SEISMIC VULNERABILITY ASSESSMENT OF "SION CATHEDRAL" (SWITZERLAND): AN INTEGRATED APPROACH TO DETECT AND EVALUATE LOCAL COLLAPSE MECHANISMS IN HERITAGE BUILDINGS

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Originally published in *Prohitech '17 proceedings (2017): 3rd International Conference on Protection of Historical Constructions - 12 - 15 july 2017 . Lisbon | Portugal*

Keywords: Seismic vulnerability assessment; Masonry structures; Local mechanisms; Architectural heritage, Non-linear time-history analysis.

Abstract. *Seismic assessment of existing heritage buildings remains a challenging task. There is a high level of complexity and uncertainty compared with the assessment of standard buildings. Heritage masonry churches are usually prone to partial collapses during earthquake due to local loss of stability, and exhibit particular seismic vulnerabilities. An important step in the seismic analysis of heritage masonry buildings is the detection of local mechanisms. The Italian Building Code provides a simplified approach (LV1-churches) to assess the vulnerability of heritage churches evaluating and comparing 28 potential mechanisms. A general index of vulnerability and hierarchy between mechanisms is thereby provided. Verification of safety against local mechanisms can also be carried out using the kinematic approach. This procedure is based on evaluating the horizontal action needed to activate out-of-plane collapse mechanisms. Based on a full-scale study (Sion Cathedral), this paper evaluates the reliability of the "LV1-church" approach and of the kinematic approach through a comparison with the results obtained with a complex 3D model using the Applied Element Method.*

1 INTRODUCTION

Seismic vulnerability assessment of existing masonry buildings is a complex task. Several difficulties need to be overcome, mainly related to heritage structures. Primary uncertainties regard structural characteristics, materials properties, design drawings and a general lack of knowledge of construction techniques. To overcome such problems in seismic vulnerability assessment of existing historical masonry buildings, chronological investigations, in situ surveys and experimental tests would be needed. Unfortunately, some of these refined investigations cannot always be carried out due to heritage preservation requirements. Approaches based on simplified mechanical, statistical or qualitative models can be a useful tool for preliminary seismic assessment.

From the large amount of heritage masonry buildings, churches stand out as particularly vulnerable for their architectural, typological and construction features [1]. Continuous additions of structural components, plan extensions, chronological overlapping of construction techniques, the presence of thrusting elements and big windows, weaken the structural response to seismic loads.

The analysis of existing masonry churches affected by previous seismic events has shown local collapse is expected instead of a global failure [2]. Poor brick corner teething, inadequate connections between structural components, absence of rigid floors and roof undermine the distribution of seismic forces between vertical resisting elements for a global response. Single components behave as rigid isolated elements undergoing out-of-plane failure [3]. Detection of local collapse mechanisms is a challenge of particular importance for seismic analysis of heritage masonry buildings. It involves many considerations and usually requires expert knowledge from previous post-earthquake assessments.

This paper includes a study on the seismic vulnerability assessment of a stone masonry heritage building, the “Sion Cathedral”. The building is located in Sion, in the Canton of Valais. The Canton of Valais contains the highest seismic-hazard region in Switzerland with a peak ground acceleration of 1.6 m/s^2. For the city of Sion a specific microzonation study is available [4]. The “Sion Cathedral” belongs to the microzone MA3. The paper compares three different methodologies, characterized by an increasing in-depth analysis. The goal is a cross-validation of the three approaches in order to obtain a possible procedure for assessing heritage masonry churches. This paper continues the study on heritage masonry buildings in the city of Sion, started with the seismic assessment of the “Ancient Hôpital” [5, 6, 7].

A preliminary method taken from the Italian Building Code [8] is firstly performed. It provides an approach, based on a simplified mechanical model, to assess the vulnerability of heritage churches. The method using a standardized form (LV1-churches) evaluates and compares 28 potential local mechanisms. The objective is to determine the hierarchy between local collapse mechanisms and to provide a general index of vulnerability, i_V. Such an index is useful for comparisons on a regional scale with other heritage churches. In addition, the most dangerous local collapse mechanisms highlighted by the LV1-form have been studied using a detailed kinematic approach [9, 10]. Verification of safety against local mechanisms can be carried out by the kinematic approach if the walls are supposed to behave as rigid body and no local disaggregation of masonry is admitted. The procedure is based on the definition of appropriate collapse mechanisms and on the evaluation of the horizontal action that is needed for their activation. Finally, a refined 3D model using the Applied Element Method (AEM) has been developed for the entire cathedral. The AEM has the implicit capacity to predict a large range of typical masonry failure mechanisms. A continuous structural element of discrete materials is simulated as virtual elements connected through springs [11]. Therefore, earthquake simulations using AEM incorporate large displacements that lead to progressive formation and opening of separation joints. The model has been developed to validate the previously obtained results.

2 INFORMATION ABOUT THE SION CATHEDRAL

The “Sion Cathedral” (or the “Cathedral of Our Lady of Sion”) is the seat of the Diocese of Sion. The current gothic cathedral has been built at the end of the 15^{th} century on the foundations of a previous Romanesque church. The building is located within the city of Sion, representing an isolated structure, without any connection with surrounding buildings.

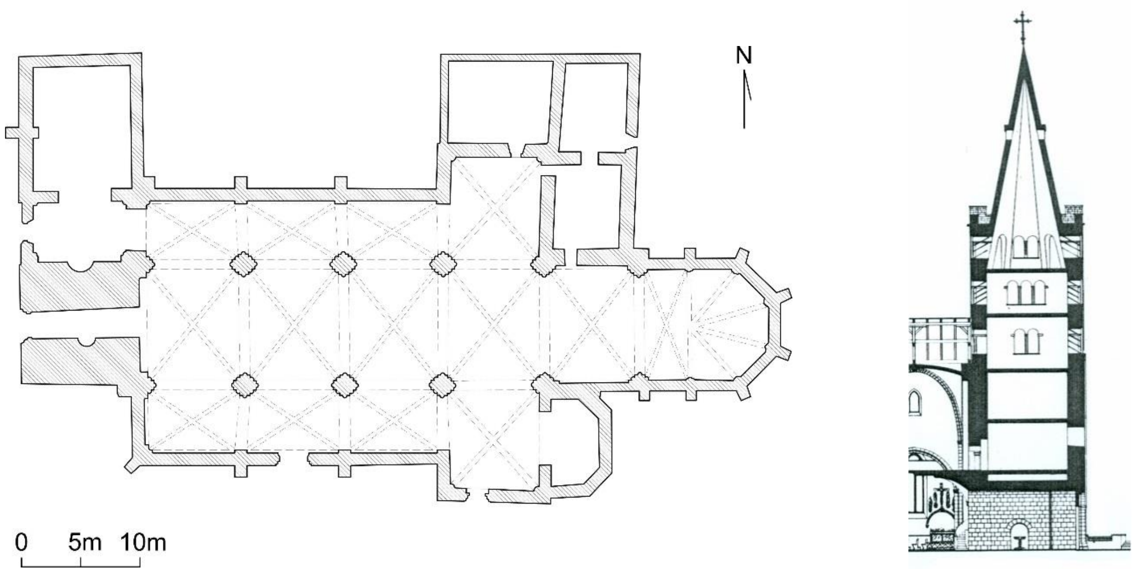
Early sources report an original Carolingian church of the 8^{th} century destroyed by fire in 1010. A second cathedral, in Romanesque style, has been built during the 11^{th} century. The massive bell tower has been built between the 12^{th} and the 13^{th} century with the addition of the last stage and the masonry spire in 1403. After a sequence of fire and rubbery, the church, still in function, needed important restauration measures. Between the end of the 15^{th} and the beginning of the 16^{th} century the current gothic church has been constructed. In 1947-1948 two bays have been added to the apse. Last restauration works took place in 1986. Under the first part of the apse, an interesting crypt, dating back to the Romanesque period, is present [12].

The “Sion Cathedral” is a stone masonry building, with overall plan maximal dimensions of 64 m x 37 m (see Figure 1a). External views of the south and west building facades are shown in Figure 2a and 2b. A cross-section of the bell tower is reported in Figure 1b.

The church is cruciform in plan with the interior covered by cross vaults. The nave, which has 3 bays, is 25 m long and 8 m wide to pillars and rises to 15 m in keystone of cross vaults. The massive cruciform pillars (2.85 m^2 in base section) separate the main nave from the two aisles. The two aisles are 4.40 m wide and rise to 9.30 m. The transept, 28 m wide, is composed of 3 cross vaulted bays, high as the nave vaults. In axis with the nave, the apse is 18.50 m long, raised at least 1 m above the level of the rest of the church. Three chapels surround the transept, two in the north side and one in the south side. The Saint-Barbe chapel, in the south east corner, is the oldest one (dated 1474). In the west side, the entrance is surmounted by a tall and massive bell tower having a height of approximately 35 / 40 m to the top of the spire. An important entrance is present in the south facade. The general thickness of the walls is about 1.00 m. Impossibility of inspection undermines the evaluation of the total external height.

The structure of the “Sion Cathedral” is composed of masonry walls defining the internal space of the church mainly covered with cross vaults. A timber and slate tile roof covers the vault system. The uncertainties about the top are due to the impossibility to visit the extrados of cross vaulted structure and to obtain clear cross section drawings. Assessing the quality of masonry typology by hole drilling has been impossible.

The external and internal outer layer of walls is covered by plaster, except in the corners where type, arrangement, state and characteristics of stones can be seen. The stones used are regularly shaped, but with different dimensions. The masonry walls show a good corner brick teething. The horizontal thrust of the roof, of arches and vaults is supported by the presence of massive buttresses all around the walls of the church. The presence of such buttresses is a good protection against seismic actions. In the transept, the presence of several metal rods contributes to the box behaviour of walls and to the protection against seismic actions and local overturning. The general state of conservation of the structure is good. Little and non-dangerous cracks are identified in the south facade, in the nave vaults and in the upper wall of the colonnade.



a) b)

Figure 1: Plan (a) and cross section of the bell tower (b) of Sion Cathedral

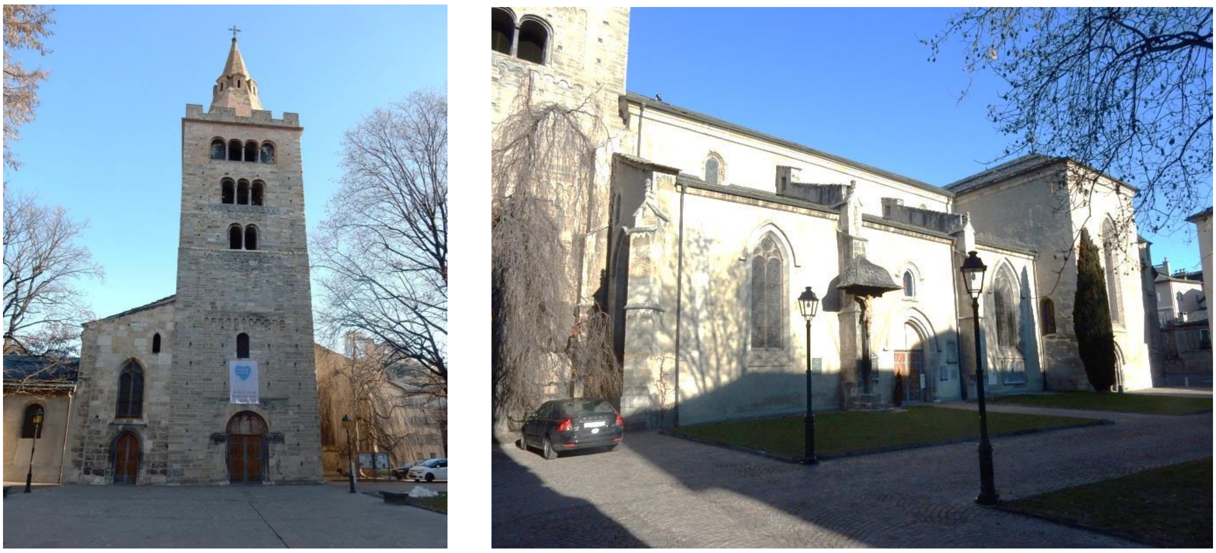


Figure 2: West facade (a) and south facade (b) of Sion Cathedral

3 SIMPLIFIED APPROACH

3.1 Analysis of a simplified index of vulnerability

The Italian Building Code, through the application of a simplified survey-form based on mechanical models, provides a qualitative method to assess the general vulnerability of masonry churches [8]. The goal of the church survey-form (LV1-churches) is the assessment of the global index of vulnerability, i_V. Such an index is useful for comparisons on a regional scale with other heritage churches. The form evaluates and compares 28 potential local mechanisms. The assessment consists of two complementary shares: vulnerability indicators and specific seismic reinforcements. Vulnerability indicators include poor masonry, presence of thrusting elements such as vaults and arches, as well as big windows that are considered as weaknesses for the portion analysed. On the other hand, seismic reinforcements comprehend buttresses, tie rods, brick teething and devices reducing the vulnerability of the mechanism. In Table 1 a specific evaluation form for the mechanism of the overturning of facade is shown.

The global index of vulnerability is given by the following expression:

$$i_v = \frac{1}{6}\frac{\sum_{k=1}^{28}\rho_k\left(v_{ki}-v_{kp}\right)}{\sum_{k=1}^{28}\rho_k}+\frac{1}{2} \tag{1}$$

where:

i_v is the global index of vulnerability;

k are the twenty-eight possible collapse mechanisms;

ρ_k is the importance weight of the mechanism (between 0.5 and 1.0; equal to 0 if the mechanism is absent) ;

v_{ki} is the generic score evaluated for the examined mechanism in term of vulnerability;

v_{kp} is the generic score evaluated for the examined mechanism in term of protection devices.

The evaluation of the v_{ki} and v_{kp} score is obtained from the survey-form and a standardization process. By the global index of vulnerability, i_v, it is possible to define the seismic limit acceleration, $a^*_{g(SLU)}$, that the structure is supposed to resist to:

$$a^*_{g(SLU)} = 0.025\cdot 1.8^{5.1-3.44\cdot i_v} \tag{2}$$

The application of the LV1-church form is shown in Table 2 with each mechanism evaluated in term of v_{ki}, v_{kp}, ρ_k and i_{vk}. The i_{vk} index is the local index of vulnerability and is defined as the difference between v_{ki} and v_{kp}. The global index of vulnerability, i_v, obtained is 0.55 (see Table 3), leading to seismic limit acceleration $a^*_{g(SLU)}$ equal to 1.606 m/s^2. The comparison with the seismic demand leads to a value of the safety coefficient α_{eff} equal to 1.24. For the microzone of Sion MA3 where the church is situated [4], the seismic demand, with an importance factor γ_1 = 1.2 and assuming a strength reduction factor q = 2, is equal to [13]:

$$a_{g(SLU)} = \frac{a_{gd}\cdot\gamma_1}{q} = \frac{1.84\cdot 1.2}{2} = 1.296\, m/s^2 \tag{3}$$

By Table 2, it is possible to prove that the most dangerous mechanisms are highlighted by high values of the local index of vulnerability i_{vk}. These are: the mechanism related to the apse roof elements (N. 21) and the mechanism of the belfry (N. 28), both with i_{vk} equal to 2.

Other important mechanisms are those with high values related to local vulnerability aspects (v_{ki} = 3); this because of the uncertainties related to the evaluation of the real capacity of same existing protection devices (especially metal rods). It is possible to identify: overturning of the south facade (N. 1); apse overturning (N. 16); in-plane shear mechanism in the apse (N. 17); mechanisms related to the bell tower (N. 27). All these mechanisms have the index of local vulnerability aspects v_{ki} equal to 3.

In relation to simplified evidences of LV1-church survey form, it must be underlined that the belfry, during the latest significant seismic event in this region in 1946, was the only section of the cathedral that has been subjected to damage. This damage is still visible by the presence in the upper mullioned windows of the bell tower of timber support elements.

Table 1: Section of the LV1-churces form related to the mechanism of the overturning of the facade.

01 - Overturning of the facade			
Possibility of activation of collapse mechanism: YES [ ] NO[ ]			
YES	NO	Seismic protection devices	Importance
[ ]	[ ]	Presence of metal chains	[ ] [ ] [ ]
[ ]	[ ]	Presence of counter-action elements	[ ] [ ] [ ]
[ ]	[ ]	Good brick corner teething	[ ] [ ] [ ]
[ ]	[ ]	…………………………………..	[ ] [ ] [ ]
YES	NO	Vulnerability aspects	Importance
[ ]	[ ]	Presence of pushing elements (arches, vaults, hip beam)	[ ] [ ] [ ]
[ ]	[ ]	Presence of big windows	[ ] [ ] [ ]
[ ]	[ ]	…………………………………..	[ ] [ ] [ ]

Table 2: Application of the LV1-church form to the Sion Cathedral

Mechanism number	Mechanism	v_{ki}	v_{kp}	ρ_k	i_{vk}
1	Overturning of south facade	3	3	1	0
2	Collapse of the top of the facade	-	-	0	-
3	In-plane mechanism of the facade	-	-	0	-
4	Narthex	-	-	0	-
5	Cross response of the church	2	2	1	0
6	In-plane shear mechanism of the side facade	2	2	1	0
7	Lengthwise response of the colonnade	2	1	1	+1
8	Nave vaults	1	0	1	+1
9	Aisles vaults	0	0	1	0
10	Overturning of the transept walls	2	3	1	-1
11	In-plane shear mechanism of the transept walls	2	2	1	0
12	Transept vaults	1	0	1	+1
13	Triumphal arches	0	1	1	-1
14	Dome - tambour	-	-	0	-
15	Lantern	-	-	0	-
16	Apse overturning	3	3	1	0
17	In-plane shear mechanism in apse	3	2	1	+1
18	Apse vaults	1	0	1	+1
19	Nave and aisles roof elements	1	2	1	-1
20	Transept roof elements	1	0	1	+1
21	Apse roof elements	2	0	1	+2
22	Overturning of chapels	1	1	1	0
23	In-plane shear mechanism in chapels	1	2	1	-1
24	Chapels vaults	0	0	1	0
25	Interaction with irregular elements	2	2	1	0
26	Projections	-	-	0	-
27	Bell tower	3	2	1	+1
28	Belfry	3	1	1	+2

Table 3: Value of global index of vulnerability, seismic limit acceleration, seismic demand and safety factor

i_v	Seismic limit $a^*_{g(SLU)}\left[m/s^2\right]$	Seismic demand $a_{g(SLU)}\left[m/s^2\right]$	$\alpha_{eff}\left[-\right]$
0.55	1.606	1.296	1.24

4 KINEMATIC ANALYSIS OF COLLAPSE MECHANISMS

4.1 Linear and non-linear approaches

Local collapse mechanisms can be evaluated by the limit equilibrium analysis according to linear and non-linear kinematic approaches with conventional rigid body mechanics [2, 9]. The procedure is based on the analysis of appropriate collapse mechanisms and on the evaluation of the horizontal load multiplier (α_0) that leads to their activation [10]. Therefore, the application of such analyse needs initial detection of mechanisms that may activate during a seismic event. These mechanisms can be identified on the basis of the presence of pre-existing cracks or by considering the damage experienced by similar structures under previous seismic actions. In addition, the quality of masonry element connections, the masonry arrangement and interlocking as well as the presence of elements such as ties or ring beams must be taken into account when defining possible collapse mechanisms.

In particular, for the linear analysis, the spectral acceleration, a_0^*, activating the mechanisms, is given by the following expression:

$$a_0^* = \frac{\alpha_0 g}{e^*} \tag{4}$$

where:

g is the gravitational acceleration;
α_0 is the load multiplier for the activation of the mechanism;
e^* is the fraction of mass participating to the mechanisms.

The linear safety of the structure against the considered collapse mechanism is satisfied if:

$$a_0^* \geq a_{g(SLU)} \tag{5}$$

where $a_{g(SLU)}$ is the seismic acceleration demand obtained from Eq. (3).

Then, through a nonlinear kinematic analysis, the capacity curve that describes the evolution of the load multiplier α for displaced configuration of the element can be determined as a function of the displacement d_k of a control point. In terms of equivalent single degree of freedom (s.d.o.f.) systems, it is possible to define the spectral capacity curve ($a^* - d^*$). The ultimate displacement capacity of the element, d_u^*, is taken as the minimum between the 40% of the spectral displacement d_0^* corresponding to the null value of a^* and the displacement that causes local instability (e.g. beams slip out of walls) [10, 13, 14]. The displacement demand, $\Delta_d = S_{De}(T_s)$, is defined on the response spectrum, as the elastic displacement demand at a certain secant period T_s. The Italian Building Code defines the secant period T_s as:

$$T_s = 2\pi\sqrt{\frac{d_s^*}{a_s^*}} \tag{6}$$

where $d_s^* = 0.4 \cdot d_u^*$ and $a_s^* = a_0^* \cdot \left(1 - \frac{d_s^*}{d_0^*}\right)$.

The non-linear safety against the considered collapse mechanism is satisfied if:

$$d_u^* \geq \Delta_d \tag{7}$$.

4.2 Verification of the local mechanisms

The analysed local collapse mechanisms are: overturning of the south facade; overturning of the apse; vertical bending of the apse; mechanisms of the central arch. These collapse mechanisms have been chosen as the most dangerous identified by the LV1-form. The central arch has been added as a term of comparison. In the case of the central arch the analysis of the mechanism has been carried out with the support of software Mc4 Loc (Mc4 Software ®). The apse roof elements (i_{vk} = 2) have not been taken into account due to lacking of information on the junction of the roof elements to the wall. The same lack of information has been found concerning the belfry (i_{vk} = 2) and the bell tower (v_{ki} = 3).

First, the linear verification is carried out and the results are summarized in Table 4. The data provided are: the horizontal load multiplier α_0 for the activation of the collapse mechanism; the participant mass M^*, evaluated considering the virtual displacement of the point of application of vertical loads as a modal shape; the fraction of mass e^* participating to the mechanisms; the seismic spectral acceleration a^*_0 activating the mechanism associated to the equivalent s.d.o.f. system; the demand in terms of acceleration $a_{g(SLU)}$; the safety factor α_{eff} is calculated considering an importance factor γ_1 = 1.2.

The obtained linear safety factor for the vertical bending of the apse is higher than one and the mechanism related to the central arch is slightly below one. The safety factors of the overturning of the south facade and of the overturning of the apse are widely under one, underlining the risk related to these mechanisms.

The verification was also carried out by a kinematic non-linear approach, taking into account the ultimate displacement capacity of the macro-elements. The results are summarized in Table 5. The data provided are: the spectral displacement d_0^* corresponding to a null value of a^*; the ultimate spectral displacement $d_u{}^*$ associated to the equivalent s.d.o.f. system and the corresponding ultimate acceleration a^*_u; the secant displacement d^*_s and the secant acceleration a^*_s, necessary for the determination of the secant period T_s; the displacement demand $\Delta_d(T_s)$.

In the case of non-linear verification, the safety factors are all above one. In general the non-linear verification provides higher safety factors than the linear verification, meaning a more reliable prediction compared to the results from numerical model (see Section 5). Only vertical bending shows a non-linear safety factor lower than that provided by linear analysis. In Figure 3 overturning of the south facade is shown.

Table 4: Results of the linear analysis

Mechanism	Load multiplier α_0 [-]	Participant mass M^* [kNs²/m]	Fraction of part. mass e^* [-]	Spectral accel. a_0^* [m/s²]	Seismic demand $a_{g(SLU)}$ [m/s2]	Safety factor α_{eff} [-]
Overturning south facade	0.095	561.33	0.98	0.948	1.296	**0.73**
Overturning of the apse	0.091	240.89	0.97	0.920	1.296	**0.71**
Vertical bending of the apse	0.362	245.05	0.98	3.618	1.296	**2.79**
Central Arch	0.149	330.37	0.85	1.276	1.296	**0.98**

Table 5: Results of the non-linear analysis

Mechanism	Spectral displ. d^*_0 [m]	Ultimate displ. d^*_u [m]	Secant displ. d^*_s [m]	Ultimate accel. a^*_u [m/s2]	Secant accel. a^*_s [m/s2]	Secant period T_s [s]	Displ. demand $\Delta_d(T_s)$ [m]	Safety factor α_{eff} [-]
Overturning south facade	0.625	0.250	0.100	0.569	0.797	2.23	0.168	**1.49**
Overturning of the apse	0.806	0.300	0.080	0.692	0.829	1.95	0.166	**1.21**
Vertical bending of the apse	0.561	0.200	0.080	2.329	3.102	1.01	0.118	**1.69**
Central Arch	0.975	0.390	0.156	0.766	1.072	2.50	0.168	**2.32**

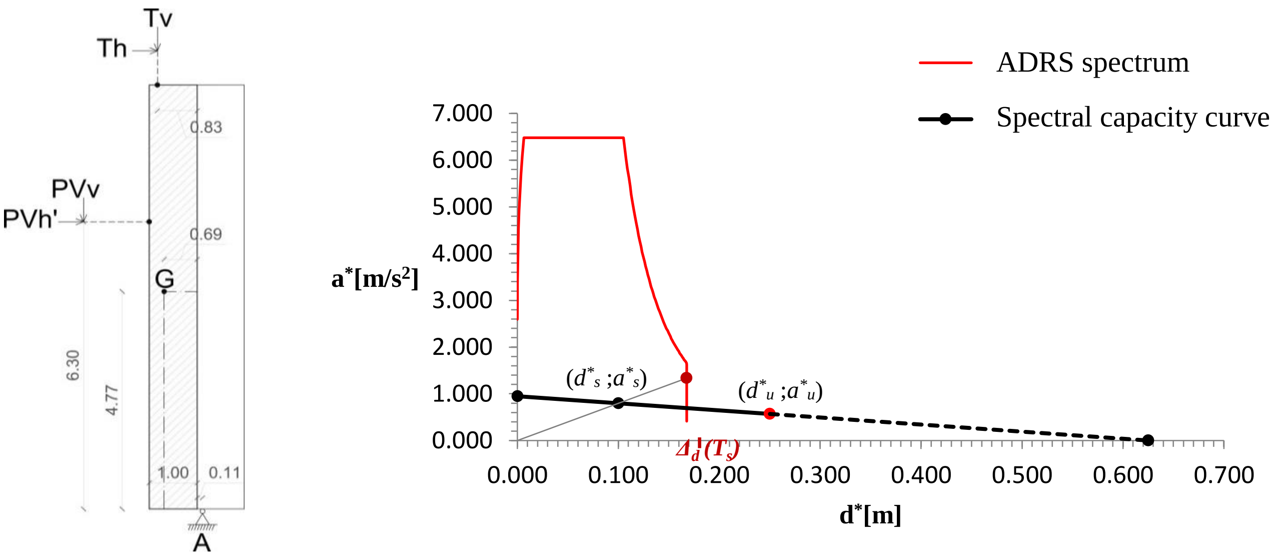


Figure 3: Overturning of the south facade mechanism: example of the mechanism and spectral capacity curve (a^*-d^*) with non-linear verification.

5 NUMERICAL MODEL OF THE CHURCH

A numerical model was built using the Applied Element Method (AEM) [15]. The AEM has the implicit capacity to describe possible failure mechanisms that are typical for masonry buildings. With AEM, the structure is modelled as an assembly of small elements. The elements are connected by normal and shear springs, located at the edges of the elements. Spring properties directly represent material features and are used to determine strains, stresses and failure criteria. The model developed using AEM thereby has the implicit capacity to predict a large range of typical masonry failure mechanisms. Therefore, earthquake simulations using AEM incorporate large displacements that lead to progressive formation and opening of separation joints. The model is here used to validate the previously obtained results.

The material behaviour is shown is Figure 4. Masonry behaviour is supposed to be similar to concrete, using Maekewa compression model [16]. The compression stress-strain diagram is defined by the initial Young's modulus, the fracture parameter and the compressive plastic strain (see Figure 4a). For the springs subject to tension, the stiffness equals the initial stiffness until reaching of cracking point. After cracking, stiffness of spring subject to tension is set to be equal to zero. The material cracks when the major principal stress reaches the tensile strength [17]. The relationship between shear strains and stresses (see Figure 4b) is supposed to remain linear till the cracking of the material. After that, the stresses drop down. The height of the drop depends on the aggregate interlock and friction at the crack surface. When the separation strain is reached, all the springs on the edge of the contact surface are cut. Separation strain represents the strain at which adjacent elements result totally separated. When elements are separated, if contact occurs again, they behave as rigid bodies. The new contacts are ruled by the normal and shear contact stiffness factors, the contact spring unloading stiffness factor and the friction coefficient [5, 6, 7].

Regarding the "Sion Cathedral", specific in situ in-depth surveys and experimental tests have not been performed. The material parameters have been chosen using Table C8A.2.1 of the Italian "Circolare Esplicativa NTC 2008 n° 617 del 02/02/2009" (pag. 403) for a masonry stone building with regular shaped stones with good arrangement and mortar. The material properties are reported in Table 6.

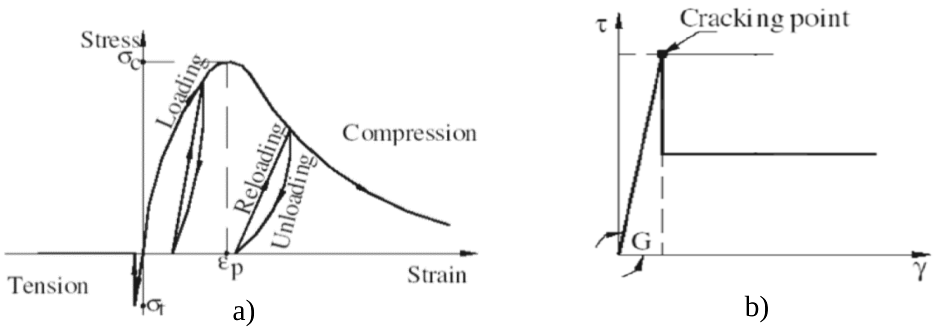


Figure 4: Assumed material behaviour: (a) tension/compression, (b) shear (pictures taken and adapted from [17]).

Table 6: Mechanical parameters for masonry

Unit weight [kN/m³]	Young's modulus [N/mm²]	Shear modulus [N/mm²]	Tensile strength [N/mm²]	Compressive strength [N/mm²]	Separation strain [-]	External damping ratio [-]
22	3840	1128	0.60	8	0.1	0

5.1 Seismic input

For the city of Sion, a specific microzonation study is available and provides the design spectra according to a return period of 475 years [4]. The "Sion Cathedral" is situated in the middle of the microzone MA3. The response spectrum of microzone MA3 is plotted as a dashed line in Figure 5b. The earthquake record chosen from international databases for the non-linear dynamic analysis has been the Record ESD 198[1], reported in Figure 5a, cut after 18 seconds. The earthquake is the same chosen for similar studies in Sion [5, 6, 7], so to have a term of comparison of the building behaviour. The response spectra in X (N-S) and Y (E-W) directions are plotted in Figure 5b.

5.2 Non-linear analysis results

The non-linear dynamic analysis was performed considering the accelerations reported in section 5.1. After such analysis, the bell tower is found to be the most vulnerable part of the church, in particular the upper part, which presents high damage and important out-of-plane failures. Most of the external walls of the church do not suffer damage. Some cracks and elements loss of connectivity can be identified in the last arch of the apse, in the upper part of the corner wall S-W and in the roof elements. The resulting damage state of the church after the application of the seismic registration can be seen at Figure 6. Some cracks and collapses appear also in the vault-resisting system that is not shown in Figure 6. It must be stressed that for computational reason the vault system has been modelled in a simplified. It is limited to provide stiffness and to connect different walls and not to simulate the real behaviour of vaults. Thus, these failures should not be considered unless a more refined and detailed model is performed.

6 SUMMARY AND CONCLUSIONS

The seismic vulnerability assessment of the "Sion cathedral" has been performed using increasingly complex methods. These different methodologies have been applied in order to provide a possible procedure to follow in the seismic analysis of such type of heritage masonry structures. The AEM model is used to cross-validate the results previously provided by a simplified approach of the LV1-church form and by a kinematic approach.

The model predictions validate the mechanism that has been identified to be the most vulnerable by the LV1-form (the mechanism related to the belfry). After a dynamic time-history simulation of an historic earthquake, the belfry (and in general the upper part of the bell tower) sustains important damage and out-of-plane failure. The other vulnerable elements stressed by the form were the apse roof elements that do not suffer direct damage. It must be stressed that some cracks are shown anyway in the last arch of the apse, meaning the weakness of this part of the church. The global index of vulnerability (i_V = 0.55) and the related safety factor (α_{eff} = 1.24) provided by the form has a general reliable evidence if related to the performance shown by the model.

The other mechanisms identified as possible vulnerable elements are studied in depth by the kinematic approach, both linear and non-linear. The results provided by the linear approach seem to be over-conservative. Three mechanisms (overturning of the south facade, overturning of the apse and the mechanism of the central arch) have safety factors that are below one, according to linear verification. In the non-linear verification, all the safety factors exceed one. Therefore, the non-linear verification seems to be more realistic and consistent with the damage configuration provided by the AEM model that does not show important cracks for the aforementioned mechanisms.

[1] Record ESD 198 (European Strong Motion Database, Montenegro Earthquake, M_S=7.1, PGA = 0.224g) [18]

To improve the seismic assessment on the “Sion cathedral”, in the next future other time-history dynamic analyses will be performed on the model with other seismic registrations that fit with the response spectrum of the microzone MA3. Furthermore, a detailed model concerning exclusively one or two bays of the nave and the aisles will be carried out in order to validate the behaviour of the vault system.

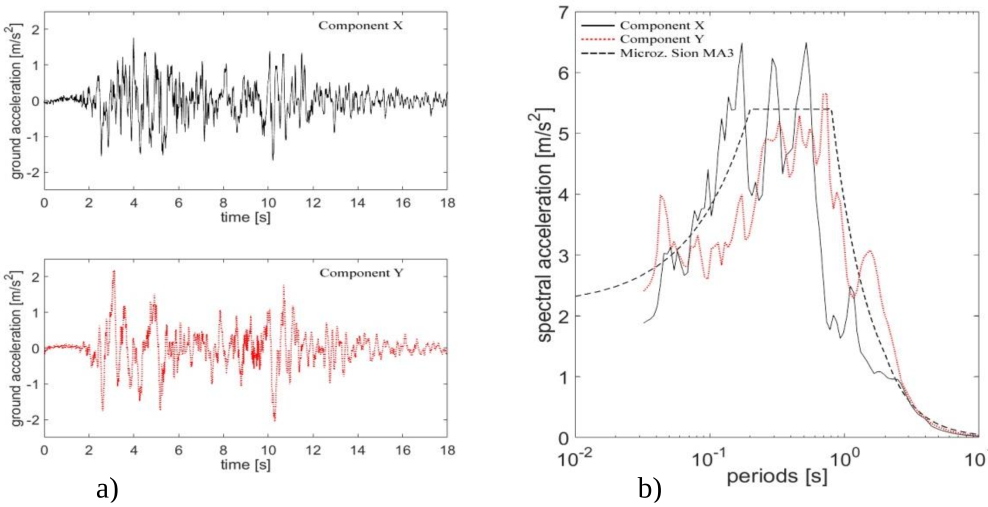


Figure 5: Record ESD 198 (European Strong Motion Database, Montenegro Earthquake, M_S=7.1, PGA = 0.224g) [18]: recorded ground accelerations in X and Y directions (a) and elastic response spectra for 5% damping together with response spectrum of microzone Sion MA3 (b).

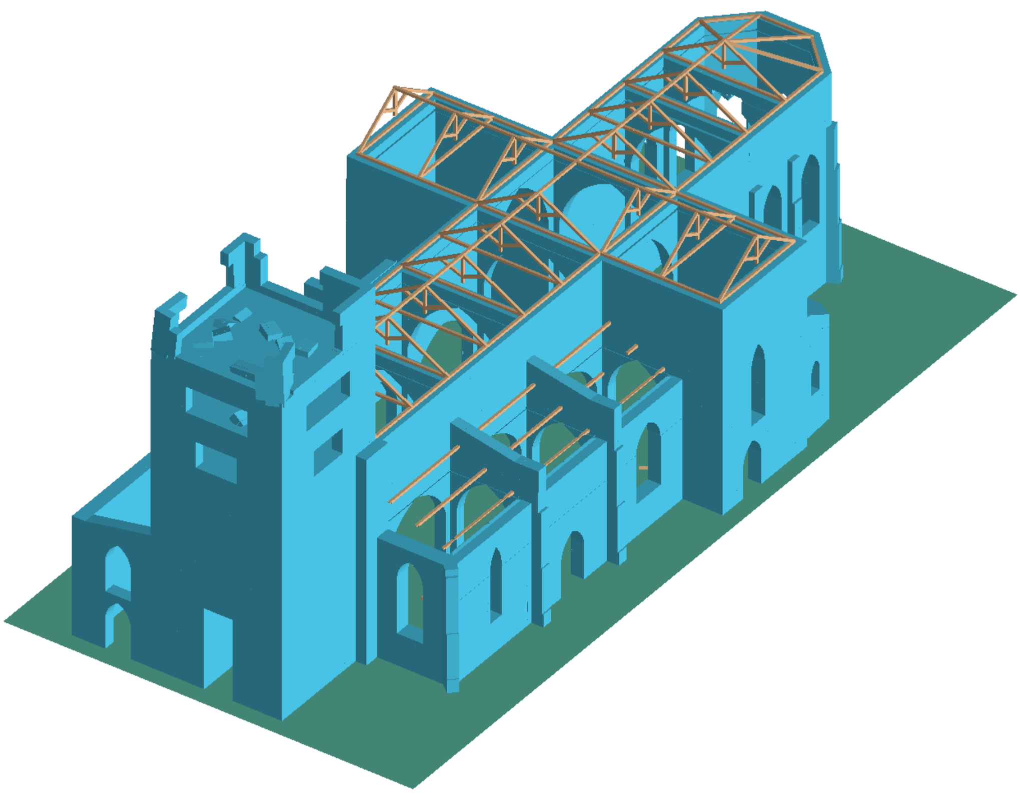


Figure 6: Damage distribution in the model after 18 seconds.

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