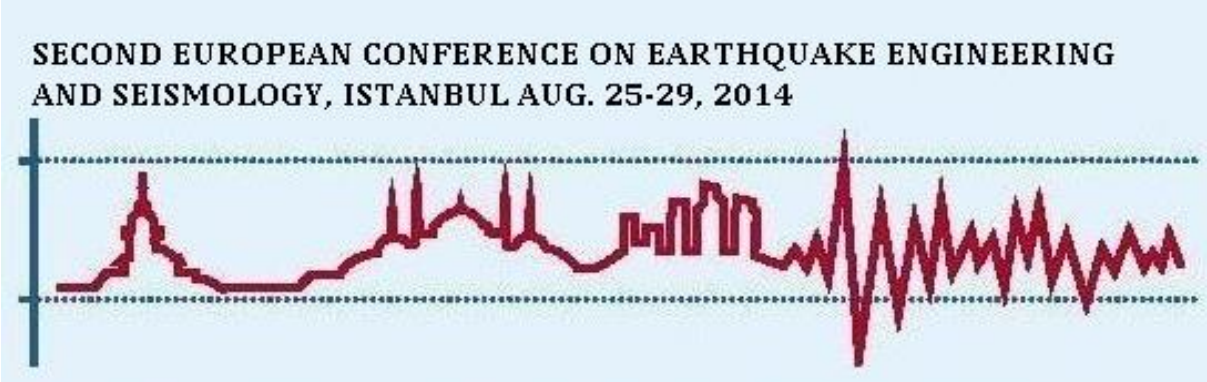

LOADING PROTOCOLS FOR EUROPEAN REGIONS OF LOW TO MODERATE SEISMICITY

Panagiotis MERGOS[1] and Katrin BEYER[2]

ABSTRACT

Existing loading protocols for quasi-static cyclic testing of structures are based on recordings from regions of high seismicity. For regions of low to moderate seismicity they overestimate imposed cumulative damage demands. Since structural capacities are a function of demand, existing loading protocols applied to specimens that are representative of structures in low to moderate seismicity regions might underestimate structural strength and deformation capacity.

To overcome this problem, this paper deals with the development of cyclic loading protocols for European regions of low to moderate seismicity. Cumulative damage demands imposed by a set of 60 ground motion records are evaluated for a wide variety of SDOF systems that reflect the fundamental properties of a large portion of the existing building stock. The ground motions are representative of the seismic hazard level corresponding to a 2% probability of exceedance in 50 years in a European moderate seismicity region. To meet the calculated cumulative damage demands, loading protocols for different structural types and vibration periods are developed. For comparison, cumulative seismic demands are also calculated for existing protocols and a set of records that was used in a previous study on loading protocols for regions of high seismicity. The median cumulative demands for regions of low to moderate seismicity are significantly less than those of existing protocols and records of high seismicity regions. For regions of low to moderate seismicity the new protocols might therefore result in larger strength and deformation capacities and hence in more cost-effective structural configurations or less expensive retrofit measures.

INTRODUCTION

Performance-based earthquake design and assessment requires reliable estimates of structural members' strength and deformation capacities. These capacities can often not be predicted accurately by analytical or numerical modelling and experimental testing is required. Most commonly, quasi-static cyclic tests are conducted where predefined displacement histories, named loading protocols, are applied at slow rates. When subjected to cyclic loading, strength and in particular deformation capacity of structural components depend on the imposed cumulative damage demand (Krawinkler et al. 2001). Hence, in order to yield realistic capacity estimates, loading protocols must reflect the estimated cumulative seismic demands for the region of interest.

Gatto and Uang (2003), for example, examined the effects of the imposed loading protocols on the strength and displacement capacities of woodframe shear walls. They observed that woodframe shear walls subjected to the SPD loading protocol (Porter 1987), which is known to overestimate seismic demands even for regions of high seismicity, had in average a 25% lower ultimate strength capacity and a 47% lower ultimate deformation capacity than woodframe shear walls tested with the

[1] Lecturer, City University, London UK, panagiotis.mergos.1@city.ac.uk
[2] Assistant Professor, EPFL, Lausanne Switzerland, katrin.beyer@epfl.ch

CUREE protocol for ordinary ground motions (Krawinkler et al. 2001), which represents better the anticipated seismic demand for regions of high seismicity. Moreover, the failure type observed for the SPD protocol was not the one developed in real earthquakes.

Several protocols have been developed in the literature for different types of structural and non-structural components. A list of these protocols includes but is not limited to: SPD protocol (Porter 1987), CUREE protocols (Krawinkler et al. 2001), EN-12512 protocol (EN 2001), protocol for short links in eccentrically braced frames (Richards and Uang 2006), FEMA-461 protocols (FEMA 2007), ISO protocol (ISO 2010) and the protocol for non-structural window systems (Hutchinson et al. 2011).

All of the above protocols have been developed for regions of high seismicity. However, earthquakes in these regions impose in average higher cumulative damage demands than earthquakes in regions of low to moderate seismicity (Kramer 1996). Hence, existing loading protocols may overestimate seismic demands for regions of low to moderate seismicity and therefore underestimate force and/or deformation capacity leading to uneconomic or even unfeasible structural designs and retrofit solutions. Furthermore, many of the existing loading protocols have not been developed to conform to the performance objectives prescribed in modern seismic design codes like EC8-Part 3 (CEN 2005). More specifically, they have been developed for seismic hazard levels corresponding to the 10% probability of exceedance in 50 years and not the 2% probability of exceedance in 50 years, which is the basis for determining displacement capacities in accordance with EC8-Part 3.

This study develops quasi-static cyclic loading protocols representative of the seismic demand in European low to moderate seismicity regions. The protocols are applicable to a wide range of structures and were developed as follows: 1) selection and scaling of ground motion records; 2) selection of representative structural systems; 3) calculation of cumulative seismic demands and 4) construction of loading protocols. The following sections outline these steps. The paper summarises the key points of a study presented in Mergos and Beyer (2014).

SELECTION AND SCALING OF GROUND MOTIONS

EC8-Part 3 defines deformation capacities Δ_{NC} at the Near Collapse "NC" performance level as the deformation related to a 20% drop of the peak strength. Deformation capacities Δ_{SD} for the Significant Damage "SD" performance level are then determined as a fraction of Δ_{NC} (e.g. 75% for concrete members and unreinforced masonry piers). Hence, in order to calculate deformation capacities for both limit states, Δ_{NC} needs to be estimated.

Unlike often assumed, force and deformation capacities of structural members are not independent of, but are rather related to demands. Hence, in order to establish by means of quasi-static cyclic testing reliable estimates of Δ_{NC} that are consistent with EC8 design objectives, the imposed loading protocol should represent the 2/50 seismic hazard level. For this reason, selection and scaling of the ground motion records in this study aim at representing the cumulative demand imposed by this seismic hazard level.

The city of Sion in Switzerland is used in this study as a representative region of low to moderate seismicity. It is situated in the Rhone Valley and the design PGA for ground type C is $0.16 \cdot 1.15 = 0.184$g for the 10/50 hazard level. For this site, de-aggregation of hazard results for the 2/50 seismic hazard level are readily available (Giardini *et al*. 2004). In total, 60 ground motion records were selected to represent the seismicity of Sion in accordance with several selection criteria. The applied selection criteria and the complete set of records can be found in Mergos and Beyer (2014). In addition to the 60 ground motion records representative of low to moderate seismicity regions, the 20 ground motion records employed for developing several protocols for high seismicity regions (e.g. Krawinkler *et al.* 2001, FEMA-461 2007) are also examined herein for comparison reasons

The selected ground motion records are scaled one by one in order to match the spectral acceleration of the horizontal elastic spectrum of EC8 for the 2/50 seismic hazard level at the fundamental period of the structure. The same procedure was adopted by Krawinkler *et al*. (2001). The target EC8 elastic spectrum is derived for soil class C. The PGA for the 2/50 seismic hazard level is calculated by multiplying the PGA for the 10/50 hazard level by the importance factor γ_I in EC8-Part 1 (CEN 2004):

$$\gamma_I = \left(\frac{P_L}{P_{LR}} \right)^{-1/k} = \left(\frac{2}{10} \right)^{-1/3} \approx 1.71 \tag{1}$$

In this equation, P_L is the target probability of exceedance in 50 years (2%) and P_{LR} is the reference probability of exceedance in 50 years (10%). The parameter k is an exponent that depends on the seismicity and which, according to EC8, is generally of the order of 3. The PGA on rock for the 10/50 seismic hazard level and the site of Sion is taken equal to 0.16g (SIA 2003), while for the high seismicity earthquakes it is taken equal to 0.40g. The latter value applied to the EC8 spectrum yields the same plateau acceleration as the response spectrum employed in the study by Krawinkler *et al*. (2001) who examined the seismic demand for regions of high seismicity for the 10/50 hazard level.

SELECTION OF REPRESENTATIVE STRUCTURAL SYSTEMS

Cumulative damage effects imposed by ground motions are strongly dependent on the type of structural system. Hence, structural systems representative of those that will be tested need to be examined when developing loading protocols. In this study, the following structural systems are considered: elastic systems, systems for which lateral resistance is provided by timber walls, reinforced concrete (RC) frames, RC walls, unreinforced masonry shear or rocking walls.

SDOF systems are employed to model the structural response. Previous studies comparing SDOF and MDOF systems (FEMA-461 2007) have revealed that for short-period MDOF systems the demand on the structural components is well correlated with the demand on the SDOF system representing the first mode. For long-period MDOF systems, higher mode effects may become more important. However, as it will be shown in the following, cumulative damage effects for long-period systems are much less significant than for short-period systems. Hence, only SDOF systems are considered within the scope of this study. Nevertheless, it should be kept in mind that the proposed loading protocols are not representative of structural systems with important higher mode effects or MDOF systems with a strong concentration of inelastic deformations (e.g. soft storeys).

To be representative of a particular structural system, the SDOF system has to be assigned an appropriate force-displacement hysteretic model. Table 1 summarizes the structural systems and the corresponding hysteretic models employed in this study. Following the suggestions by Priestley *et al.* (2007), the ‘fat’ Takeda hysteretic model is applied for RC frames and the ‘thin’ Takeda hysteretic model for RC walls. The latter can also be used as rough approximation of the hysteretic response of unreinforced masonry shear walls (Aldemir *et al.* 2013). For rocking masonry walls a flag-shaped hysteretic model is chosen. The Wayne Stewart hysteretic model is adopted for timber walls with the hysteretic parameter values that Stewart (1987) proposed for plywood sheathed timber walls. The elastic model is used for all structural systems expected to respond in the elastic domain even for the 2/50 seismic hazard level. The exact values of the parameters adopted for each hysteretic model can be found in Mergos and Beyer (2014).

Table 1 summarises the range of periods of vibration T and post-yield stiffness ratios r (ratio of post-yield to elastic stiffness) of the SDOF systems that are considered in this study. The period range reflects typical fundamental periods of a large portion of the existing building stock in Europe. The lowest period for RC frames is taken equal to 0.15s and not 0.10s as for the other structural systems. This is in line with the empirical formula in EC8-Part 1 (§4.3.3.2.2(3)) for estimating the fundamental period of vibration for single storey RC frames. Moreover, higher post-yield stiffness ratios have been adopted for timber walls than for other structural systems in accordance with experimental results by Stewart (1987).

The q-factors have been chosen following the recommendations in EC8-Part 1. The yield strength F_y of the SDOF systems is calculated from the ordinate of the EC8 design spectrum for the 10/50 seismic hazard level, the period T and the q-factor of the SDOF system. The viscous damping ratio ζ is assumed equal to 5% for all structural systems. In total, 567 different SDOF systems are examined.

Table 1. SDOF systems representative of a large part of buildings in Europe

Structural systems	Hysteretic model	Period T (sec)	Hardening ratio r	Behaviour factor q
Infinitely elastic	Elastic (EL)	0.10, 0.20, 0.30, 0.50, 0.75, 1.00, 1.50	-	-
Timber walls	Wayne Stewart (WS)	0.10, 0.20, 0.30, 0.50, 0.75, 1.00, 1.50	0.001, 0.01, 0.10, 0.40	1.0, 2.0, 3.0, 4.0, 5.0
RC frames	'Fat' Takeda (FT)	0.15, 0.30, 0.50, 0.75, 1.00, 1.25, 1.50	0.001, 0.01, 0.05, 0.10	1.0, 2.0, 3.0, 4.5, 6.0
RC and masonry shear walls	'Thin' Takeda (TT)	0.10, 0.20, 0.30, 0.50, 0.75, 1.00, 1.50	0.001, 0.01, 0.05, 0.10	1.0, 2.0, 3.0, 4.5, 6.0
Masonry rocking walls	Flag shaped (FS)	0.10, 0.20, 0.30, 0.50, 0.75, 1.00, 1.50	0.001, 0.005, 0.01, 0.05	1.0, 1.5, 2.0, 2.5, 3.0

CALCULATION OF SEISMIC DEMANDS

This section evaluates the cumulative seismic demands imposed on the structural systems by the scaled ground motion records. To serve this goal, an application named Protocol.m is developed in MATLAB v7.11 (2010). The steps followed in order to calculate cumulative seismic demands are outlined in the following.

Initially, linear and nonlinear time history analyses were carried out by means of the software RUAUMOKO (Carr 2012) using the Newmark constant acceleration integration algorithm and an analysis time step of 0.001s. Tangent stiffness proportional damping was applied as recommended by Priestley and Grant (2005). For each combination of SDOF system and ground motion record, Protocol.m writes the input file, executes RUAUMOKO and reads the output results. In total, 567 (SDOFs) x 80 (ground motions) =45360 time history analyses were conducted.

Cumulative seismic damage effects are a function of the number, ranges, means and sequence of the imposed deformation cycles (Krawinkler *et al*. 2001). To determine the first three parameters, all displacement responses obtained by time history analyses of the SDOF systems are re-arranged using the simple rainflow cycle counting algorithm by Downing and Socie (1982). This method identifies cycles as closed hysteretic loops and provides their ranges (difference between maximum and minimum peak) and means (average value of minimum and maximum peak).

The calculated cycle ranges are centred with respect to zero and normalized with respect to the maximum cycle range divided by two. This assumes that the cycle means are close to zero and the displacement history can be approximated by symmetric cycles around a zero mean. Finally, normalized cycle ranges are arranged in descending order. The implications of the assumptions adopted in the afore-described methodology are explained in detail in Mergos and Beyer (2014).

Fig. 1 summarizes the adopted methodology for a timber wall SDOF system with fundamental period T=0.20s, post-yield stiffness ratio r=1% and q-factor=1, which is subjected to the Umbria Marche (1997) aftershock ground motion record (M_w=5.6, R=13km, PGA=0.09g, Soil type C). Fig. 1a presents the lateral displacement response of the SDOF system. In the same figure, the pre-peak response that will be used for determining the imposed cycle demands is highlighted. Fig. 1b presents the force vs. displacement hysteretic response. Following Wayne Stewart's hysteretic model, this response is characterized by significant pinching and cyclic strength deterioration. Note that inelastic response is developed despite the fact that this SDOF system was designed for q=1. The SDOF system responds in the inelastic range because it is examined for the 2/50 seismic hazard level while it was designed for the 10/50 seismic hazard level.

Fig. 1c presents displacement cycle amplitudes, which are defined in the following as cycle ranges divided by 2. Cycle ranges are determined by the rainflow cycle counting method for the pre-peak displacement response of Fig. 1a, then they are centred with respect to zero and finally they are placed in descending order. For example, using rainflow counting, the range of the maximum cycle of the pre-peak displacement response in Fig. 1a was calculated to be 0.022m. This results in a symmetric cycle with a displacement amplitude of 0.011m around a zero mean. In addition, Fig. 1d shows the same amplitudes normalized with respect to the maximum amplitude. As a result, normalized amplitudes of the first cycle are equal to 1 and of the remaining cycles less than one.

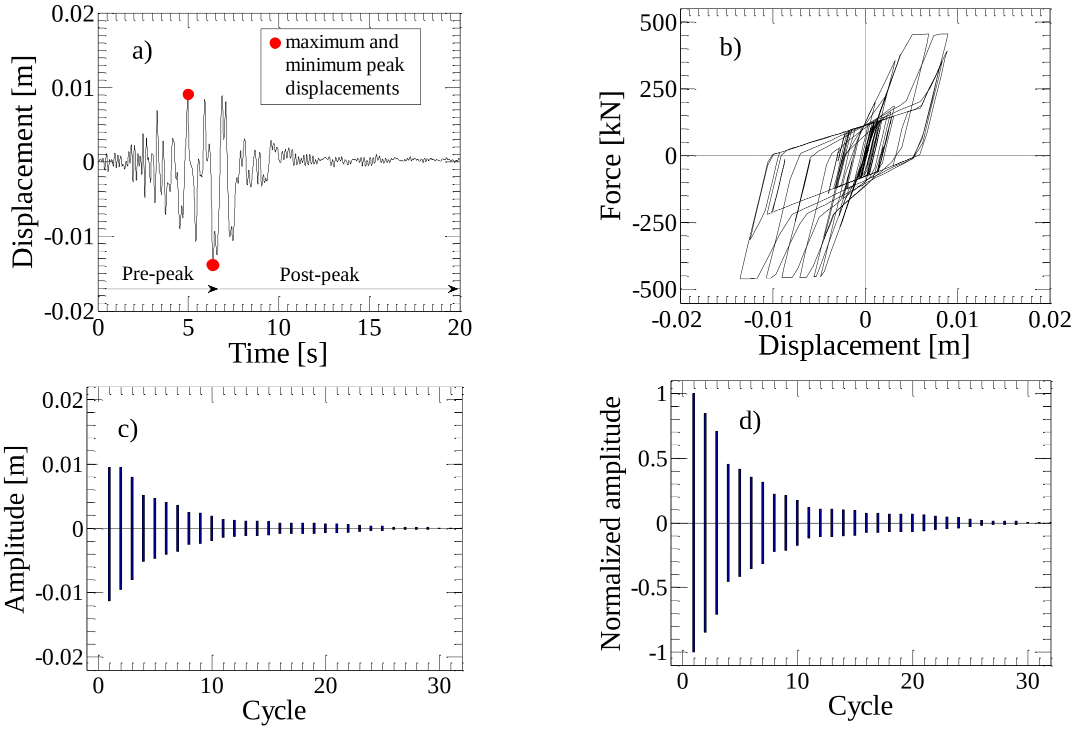

Figure 1: Seismic demand on an example SDOF system representing a timber wall building with T=0.20s, r=1% and q=1 subjected to the Umbria Marche (1997) aftershock record: a) lateral displacement response; b) force-displacement hysteretic response; c) ordered cycle amplitudes; d) ordered normalized cycle amplitudes

As proposed by FEMA-461, the loading protocols will reflect the median values of the normalized cycle amplitudes. This is in good agreement with EC8-Part 1 (§4.3.3.4.3(4)) which allows that the average value of all analyses is used as design value if the response is obtained from more than 7 different accelerograms.

To analyse the data of each SDOF system, the median values of the normalized cycle amplitudes of the two sets of records are evaluated. The first set comprises the 60 ground motion records for the low to moderate seismicity case and the second set the 20 ground motion records for the high seismicity case (Krawinkler *et al*. 2001). The median normalized cycle amplitudes are calculated as the median of the 1st, 2nd, 3rd … largest cycle of all ground motion records of one set (FEMA-461 2007). As all amplitudes have been normalized by the maximum amplitude and arranged in descending order, the amplitudes of all first cycles are equal to one and therefore also their median is equal to one. For the 2nd, 3rd … largest cycle the median values of the normalized amplitudes are always smaller than one.

After evaluating the statistical measures of normalized cycle amplitudes, parametric analyses are conducted in order to determine the most critical SDOF systems in terms of cumulative seismic demands. Two important cumulative demand parameters are examined, namely the number of damaging cycles N and the sum of normalized cycle amplitudes $\Sigma\delta_i$ as determined by the median normalized cycle amplitude sequences of the SDOF systems. The same parameters for determining cumulative damage demands have been used in several previous loading protocol studies (e.g. Richards and Uang 2006). The calculated variations of the cumulative damage parameters with the vibration period, behaviour factor q, hardening ratio r and hysteretic model can be found in Mergos and Beyer 2014.

Fig. 2 compares the cumulative demand parameters of the median normalized cycle amplitude sequences as derived from the 60 low to moderate seismicity ground motion records with those from the 20 high seismicity records (Krawinkler *et al*. 2001). The figure clearly underscores that high seismicity records impose higher cumulative demands than low to moderate seismicity records. This applies in particular to the elastic systems or systems responding in the low ductility range, which are also the systems subjected to the largest cumulative demands and which will therefore govern the design of loading protocols. This finding advocates the usage of different loading protocols for low to moderate seismicity regions and high seismicity regions. It is recalled that Fig. 2a refers to the sum of normalized cycle amplitudes with respect to the maximum displacement Δ_{max}. A comparison of the sum of non-normalized cycle amplitudes $\Sigma\Delta_i$ would of course be much more severe for the high seismicity records.

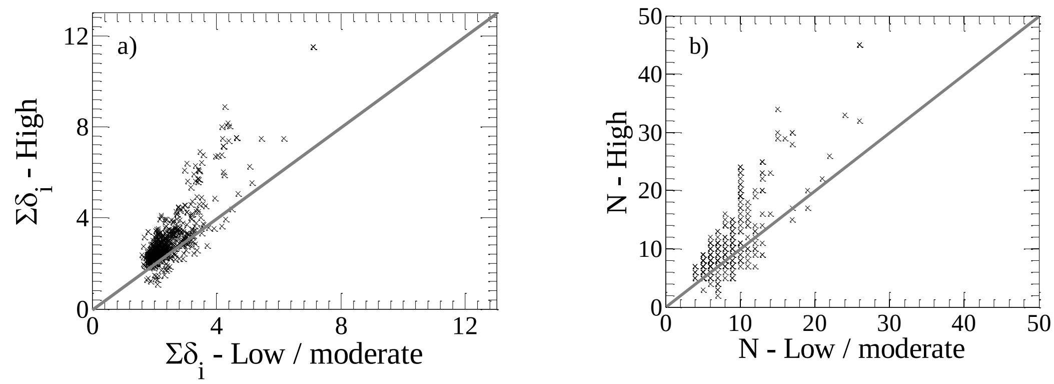

Figure 2: Comparison of cumulative seismic demand parameters calculated for low to moderate and high seismicity regions: a) $\Sigma\delta_i$; b) N. Each point represents the cumulative damage parameters of a particular SDOF system calculated from its median normalized cycle amplitude sequence.

CONSTRUCTION OF LOADING PROTOCOLS

The algorithm for constructing loading protocols developed in this study aims at describing the normalized ordered amplitude sequence of the SDOF system as an analytical function with empirical coefficients. The method is based on similar procedures developed in previous studies on loading protocols (Richards and Uang 2006; Hutchinson *et al*. 2011). Unlike in previous studies, however, cycle amplitudes of the loading protocol are expressed as analytical functions of the load step, which allows describing different loading protocols for different structural systems by only two parameters.

Each loading protocol consists of n load steps with n_1 cycles of the same amplitude per step. The loading protocol comprises therefore in total $n_{tot}=n \cdot n_1$ cycles. Before constructing the loading protocol, the number of cycles per step n_1 is chosen. Typically, two (e.g. FEMA-461) or three (e.g. ISO-21581) cycles per load step are assigned, which allows investigating the stiffness and strength degradation of the structural component that is tested. As the number of equal cycles per step decreases, the SDOF's ordered amplitude sequence obtained from time history analysis can be represented with higher accuracy. As a limit case, when each cycle is assigned a different amplitude, the actual SDOF's amplitude sequence can be obtained. In order to give the applicant the largest possible choice with regard to the form of the loading protocol, loading protocols for all three options (one, two and three cycles per step) are developed.

The SDOF system's normalized amplitude sequence is obtained using the methodology described in previous sections and the corresponding empirical cumulative distribution function (CDF) is constructed. The latter reflects the distribution of the median values of the normalized cycle amplitudes. Additionally, the cumulative damage effect (CDE) of the SDOF system cycle sequence is calculated. The basis for calculating the CDE is the following general damage model, which is based on Miner's rule (Krawinkler *et al*. 2000, Richards and Uang 2006):

$$CDE = C \cdot \sum_{i=1}^{N} \left(\Delta_i\right)^c = C \cdot \left(\Delta_{\max}\right)^c \cdot \sum_{i=1}^{N} \left(\delta_i\right)^c \tag{2}$$

where C and c are structural performance parameters. The parameter c is typically greater than 1 reflecting the fact that larger cycles cause more significant damage than small cycles (Richards and Uang 2006).

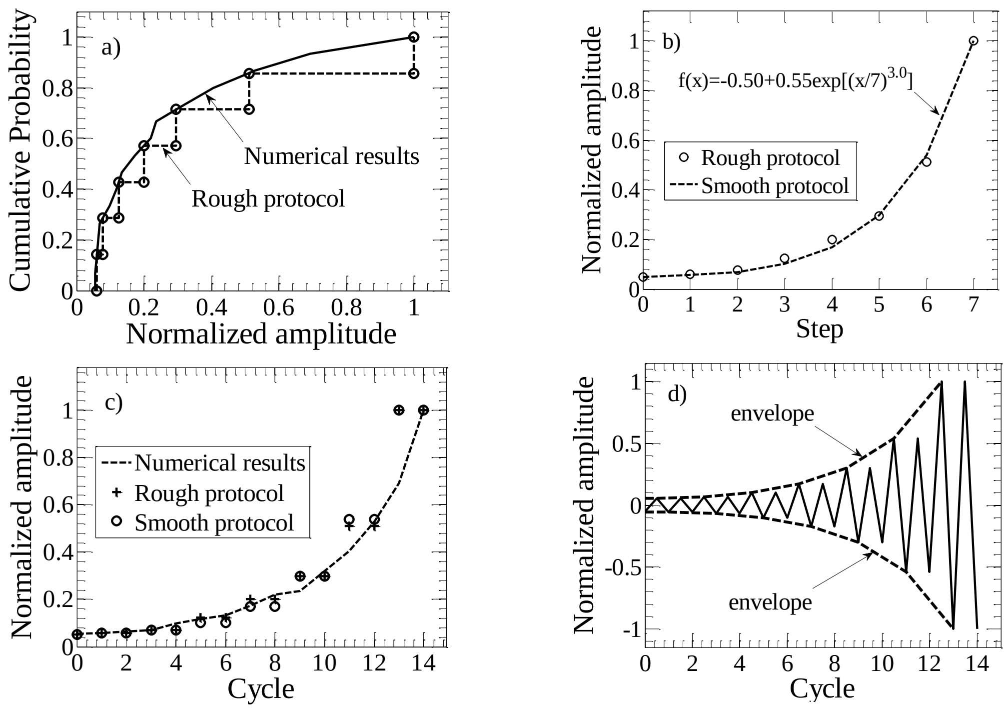

Figure 3: Loading protocol construction: a) comparison of loading protocol and numerical results normalized cycle amplitude CDFs; b) comparison of rough and smooth protocol normalized load step amplitudes; c) normalized cycle amplitude sequences of the numerical results, the rough and the smooth protocol and d) derived normalized loading protocol

As a first step when constructing the loading protocol, a while-loop is launched, where the number of total steps n progressively increases. For each value of n, first the protocol cycle step amplitudes are determined to match SDOF's and protocol's CDF for each load step (see Fig. 3a) and then protocol's CDE is calculated. The while-loop terminates when protocol's CDE exceeds for the first time SDOF's CDE.

For the construction of loading protocols, the value of c is assumed as unity. If a protocol's CDE exceeds the SDOF's CDE for c=1, then the same holds for all values of c>1. This applies because the proposed methodology for deriving the loading protocol tends to impose more cycles with large amplitudes than resulted from the numerical analyses of the SDOF systems (Fig. 3a). Hence, c=1 may be considered a conservative assumption. As only the relative and not the absolute magnitude of the CDE is of interest, the choice of C is irrelevant.

Fig. 3 presents the loading protocol development for the median normalized amplitude sequence of the SDOF system described in Fig. 1. For two cycles per step, the algorithm yields seven steps (14 cycles in total). Fig. 3a presents for this SDOF system the comparison of the CDF as obtained from the numerical results and as calculated from the derived protocol. The loading protocol CDF meets the SDOF's CDF at the end of each load step (every two cycles). In this manner, the loading protocol's CDF approaches and remains always below the SDOF's CDF. This is on the conservative side since it indicates that the protocol comprises always a higher percentage of large amplitude cycles, which are more damaging than small amplitude cycles.

The previous methodology yields arbitrary loading protocol cycle amplitudes which may change abruptly between two subsequent load steps ('rough' loading protocol). In order to smooth the loading protocol curve, the following general exponential function is fitted to the rough protocols:

$$f(t) = \frac{1}{e-1} \cdot \left[\delta_o \cdot e - 1 + (1-\delta_o) \cdot \exp\left(t^a\right) \right] \tag{3}$$

where δ_o is the threshold for damaging cycles (assumed 0.05 herein), t=x/n, x is the current load step, n is the number of load steps and α is a parameter describing the rate of amplitude increase. The proposed function approaches for t=0 δ_o and for t=1 unity. Hence, it always satisfies the boundary conditions of the loading protocols proposed in this study. The form of Eq. (3) was chosen because it yields in almost all cases superior fits than polynomial or power functions. Substituting δ_o=0.05 and t=x/n into Eq. (3), one obtains:

$$f\left(x\right) = -0.50 + 0.55 \cdot \exp\left[\left(\frac{x}{n}\right)^{\alpha}\right] \tag{4}$$

Eq. (4) requires only two parameters (i.e. n and α) for fully determining the normalized loading protocol sequence. The parameter α is calculated in order to provide the best fit between the 'rough' and the 'smooth' protocol, which minimizes the sum of squared errors between the predictions of Eq. (4) and the normalized amplitudes of the 'rough' protocol.

Fig. 3b compares for the example SDOF system the predictions of Eq. (4) for n=7 and α=3.00 with the normalized amplitudes of the rough protocol and shows that the amplitudes of the rough and smooth protocol do not differ significantly. Furthermore, Fig. 3c compares the normalized cycle amplitudes of the SDOF system as derived from the numerical analyses (placed now in ascending order for comparison purposes), with the normalized cycle amplitudes of the rough and the smooth protocol. The protocols follow closely the SDOF's median response, yet remaining conservative for the large cycle amplitudes. Fig. 3d illustrates derived smooth normalized loading protocol. It consists of seven load steps of two equal cycles yielding fourteen cycles in total. The amplitudes are determined by the envelope function defined by Eq. (4) for n=7 and α=3.00. Note that x in Eq. (4) is the load step and not the cycle.

PROPOSED LOADING PROTOCOLS

Most existing loading protocols were developed in order to meet the demands on the structural system that is subjected to the largest cumulative damage demand. However, this results inevitably in overly demanding protocols for all other structural systems. Existing protocols feature further a fixed number of cycles per load steps. The new loading protocols limit these drawbacks by developing the loading protocols as functions of seismicity (low to moderate vs. high), period and hysteretic model. For each of these combinations, the loading protocol is developed for the pair of q-factor and post-yield stiffness ratio that yields the largest CDE. In addition, the new loading protocols allow to choose between one, two and three cycles per step.

Table 2 summarizes the resulting protocol parameters n and α that were derived from the median values of cumulative damage demands for different structural configurations, levels of seismicity and cycles per load step. It is recalled that α describes the increase in amplitude with load step and n the number of load steps. If, for example, two cycles per load step are assigned, the total number of cycles n_{tot} is 2n.

For short natural periods, cumulative damage demands decrease with period. For periods longer than T=0.5s, however, cumulative damage demands tend to converge towards a constant value. Hence, for systems with T≥0.5s, protocols derived for T=0.5s will be adopted. The slight conservatism resulting for longer period structures may compensate partly for the higher mode effects of long-period MDOF systems. It is however recalled that the proposed loading protocols cannot represent structural systems with significant higher mode effects or MDOF systems with a significant concentration of inelastic deformations (e.g. structures forming soft storey mechanisms).

The loading protocols proposed in Table 2 are all normalized with respect to the maximum displacement Δ_{max}. Before performing a quasi-static cyclic test, Δ_{max} needs to be estimated. Since the cumulative demand was determined for the seismic hazard corresponding to the NC limit state, the parameter Δ_{max} corresponds to the displacement capacity of the specimen which EC8-Part 3 (2005) defines as the displacement associated with a strength loss of 20% of its maximum strength. This displacement can be estimated by analytical, numerical or empirical models or by performing first a monotonic test and then assigning an appropriate reduction factor, which relates cyclic to monotonic displacement capacities.

Table 2. Proposed loading protocol parameters

Structural system-Hysteretic model	Vibration period	Low to moderate seismicity			High seismicity		
		n_1=1	n_1=2	n_1=3	n_1=1	n_1=2	n_1=3
Infinitely elastic-Elastic (EL)	T=0.1s	n=26 α=3.05	n=12 α=3.05	n=8 α=3.01	n=45 α=3.24	n=22 α=3.22	n=14 α=3.25
	T=0.2s	n=14 α=1.96	n=6 α=2.00	n=4 α=1.87	n=25 α=2.42	n=12 α=2.44	n=8 α=2.36
	T=0.3s	n=10 α=1.49	n=5 α=1.45	n=3 α=1.45	n=24 α=2.51	n=12 α=2.49	n=7 α=2.52
	T≥0.5s	n=7 α=1.58	n=3 α=1.56	n=2 α=1.60	n=11 α=2.01	n=5 α=1.98	n=3 α=2.03
Timber walls-Wayne Stewart (WS)	T=0.1s	n=27 α=3.94	n=12 α=3.97	n=7 α=3.81	n=32 α=3.62	n=15 α=3.58	n=9 α=3.49
	T=0.2s	n=15 α=2.96	n=7 α=2.93	n=4 α=2.85	n=34 α=3.22	n=16 α=3.21	n=10 α=3.21
	T=0.3s	n=13 α=3.16	n=6 α=2.98	n=3 α=2.71	n=23 α=2.44	n=11 α=2.4	n=7 α=2.45
	T≥0.5s	n=11 α=3.16	n=5 α=3.07	n=2 α=2.48	n=14 α=2.91	n=6 α=2.75	n=3 α=2.56
RC frames-Fat Takeda (FT)	T=0.15s	n=16 α=3.37	n=7 α=3.3	n=4 α=2.93	n=30 α=2.82	n=14 α=2.80	n=9 α=2.78
	T=0.3s	n=10 α=1.98	n=5 α=1.96	n=2 α=1.85	n=20 α=2.0	n=10 α=1.94	n=6 α=1.9
	T≥0.5s	n=6 α=2.06	n=2 α=1.66	n=2 α=1.66	n=12 α=2.57	n=5 α=2.40	n=3 α=2.43
RC & masonry shear walls-Thin Takeda (TT)	T=0.1s	n=24 α=4.23	n=11 α=4.17	n=6 α=4.03	n=33 α=4.24	n=16 α=4.19	n=10 α=4.11
	T=0.2s	n=13 α=2.3	n=6 α=2.26	n=3 α=2.2	n=23 α=2.63	n=11 α=2.66	n=7 α=2.55
	T=0.3s	n=10 α=2.15	n=5 α=2.16	n=2 α=2.22	n=20 α=2.3	n=10 α=2.28	n=6 α=2.3
	T≥0.5s	n=7 α=1.7	n=3 α=1.63	n=2 α=1.69	n=13 α=2.23	n=6 α=2.27	n=3 α=2.06
Masonry rocking walls-Flag-shaped (FS)	T=0.1s	n=8 α=1.2	n=4 α=1.21	n=2 α=1.21	n=15 α=2.3	n=7 α=2.25	n=4 α=2.38
	T=0.2s	n=12 α=2.28	n=5 α=2.25	n=3 α=2.36	n=16 α=3.05	n=7 α=2.96	n=4 α=2.92
	T=0.3s	n=9 α=1.89	n=4 α=1.83	n=2 α=1.85	n=17 α=2.85	n=8 α=2.86	n=5 α=2.83
	T≥0.5s	n=6 α=1.51	n=3 α=1.63	n=2 α=1.31	n=10 α=2.02	n=5 α=2.03	n=2 α=1.73

As an alternative Δ_{max} can be taken as the target displacement demand for which the structural component is to be qualified (Krawinkler 2009). This displacement may be determined by nonlinear time history analyses or simpler methods like the capacity spectrum method or the displacement coefficient method (FEMA-273 1997). In this case, the loading protocols can be used to verify the adequacy of the test specimen for the specific seismic demand.

As example, loading protocols for a structure with RC shear walls and T=0.2s are constructed. Table 2 shows the corresponding loading protocol parameters for one to three cycles per load step: n=13 and α=2.3 when n_1=1, n=6 and α=2.26 when n_1=2 and n=3 and α=2.2 when n_1=3. Using the approach in EC8-Part 3, the NC chord rotation capacity of the RC shear wall is estimated as 1.8%. The resulting loading protocols for this SDOF system are presented in Fig. 4.

The amplitudes of the load steps are:

n_1=1: 0.10, 0.11, 0.13, 0.17, 0.21, 0.28, 0.37, 0.48, 0.63, 0.82, 1.07, 1.38, 1.80%

n_1=2: 0.12, 0.18, 0.33, 0.59, 1.03, 1.80%

n_1=3: 0.19, 0.60, 1.80%

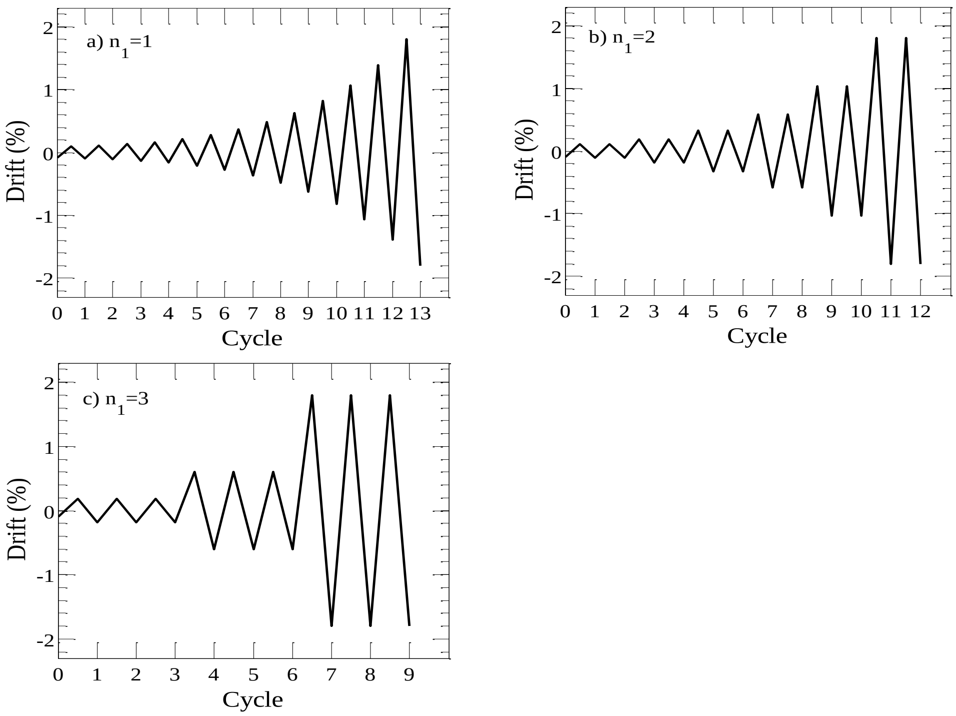

Figure 4: Example loading protocols for an RC shear wall structure with elastic period of vibration T=0.2s in a region of low to moderate seismicity a) one cycle per step; b) two cycles per step; c) three cycles per step

COMPARISONS WITH EXISTING PROTOCOLS

This section identifies trends in the proposed loading protocols and compares them to three well established loading protocols for quasi-static cyclic testing: the CUREE protocol developed for woodframed shear wall structures and ordinary ground motions (Krawinkler *et al*. 2001); the FEMA-461 displacement controlled protocol for drift sensitive non-structural components (FEMA, 2007); and the ISO-21581 (ISO 2010) protocol for timber shear wall structures. All these protocols express the loading history as a function of the peak displacement which facilitates the comparison.

Fig. 5 compares the new and existing protocols in terms of the sums of normalized displacements $\Sigma\delta_i$. This cumulative damage parameter is chosen because it contains information on the number and amplitudes of the cycles in the loading protocol. In this figure, structural systems are annotated with two letters followed by a decimal number. The two letters identify the hysteretic model (see Table 1) and the decimal number represent the natural period in seconds. Note that – unlike the new protocols – the CUREE, FEMA-261 and the ISO-21581 protocols are all independent of the structure's fundamental period. The new protocols are all evaluated for two cycles per load step.

The figure shows that the new protocols for low to moderate seismicity impose always significantly lower cumulative damage demands than the new protocols for high seismicity. Fig. 5 shows further that $\Sigma\delta_i$ tends to decrease as the period of vibration increases. As a result, the $\Sigma\delta_i$ demands for periods equal to or longer than 0.5s are significantly smaller than the $\Sigma\delta_i$ demands for periods between 0.1s and 0.3s.

When the new protocols are compared to the existing ones (CUREE, FEMA-461 and ISO-21581), one notices that the new protocols for regions of low to moderate seismicity are, as expected, significantly less demanding than the existing loading protocols. Hence, the application of the new protocols for low to moderate seismicity may lead to less conservative estimations of structural capacities. The CUREE and FEMA-461 loading protocols impose similar cumulative demands than the new protocols for high seismicity if the period of vibration is less than 0.5s. CUREE and FEMA-461 are less demanding for stiff elastic systems (T=0.1s) in high seismicity regions and more demanding for all flag-shaped hysteretic systems. Note, however, that the CUREE protocol includes primary and secondary cycles and therefore the parameter $\Sigma\delta_i$ overestimates its actual CDE since secondary cycles generate less damage than primary cycles. The ISO-21581 protocol imposes a significantly larger CDE than the new protocols on all structural systems apart from the stiff elastic system with T=0.1s in high seismicity regions.

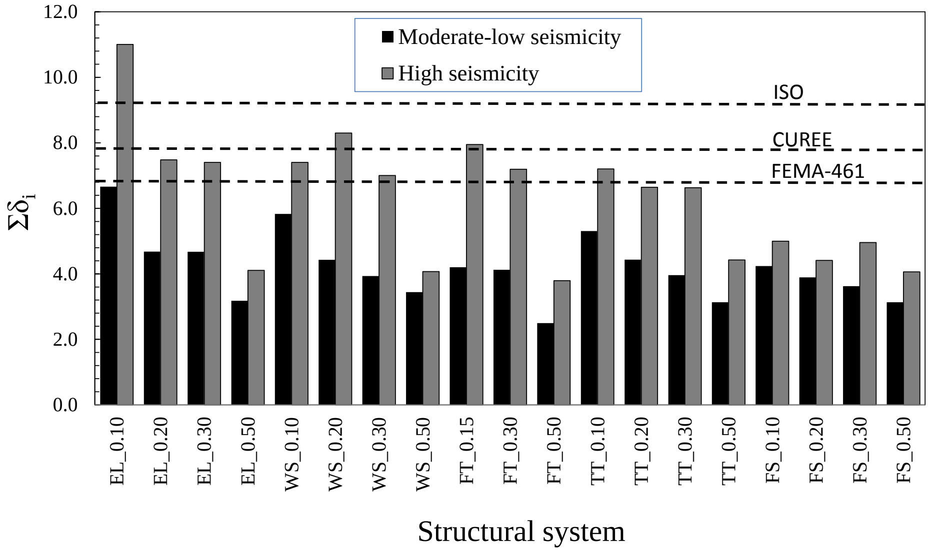

Figure 5: Comparison of proposed and existing loading protocols in terms of $\Sigma\delta_i$

CONCLUSIONS

Seismic strength and deformation capacities of structural members are often quantified by means of quasi-static cyclic tests. In these tests, predefined displacement histories, named loading protocols, are imposed at slow rates. Since strength and in particular deformation capacity of structural members are dependent on the cumulative damage demand, loading protocols should impose cumulative damage demands similar to the ones imposed by real earthquakes.

In this study, two different ground motion sets are employed. The first set consists of 60 records and is representative of low to moderate seismicity regions in Europe for the hazard level 2/50 (Mergos and Beyer, 2014). The second ground motion set is a set that was used in previous studies on loading protocols for high seismicity regions (Krawinkler *et al*. 2001). In a parametric study, the ground motions are applied to a large variety of SDOF systems representing the majority of buildings in European regions. The results reveal the strong dependence of the cumulative seismic demand on the level of seismicity (low to moderate vs. high) as well as on several structural parameters of the SDOF systems such as the period of vibration, the behaviour factor (as a measure of the inelasticity the system is subjected to), the post-yield stiffness ratio and the type of the hysteretic response.

Using a new algorithm, loading protocols are developed as a function of the seismicity, the hysteretic model, the fundamental period and the number of cycles per load step (one, two or three). All protocols follow the same analytical form which requires only two parameters to define amplitudes of each load step. Adopting this approach instead of proposing a single protocol provides more representative and less conservative loading protocols for the different structural systems and levels of seismicity. The new protocols allow, in addition, to choose between one to three cycles per load step.

Comparisons of the proposed loading protocols for regions of low to moderate seismicity with protocols well established in experimental testing (CUREE 2001, FEMA 2007, ISO 2010) show that the latter impose significantly higher cumulative damage demands. This may lead to an underestimation of the test specimen's strength and especially deformation capacity for regions of low to moderate seismicity. For regions of high seismicity, existing (CUREE 2001, FEMA 2007) and proposed loading protocols impose similar cumulative demands for the majority of structural systems. This is expected since existing protocols were derived for high seismicity regions. However, since existing protocols are not dependent on the fundamental period of the structure, they yield for long period structures a larger cumulative damage demand than the new loading protocols for high seismicity regions.

REFERENCES

Aldemir A, Eberik MA, Demirel O, Sucuogˇlu H (2013) "Seismic Performance Assessment of Unreinforced Masonry Buildings with a Hybrid Modeling Approach," *Earthquake Spectra*, 29(1):33–57.

Ambraseys NN, Douglas J, Rinaldis D, et al. (2004) Dissemination of European Strong Motion Data. Vol. 2, Engineering and Physical Sciences Research Council, UK

Carr AJ (2012) Ruaumoko – A computer program for inelastic time history analysis, Department of Civil Engineering, University of Canterbury, New Zealand

CEN (2004) Eurocode 8: Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, European Standard EN 1998-1, Brussels, Belgium

CEN (2005) Eurocode 8: Design of structures for earthquake resistance, Part 3: Assessment and retrofitting of buildings, European Standard EN 1998-3, Brussels, Belgium

Dowing SD, Socie DF (1982) "Simple rainflow cycle counting algorithms," *International Journal Fatigue*, 4(1):31-40

EN-12512 (2001) Timber structures-Test methods. Cyclic testing of joints made with mechanical fasteners, European Committee for Standardization, Brussels, Belgium

Fardis M (2009) Seismic design, assessment and retrofitting of concrete buildings, Springer

Filiatrault A, Isoda H, Folz B (2003) "Hysteretic damping of wood framed buildings," *Engineering Structures*, 25:461-471

FEMA-273 (1997) NEHRP guidelines for the seismic rehabilitation of buildings, Federal Emergency Management Agency, Washington, DC

FEMA-461 (2007) Interim protocols for determining seismic performance characteristics of structural and non-structural components through laboratory testing, Federal Emergency Management Agency, Washington, DC

Gatto K, Uang C (2003) Effects of loading protocol on the cyclic response of woodframe shearwalls, *Journal Structural Engineering*, 129(10):1384-1393

Giardini D, Wiemer S, Fäh D, Deichmann N (2004) Seismic hazard assessment of Switzerland, Swiss Seismological Service, Zurich, Switzerland

Hutchinson T, Zhang J, Charles E (2011) "Development of a drift protocol for seismic performance evaluation considering a damage index concept," *Earthquake Spectra*, 27(4):1049-1076

Iervolino I, Maddaloni G, Cosenza E (2008) "Eurocode 8 compliant real record sets for seismic analysis of structures," *Journal Earthquake Engineering*, 12(1):54-90

ISO-21581 (2010) Timber structures-Static and cyclic lateral load test methods for shear walls, International Standards Organization, Geneva, Switzerland

Kramer SL (1996) Geotechnical earthquake engineering, Prentice Hall, New Jersey

Krawinkler H, Parisi F, Ibarra L, Ayoub A, Medina R (2001) Development of a testing protocol for woodframe structures, CUREE publication No. W-02

Krawinkler H (2009) "Loading histories for cyclic tests in support of performance assessment of structural components", *Proceeding of 3rd International Conference on Advances in Experimental Structural Engineering*, San Francisco

MATLAB 7.1 (2010) The MathWorks Inc., Natick, Massachusetts

Mergos PE, Beyer K (2014) "Loading protocols for European regions of low to moderate seismicity," *Bulletin of Earthquake Engineering*, published online

Porter ML (1987) "Sequential phased displacement procedure for TCCMAR testing", *In Proceedings of the 3rd meeting of the joint technical coordinating committee on masonry research*, US-Japan coordinated program.

Priestley MJN, Calvi GM, Kowalsky MJ (2007) Direct displacement based seismic design of structures. IUSS Press, Pavia, Italy

Priestley MJN, Grant DN (2005) "Viscous damping in seismic design and analysis," *Journal Earthquake Engineering*, 9(sup2):229-255

Richards PW, Uang C (2006) "Testing protocol for short links in eccentrically braced frames," *Journal Structural Engineering*, 132(8):1183-1191

SIA 261 (2003) Actions on structures, Swiss Society of Engineers and Architects, Zurich, Switzerland

Stewart WG (1987) The seismic design of plywood sheathed shear walls, PhD Thesis, University of Canterbury, New Zealand