Capacity Design Procedure Evaluation for Buckling Restrained Braced Frames with Incremental Dynamic Analysis

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SUMMARY:
The objective of the presented research is the comprehensive numerical modelling of Buckling Restrained Brace (BRB) behaviour and the development of European standardized specifications for design and verification of frame structures employing diagonal BRB elements. A series of quasi-uniaxial cyclic load tests have been performed to evaluate the main structural characteristics of the braces according to European regulations. Test results are also used to develop and verify a numerical element model designed for the incremental dynamic analysis of BRB frames that is presented in this paper. The framework from FEMA P695 is the basis of a design procedure evaluation methodology, which is used to optimize design parameters and regulations for BRBs. The objective is to eventually cover the complete design space of conventional BRB solutions and a short application example is also presented in this paper.

Keywords: buckling restrained braces, standardized design procedure, incremental dynamic analysis

1. INTRODUCTION

Buckling Restrained Braces (BRB) are displacement dependent anti-seismic devices capable of significant energy dissipation when subjected to extensive cyclic loading (López and Sabelli, 2004). Therefore, seismic load on braced frames can be reduced considerably by using BRB elements as diagonal members. Although design of BRB is already standardized in the United States and Japan, there is no standard design procedure available in Europe yet. The objective of the presented research is the proposal of standardized specifications for design and verification of Buckling Restrained Braced Frames (BRBF).

Nonlinear Displacement Dependent Devices (NLD) like BRBs are currently regulated in Europe by the EN 15129 standard for anti-seismic devices (CEN, 2010). Its requirements are the basis of a series of laboratory experiments on BRBs performed at the Budapest University of Technology and Economics (BME) since 2010 (Zsarnóczay and Dunai, 2011). Test results on the one hand led to proposal of a theoretical bilinear model for BRBs that could be used both in static analysis and for quality control in Europe. On the other hand a detailed numerical BRB model has been developed for dynamic analyses of braced frames.

Since there is significant difference between the European and the US approach in structural design regulations, direct application of US design specifications for BRB-based structures in Europe is not recommended. A new procedure shall be developed based on existing information, but tailored to fit in the European way of designing structures. Thus, the proposed methodology is based on existing regulations in Eurocode 8 (EC8) (CEN, 2008) for design of dissipative frame structures. The unique characteristics of BRBs are taken into account by applying BRB-specific values for key parameters (e.g. behaviour factor, overstrength factor) and parameters from other standards are also introduced (e.g. tension and compression strength adjustment factors). The procedure is intended to use only linear static analysis results for the design of BRBFs.

FEMA P695 (ATC, 2009) presents a methodology for detailed numerical analysis of structural seismic performance. It proposes the evaluation of seismic performance through numerical modelling and complex analyses of several archetypical structural configurations. The methodology involves the use of both linear and nonlinear static and nonlinear dynamic analyses and requires accurate modelling of the main dissipative structural members. It has already been used to verify several different structural solutions in the US, including BRBFs (NEHRP Consultants Joint Venture, 2010). This research also assesses BRBF performance based on FEMA P695, but a few modifications are made to adapt to European practice. The paper presents the evaluation of a group of concentrically braced BRBF archetypes and shows how the methodology can be used for design procedure optimization.

2. LABORATORY EXPERIMENTS

2.1. Experimental setup, specimen characteristics

Six laboratory tests have been performed on small capacity BRB specimens at BME. Table 2.1 summarizes the main characteristics of the specimen. The quasi-uniaxial cyclic load tests were designed in accordance with EN 15129 and recommendations of ECCS (ECCS, 1986), AISC (AISC, 2005) and US professionals (Sabelli, 2005). A custom loading frame was built for the tests where the braces are installed in a slightly tilted position to simulate the effect of second order moments.

Specimens are loaded by hydraulic jacks using an extensive displacement controlled loading protocol (Figure 2.1) that significantly surpasses the requirements of EN 15129. The large number of load cycles at various large displacement levels in this protocol gives essential information for numerical modelling about the cyclic hardening behaviour of the braces. Figure 2.2 shows typical resulting experimental force-displacement curves.

Table 2.1. Main characteristics of tested BRB specimens

		EWC 500 A EWC 500 B	EWC 800 A EWC 800 B	600 BCE	825 BCE
specimen(s)					
total brace length [mm]		2960	2960	2760	3120
yielding zone	**length [mm]**	2000	2000	1802	2198
	cross-section [mm]	20 x 25	20 x 40	15 x 40	15 x 55
core material		S235	S235	S235	S235
actual load bearing capacity tension/compression [kN]		199.5/232.1	323.7/442.0	230.9/270.9	317.3/392.3

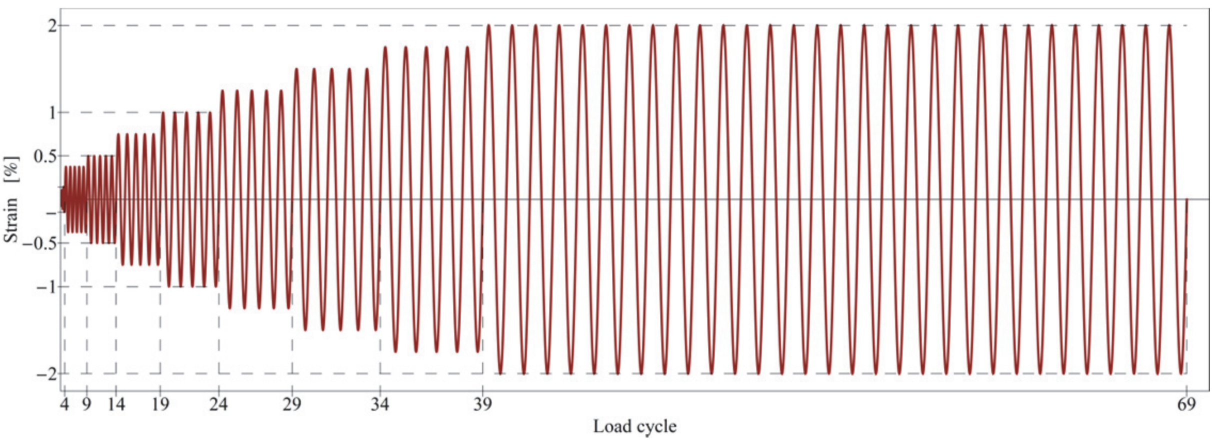

Figure 2.1. Displacement dependent load protocol used for testing BRB specimens

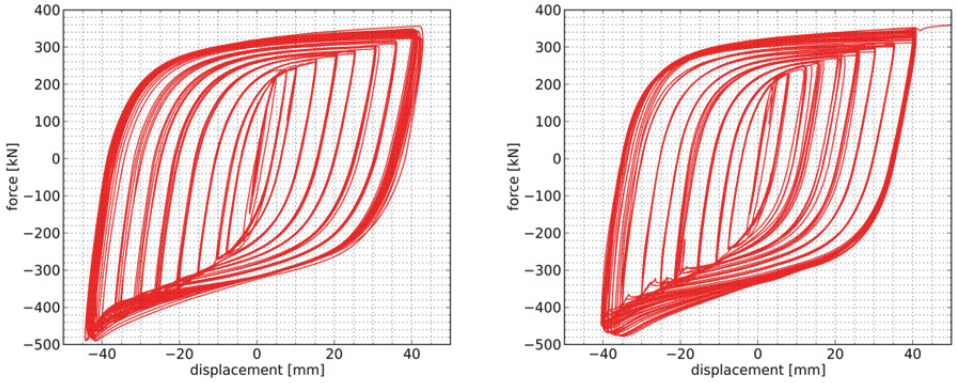

Figure 2.2. Typical hysteresis loops from experiments

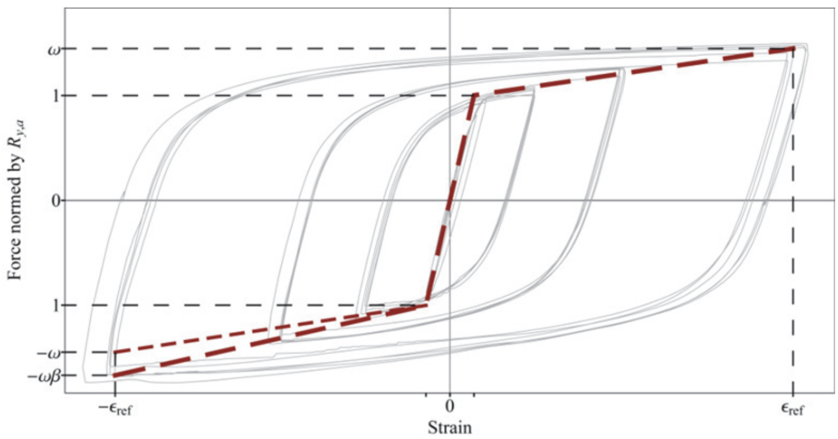

Figure 2.3. Characteristic bilinear stress-strain relationship defined by compression and tension strength adjustment factors with one of the experimental force-displacement data in the background

2.2. Result evaluation

Test results were initially evaluated as per EN 15129 specifications. All of the tested specimens showed stable and balanced cyclic hardening behaviour with energy dissipation capacity significantly surpassing the AISC requirement of 200. Definition of post-yield hardening for NLDs was thoroughly investigated and it was shown that current regulations lead to ambiguous requirements that depend more on values selected by the designer than on parameters of the specimens themselves (Budaházy, Zsarnóczay and Dunai, 2011). Therefore, an alternative method is suggested and used for modelling the cyclic behaviour of BRBs based on the combination of tension and compression strength adjustment factors (ω and β) from AISC 341-05 (AISC, 2005) and partial factors in line with EN 15129 and EC8 for calculating material and structural overstrength (γ_{ov} and Ω).

Figure 2.3 displays the proposed bilinear relationship and its relevant parameters. This relationship is similar to the so-called theoretical bilinear cycle in EN 15129 and the backbone curve in AISC. The strain hardening rate with respect to $R_{y,a}$ is expressed by ω, while β represents compression overstrength with respect to tensile strength. $R_{y,a}$ – the actual cross-section resistance – is the product of the actual yield stress of the steel material and the net yielding BRB cross-sectional area. An important difference from the other two bilinear approximations is that strains refer to the deformation of the yielding zone only, excluding the effect of elastic BRB components on brace deformation. This separation of inelastic and elastic behaviour eliminates the variance in hardening for different BRB geometries. Therefore, behaviour of BRBs with different proportions and capacities are directly comparable with this relationship. With appropriate limits defined for acceptable variance, BRB manufacturers can use these curves to control the quality of their products.

3. COMPREHENSIVE NUMERICAL BRB MODEL

3.1. Material setup

The aforementioned bilinear BRB material model is sufficiently accurate for even nonlinear static analysis, but it has important flaws when it comes to dynamic examinations, especially under cyclic loading conditions. The following phenomena are not or not appropriately modelled with the simple backbone curve:

- Bauschinger effect
- nonlinear nature of plastic strain hardening under tension and compression
- cyclic hardening even after a large number of load cycles at the same displacement level
- asymmetric hardening under tension and compression
- monotonic loading curve

A complex BRB model made of solid elements has already been developed and provides accurate results for detailed element analysis. While it models the above phenomena appropriately, this model is too expensive computationally to be applicable in the global analysis of large frame structures with several often different braces. The efficient method in this case is the use of a simple beam model and a sufficiently complex custom BRB material for the analysis.

The material model was developed in the OpenSEES FEM environment (UC Regents, 2006). The numerical BRB model configuration is displayed on Figure 3.1. Elastic BRB parts and gusset plates are modelled with their appropriate cross-sectional areas and a standard linear elastic steel material. Joints also take up part of the work-point-to-work-point (wp-wp) length of BRBs. They are approximated as rigid bodies.
The custom material is created only for the yielding zone. Its behaviour is described by connecting three distinct materials in parallel. This parallel placement means that strain in a given finite element equals strain in each component, while stress is the sum of component stresses.

3.2. Calibration and verification

Although the use of three materials in parallel lead to a large number of unknown parameters, the purpose of each component in the model is clearly defined, facilitating the calibration process. The numerical BRB model was first calibrated for a subset of the experimental results and the parameters sensitive to changes in BRB characteristics were identified. Each of these key parameters was expressed as a function of available BRB characteristics (e.g. ω, β, γ_{ov}). The resulting material model was verified by applying it to the remaining experimental results without calibration. The developed model provides an accurate approximation of BRB behaviour within the range of experimental results. A comparison of experimental and numerical force-displacement data is shown on Figure 3.2.

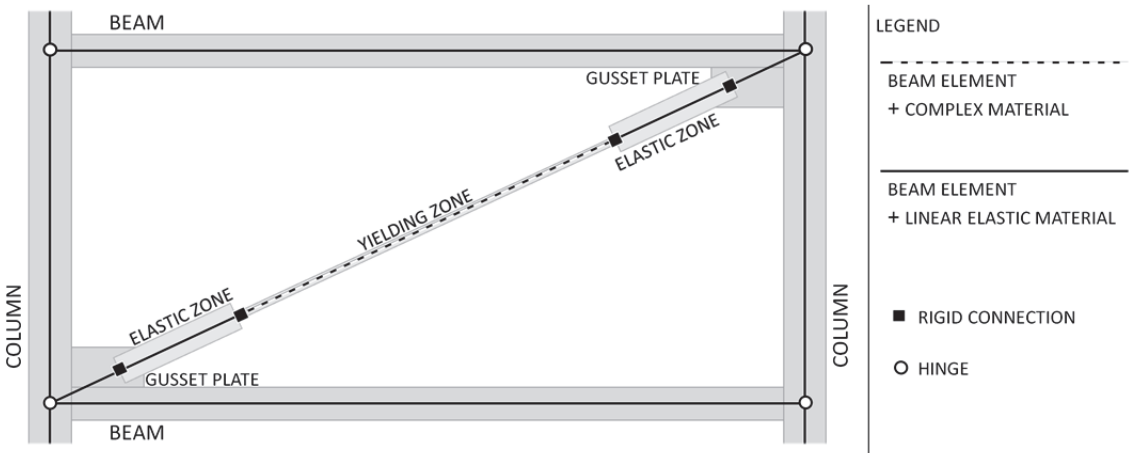

Figure 3.1. BRB model configuration for numerical material development

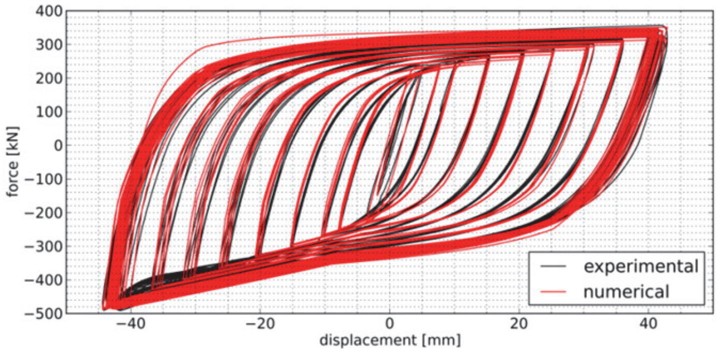

Figure 3.2. Comparison of numerical and experimental results of a cyclic load test.

The numerical model on Figure 3.1 consists of five elements, which unnecessarily increases the number of degrees of freedom in the global structural model and poses possible convergence and modelling difficulties for BRBs with pinned connections. Since only the yielding zone behaves in a nonlinear manner, the linear elastic properties of the other four elements can be combined with the developed complex parallel model as if consecutive elastic members with various cross-sections and Young moduli were replaced by a single one with an equivalent stiffness and/or cross-sectional area. This calculation is a preprocessing step and as a result the BRB can be represented as a single element in the global model. This procedure has to be performed for each BRB separately, since brace geometry and connecting beam and column sizes influence the resulting material properties. The single beam element provides accurate BRB stress-strain curves without sacrificing solution stability or computational resources.

4. BRBF ARCHETYPE SELECTION AND DESIGN

4.1. Archetype design space

Design procedure evaluation requires the definition of a set of archetypes from the structural group of interest. The performance of these archetypes under various levels of seismic excitation is proportional to the quality of the design procedure in consideration. Each archetype shall represent a specific structural configuration common in engineering practice. Optimization and reliable evaluation of a design procedure requires the analysis of the full archetype design space. It is important to note that a large number of unrealistic configurations can be excluded from the full set during analysis by practicing engineering judgement.

Conventional BRB solutions include diagonal braces in various configurations in steel frames. Frame joints can either be moment resisting characterized by a typically multilinear moment-rotation relationship or hinges. The two systems require fundamentally different approaches from an engineer. The difference between their behaviour is also reflected in the separate set of design coefficients (e.g. response modification coefficient: R) assigned to them in AISC 341-05. Therefore each solution is analysed with its own set of archetypes and a custom design procedure.

Examples in this paper are frames with non-moment-resisting beam-column connections. The parameters in Table 4.1 describe the current design space for this type of BRBF solution. It is important to emphasize that these values are subject to change during the course of analysis. For instance the number of stories will probably be reduced, because design of high-rise BRBFs without moment-resisting connections is almost always controlled by displacement constraints (e.g. limited interstorey drift). This leads to unnecessarily large members, which could be avoided by using moment-resisting connections. The final ranges of parameters define the applicability of the developed design procedure. Therefore it is important to start with a large space and shrink it appropriately as new information becomes available.

Table 4.1. Parameters of the design space for non-moment-resisting BRBF solutions

parameter	description	range / options
number of stories	the range of common building heights in Europe.	1-20
bay configuration	a set of frequently used combinations are selected from the height and width ranges	height: 3-5 m width: 4-8 m
gravity loading	provides different mass possibilities for buildings with the same height while influencing non-BRBF column sections	dead load: 3.5 – 12.0 kN/m^2 live load: 2.0 – 4.0 kN/m^2
braced area	size of floor area that is supported by a single braced frame this approach makes the actual building layout irrelevant as long as it is regular both in plan and in elevation	150-650 m^2
seismic intensity	the peak ground acceleration corresponds to 475 year return period according to European design practice the type of spectra separates near-field and far-field records common soil classes are B, C and D with D corresponding to the least favourable acceleration response	a_{gr}=1.0-4.0 m/s^2 type I or type II spectrum soil class B, C or D
non-structural elements	influences the interstorey drift limitation the values after the element types are the limits under 475 year return period seismic excitation	brittle – 0.01*h* ductile – 0.015*h* independent – 0.02*h*

The type of framing topology (e.g. single diagonal, V, X, Z) is not an archetype parameter as it shall be a design consideration and the design procedure shall provide guidance for the engineer in this topic as well. Similarly, deciding the geometry and configuration of the buckling restrained braces used shall either be the responsibility of the engineer or more often the brace manufacturer based on project characteristics.

This paper presents an illustration of the methodology through six BRBF archetypes. The structures are assembled into so-called Performance Groups. Each performance group contains structures with similar parameters and thus probably similar behaviour. All structures are low-rise buildings to avoid the aforementioned displacement-dependency in design. The number of stories modelled ranges from two to four with typical gravity loading. The objective of the presented analysis is to investigate how the bay configuration affects the performance of structures with otherwise identical parameters. The braced floor area is chosen according to the bay configuration for each performance group and ductile non-structural elements are assumed. A very large seismic intensity is selected that gives a design spectral acceleration level similar to that of D_{max} in the US. Archetype parameters and performance groups are summarized in Table 4.2.

Table 4.2. Parameters of the six BRBF archetype structures

<table>
<tr><th>ID</th><th>stories</th><th>bay config</th><th>braced area</th><th>gravity load</th><th>period domain</th><th>seismic intensity</th></tr>
<tr><td colspan="7">Performance Group I</td></tr>
<tr><td>1</td><td>2</td><td rowspan="3">4 m x 3 m</td><td rowspan="3">150 m2</td><td rowspan="3">typical</td><td rowspan="3">short</td><td rowspan="3">high</td></tr>
<tr><td>2</td><td>3</td></tr>
<tr><td>3</td><td>4</td></tr>
<tr><td colspan="7">Performance Group II</td></tr>
<tr><td>4</td><td>2</td><td rowspan="3">6 m x 4 m</td><td rowspan="3">250 m2</td><td rowspan="3">typical</td><td rowspan="3">short</td><td rowspan="3">high</td></tr>
<tr><td>5</td><td>3</td></tr>
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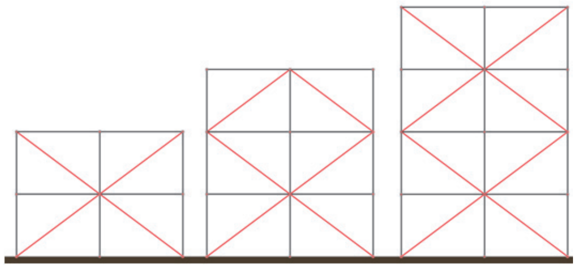

Figure 4.1. BRB layout for 2-, 3-, 4- story archetypes (BRBs are coloured red)

4.2. Design procedure

Figure 4.1 displays the three braced frame configurations. The other structural members without lateral stiffness are considered by connecting a so-called leaning column to the BRBF and applying the gravity load of the corresponding braced area on it. Although three dimensional frame models are not developed, the effect of accidental eccentricity and resulting torsional moments is taken into account by increasing the seismic load by 10% on the frames. This is in line with the recommendation of professionals for cases when the braced frame is at the perimeter of the structure.

Design of structural members is based on the capacity design procedure for DCH (highly ductility) structures described in detail in EC8. Although BRBF is not included in the standard, the proposed methodology is evaluated as if it was already standardized with a behaviour factor assumed based on specifications of AISC 341-05. Every structure is designed with modal response spectrum analyses in spite of the applicability of the lateral force method in some cases to avoid bias in results. Sections of non-BRBF structural members are also calibrated based on the gravity load level and bay sizes. These non-BRBF member sizes are used to calculate the structural mass which is applied as additional gravity load on the leaning column.

BRB elements are designed to remain elastic under internal forces from the reduced seismic load. The spectral acceleration value is decreased by the behaviour factor (response modification coefficient). Variance of brace utilization (i.e. acting force over resistance) $\mathbf{\Omega}_i$ for brace *i* is limited by the following equation:

$$\frac{\Omega_{\max}}{\Omega_{min}} - 1 \leq 0.25 \tag{4.1}$$

The above regulation aims to prevent the design of outlier members in buildings with several braces that might initiate an unwanted failure mechanism (e.g. soft storey). Non-dissipative members of the BRBF are designed by applying structural overstrength as a product of the following:

$$1.1\gamma_{ov}\Omega\omega\beta\gamma_{\omega\beta} \tag{4.2}$$

where γ_{ov} is the overstrength factor, ω is the tension and β is the compression strength adjustment factor with their partial factor $\gamma_{\omega\beta}$, while $\mathbf{\Omega}$ is the minimum of $\mathbf{\Omega}_i$ from Equation 4.1. The strength adjustment factors and the overstrength factor are determined from the experimental results and evaluation procedure presented in Section 2.

Displacement-related constraints have to be verified on displacement levels multiplied by q_d (equals q in practice). The interstorey drift level on each storey is calculated and has to be below the allowed limit.

The final step is the verification of BRB deformation. Strain in the yielding zone shall remain below a maximum allowable limit. This limit is not yet defined in European standards. Based on experimental results the strain in the yielding zone of the core is maximized in 3.0%. Note that this is not a regulation of the European standard, but a preliminary proposal for avoiding brace rupture.

5. PERFORMANCE EVALUATION

5.1. Incremental Dynamic Analysis

The developed design procedure is evaluated in this final step through the performance of the archetypes in the so-called Incremental Dynamic Analysis (IDA). Each archetype is subjected to 44 different ground motions several times. The earthquakes were carefully selected by the creators of FEMA P695 to provide as diverse set of high quality accelerograms as possible. The record set is first normalized by the peak ground velocity of each record to reduce its variance to an acceptable level. Afterwards, the set as a whole is scaled so that its median acceleration response spectrum equals the response spectrum used for archetype design at the fundamental period of the structure under investigation. Figure 5.1 illustrates the result of scaling the set to the standard EC8 response spectrum for soil type D at a natural period of one second. In the end a set of ground motions is available that produces the same median spectral acceleration level as the theoretical design spectrum, but in nonlinear dynamic analysis.

Structural performance can be expressed in a probabilistic manner with the help of this record set. The probability of collapse at any given spectral acceleration level is considered proportional to the number of earthquakes that lead to collapse from a group that is scaled to that level. The collapse probability of the archetypes is investigated at several levels of increasing spectral acceleration. At each level the maximum interstorey drift experienced by the structure from each ground motion record is plotted (Figure 5.2). When a structure collapses, its interstorey drift rapidly increases (quasi-horizontal lines on the figure). The median collapse level (S_{CT}) is the spectral acceleration where half of the ground motions collapse the structure.

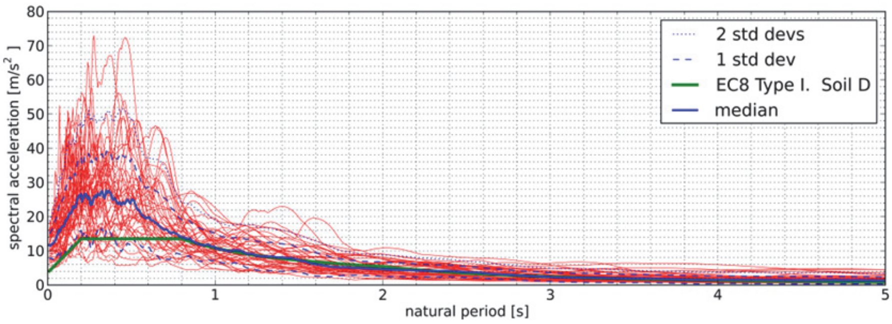

Figure 5.1. Set of ground motion records scaled to a pre-defined spectral acceleration level at T=1s

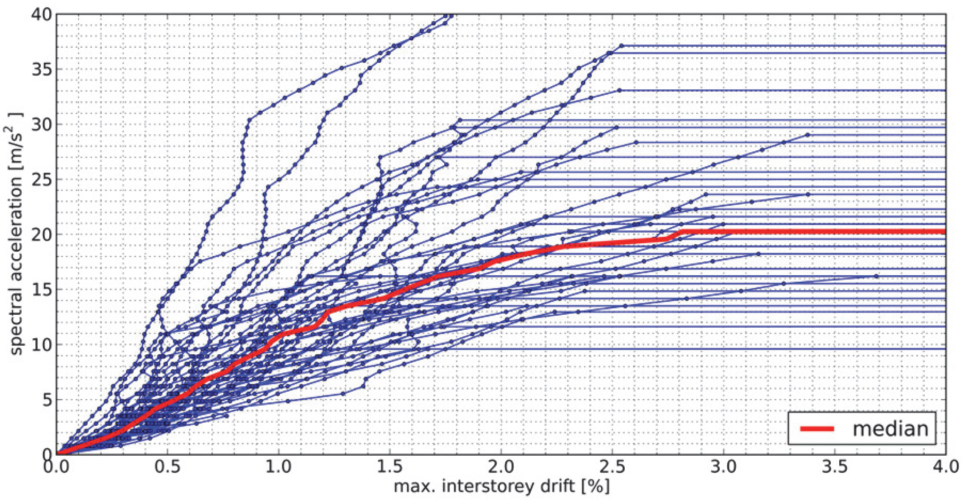

Figure 5.2. Maximum interstorey drift values from each ground motion record at different spectral acceleration levels for archetype no. 1

The probability of collapse versus the corresponding spectral acceleration level gives the fragility curve of the structure under examination. Such a fragility curve is shown in dashed blue on Figure 5.3 for archetype 1. The fragility curve from the analysis is a lognormal cumulative distribution function with a constant standard deviation of β=0.4 and a mean of S_{CT} with the selected record set. Therefore, knowing S_{CT} is enough to draw the complete fragility curve, which can save a large amount of computational resources. 0.4 expresses the magnitude of uncertainty in ground motion properties and the fragility curve shows how this uncertainty affects structural performance.

The methodology suggests to take other sources of uncertainty into account by an appropriately increased (or in some cases decreased) β. The three other sources to consider are the quality of test data, design requirements and modelling. Good model and design requirement quality and superior test data yields β=0.5 for example. The dashed red curve corresponds to this increased uncertainty on Figure 5.3. Notice how the increased β increased the probability of collapse at low spectral acceleration levels.

The final step of evaluation is consideration of the so-called spectral shape effect. Spectrum of ground motions of large intensity are usually peaked at a certain period, from where they drop off quickly. This leads to overestimation of damage from these rare records. The error is corrected by the spectral shape factor, which moves the fragility curve of the structure in the positive direction along the acceleration axis, therefore reduces the probability of failure at every acceleration level. The corrected curve is plotted by green on Figure 5.3

5.2. Archetype Performance

Performance of the examined archetypes is summarized in Table 5.1. (Note that the complete methodology also requires pushover analysis results for each archetype) S_{MT} is the design ground motion intensity (before division by q). CMR is the collapse margin ratio that expresses the ratio of collapse and design spectral acceleration. SSF is the spectral shape factor corresponding to the structure. ACMR is the product of CMR and SSF. The methodology limits the probability of failure for any archetype in 20% and the mean of archetype ACMR values shall lead to a collapse probability less than 10%. These limits are based on US safety standards and shall be revised and adapted to European safety regulations in the future.

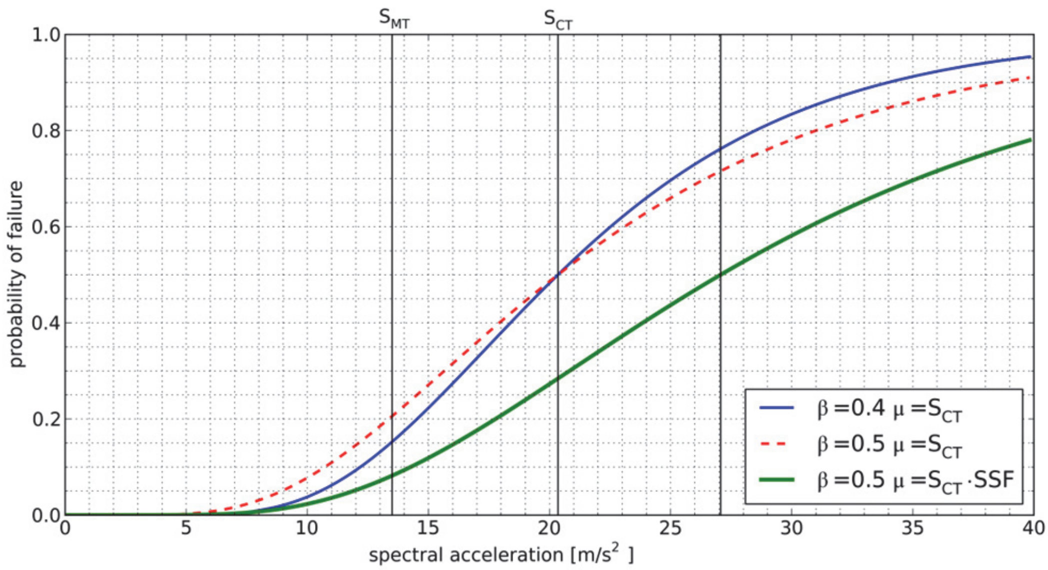

Figure 5.3. Fragility curve and corresponding quantities for archetype no. 1

All archetypes passed both the 20% and the 10% criteria. However, it is interesting to see how close the first performance group is to failing, because larger uncertainty in any of the input data would increase the probability of failure enough to make its expected probability of failure higher than 10%. On the other hand, Performance Group II with increased bay sizes easily passed the criteria. Note that these are results of a preliminary design procedure and they are not yet representative concerning BRB performance. The objective of the presented example is to show the steps of the methodology rather than evaluate the applicability of a complex anti-seismic solution with six archetypes.

Table 5.1. Analysis results for the investigated archetype configurations

Archetype			IDA results				Acceptance check	
ID	stories	S_{MT}	S_{CT}	CMR	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group I								
1	2	13.5	20.35	1.51	1.33	2.01	1.52	Pass
2	3	13.5	19.00	1.41	1.33	1.87	1.52	Pass
3	4	13.5	18.28	1.35	1.33	1.82	1.52	Pass
mean of performance group						1.90	1.90	Pass
Performance Group II								
4	2	13.5	23.05	1.71	1.33	2.27	1.52	Pass
5	3	13.5	21.71	1.61	1.33	2.17	1.52	Pass
6	4	13.5	23.73	1.76	1.33	2.41	1.52	Pass
mean of performance group						2.28	1.90	Pass

6. CONCLUDING REMARKS

The presented methodology for design procedure evaluation is implemented into a custom-made application for BRB performance evaluation based on the OpenSEES FEM environment. This application is capable of archetype design and design procedure evaluation through IDA. The next step is a comprehensive BRB performance analysis and design procedure development using the methodology presented in this paper.

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