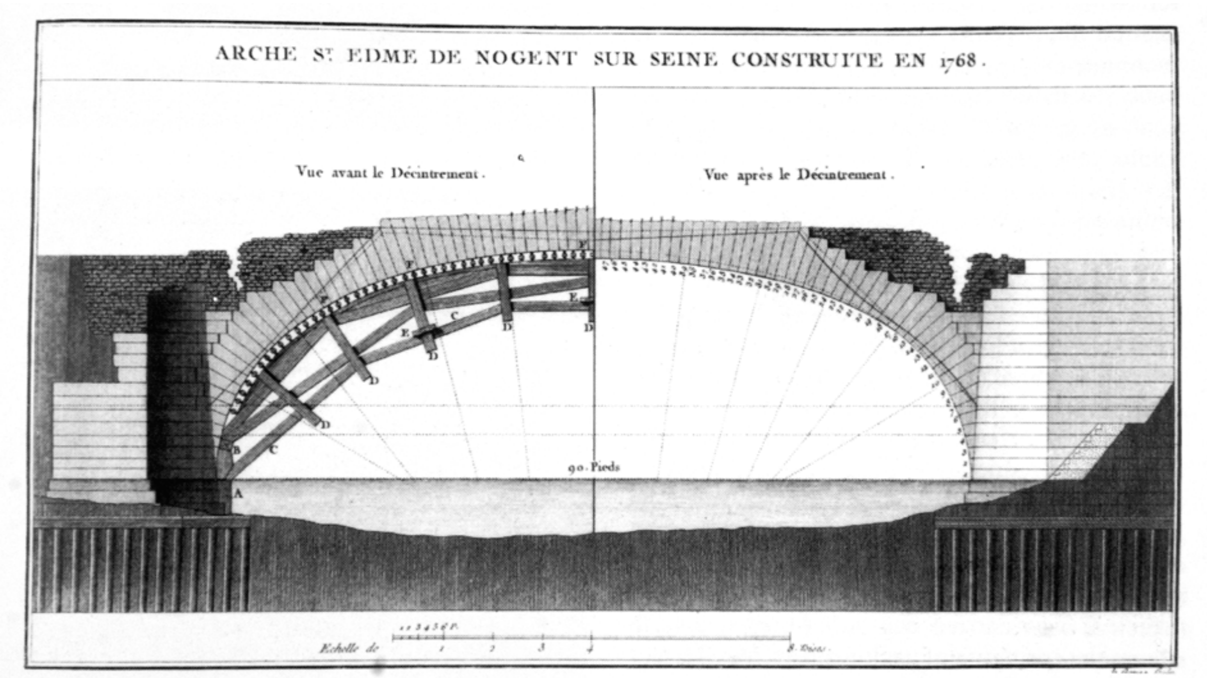


Fig. 21: Deformations of an arch on removal of the centering. Jean-Rondolphe Perronet, 1773. (Mainstone)

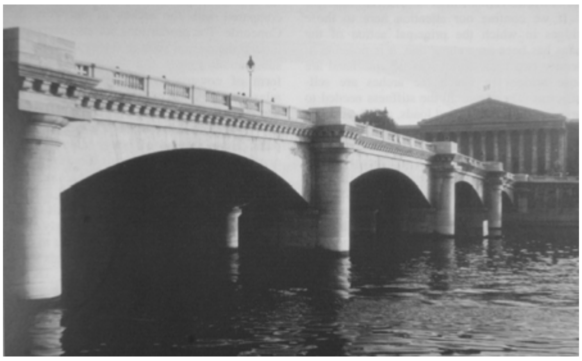


Fig. 22: Pont de la Concorde, Paris. Jean-Rondolphe Perronet, 1791. (Mainstone)

Fig. 23: Stone viaducts built at the turn of the twentieth century. Langwasser Viaduct (1902) and the Wiesener Viaduct (1908). These are among the slenderest constructions of their type. Both are located in Graubünden, Switzerland.

Fig. 24: Prestressed stone arch of the Pavilion of the Future, Seville. David Mackay and Peter Rice, 1992. (Rice)

Fig. 25: Stone mast of a cable supported pedestrian bridge in Bad Homburg, Germany. Schlaich Bergermann & Partner, 2002. (Russell)

Fig. 26: Stressed ribbon bridge using stone and stainless steel prestressing bands. Suransuns Bridge, Graubünden, Switzerland. Jürg Conzett, 1999. (Dooley)

9.3 Iron, 1556-1854

1556 Georgius Agricola's *De Re Metallicca* is post-humously published. Probably the first book to describe methods and processes of iron making. No other comparable reference was published for two hundred years. (Goodale, p40)

1558 Queen Elizabeth issues decree in England limiting the use of timber for fuel in iron manufacture because of strains on the timber supply for the navy. The growth of the industry had led to severe deforestation. (Goodale, p40)

1606 Waterpower used to cut iron into nail rods in England. (Goodale, p42)

1613 Rovenson invents reverberatory furnace. The oldest record of the cementation process, by which iron is converted into steel, exists in the form a patent granted to two Belgian arms makers. (Goodale, p42)

1615 and 1617 Faustus Verantius publishes a chain bridge proposal in his book *Machinae novae*. (**Appendix A-02, p.A.34, Fig. 10**)

1619 Dud Dudley of Worchester, England obtains patent from King James for a method of smelting iron using pit or sea coal. Dudley's invention was not commercially successful because of punitive measures directed at him by an organization of charcoal iron masters opposed to his invention. Dudley could produce up to 7 tons/week. [assume 1 furnace] (Goodale, p43)

1627 Gunpowder used in mining by Caspar Weindle in Hungary. First used in England around 1700. (Goodale, p44)

1638 Galileo Galilei's *Two New Sciences* is published. He presents first known published theory of stress distribution in a cantilevered beam. Galileo incorrectly concluded that stress is distributed evenly across the section of a beam. (Timoshenko, p12.)

1651 English first in Western world to substitute coke (a product of coal) for charcoal (a product of timber) to fuel smelting operations. Coke had been used for far longer in China. First patent issued to Jeremiah Buck. (Goodale, p46)

1660 Robert Boyle, an Englishman, discovers Boyle's Law of gases, which says that volume is inversely proportional to pressure. This knowledge was useful to the most pressing technological problem of his time, the chemistry of combustion. (Goodale, p47)

1665 Dud Dudley publishes *Metallum Martis*, in which he describes his process for using pit coal to manufacture iron. (Goodale, p47)

c1666 Wire drawn form strips of iron sheared from rolled charcoal-iron plates. (Goodale, p48)

1667 Kircher publishes book in Europe with illustration of an the Lan Jing Bridge, an iron-link chain bridge in Yunnan, China. This image was reproduced in an 1800 travelogue by Turner. The Lan Jing Bridge dates from 65 AD, however it clearly had little influence on Western developments before publication of Kircher's book. (**Appendix-02, p.A.32, Fig. 5**)

1678 Robert Hook publishes a paper entitled *De Potentiâ Restitutiva* containing 'Hooke's Law' relating the magnitude of forces and the deformation they produce.

1686 Mariotte's theory of stress in a beam is published post-humously by M. de la Hire. Based on various bending and tensile strength tests he concludes that stress is distributed over the section, with compression in the upper fibers and tension in the lower. Mariotte made an error in his mathematics that prevented him from accurately calculating the magnitude of stress. He also erroneously assumed that tensile and compressive strength were equal for all materials – an error later repeated by Thomas Tredgold.

1698 Thomas Savery invents steam engine, introduces power measurement of horsepower. (Goodale, p50)

1712 First Newcomen engine built. Used to pump water from mining pits. Mining was one industry that could not accommodate the limitations of waterpower. Textile mills could be located anywhere, but minerals lay were they lay, and if there is no convenient water source to provide power other means had to be employed. (Basalla, p149)

Blast furnace in New England could produce 14.5 tons/week of charcoal iron. (p51)

1729 Peter van Musschenbroek, a Dutch physicist, publishes first accurate results on the strength of iron. (Goodale, p55)

1734 Swedenborg publishes *Opera Philosophica et Mineralia*, which is considered the first textbook on iron mineralogy.

1735 Abraham Darby first uses coke in a blast furnace.

1740 Benjamin Huntsman, a Quaker clockmaker, develops a method for producing better quality steel which entails re-melting the steel. (Goodale, p57-58)

1741 Winch Catwalk, a short, iron-link chain bridge first built in County Durham, England.

1742 A Pennsylvania blast furnace was producing 35-40 tons/week of charcoal iron.

1754 Henry Cort builds first iron rolling mill. (Goodale, p61)

John Wilkinson, of Wales, first to build a coke blast furnace for commercial production.

1760 John Smeaton invents the first large cast-iron blowing cylinders with tight-fitting pistons, constituting an important improvement over the bellows for use in the blast furnace. (Goodale, p62)

1769 James Watt improves the steam engine. (Goodale, p65)

1770 Use as reinforcement of stone structure of Eglise de Sainte-Geneviève, Paris. Jacques-Germain Soufflot and Jean-Baptiste Rondolet. (**Appendix A-03, p.A.95, Fig. 20**)

1773 C.A. Coulomb publishes paper titled *Sur une Application des Régles de maximis et minimis à quelques problèmes de statique relatifs à l'architecture* in which he corrects

Marriotte's error of assuming materials to be equally strong in tension and compression, and, in general, greatly advances the state of mechanics of materials.

1779 First known all-cast-iron bridge in the world completed by Abraham Darby Jr. The bridge spans the Severn River at Coalbrookdale, England. The semi-circular arch has a span of 102 ft. (Goodale, p68) (**Appendix A-02, p.A.41, Fig. 17**)

1781 Torbern Bergman, of Sweden, is the first to define the difference between wrought iron, steel, cast iron by variations in the percentage of carbon present; wrought iron containing from 0.05% to 0.20%, steel 0.20% to 0.80%, and cast iron form 1% to 3%. (Goodale, p71)

Jacques-Germain Soufflot, a Paris architect, made many investigations into the strength of building materials, including iron and steel. (Goodale, p72)

1782 von Sickingen, of Sweden, publishes some of the earliest figures on strength of materials. (Goodale, p72)

1783 Henry Cort takes first patent in England for grooved rolls. (Goodale, p73)

1784 Henry Cort patents puddling process using a reverberatory furnace with a sand bottom for converting pig iron into wrought iron. (Goodale, p73)

1785 As early as this date, the American Oliver Evans conceives of ideas for steam powered boats and wagons. (Goodale, p73)

1786 Victor Louis designs wrought-iron roof structure of the Théâtre Français, Paris. (**Appendix A-03, p.A.95, Fig. 21**)

1789 Wrought-iron roof structure erected over salon of the Grande Galerie in the Louvre, Paris. Auguste Rénard. (**Appendix A-03, p.A.95, Fig. 22**)

1793 William Strutt uses cast-iron columns in his textile mill in Milford England. (**Appendix A-03, p.A.96, Fig. 23**)

1795 Thomas Telford builds the Buildwas Bridge over the River Severn in England. This bridge has two super-imposed cast-iron arches. (**Appendix A-02, p.A.42, Fig. 18**)

John Nash uses hollow-box cast-iron voussoirs cast in the shape of elongated stone ones, bolted together at adjoining faces to form a stiff, cellular arch.

1796 Thomas Telford completes the Longdon Viaduct, and all-cast-iron structure with tree-like arrangement of compression struts for the piers (evocative of timber design at the time) and a canal trough made of cast-iron plates. (**Appendix A-02, p.A.44, Fig. 20**)

Thomas Wilson completes the Sunderland Bridge using cast-iron voussoir blocks first developed by Thomas Paine. Wrought-iron straps held the blocks together.

1797 Charles Bage uses cast-iron for beams and columns on interior of brick exterior walls of textile mill in Shrewsbury, England. (**Appendix A-03, p.A.97, Fig. 24**)

1798 David Mushet, of Glasgow, patents an improved method of casting steel. His discovery is predated by its use in India. (Goodale, p79)

1801 James Finley builds the first of about forty bridges using a patented iron chain-link suspension design. Unlike the winch catwalk, Finley's bridge had a timber stiffening truss that doubled as handrails. (**Appendix A-02, p.A.36, Fig. 12**)

1804 L.A. de Cessart and J.V.M.L. Dillon complete the wrought-iron Pont des Arts over the Seine in Paris.

1805 Pont Cysyllte Viaduct completed. Cast-iron arches cast in two halves support a cast-iron trough composed of plates connected at diagonal, voussoir-like, joints with bolts. Piers are stone. (**Appendix A-02, p.A.44, Fig. 21**)

1806 M.C. Lamandé builds the Pont d'Austerlitz with cast-iron blocks derived from the Sunderland Bridge.

1808 First all-wrought-iron bridge, the Pont sur la Crou, is built near Saint-Denis, France, by Louis Bruyère. (**Appendix A-03, p.A.108, Fig. 37**)

1810 Thomas Telford builds the Craigellachie Bridge, a cast-iron arch composed of cast-iron arch segments detailed as five panels of latticed voussoirs. The lattice form lightens the structure and combining five 'voussoirs' together into one pre-cast unit greatly simplifies construction. The arch segments were bolted together. (**Appendix A-02, p.A.44, Fig. 19**)

US pig iron production: 53,000 tons (p86)

1815 Southwark cast-iron arch bridge built over Thames River (Goodale, p91) John Rennie builds the Southwark Bridge. (**Thesis main body, p141, Fig. 5.13**)

1817 First steamboats on Great Lake come into service. [important because they transported iron ore from west to iron making centers in the east of the country. (Goodale, p93-94)

1818 William Fairbairn improves mill gearings, creating a revolutionary change in power transmission systems. (Goodale, p94-95)

Samuel Brown completes the Union Bridge in England with welded eye-bar chains. (**Appendix A-02, p.A.37 Fig. 13 and p.A.60, Fig. 29**)

The first all-iron ship, the Aaron Manby, was built in Scotland.

1819 Between 1800 and 1819, a rolling mill in France produced the first wrought-iron angles.

1820 US production of pig iron: 20,000 tons (low figure partly attributed to depressed economy. (Goodale, p95-59)

Ithial Town patents his lattice truss. First produced in timber. His success led to many other patent designs, including those by Haupt, Long and Burr. (**Appendix A-03, p.A.86-A.89, Figs. 12-15**)

Alphonse Duleau discovers the benefits of the I-form in comparison with a solid rectangular form.

1822 Duleau investigates strength of wrought iron. His work, and that of Tredgold in 1824, and Peter Barlow in 1817, are first systematic investigations of cast and wrought iron. (Goodale, p98)

1824 Thomas Tredgold proposes a cast-iron beam with an **I**-section. The flanges are symmetrical and the thickness of the flanges and web are equal. (**Appendix A-03, p.A.100, Fig. 30(a)**)

1825 First railroad in the world opens, the Stockton and Darlington RR. It was 37 miles long. (Goodale, p101)

1836 C.L.M.H. Navier publishes *Leçons…*, in which he correctly places the neutral axis at the center of gravity of a beam under elastic bending stress.

1828 James Neilson, of Glasgow, invents the hot blast. It is first used at the Clyde Iron Works in 1829. Until the introduction of the hot blast, the avg. output of a blast furnace would not be over 24 to 30 tons/week. (Goodale, p104)

1830 US production of pig iron: 165,500 tons (Goodale, p106)

Eaton Hodgkinson designs an 'ideal' beam section for cast iron based on extensive material strength testing. (**Appendix A-03, p.A.100, Fig. 30(b)**)

1831 Robert Stevens invents the American **T**-Rail. (**Appendix A-03, p.A.98, Fig. 26(d)**)

1837 William Fairbairn builds a riveting machine. With this machine, two men and one boy could do in one hour what formerly took three men and a boy twelve hours. Fairbairn was motivated to make such a machine after a strike of boilermakers. (Goodale, p116)

1839 William Fairbairn develops a built-up wrought-iron **I**-section with asymmetrical flanges similar to the proportions of Eaton Hodgkinson's 'Ideal' cast-iron beam section. (**Appendix A-03, p.A.99, Fig. 28**)

1840 286,903 long tons of pig iron produced in US. (Goodale, p119)

Blast furnace in Pennsylvania could produce 50 tons/week. (Goodale, p120)

1841 Squire Whipple patents an all-iron bowstring truss in America.

1844 Kennedy and Vernon patent the bulb-tee for shipbuilding. (**Appendix A-03, p.A.99, Fig. 29**)

c.1845 Around this time, cast-iron girders in Britain started to be reinforced with wrought iron. (**Appendix A-03, p.A.102-A.105, Figs. 33-35**)

1847 Failure of the Dee Bridge… wrought iron trussed cast-iron girder bridge. (Goodale, p72-73)

c.1847 Squire Whipple writes *A Work on Bridge Building…*, in which he presented an early mathematical method of designing trusses.

1848 Richard Turner completes the Palm House at Kew Gardens. The structure is wrought iron except for the columns and their brackets. (Peters[2], p218)

1849 Ferdinand Zorés rolls the first wrought-iron **I**-beam.

1850 Max production of a US blast furnace was 150 tons/week. (Goodale, p131)

The Britannia Bridge is completed by Robert Stephenson, with the aid of William Fairbairn and Eaton Hodgkinson. Its all-wrought-iron tubular form led to the development of the plate girder by William Fairbairn and marks the transition from cast-iron to wrought iron as the preeminent structural material in the West.

1854 World production of pig iron estimated at 6 million tons. (Goodale, p139)

9.4 Aluminum, 1808-Present

Unless otherwise noted, information is from Richards.

1760 Existence of aluminum first determined by Baron, a chemistry professor in Paris.

1807 Humphrey Davy unsuccessfully tries to use a battery to isolate aluminum as he had potassium and sodium. He is about 70 years ahead of his time trying to use electrolytic reduction to produce aluminum.

1827 Aluminum is first isolated by Frederick Wöhler, a professor of chemistry at the University of Göttingen, Germany, however he could not obtain pure samples to determine its properties. (**Appendix A-04, p.A.158, Fig. 12**)

1854 H. Sainte-Claire Deville, a professor of chemistry at the Ecôle Normale in Paris, isolates almost perfectly pure aluminum using potassium as a reducing agent and establishes its properties. Devotes himself to the development of an aluminum industry using a chemical process of reduction because the cost of the batteries necessary for electrolytic reduction was too high.

Deville turns to sodium for reducing aluminum because it is less expensive than potassium and less material is required. Deville helps to refine the industrial process of making sodium, reducing the cost from FFr. 2000/kilo in 1855 to FFr. 10/kilo in 1859.

Deville presents the Emperor Napoleon III with a medal of aluminum in August 1854. Napoleon authorized experiments to continue at his own expense on a large scale because of the possible military applications of the metal.

1855 Deville produces large bars of the metal. Aluminum samples and some products of were exhibited at the Paris Exposition of 1955 in the Palais d'Industrie.

1855 Cost of aluminum, FFr. 900/kilo .

1856 Cost of aluminum, FFr. 300 /kilo ($60/kilo).

1856 Baby rattle designed by Charles Rambert for Napoleon III's son. The first article made of aluminum. (Nichols, p192) (**Appendix A-04, p.A.163, Fig. 14**)

1858 Stand of chased and gilded aluminum, designed by Charles Christofle. (**Fig 27**)

1859 First aluminum works in England started.

1860 Bracelet, Armand Dufet. (**Fig. 28**)

1866 Fan, aluminum, silk and paper. August Edouard Achille Luce. (**Fig. 29**)

1867 Sheet, wire, and foil products are produced.

Aluminum bronze alloy first appears.

1878 Several reasons why the metal is not used by mathematical instrument makers and others is: the price; the methods of working it are not everywhere known; no one knows how to cast it.

1878 Cost of aluminum, FFr. 130/ kilo; Cost of aluminum-bronze, FFr. 18/kilo with 10% aluminum.

1883 Walter Weldon proposes following measures to reduce the cost of aluminum:

1. cheapen the production of aluminum chloride, or of aluminum-sodium chloride.
2. Substitute these chlorides for some other cheaper anhydrous components of aluminum not containing oxygen
3. cheapen sodium
4. replace sodium with cheaper reducing agent.

[price of bauxite, the raw material of aluminum was literally dirt cheap]

Weldon stated the relative cost of the materials used to make aluminum as follows:

- Producing alumina: 10%
- Producing double chloride: 33%
- Producing sodium and reducing therewith 57%

1884 The Washington Monument is capped with a 2.8 kg (100 oz.) pyramidal casting of pure aluminum, the largest such casting to that date. William Frishmuth, of Philadelphia, produced it. (**Fig. 30**)

1886 H.Y. Caster, of New York City, invents method of producing sodium helps lower cost of sodium from $1/pound to about 20 or 25 cents.

1886 Ed. Kleiner, of Zurich, develops electrolytic process of making aluminum, made possible by recent developments in the dynamo. A works was set up at the Rhein waterfall at Neuhausen, which could produce 15,000 horsepower.

Charles M. Hall applies for a patent in the US using a novel electrolytic process.

1887 Paul T. Heroult independently develops a process for making alloyed-aluminum. His method is used to improve the works at Neuhausen, generating great success. Using the economy of the waterpower, the Neuhausen works was able to produce aluminum for FFr. 4.5/kilo ($0.50/pound), which meant that they could sell below the market cost.

1888 Herault's process is improved by Kiliani, so that pure aluminum can be made. The process is essentially the same as Hall's.

1889 Sculpture, *Venus de Milo*, F. Barbedienne Foundry. Cast aluminum.

1889 Charles M. Hall, of Oberlin, Ohio, patents (he applied in 1886) novel electrolytic process for producing aluminum that becomes the standard of the industry. His invention was used to form the Pittsburgh Reduction Company, which would later become Alcoa.

1889 Cost of aluminum was $4.50/pound from the Pittsburgh Reduction Company.

1889 Paris Exhibition: alloy of aluminum bronze, aluminum brass, ferro-aluminum shown. Products included a bell, springs, statutes, aluminum plate, round and square tubes, wire, sheet, etc.

c1889 First aluminum boats produced by Swiss company Escher-Wyss. (**Appendix A-04, p.A.164, Fig. 17**) The largest boat built prior to 1897 was a French torpedo boat, not built by Escher-Wyss, that was 18.3 m (60 ft) long.

1890 Cost of aluminum was $2/ pound after the Pittsburgh moved and enlarged.

1892 Cost of aluminum was $0.50/pound after the Pittsburgh works was enlarged again.

1892 Elevator enclosures, staircase, Monadnock Building, Chicago. Burnham and Root, architects. (**Fig. 31**)

Cost statistics, 1956-1995:

date	place	FFr. per kilo	$ per pound
1856 (spring)	Paris	1000	90.90
1856 (August)	"	300	27.27
1859	"	200	17.27
1862	"	130	11.75
1862	Newcastle (UK)		11.75
1878	Paris	130	11.75
1886	"		12.00
1887	Bremen		8.00
1888	London		4.84
1889	Pittsburgh		2.00
1891	"		1.50
1892	"		1.00
1893	"		0.75
1894	"		0.50
1895	Switzerland		0.35

1894 and 1897 Two airships were built by David Schwartz, an Austrian, and Carl Berg, a German aluminum manufacturer. These airships had aluminum frames and were clad by thin aluminum sheeting. They first ship was destroyed before flight trials and the second ship crashed during its first flight trial. The Schwartz airship was 38.3 m long and the largest diameter of the hull was 18.2 m. (**Appendix A-04, p.A.192, Fig. 20**)

1895 Used widely in cooking ware.

1900 Ferdinand von Zeppelin, a German aristocrat, constructs the first successful rigid-type airship. It was 147 m long and the hull had a diameter of 11.65 m. The airframe was of aluminum and the ship was clad in cotton cloth. (**Appendix A-04, p.A.196, Fig. 21 and p.A.197, Fig. 22**)

1903 The Orville and Wilbur Wright use an aluminum motor block in their Wright Flyer, the first airplane to achieve controlled flight. (Nichols, p23)

1909 Alfred Wilm invents duraluminum, a high-strength aluminum alloy used to build Zeppelin airships and was instrumental in aluminum's application to heavier-than-air craft as well. (DRP 244 5554, 10 March 1909 / 09 March 1912) (Nichols, p23)

1914 Giuseppe Merosi designs the 40/60 Aerodynamica, a car made by Alfa Romeo with a futuristic, streamlined aluminum body.

1916 US Navy approaches Alcoa to develop a strong alloy for use in airships, resulting in a strongly directed, systematic research and development program to begin at that company.

1919 Hugo Junkers, a German airplane manufacturer, constructs a five-passenger airplane constructed entirely of duralumin. Duralumin proved to be problematic over time because under cyclic stress conditions it becomes brittle. (Nichols, p23)

1920s US military subsidizes experimental research into aluminum alloys. This research resulted in the development of aluminum alloys better suited to aeronautic applications. Among the earliest planes built with these new alloys was the Boeing 247, built in 1931. Production of the Douglas DC-3, one of the most successful commercial airplanes in history, began in 1934. (**Fig. 32**)

1929 Buckminsiter Fuller publishes plans for a low-cost, housing system called the Dymaxon House. Between 1941 and 1946 he refined his design using aluminum alloys and incorporating construction practices used in airplane construction. He partnered with the Beech Aircraft Company to produce the houses but the endeavor failed after only one prototype was built. (**Fig. 33**) (Nichols, p240)

1936 The Airstream Clipper trailer went on the market. It was an airplane construction inspired monocoque aluminum body. (**Fig. 34**)

1936 The Hindenburg, the largest Zeppelin rigid-type airship, was built. It was 245 m long and had a diameter of 41.2 m. It had a gas volume of 200,000 m^3. The Hindenburg was destroyed by fire while landing at Lakehurst, New Jersey in 1937. This event effectively ended the airship era and airplanes became the indisputable preference for aviation. (**Fig. 35**)

c.1950 Prefabricated housing for developing country. House illustrated is in Jamaica. (**Main body of thesis, p39, Fig. 3.1**)

1950 The Arvida Bridge, an all-aluminum arch structure, is built in Quebec, Canada. The span of the arch is 91.5 m. Aluminum was chosen for its resistance to corrosion. (**Fig. 36**)

1951 The Dome of Discovery, an aluminum, geodesic lattice dome with a diameter of 111 m (365 ft), was built in London. The engineering firm Tubbs, Powell & Moya designed it. (**Fig. 37**)

1953 An aluminum monocoque dome with 100 m diameter is built at Longview, Texas. It was designed by the R.G. LeTourneau Company. (**Main body of thesis, p78, Fig. 4.26**)

1954 French technologist Jean Prouvé constructed the Pavillon de l'Aluminium with sheet aluminum girders and cast aluminum fasteners. (**Fig. 38**)

1955 The De Havilland Avion Hanger is built suing a trussed aluminum frame structure that was 45 ft high and spanned 200 ft. This structure was 1/7th the weight of an equivalent steel structure. (**Fig. 39**)

1956 First aluminum can introduced by Kaiser Aluminum.(**Fig. 40**) (Nichols, p153)

1998 Media Center at Lord's Cricket Grounds, monocoque structure. Shipbuilders were contracted to fabricate the structure. (**Fig. 41**)

2000 International Space Station is structured around an aluminum truss. The truss had to be manufactured in modular units that could fit in the cargo bay of a US relaunchable space shuttle, explaining why it is wider than it is tall. Each section is bolted to the next using only four bolts. The transversal components of the truss are machined from a single piece of aluminum weighing 5,450 kg (12,000 pounds) and measuring 3.4 m x 4.3 m (11 x 14 ft) and is 115 mm (4.5 in) thick. Each of these components costs several hundred thousand dollars and takes up to a week to machine. (**Appendix A-04, p.A.161, Fig. 13**)

Fig. 27: Stand designed by Charles Christofle, 1858. (Nichols)

Fig. 28: Bracelet designed by Aramand Dufet, 1860 (Nichols)

Fig. 29: Fan designed by August Edouard Achille Luce, 1866. (Nichols)

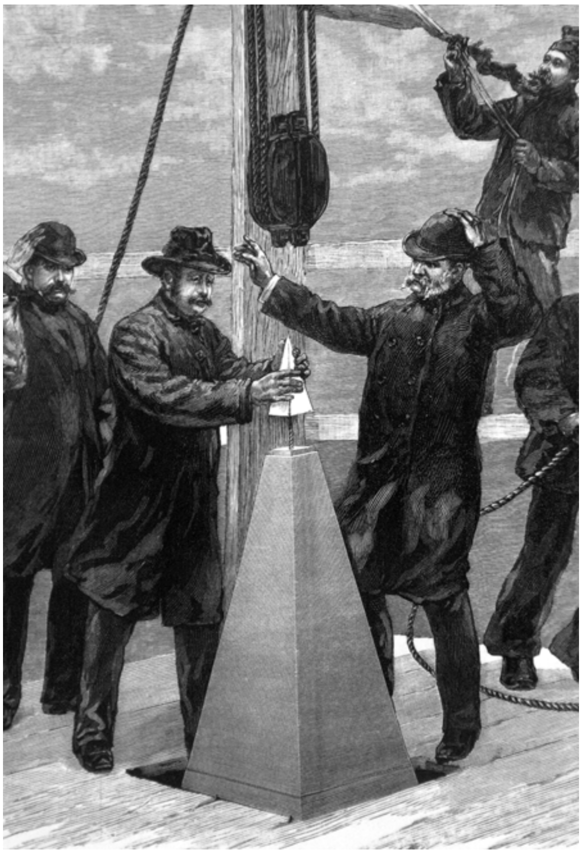


Fig. 30: 100 oz. pyramidal casting capping the Washington Monument, 1884. (Nichols)

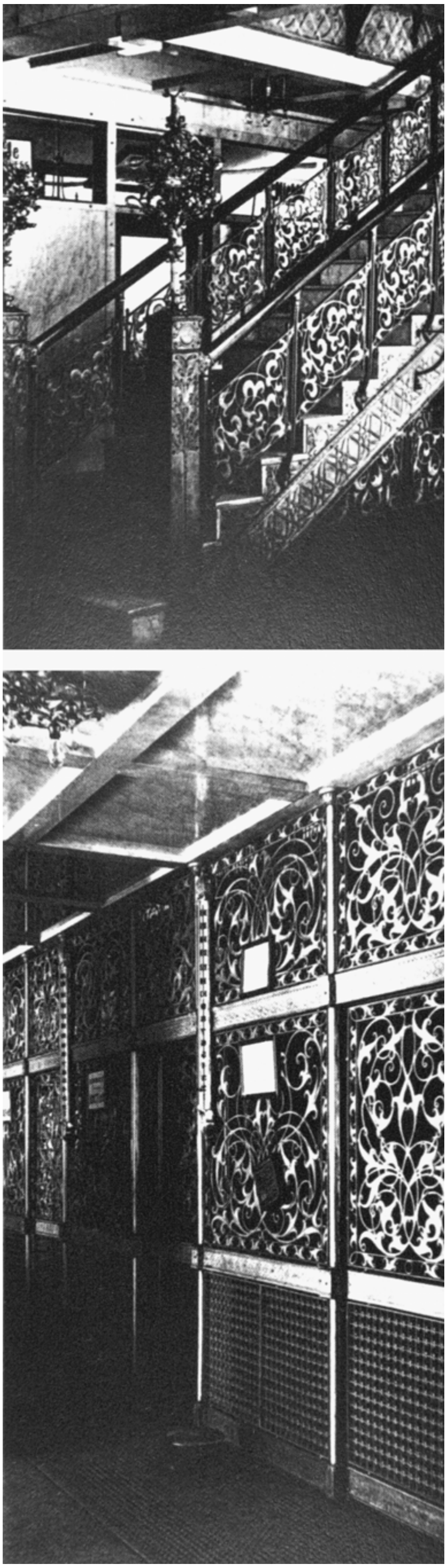


Fig. 31: Aluminum staircase and elevator enclosure, Monadnock Building, Chicago, 1892. (Nichols)

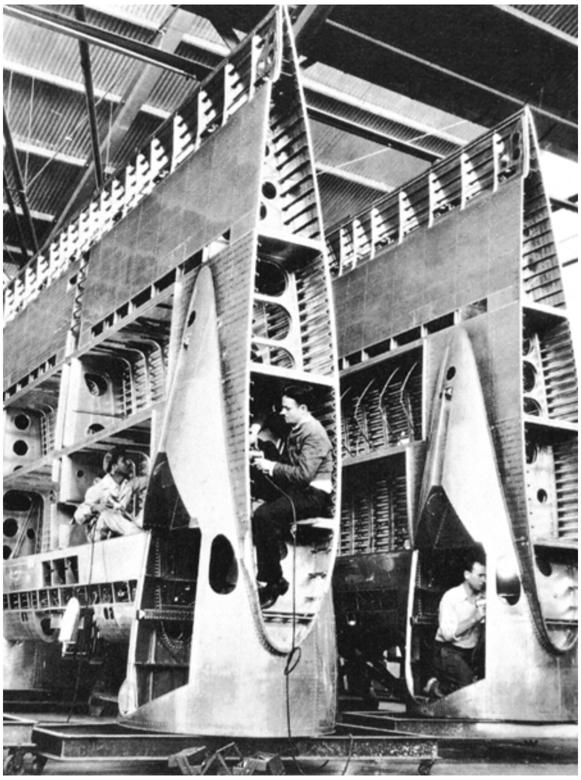


Fig. 32: Douglas DC-3 wing, 1934. (Nichols)

Fig. 33: Wichita ‘Dymaxion’ House, Buckminster Fuller. (Nichols)

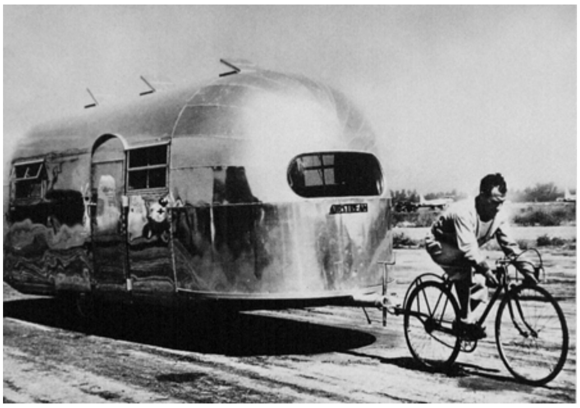


Fig. 34: Airstream Clipper mobile home trailer, 1836.
(Nichols)

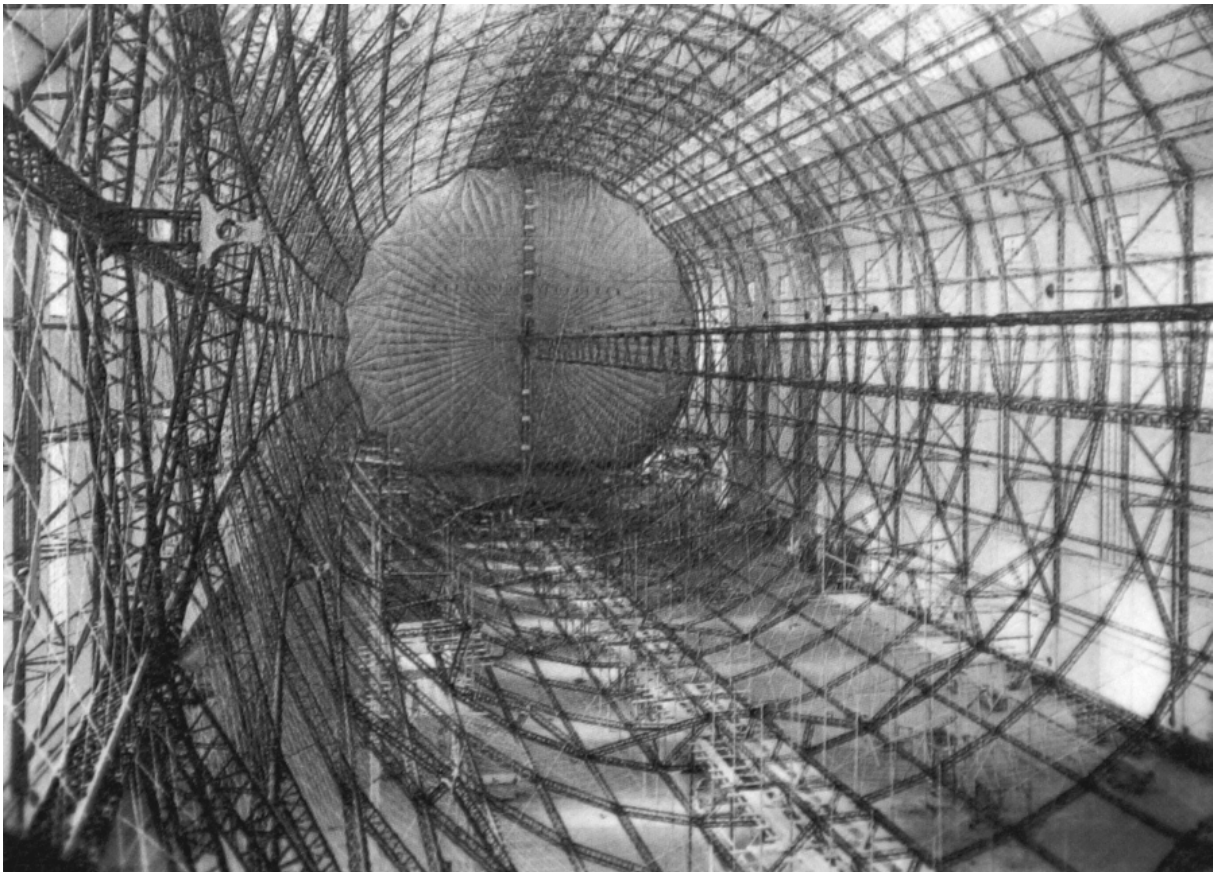


Fig. 35: Interior of the Hindenburg airframe, 1936. (Nichols)

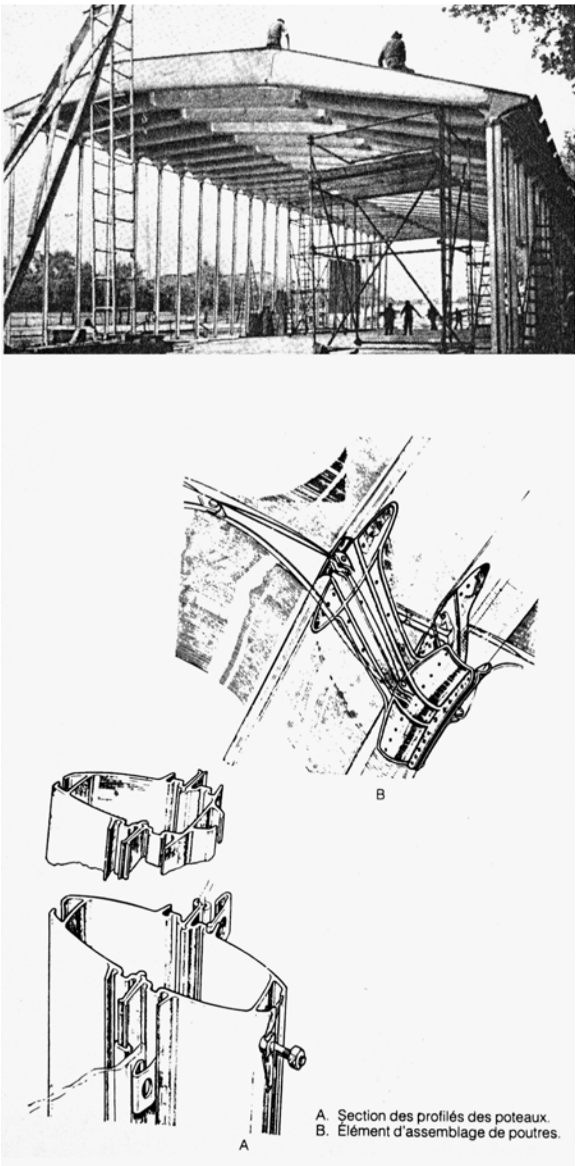


Fig. 38: Construction detail of Jean Prouvé’s Pavillon de l’Aluminium, 1954.

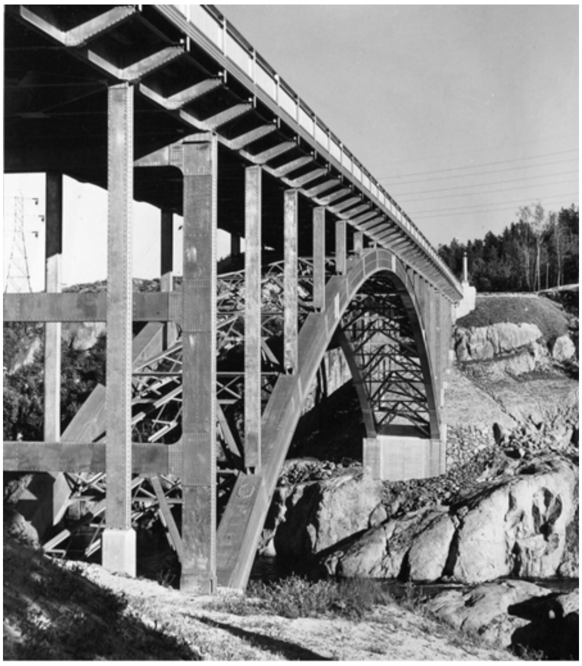


Fig. 36: The Arvida Bridge, Quebec. All-aluminum arch, span = 91.5m. 1950. (Dominion Bridge)

Fig. 37: The Dome of Discovery, London. Diameter = 110 m. Tubbs, Powell & Moya, 1951. (Peter)

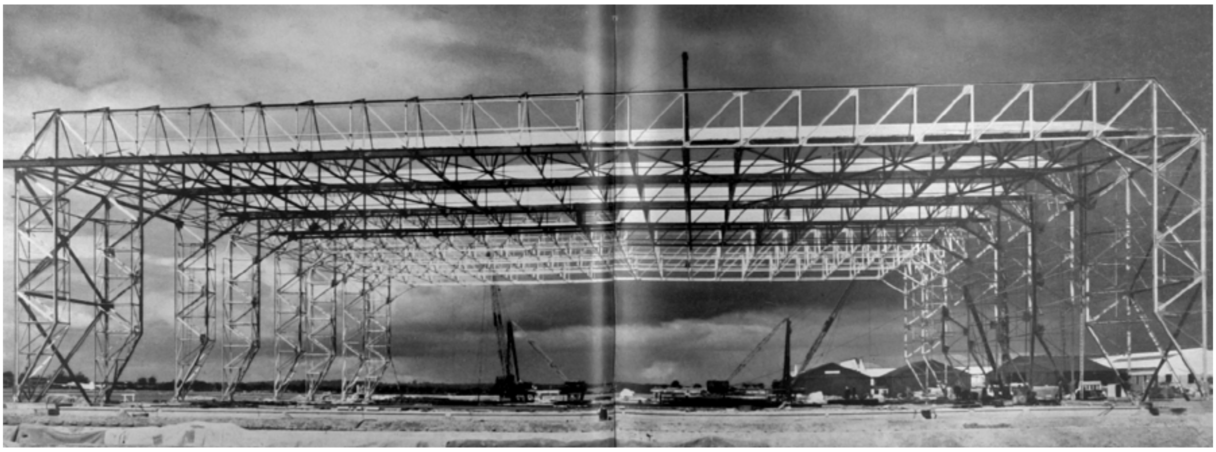


Fig. 39: De Havilland Avion Hanger, trussed aluminum frame structure spanning 61 m. James M. Monro & Son, 1955. (Peter)

Fig. 40: Aluminum can first introduced 1956. (Nichols)

Fig. 41: The Media Center at Lord's Cricket Ground, London, 1998. Future Systems and Büro Happold. Semi-monocoque structure. (Future Systems)

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MATERIAL EVOLUTION GROUPS A-10

10.1 Classifying Material Evolution

The nineteenth century, synonymous with the Industrial Revolution, is a pivotal period in the development of structural materials. This period is marked by the development of structural theory and materials science. More generally, the economy of Western nations grew with the development of first the steam engine, then electricity, and finally the internal combustion engine. These technologies in turn led to the development the railroads, steam ships, and industrial manufacturing. All of these developments in some way are products of the ascendance of iron and steel as the primary materials of the new age, replacing stone and timber. During the nineteenth century, modern plywood was developed, aluminum was discovered, the first plastics invented, and reinforced concrete emerged as a distinct material. These materials variously challenged the primacy of iron and steel in the twentieth century.

Each structural material throughout history has evolved in particular ways influenced by the socio-political and economic context in which they were discovered and used, the state of knowledge about material properties and structural mechanics, and the tools available to work the materials. In the course of my research, I have noticed some patterns that are shared by several groups of materials. These groups are roughly defined by whether the materials were discovered before or during the Industrial Revolution, however I have identified four specific group categories that I think can be useful in examining the development of new materials today. Those groups are:

- *Found Materials*
- *Ferric Metals*
- *Non-Synthetic Composite Materials*
- *Materials of Science*

Found Materials are those materials that are naturally apparent in usable form. They include stone, wood, and other organic materials such as vines and bamboo. These materials have generally evolved gradually over a long period.

Ferric metals belong in class of their own. Discovered before 1000 BC, in Western Asia and Egypt, iron and steel have also evolved slowly. However, these materials were instrumental in the great technological advances made during the Industrial Revolution. Their ascendance coincides with a transitional period during which material science and structural theory grew as distinct fields and the knowledge produced was applied to the design of structures.

Non-Synthetic Composite Materials are comprised of those materials that have been intentionally developed to address deficiencies of a base material. These include: fiber reinforced brick, plywood (and other engineered lumber products), reinforced concrete and prestressed concrete. Reinforced concrete does not fit cleanly into this model because, initially, it was not the product of intentional development as a distinct material. Plain concrete dates back to the Romans, and properly belongs in Group I, since it was surely discovered accidentally. Iron was first embedded in concrete to protect the iron from corrosion and fire. It is only when Frenchmen Joseph-Louis Lambot and Joseph Monier used reinforced concrete to make large flowerpots subject to hoop tension from the pressure of plant roots that this material was clearly being used as a distinct material whose properties were distinct from the properties of the constituent materials.

Materials of Science are those that are products of scientific knowledge, inquiry and development. These include aluminum and fiber reinforced plastics (FRP). I will only examine aluminum in detail here. The development of FRP will be analyzed in **Chapter 07**.

In this chapter, I will describe the characteristics of each of these evolutionary groups through specific material case studies. At the end of this chapter I will discuss the issue of *substitution* and, finally, I will propose how knowledge of these characteristics might be applied to the development of new materials.

10.2 Group I: Found Materials

10.2.1 Introduction

Man's first structural materials were those readily at hand: stone, wood, earth, vines, bamboo, and other naturally occurring materials that could be fashioned into shelter or fortification. The developments of structures made of earth, vines, and bamboo are interesting but outside the scope of this project. Rammed earth, or *pisé*, clearly had an influence on the development of reinforced concrete as François Coignet adapted the formwork of that traditional building method to the construction of concrete structures.[1] Vines and bamboo were both reconstituted to make long cables for suspension structures in South America and China.[2] However, these materials have not had a greater influence on the developments of structural form due to their lack of durability.

Stone and wood have been among the most important engineering materials throughout history. Wood continues to be used today, though the structural use of stone is very limited. In this section, I will review the evolution of stone only. Wood, in its unmodified form, shares similar evolutionary characteristics with stone, making it unnecessary to go into detail about its evolution as an engineering material here. Both wood and stone were used in post and lintel construction. Over time, stone was developed into a material for spanning space through arch forms, while wood was made to span spaces longer than any individual piece of wood through the development of the truss. Though the system form evolved significantly over time, the component forms changed little. Wood trusses are made of squared, linear

[1] **Ref. Appendix A-05, p.A.239**.
[2] **Ref. Appendix A-02, p.A.30-A.32**.

length of lumber, just as the simple roof structures of Greek temples or Roman basilicas were. Greater advances in System, Component, and Element Form have occurred from the creation of engineered wood products such as plywood, glue-laminated timber, and oriented strand board. I will discuss plywood in the section on Group III materials. The information in this section is drawn from **Appendices A-01 and A-09**. Additional references are noted as necessary.

10.2.2 Material Evolutionary Study: Stone

The first stone structures were probably made by stacking found rocks to make walls. Megaliths are evidence that men learned how to maneuver large stones, using earth ramps to make such distinctive structures as dolmens. (**Appendix A-09, p.A.361, Fig. 2**) As men became skillful at stacking they began to make corbelled arches and domes. (**Appendix A-09, p.A.361, Fig. 3; Appendix A-01, p.A.24, Fig. 26**)

Tools to work the stone improved during the Bronze Age as evidenced by the squared surfaces of the beams and columns of Stonehenge and the great blocks used to construct the pyramids in Egypt. (**Appendix A-09, p.A.361, Fig. 5**) The Egyptians built simple arches, two stones leaned against one another, in the construction of the pyramids, and at some time they began to make arched vaults with brick. (**Appendix A-09, p.A.361, Fig. 6; Appendix A-01, p.A.24, Fig. 25**)

The improved detail of stone carving during the course of Greek and Egyptian civilizations can be attributed to better tools made of iron. (**Appendix A-01, p.A.3, Fig. 1 and p.A.5, Fig. 5**) The Greeks became aware of the arch as early as 300 BC, however there are scant examples of its use.[3] They either did not appreciate the possibility of spanning greater lengths with the arch than with post and lintel construction, or they refused to adapt their established system of architectural design and proportion to this new form.

The temple architecture was influenced by its early development when the Greeks were building primarily with timber.[4] (**Fig. Appendix A-01, p.A.4, Fig. 2**) The choice to use stone was probably a combination of economic ability, improved tools and knowledge about working with the material, and, most of all, for its durability. In this respect, the use of stone was appropriate if we accept the principal selection criteria as being non-structural. It must be recognized that there were few material choices available at the time. Besides timber and stone, there was probably earth and brick, each being no more durable than timber because of their susceptibility to degradation caused by rain. We can conclude that the Greeks used stone in application appropriate ways as long as we understand their choice was made based on a cultural value for durability and that material strength was a secondary concern. As it were, stone architraves were not stressed significantly. The beams of the Propylaia in the Acropolis at Athens spanned about 2.5 m and each architrave carried about 2.9 metric tons. The architraves were 850 mm high by 500 mm wide. The calculated bending stress for an assumed point load at mid-span is 0.7 MPA, well within the tensile strength of marble equal to 7 MPa.[5] Therefore, stone was structurally adequate for the application and should

[3] **Ref. Appendix A-01, p.A.25-26**.
[4] **Ref. Appendix A-01, p.A.4-A.5**.
[5] **Ref. Appendix A-01, p.A.20-A.21**.

not be dismisses as inappropriate just because it is not efficient with respect to mass. The Greeks wanted the mass anyway because of their proportional system of aesthetics.

Around 200 BC, the Romans conquered the Greeks, and their empire spread. The Romans had a different civic function for their large buildings than did the Greeks. The Roman basilicas and baths were meant for public gathering, which the Greek temple design did not accommodate comfortably because of the typical density of columns. Therefore, there was a functional imperative to increase spans. In addition to the basilicas, the Romans built up an enormous infrastructure to support their empire. The aqueducts demanded many bridges, again requiring a spanning system that could not be satisfied by post and lintel or solid wall construction. The arch was eminently suitable to address all of these functions and was correspondingly exploited by the Romans. (**Appendix A-09, p.A.362, Fig. 9 and p.A.363, Fig. 11**) The form of the arch as the Romans employed it was of a semi-circle, no doubt because of its simple geometry, which also facilitated its construction because it was easy to lay out the centering and the shape of the stone voussoirs.

From the arch there developed logical translations of the simple form whereby the arch was extended linearly to make vaults or rotated radially to make domes. (**Appendix A-09, p.A.362, Fig. 10**) The Romans made use of concrete, brick and rubble to speed construction, a necessity driven by the speed at which the empire expanded and built up its infrastructure.

In the Middle Ages, the arch form was manipulated to its pointed, Gothic form. The cathedral builders pushed the limits of stone construction, building high, slender walls with large openings for light to pass through. Out of this development the flying buttress was invented. (**Appndix A-09, p.A.363, Fig. 12**) The form of the vaults and the development of the ribbed vault all indicated increased proficiency of the stonemasons to shape the stones using complex geometry. A sexpartite vault replaced the quadripartite vault in some cathedrals. (**Appendix A-09, p.A.363, Fig. 14**) Robert Mark explains that the cathedral builders probably used the sexpartite vault because it exerted lower horizontal thrusts in the plane of the wall during construction.[6]

Iron was used as reinforcement in some Middle Age structures. Iron reinforcement was tried in the arches of some Middle Age structures, though this application did not expand widely because the when the iron rusts, it oxidizes and causes the stone to crack if the iron is fit snuggly in the stone. The example shown is from Saint Chapelle, in Paris, completed in 1247.[7] (**Appendix A-09, p.A.363, Fig. 13**)

During the Renaissance, Italian architects restrained arches supported by slender piers with wrought-iron ties. Finally, the ultimate example of the use of iron-reinforced stone was in the Eglise de Ste.-Geneviève, in Paris, completed in 1770. This building is of great scale and has flat arches that span greater than normal distances.[8] (**Appendix A-03, p.A.95, Fig. 20**)

In the 16th and 17th centuries, French mathematicians developed a mathematics based arch theory, leading to knowledge about the relationship between the form of the catenary under tension and the reciprocal form of the thrust line in an arch. The mathematicians also

6 Mark, p163.
7 Hamilton, p42-43.
8 **Ref. Appendix A-03, p.A.94-A.95**.

examined the failure of arches by rotation, the settlement of an arch when the centering is removed, and methods for more precisely calculating the necessary thickness of the piers of an arch. With this knowledge, arches were built with higher span to depth ratios, thus introducing flatter arches.[9]

Little further development was possible, particularly with unreinforced stone. Stone continued to be used into the early twentieth century, though more rarely as iron, steel and reinforced concrete accounted for most of the market. The development of the stone arch can be said to have culminated in the construction of railroad bridges in Graubünden, Switzerland, the Landwasser (1902) and Weisener Viaducts (1908). (**Appendix A-09, p.A.366, Fig. 23**)

Stone is rarely used structurally today. It is primarily used in novel composite structures such as Jürg Conzett's Suransun's Bridge (1999), a stressed ribbon pedestrian bridge made of stone plates stressed by two stainless-steel bands. (**Appendix A-09, p.A.367, Fig. 26**) Stone is also used in pylons or piers of bridges and buildings. The British engineer Peter Rice designed a prestressed arch system for the world expo in Seville of 1992. (**Appendix A-09, p.A.366, Fig. 24**)

Stone was probably first used for constructabilty and durability reasons. As tools for working the material improved, the finish of the stone was refined. The Greeks undoubtedly chose stone for its durability, a cultural imperative for its application in temple construction. However, its use as beams subject to flexure was a function of cultural norms of building design not directly linked to the structural properties of the material. In any case, the Greeks were relatively conservative in their designs, and the beams were normally proportioned adequately for the stresses to which they were subject.[10]

Since the arch was applied to arch construction, little further development has occurred except refinements of form. These refinements reflect increased knowledge of geometry and an empirical understanding of the forces during the Middle Ages, and the development of arch theory in France during the sixteenth and seventeenth centuries. Today, stone's compressive strength is used compositely with other materials, being especially suitable to use with prestressing.

10.2.3 Summary of Evolutionary Characteristics of Group I Materials

- Found material, not the product of intentional invention.
- Slow evolution, empirical developments.
- Development of processing tools and construction methods led to refinement and creation of new forms.
- Structural forms precede knowledge of structural theory and material properties.
- Structural forms and general use conditioned over time. Principle effect of increased knowledge is to refine System Form. Component form changed very little.

[9] **Ref. Appendix A-09, p.A.359-A.360**.
[10] Cotterell and Kamminga, p114.

- Forms and applications were not significantly improved during twentieth century in construction.
- Used today principally in composite structures or as reconstituted materials.

10.3 Group II: Ferric Metals

10.3.1 Introduction

The ferric metals include cast iron, wrought iron, and steel. The first irons were wrought iron and eutectic steel, and were produced using furnaces fueled by charcoal. It is estimated that the earliest pre-historic furnaces could only produce blooms weighing about 50 pounds and there was a lot of waste because of the quantity of slag. The first iron was probably used for weapon making and later for tools. Processing technology improved sufficiently during the Greek period such that they made lifting devices and stone ashlar clamps by hammer welding several small pieces of iron together. It is estimated that by Imperial Roman times, blooms weighing about 100 pounds could be produced. Waterpower needed to be developed before iron furnaces could produce larger quantities of iron at lower cost. Vitruvius described an under-shot waterwheel in the first century BC, but its use was limited to corn milling.

The first recorded application of waterpower to the making and forging of iron was in the twelfth-century by Cistercian monasteries in France. There was a steady development of iron production in Europe from the twelfth century onwards. A twelfth century furnace could produce blooms weighing up to 140 pounds using 4.5 times that weight in charcoal. The Stückofen, a furnace 10 to 14 ft high was developed in Germany and was the most powerful furnace developed to that time. By 1430, A Stückofen could produce a bloom weighing 800 pounds using 2.5 times that weight in charcoal, and 1,200 pounds in 1600. Cast iron, or pig iron, could have been produced if there was a powerful enough blast and a slag hole provided but that was seldom done before the fifteenth century. Waterpower operated bellows provided the powerful blast. Waterpower was also used to operate a tilt hammer to remove the slag and shape the bloom. To convert pig iron into wrought iron, it had to be reheated in a more open furnace, called a finery, with charcoal and more blast to remove the combined carbon and as much as possible of any sulfur and phosphorous present.[11]

Cast iron, wrought iron, and steel were essential to the technological progress made during the Industrial Revolution. The steam engine was only practical if the boiler were made of iron. In turn, this new source of power made iron production even more efficient, raising production and decreasing cost. The combination of steam engine and more economical steel led to further technological developments such as the locomotive and steamships. More directly, better processing technologies were invented that took advantage of the power of the steam engine. These factors contributed to an almost self-perpetuating situation where new technologies and more efficient production methods cyclically improved the economy, leading to an expansion of the iron making industry to meet demand, which in turn caused competition and resulted in further improvements of processing technology. All the

[11] Hamilton, p29-31 and 42-44.

while, these new functions, the economy, and the socio-economic climate demanded longer spanning roofs and bridges, which resulted in the development of new structural forms adapted to the knowledge, materials and processing technologies of the time.

In this section, I will focus on evolution of cast iron from the sixteenth century to the mid-eighteenth century in England, when cast iron was supplanted by wrought iron. This period is variously covered in **Appendices A-02, A-03, and A-09**.

I do not include later developments in wrought iron and steel because by the mid-nineteenth century the principal component forms of iron, and later steel, had been established. Further refinements were the result of improved processing technologies and knowledge of mechanics. The principal changes of form occurred at System and Detail level except for the application of cables, particularly high-strength steel cables used for prestressing and the construction of cable nets. I include the use of iron and steel in reinforced and prestressed concrete in the next section on Group III materials.

10.3.2 Cast Iron in England

In 1558, Queen Elizabeth issued decree in England limiting the use of domestic timber for fuel in iron manufacture because the strained timber supply had to be preserved for the navy. The growth of industry and the navy had led to severe deforestation. This decree effectively crippled the English iron industry, though a certain level of production was maintained. The restriction had a lasting benefit however because it compelled people like Dud Dudley in 1619, and Abraham Darby in 1735, to develop coal as an alternative fuel.

Prior to 18th century, the primary structural uses of iron were as ties for arches, mechanical connections of wood, and reinforcement for stone construction. All of these applications used wrought iron. The introduction of coal as a fuel in England resulted in the production of cast iron, because the coke, made from coal, burns hotter than charcoal, made from wood. There was clearly resistance to the introduction of cast iron into the English market. Charcoal using iron founders who felt threatened by Dud Dudley's invention attacked him. No major cast-iron structure was made until 1779, when Abraham Darby, Jr. built Ironbridge, the first known all-cast-iron structure in the world. (**Appendix A-02, p.A.41, Fig. 17**) It can be assumed that Darby built Ironbridge as a means of advertising what his product could do, investing in the bridge to encourage the use of cast iron in construction, though the next significant cast-iron structure was not built until 1795.

The form of Ironbridge is a semi-circular arch made of three concentric rings attached to one another by radial struts. The striking aspect of Ironbridge is the slenderness of the cast-iron components. This bridge may have the overall form of a stone bridge but it far lighter both visually and materially. The lightness of the arch was a contributing factor to problems that occurred with movement of the embankments that pushed up the crown of the bridge. Thomas Telford addressed this problem in his Buildwas Bridge, built just upstream from Ironbridge, in 1795. (**Appendix A-02, p.A.42, Fig. 18**) Telford used two super-imposed arches; the flatter of the two was designed to act against movement of the embankment. Telford was a trained mason, and expert in the construction of masonry arches. It seems natural that he would use this new material in a form he was familiar with, yet he recognized the fundamental difference between the two materials with respect to their weight and processing attributes. In any case, structures subject to compression are appropriate to cast iron because not only is its tensile strength inferior, the material's brittleness makes it susceptible to fatigue failure.

From the 1790s to the 1840s, cast iron beams were developed in England. The first known cast-iron beams were designed by Charles Bage and installed in his textile mill at Shrewsbury in 1797. (**Appendix A-03, p.A.97, Fig. 24**) Bage tested his beams for strength and later made experiments with different sized beams so that he could calculate which size beam he needed for a given span and load. His beams already had the lower flange on them however and there is no information that would indicate that Bage was aware of the unequal strength properties of cast iron even though the form of his beam had an appropriate disposition of material. The flange was a detail of function integration. Its purpose was to provide a support for the brick vaults that spanned between beams and supported the floor.

The choice of cast iron for a mill beam is not inappropriate within the context of the time. The cast iron then being produced had essentially the same tensile strength as timber but its stiffness is much higher and is not susceptible to creep like timber is. The only disadvantage to using cast iron was its weight, but that cannot be considered a disadvantage in a building. Furthermore, the choice between timber and cast iron was not actually driven by structural concerns. Rather, Bage and other mill owners turned to cast iron in an effort to protect their buildings from fire. It was only when cast iron beams were used in railroad bridges subject to fatigue inducing dynamic loads that the material's inadequacy revealed itself.

Over time, the sections of the cast-iron mill beams were developed. The principle improvements occurred with the introduction of the fish-belly form, which better reflected the moment distribution in the beam, and the details of the connections to the columns that made the beams act continuously over the supports, introducing unwanted negative moments. Connections were devised to relieve that stress.

At the beginning of the nineteenth century, materials science began to develop as a distinct field of scientific inquiry. The first accurate results on the strength of iron were published by Peter van Musschenbroek, a Dutch physicist, in 1729, however his work was of limited value, particularly to the use of cast iron. In 1824, Thomas Tredgold published results from tests he made based on the deflection of cast-iron beams under their own dead load. His information was not especially useful, but he did propose an I-section form as the most appropriate for cast iron. He erroneously assumed cast iron to have equal compressive and tensile strength. (**Appendix A-03, p.A.100, Fig. 30(a)**) This was not the first discovery of the benefits of the I-section, though by all accounts Tredgold came to his conclusion independent of the fact that Alphonse Duleau had performed experiments demonstrating the advantage of the I-form over a simple rectangular section in France in 1820. Tredgold also proposed that the flanges and web be of the same thickness to avoid cracking due to stress concentrations caused by uneven cooling of the cast component. Thus, processing was an intrinsic consideration in the form.

Eaton Hodgkinson was the first to correctly determine the strength properties of cast iron around 1830. He demonstrated the different strength properties by simply testing a beam with a **T**-section oriented with the flange up and inverted. He then used a scientific process

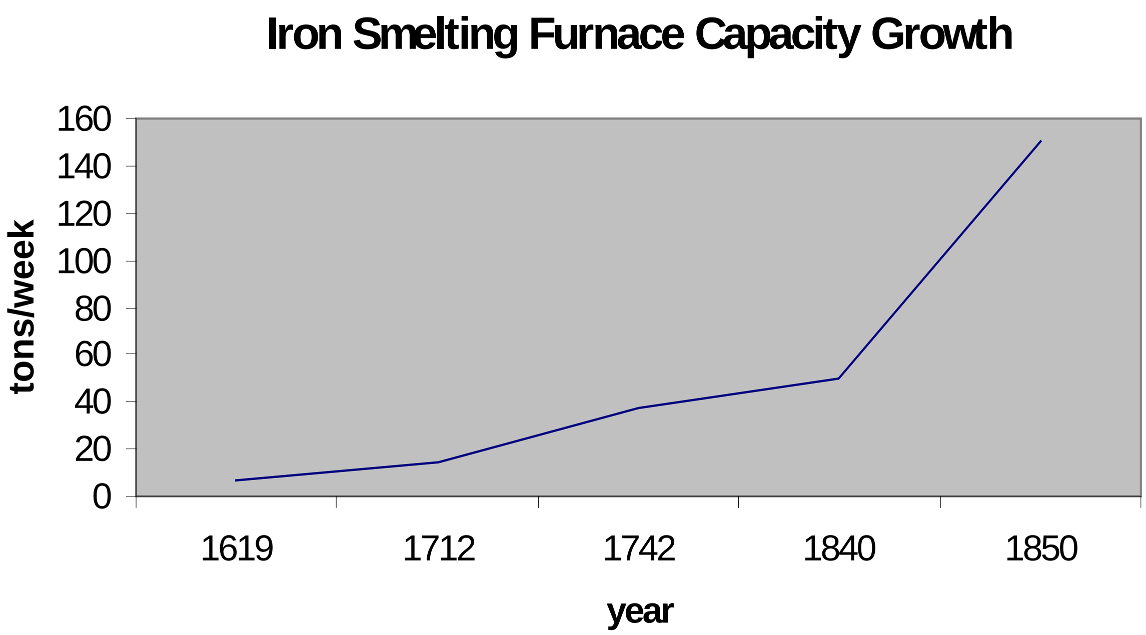

Fig. 1: Growth of production volume of an iron furnace from 1619 to 1850. (Dooley)

of parameter variation to determine the relative difference in strength. The result was a beam with asymmetrical flanges and a tapered web. (**Appendix A-03, p.A.100, Fig. 30(b)**)

In 1823, the Stockton and Darlington Railroad became the first railway in the world. This led to the need for bridges that could support higher live and dynamic loads than was necessary ever before. Cast iron beams were used for short spans, but their brittleness led to fatigue failures. To address the problem, engineers started to reinforce the cast iron beams with wrought iron.[12]

The quantity of wrought iron increased until development on the Britannia Bridge started and all-wrought-iron beams began to be produced. Both William Fairbairn and Robert Stephenson, who worked together to develop the tubular concept of the Britannia Bridge, initially designed wrought iron beams that had sections with asymmetrical flanges like Hodgkinson's cast iron section. This can be attributed to the fact that only the tensile strength properties of wrought iron were known at the time. Theretofore, wrought iron had been used almost exclusively in England in structures subject to tension, either as chains in a suspension bridge or as plates in boilers. This lack of knowledge was quickly rectified however, and the completion of the Britannia Bridge marks the transition from cast iron to wrought iron as the main structural material in England. Cast iron was still used in columns and other applications where it was under axial compression, however it did not develop further.

Henry Cort patented a method for converting pig iron into wrought iron using a reverberatory furnace in 1784. However, wrought iron was not used extensively in England the process of making wrought iron was more expensive than cast iron since the wrought iron was derived

[12] **Ref. Appendix A-03, p.A.102-A.104**.

from pig iron. James Neilson, of Glasgow, invented the hot blast in 1828 and it was first used in 1829. Before the hot blast, the average output of a blast furnace was not over 30 tons/week. By 1840, a hot blast furnace could produce 50 tons/week, and by 1850, 150 tons/week. This great increase in production probably corresponded with decreased prices. Any price decrease would offset the additional costs of wrought iron, thereby making that material more affordable for a wider number of applications, including construction.

The increased capacity of the blast furnace is measured over time in **Figure 1**. The interesting thing about this graph is the correlation between the growth of iron production and the development of structural form, processing technologies, and knowledge. While I do not have further information to present here, I think that this relationship can be quantified further to show that this is a paradigm of material development. This paradigm does not apply to Group I materials however, and I think that is because of their limited material properties and the limited forms in which they can be used. However, I will show below that this trend is applicable to other materials, particularly reinforced concrete and aluminum.

10.3.3 Summary of Evolutionary Characteristics of Group II Materials

- The development of power was important to the development of the ferric metals. Even steel benefited from the greater motive power because it was more difficult to roll than wrought iron. The wide-flange beam, standard today, could not be rolled until the 1920s when the motive power and a new arrangement of rollers was developed to handle accomplish the job. The benefits of this form of flange over the iconic tapered flange of the stander **I**-section were known earlier, however it was not possible to produce it.
- Ferric metals were first developed empirically and then materials science and structural theory were applied to their development.
- There is a direct correlation between production volume, cost, and the development of the material. Greater production lowers costs, making the material affordable for use and further development in a wider range of applications. The I-section was variously developed in buildings, ship construction, rails for the railroad, and through theoretical (Duleau) and applied science (Hodgkinson and Fairbairn) research.
- The basic component forms of ferric metals were established before 1850. Systems of construction developed around the use of economical, linear elements used to make trusses, building frames, or lattice domes.
- Today, CAD/CAM technology is making it increasingly possible to challenge the institutionalized design culture of steel. Casting, laser cutting, and possibly, in the future, nanotechnology, may start a new period of development leading to more three-dimensional structural components in steel. Motive power is now giving way to computing power, and may be the technological boost these materials need to be better exploited.

10.4 Group III: Non-Synthetic Composite Materials

10.4.1 Introduction

In this section, I will review the evolution of man-made composite materials. Fiber reinforced bricks were probably the first composite materials, though fibers were added to sun-baked bricks to primarily control cracking, not improve strength characteristics. The use of brick did not differ greatly from stone, except it was easier to procure and the building units were generally smaller. To what extent knowledge of this material and the benefits of adding fibers to a brittle matrix material may have influenced the development of reinforced concrete or wood-flour reinforced Bakelite, one of the first fiber reinforced plastics, is not known. The use of iron to strengthen stone structures from the Middle Ages to the eighteenth century, and the use of wrought iron to reinforce cast iron indicates that by the mid-nineteenth century, men had ample experience to understand how two materials could be combined to augment the deficiencies of one or more of the constituent materials.

I will examine the evolution of two composite materials, reinforced concrete and plywood, in more detail below. Fiber reinforced plastics are not included in this section because they are direct products of science, whereas the materials included here have been developed by craftsmen and technologists from readily available materials. Therefore, FRP materials belong in Group IV. I have not included prestressed concrete because it, like plywood, is the product of dissatisfaction with the base material. Prestressing was developed in response to the problems of cracking in normal concrete structures due to shrinkage and when the concrete is subject to appreciable tensile stress. Its development is recorded in detail in **Appendix A-06**. The information on reinforced concrete is referenced from **Appendices A-05 and A-06**. Plywood is covered in **Appendix A-04**. Additional references are noted as necessary.

10.4.2 Reinforced Concrete

The idea of reinforced concrete evolved out of various applications in which iron was embedded in a ceramic material, such as cement or plaster of Paris, in order to protect the iron from fire. Over time, concrete was used in place of brick or clay pot arches between floor beams to make monolithic floors, however this has to be considered a composite structure, and not a composite material. (**Appendix A-05, p.A.247-A.248, Figs. 18-22**)

Reinforced concrete emerged as a distinct material as if there was a coalescence of influences that made the particular possibilities of reinforced concrete apparent to a number of persons. Defining when the history of reinforced concrete begins is obscured by the question of when the iron–concrete composite floors mentioned above could be defined as reinforced concrete. However, it is generally accepted that two Frenchmen, Joseph-Louis Lambot and Joseph Monier, share the honor of being the co-founders of reinforced concrete. Interestingly, both men initially used the material to make flowerpots. One questions whether Monier in some way stole Lambot's idea and whether they were competitors in the flower pot making business since they allegedly made their discoveries within a year of each other in 1848 and 1849 respectively. Lambot exhibited a small boat made of reinforced concrete at the Paris Exhibition of 1855 and applied for a patent in Brussels for a system using his 'iron cement' in structures that have to withstand moisture, such as ships, water tanks, and tubs for orange trees. Lambot's commercial success was limited but his boat surely captured the imagination of a number of visitors to the Paris Exhibition.[13]

In the early 1850s, two other men, William Boutland Wilkinson of England, and François Coignet of France, also developed reinforced concrete construction systems, most likely independent of knowledge about Lambot or Monier. Wilkinson patented a system of construction in 1854 that was listed as "improvements in the construction of fireproof dwellings, warehouses and other buildings."[14] (**Appendix A-05, p.A.237, Fig. 3**) Wilkinson's system using a concrete beam reinforced with an iron cable cannot be called anything but reinforced concrete. However, I have found no evidence that Wilkinson actually built any structures using his patented system François Coignet adapted unreinforced concrete to the rammed earth construction method called *pisé*, which utilizes formwork similar to that later adopted for reinforced concrete construction and still recognizable today. (**Appendix A-05, p.A.238, Fig. 5**) It does not seem that Coignet reinforced concrete with iron until after he published a book in 1861 entitled *Béton Agglomeré*, in which he suggests using concrete with an armature of iron. However, Coignet can be credited with advancing the method of concrete construction and improving knowledge about the effects of different water contents in the concrete mix.[15]

If Labot can be credited with actually fabricating the first reinforced concrete artifacts, Joseph Monier has to be credited with being the father of the patent-protected proprietary construction system that became the model of future reinforced concrete development until after the turn of the century. Monier received patents in 1867 for his flowerpots, followed in 1868 by a patent for water tanks, for bridges in 1873, and beams in 1877. Again, it seems further research should determine how much Monier actually capitalized on the ideas of Lambot. Nevertheless, Monier has to be given credit for commercializing his system first. Monier built his first water tank in 1868, and the first known reinforced concrete bridge in 1875, at the castle park of Chazelet, France.[16]

If stone structures developed under the personage of governments and the religious establishment, and iron developed under the personage of industrialists and businessmen, then reinforced concrete is the first material of the entrepreneur. Unlike iron, which requires costly processing under controlled conditions, reinforced concrete is made with readily acquired, cheap materials. Cement was already being produced for application in foundations, mortars, and unreinforced concrete, so the industry did not have to change significantly to support the increased demand by builders of reinforced concrete structures. This meant that the industry was widely accessible to those without great financial means, and, like with the wooden truss designs in America at the beginning of the century, a plethora of people patented hundreds of systems, the primary differences being in the arrangement and geometry of the reinforcing iron.[17]

The German firm of Wayss & Freytag were Monier licensees. However, their interest in Monier's system laid more in avoiding patent infringement suits than the technical merits of

[13] **Ref. Appendix A-05, p.A.238-A.239**.
[14] Cassie, W. Fisher. "Early Reinforced Concrete I Newcastle-upon-Tyne", from *Structural Engineer*, Volume 33, April 1955, pp134-137.
[15] **Ref. Appendix A-05, p.A.239**.
[16] **Ref. Appendix A-05, p.A.239**.
[17] **Ref. Appendix A-03, p.A.86-A.90**; **Appendix A-05, p.A.242-A.249**; Pauser.

Monier's System. They used Monier's system as a starting point for research and development based on much more sound scientific and mathematical reasoning. From the late 1880s, Wayss and Freytag were building thin arches theretofore unprecedented. (**Appendix A-05, p.A.240-A.241, Figs. 8 and 9**) This firm had a monopoly on the German market, however this monopoly does not seem to have made the firm any less innovative. However, the unanswerable question remains about the extent to which innovation was hindered because of the lack of competition.

While Wayss & Freytag had a regional monopoly, François Hennebique had a virtual monopoly on the reinforced concrete market in the rest of Europe by 1900. In America, the Hennebique System had to compete more directly against the Ransome and Melan systems. Hennebique patented his comprehensive reinforced concrete building system in 1892. Theretofore, most patents addressed individual building components, like beams or floors. Hennebique's success lay in marketing a complete building system rather than just components. He tightly controlled design and construction throughout his licensee network to control quality – a decided advantage because early builders of reinforced concrete often used the cheapest aggregates and did not appreciate the detrimental effects on strength by using too much water in the concrete mix.

In 1900, reinforced concrete was grandly displayed at the Paris Exhibition. The exhibition-showcased works built using proprietary systems, which both impressed the public and construction industry, and engendered discontent that proprietary patent holders restricted this material's use. This led to the growth of public design codes and methods during the first decade of the twentieth century, opening up the market such that engineers became more important to the material's development. This movement was led by lectures published by the French professor Charles Rabut in 1897. The first design codes in Prussian and Switzerland were written between 1901 and 1906. The American Concrete Institute was founded in 1905 with the purpose of regulating the design of reinforced concrete structures. Thus, reinforced concrete design began to move from the realm of the proprietorship to that of the engineering community. The engineer's education and knowledge of structural theory could now be collectively, and more freely, brought to bear on the challenges and potential of this material for the first time.

The consequence of opening the design market of reinforced concrete structures is notable. In the first decade of the twentieth century the flat slab was invented for building structures, and the concrete box section and deck stiffened arch were invented for bridge structures. The first concrete thin shells were being built using simple geometries from the 1929s; by 1956, Heinz Isler was making free-form shells. In the 1920s and 30s, prestressed concrete, a new material, was invented in response to problems with normally reinforced concrete due to shrinkage and creep. These properties of concrete were little understood until Eugène Freyssinet and Franz Dischinger studied them and quantified their behavior. It can be argued that this knowledge became more important as the reinforced concrete design market opened and longer bridge spans were constructed.

To summarize, reinforced concrete evolved out of construction practices concerned with protecting iron structure from fire. Once the complimentary properties of the materials was recognized and applied structurally, it was used in material adapted ways, notably Lambot's thin-shell boat and the thin-arched bridges from Wayss & Freytag. While advances were made in this early period of the material's development, the proprietary system supported by patent protections surely hindered innovation, though I cannot not say how much without further research. Clearly, once the design of reinforced concrete was freed of patent restraints and put out into the 'open market', the greatest advances were made with respect to the invention of the flat slab, thin shells and prestressed concrete.

Nevertheless, it could be argued that this phase of proprietary systems is beneficial to long-term material development. The proprietary system rewarded those who invested in the material. More research may have to be done, but it can be argued that a young industry needs this kind of protection during a period when it is trying to establish itself with clients. However, perhaps this security comes at the price of innovation. Hennebique's widely successful system was also highly conservative, expressing none of the daring so powerfully displayed by Wayss & Freytag's thin arches.

10.4.3 Plywood

The earliest antecedents to modern plywood can be traced back to ancient Egyptian civilizations. The Egyptians used veneering to decorate coffins in order to make the most of rare woods. There is one instance in which they crossed smaller pieces of wood in the panels of a coffin, thus making a primitive form of plywood in which the wood grain oriented in two directions counteracts each other's movement due to changes in humidity. Evidence from an Egyptian stone carving indicates that the Egyptians used animal based (hide, bone) adhesives, which require heating before application. The Greeks and Romans used veneering, but its use seems to have died out during the Middle Ages. Veneering was reintroduced during the Renaissance and the mastery of veneering reached a high point in furniture making under the reign of Louis XV in France and in Elizabethan England. The skilled furniture makers of this period clearly had an understanding of the behavioral characteristics of wood and purposefully cut the wood into smaller pieces and reassembled them such that the grain orientation of one piece counteracted the movement of the pieces adjacent to it.[18]

Modern plywood was developed around the 1830s for piano making. The first application was for pin and wrest planks, which hold the tuning pins. By crossing thin sawn maple sheets the pins could be in constant contact against the end fibers of the drilled wood on all sides such that the pins would not slip but could still be turned to tune the wires. In 1860, Steinway and Sons introduced laminated wood technology for fabricating the rims of grand pianos, demonstrating the pliability of laminated wood products. Throughout the rest of the nineteenth century, plywood was used in chairs, interior paneling, organs, sewing machines and other furniture items. It could not be used for exterior applications because all of the adhesives at the time were organic based and susceptible to degradation when exposed to moisture or heat. Plywood earned a bad reputation when used in exterior applications.

In the first decade of the twentieth century, casein, an adhesive made from the whey of milk, was used for the first time to make plywood. For a natural adhesive, casein is a high performing adhesive that is resistant to moisture. The first significant structural application of plywood that I have found is in the construction of rigid airships manufactured by

[18] **Ref. Appendix A-04, p.A.209-A.212**.

Luftschiffbau Schütte-Lanz, a German competitor of Luftschiffbau Zeppelin, which used aluminum to construct its airships. Without significant precedents, the use of plywood in such an ambitious structure is surprising. The first Schütte-Lanz airship was 131 m long. (**Appendix A-04, p.A.226, Fig. 59**) However, by this time most of the processing technologies used by Schütte-Lanz had been developed, particularly plywood molding techniques. Schütte-Lanz continued to build plywood airframes throughout World War I. I have found no other significant examples of the use of structural plywood until after the war started. During the war, Schütte-Lanz airships suffered greatly in the wet environment of the North Sea, and the limits of casein became apparent. Casein, being a type of cheese, is susceptible to fungal attack, which will obviously develop if the structure is moist for an extended period of time. This led Schütte-Lanz to search for alternatives, and they were active in the early development of synthetic adhesives. Schütte-Lanz was liquidated at the end of the war as a condition of the German disarmament.[19]

During World I, plywood was used in the manufacture of aircraft. Plywood was used in the first stressed-stressed skin cantilevered wings and monocoque fuselages around 1912 and 1913. In 1918, the Loughead brothers patented a process for forming fuselage half-shells out of spruce veneer in large concrete molds fitted with a rubber bladder. (**Appendix A-04, p.A.219, Fig. 52**) The veneers were set in place, casein glue was applied, and the bladder was inflated to force the wood into the mold.[20] However, casein's vulnerability to extended periods of exposure to moisture drove the development of metal skinned structures, which started to supplant plywood entirely at the end of the war until the synthetic glues were developed that made plywood nearly impervious to moisture. Plywood was the first material to be used to make to construct aircraft, gliders, and boats during World War II.[21]

After the war, the plywood industry was consumed by the new demand for plywood sheeting in building construction. The industry developed this part of their business at the expense of its research and development of high-performance structures for aviation and marine applications. By the 1950s, fiberglass construction accounted for 60% of the small boat market and aluminum dominated airplane construction. The use of plywood has not matched its early development since. This is an under-developed material.

Plywood was developed specifically to address the deficiencies of natural wood. It was first developed by craftsmen, refined in industrial manufacturing, and finally reached the height of its development as a result of concerted, scientific research to find a water-resistant adhesive. This research aided the development of plywood for a short time in high-performance structures, however it also made the wood even more appealing as a low-cost building material, and its development was effectively halted. Today plywood plates are used as shear walls in construction, and in engineered wood products like the webs of wood I-joists. These may be appropriate applications, but do not realize the material's potential by far.

[19] **Ref. Appendix A-04, p.A.229-A.230**.
[20] Jakab, p916.
[21] Wood, p387.

10.4.4 Summary of Evolutionary Characteristics of Group III Materials

- Group III, composite materials were developed specifically to address the deficiencies of one or more of the base materials. Except for the steel cables used in prestressed concrete, these materials are made of relatively inexpensive material components.
- Patents were used in the early development of reinforced concrete and prestressed concrete that served to protect and encourage entrepreneurs to develop these materials when they were new. Structural forms advanced rapidly once an open design market replaced the patent protected proprietary system. The design of reinforced concrete structures transitioned from the entrepreneur and proprietary controlled systems to engineers.
- Plywood's development was restricted until a water resistant adhesive was found. Once that adhesive was found, structural applications developed rapidly in high performance structures that exploited the materials surface-active structural characteristics and processing attributes, particularly the material's suitability for doubly curved forms.
- Ironically, the invention of synthetic adhesives that made the material nearly impervious to water first extended the development of the material in advanced structures, but then made it applicable to the more mundane application as a flat-panel building material. The building market was far larger than for aviation, and the industry unfortunately abandoned development of advanced structural applications of the material. Thus, unlike concrete, plywood transitioned from a material used by engineers, to one used by craftsmen.

10.5 Group IV: Materials of Science

10.5.1 Introduction

The field of science became influential to material development in the nineteenth century. The first structural material to be the pure product of science was aluminum, isolated in 1827. Aluminum was the product of advances in chemistry. Materials science also emerged as a distinct field during the nineteenth century. The later invention of fiber-reinforced plastics is the result of research and development done in both chemistry and materials science. In this section, I will examine the discovery and development of aluminum. The information on aluminum is primarily drawn from **Appendices A-04 and A-09**. Additional references are noted as necessary.

10.5.2 Aluminum

Aluminum is the first structural material that is a direct product of science. Aluminum cannot be found in its metallic state in nature. Advances in chemistry in the late eighteenth and nineteenth centuries led first to the identification of aluminum as an element, and to its subsequent reduction from aluminum oxide, Al_2O_3. Frederick Wöhler first successfully isolated aluminum, though his sample was impure and he could not determine the properties of the material. H. Sainte-Claire Deville deserves his title as 'father' of the aluminum industry. He isolated a nearly pure sample of aluminum in 1854. Deville, a chemist, astutely appreciated the commercial potential of the material, but the process to make the material was costly. In early 1856, aluminum was sold for FFr. 1000 per kilo, or the equivalent of $90 per pound, adjusted for inflation to be comparable to 1894 prices.[22]

Deville initially secured the financial backing of Napoleon III, who recognized the possible military applications of aluminum for lightening equipment. With this funding, Deville concentrated on bringing down the cost of aluminum by focusing on the chemicals used in its production. Aluminum ore, primarily bauxite, is relatively cheap. To reduce the alumina in bauxite to aluminum requires a large amount of energy. Electrical power was impractical at the time. Deville did use a battery to electrolytically reduce the alumina, but the batteries were too costly. Deville therefore used a chemical process. He first used potassium, but changed to sodium because it was somewhat more stable and held more promise for developing cheaper processes to manufacture it. Between 1854 and 1859, Deville helped sodium manufacturers improve their processes to such an extent that sodium prices plummeted from a high of FFr. 2000/kilo to FFr. 10/kilo. Deville, and others, analyzed the whole process of making aluminum, looking for any point in the process that improvements would have the greatest effect on reducing the gross price of the finished product. In 1883, Walter Weldon determined that the sodium still accounted for 57% of the cost of finished aluminum, while producing alumina was only 10%. The remaining 33% was the cost of producing double chloride, another chemical used in the reduction process. In 1886, H.Y. Caster, of New York City, invented a method or producing sodium that helped lower the cost from $1/pound to about 25 cents. This steady process of value analysis and targeted process improvements resulted in the cost of aluminum decreasing to FFr. 59 per kilo, or $4.84 per pound, by 1888.

Up to 1888, aluminum was used in high-end goods such as jewelry, statuary and tableware. In 1889, Charles Hall, of Ohio, received a patent for a novel process using electrolytic reduction to make aluminum. Though both Wöhler and Deville had used electrolytic reduction, it was not a practical technology until the invention and improvement of the dynamo. A similar process was independently developed at the same time in Europe by Paul T. Herault, however, Herault was more interested in making aluminum alloys rather than pure aluminum like Hall because at the time the most successful aluminum product was aluminum-bronze, which is actually a bronze alloy because it contains only 10% aluminum.

The cost of aluminum was nearly cut in half, from $8 per pound to $4.50, by the use of the electrolytic process. Both the Pittsburgh Reduction Company in America and the aluminum works at Neuhausen, Switzerland exploited the power of the largest waterfalls on both continents to expand their production plants. The economy of scale led the price to drop to just 35 cents per pound by 1895. (**Fig. 2**)

The number of aluminum products expanded from the time the electrolytic process was introduced. Aluminum was used in cookery, window frames, and other household products. The Russians tried using aluminum in ammunition carts and the French built an aluminum torpedo boat that was 18.3 m (60 ft) long. In 1891, Ferdinand von Zeppelin determined that aluminum’s strength to weight properties and its resistance to corrosion made it the best

[22] All price data comes from Richards (1896).

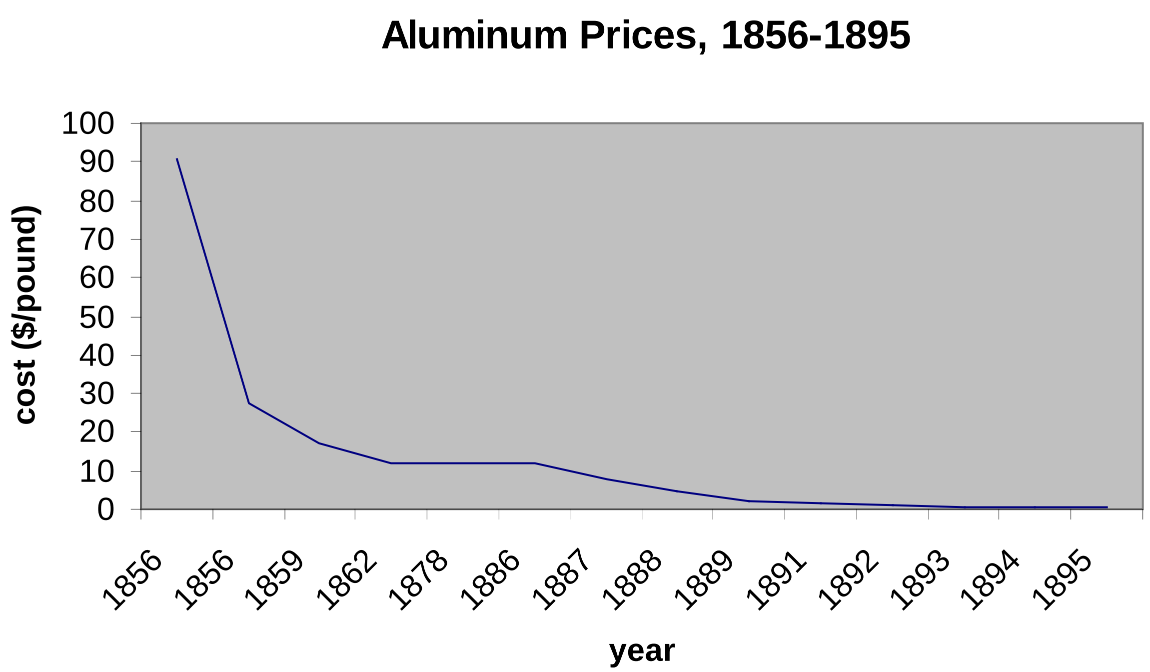

Fig. 2: Aluminum cost history from 1856-1895. (Dooley)

choice for constructing his idea for a rigid airship. David Schwartz was the first to build an aluminum rigid airship in 1894, and another in 1897. These airships were 38.32 m long, with an elliptical cross section measuring 15.4 m wide and 18.2 m tall. The first ship was allegedly destroyed before test trials and the second ship lost control and crashed shortly after take-off; thus why he does not receive credit for building the first dirigible rigid airship and is but an obscure footnote in the history of dirigible flight.

Ferdinand von Zeppelin pioneered the use of aluminum in the development of aviation when he built the first dirigible, rigid-type airship in 1900. This airship, LZ-1, was 128 m long with a diameter of 11.65 m. (**Appendix A-04, p.A.196-A.197, Figs. 21 and 22**) Over the next forty years, the Luftschiffbau Zeppelin company, under the technical direction of its chief engineer, Ludwig Dürr, made great advances in the development of thin-walled, aluminum structural components used to make the spatial trusses that were used to construct the three-dimensional airframe. (**Appendix A-04, p.A.206-209, Figs. 31 to 38**) The knowledge created at Luftschiffbau Zeppelin was transferred to the development of the airplane through Claude Dornier, who first developed thin-walled aluminum connection plates at Luftschiffbau Zeppelin before Ferdinand von Zeppelin helped finance the airplane manufacturer to be run by Dornier that would ultimately bear his name. Among Dornier's innovations was the introduction of lightening holes in thin-walled aluminum components.

The development of aluminum structures and airplanes is nearly synonymous. Even the Wright Flyer sported an aluminum engine block. Other early aviation pioneers, such as Samuel P. Langley and Hiram Maxim, tried using aluminum structurally but with little success. Motor power was probably one reason, since the power-to-weight ratio is equally important in aircraft as the structural material's strength-to-weight ratio is.[23] The German, Hugo Junkers, was the first to develop all-metal airplanes. His first two planes, built in 1915 and 1916, were made from welded tin-plate manufactured as a stressed-skin structure. These planes proved too heavy and were difficult to maneuver. In 1917, Junkers switched to duraluminum, and used it in corrugated sheet form over a load-bearing structure of the same material. This configuration did not fully stress the sheet, limiting its contribution to structural strength.[24]

Starting in 1916, the US government subsidized intensive research and development programs with the intention of creating strong aluminum alloys for aeronautic applications. This resulted in Alcoa, the largest aluminum producer in America, to establish a strongly directed, systematic research and development department within the company. Unlike the plywood manufactures, Alcoa and the industry as a whole continued to develop aluminum for aeronautic applications and in a diverse range of other markets as well, including construction. Throughout the 1950s, there were many notable examples of innovative aluminum structures built. Buckminster Fuller was a particularly instrumental figure in exploiting aluminum to construct his geodesic domes and develop the aircraft inspired Dymaxion House construction system. (**Appendix A-09, p.A.379, Fig. 33 and p.A.381, Fig. 37**) Curiously, this development seems to have stopped suddenly at the end of the 1950s. Coincidentally, the aluminum can was introduced in 1956. Could the aluminum manufacturers made the same decision as the plywood manufactures to abandon a more advanced application market for a mass production market? The parallels are by no means clear since aluminum was still being developed in the aeronautic industry. It is also interesting to note that the first all-FRP house was built in 1957, and the innovative building types constructed throughout the 1960s were primarily FRP. Perhaps the market for such innovative buildings could only support one material, and aluminum's visible use in such a mundane object as a beer can may have tarnished its 'high-tech' image. This is conjecture at this point, but further research may give credence to my theory.

Since the 1950s, aluminum has primarily been developed for use in siding, façade systems, and window and doorframes. Several aluminum bridges were built in the 1950s, the most substantial being the Arvida Bridge in Quebec, Canada. (**Appendix A-09, p.A.381, Fig. 36**) The bridge is an arch and carries road traffic. Most of the other bridge examples are footbridges. The Arvida Bridge was built of aluminum for its corrosion resistance and the speed with which it could be built in an area with a short construction season because of cold temperatures and snow. No significant bridges have been built since then except mobile bridges used for temporary emergencies or by the military.

I have found only two significant structure built of aluminum in recent years. The first is the Media Center at Lord's Cricket Ground in England. (**Appendix A-09, p.A.382, Fig. 41**) The building, designed by Future Systems and Büro Happold, is a semi-monocoque structure of riveted aluminum. Shipbuilders were employed to build it.[25] The second significant structure is the main truss of the International Space Station. (**Appendix A-04, p.A.161, Fig. 13**) The truss is made of machined aluminum components. Aluminum was chosen other materials,

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[23] Jakab, p914-915.
[24] Brooks, p809.
[25] from website of Büro Happold, http://www.burohappold.com/project/lords.htp

such as FRP, because of its good resistance to extreme thermal cycling, its stiffness, and its durability because it is bombarded by atomic oxygen from the earth's atmosphere.[26]

The beneficial properties of aluminum were recognized by Deville immediately, however the commercial applications he envisioned were impractical as long as the cost of the material was high. Like iron, the use and diversity of applications increased as the price decreased. The critical cost level appears to have been about $4 a pound, in 1888, when the electrolytic process was first introduced. In this year, the Escher & Wyss Co. of Zürich, built the first all-aluminum boat. Zeppelin had determined that he could afford to pay $1.50 a pound to build his airship in 1891, though he actually paid less than 50 cents by the time he started construction in 1899. However, as long as iron could offer superior strength and stiffness for a lower prices, aluminum could not compete in static, civil infrastructure applications where the material's lightness are of less significant value. American architects Burnham and Root exploited aluminum's resistance to corrosion, formability and aesthetic attributes in Chicago's Monadnock Building (1892), as did Austrian architect Otto Wagner at the Vienna Post Office (1906). However, these were secondary building elements.

Aluminum was first exploited in structures that could exploit aluminum's favorable strength-to-weight ratio, such as small to medium size boats, military vehicles, and aeronautic applications. The development of spatial structures and the application of airplane construction techniques to buildings led to a period of experimentation after World War II. This period lasted about ten years before it came to a sudden halt. I have already posited that aluminum may have only filled a niche market for innovative structures that was subsequently supplanted by FRP materials. Further research should be able to answer why the structural use of aluminum was halted. Since then, aluminum has been relegated to secondary elements such as window frames and façade systems. The complex geometries that can be mass produced economically in aluminum by the extrusion process, and the material's resistance to corrosion do make the material appropriate for the complex details of façade systems, however pultruded FRP materials seem likely to encroach on this market because they can be formed into similarly complex profiles, comparably resist environmental degradation, and have superior thermal conductivity properties.

10.5.3 Summary of Evolutionary Characteristics of Group IV Materials

- Material has good strength-to-weight ratio, resistance to degradation.
- Potential of material hindered by high initial costs.
- Initial development supported by government funding.
- Governments supported research and development for military applications.
- Material is of limited valued in traditional building applications as long as material costs more.
- Non-structural properties must be exploited in structural applications by function integration, thereby reducing structural complexity. Holistic and life-cycle cost comparison has to be introduced to compare and compete with conventional materials.

[26] Fortner.

- Marketing material of material is important. Aluminum was seen as a progressive, wonder material after World War II, leading to the development of innovative structures. However, this development was halted before aluminum established a solid market, possibly because it lost its appeal as a ‘wonder’ material when it was marketed as ideal for beer cans.

10.6 The Application of Material Evolutionary Models

This chapter gives a broad overview of the stages of material development for several structural materials and defined four evolutionary groups that not only divide materials into groups defined by the period in which they were discovered and the time over which they were developed, but also show certain common structural and constructabilty characteristics. Furthermore, these evolutionary groups address the socio-political and economic context in which the various structural materials were developed. These evolutionary models provide a useful tool for separating contextual and material specific influences on the general development of materials from those influences that transcend time and material.

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CURRICULUM VITAE

Sean C. Dooley

Nationality	American
Education	1994 - 1998 Lehigh University, Bethlehem USA **Bachelors of Science / Civil Engineering** **Bachelors of Arts / Architecture** • Cumulative GPA 3.25 / 4.00
Languages	English / French / German
Professional experience	2003-2004 Community Development Center, Sierra Leone **Program Manager and Designer** of 2,300 m^2 building for a NGO. 2003 Restoration of Christ Church, Lausanne CH **Owner's Representative**, supervised CHF 400,000 restoration of a nineteenth century church. 2000 Competition for a Public Works Depot, Pfäffikon CH **Designer**, with Lucienne Di Biase Dooley. 1998 Conzett Bronzini Gartmann AG - engineers, Chur CH **Draftsman**, manual and CAD production of construction documents. 1995, summer Artefact, Inc. - architects, Bethlehem USA **Draftsman**, manual production of design and construction documents.
Academic experience	2002, spring Architectural Studio - Brigitte Shim, EPF-Lausanne CH **Structures Consultant / Visiting Critic** 2001 and 2002 Structural Materials - Prof. T. Keller, EPF-Lausanne CH **Lecturer on Aluminum** 2001, spring Structures II - Prof. M. Hirt, EPF-Lausanne CH **Teaching Assistant**, assisted with student exercises and questions. 1996, summer ATLSS - Prof. K. Kasai, Lehigh University USA **Assistant Researcher**, project for a steel seismic connection.
Volunteer experience	2002 - present Christ Church, Lausanne CH **Church Council / Church Restoration Committee Member**
Publications	Th. Keller, S. Dooley, J. de Castro and V. Dubois. *Use of Fibre Reinforced Polymers in Bridge Construction*. 2001. Sean Dooley, Sharon Khosla and Mark Malseed, co-editors. *Eyelevel: A Critical Journal of the Visual Arts, vol. 3*. 1999.