DESIGN OF FRAMES WITH BUCKLING RESTRAINED BRACES
FEMA P695 based Evaluation of a Eurocode 8 Conforming Design Procedure

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INTRODUCTION

Buckling Restrained Braced Frames (BRBF) discussed in this paper are concentrically braced steel frames using continuous columns with rigid supports and BRBs as diagonal members [1]. The BRB is an innovative brace characterised by significant energy dissipation capability under cyclic loading. Its superior hysteresis behaviour compared to conventional steel braces stems from its special configuration. A highly slender central steel core is responsible for energy dissipation. The core is surrounded by a rigid casing to prevent global flexural buckling of the brace.

Because of their beneficial energy dissipation capability, BRBF are often more economical alternatives to conventional steel frame solutions. However, their application in Europe is hindered by the absence of a standardised design procedure. This paper is based on a research effort that aims to propose such a procedure that is compatible with the concepts and methodology of the Eurocode 8 standard (EC8) [2].

Evaluation of a design procedure is challenging, because of the vast number of possible design scenarios required to consider the variability in structural geometry and the seismic hazard. Until recent years the lack of computational resources impeded direct consideration of demand and capacity variability and led to simplified studies using only a small set of scenarios and crude models (e.g. [3,4]). The authors believe that besides being economical, the primary target of a good design procedure shall be assurance of sufficiently low failure probability with high confidence. Therefore, merits of a design procedure can only be judged by robust and reliable evaluation of collapse probability. This requires a framework for probabilistic assessment of the performance of a large number of typical structural solutions under various seismic hazard scenarios.

The authors developed an extended version of the framework proposed in FEMA P695 [5] to assess the seismic performance of an EC8 conform BRBF design procedure. This paper briefly presents the design procedure, the extended framework and the results on a set of BRBF archetypes.

1 DESIGN OF BUCKLING RESTRAINED BRACED FRAMES

1.1 BRBF design – state of the art

Design of BRBF is typically based on the so-called backbone curve: a simplified bilinear representation of BRB force-displacement behaviour. The primary objective of BRBF design is to ensure that braces yield and other structural members remain elastic under the design earthquake so that an advantageous global mechanism can develop.

Braces are designed to have sufficient elastic capacity to resist a reduced design seismic load. The level of reduction is justified by the energy dissipation capabilities of the braces and it is reflected by the behaviour factor. As well as having sufficient load bearing capacity, a BRB also needs to have high ductility to accommodate the development of expected deformations.

Besides appropriate BRB design, prevention of inelastic deformation of other frame elements is an important prerequisite of the desired advantageous global mechanism. This objective is achieved by assignment of sufficiently high capacity requirements to the elements that are meant to stay elastic. The required capacity of elastic members is based on the maximum possible axial force in the connecting braces at the design deformation level. This calculation takes inelastic BRB hardening into account and uses the aforementioned backbone curve to determine the axial load intensity that corresponds to the design brace deformation. The required increase in the capacity of elastic members is reflected in the overstrength factor.

The above considerations form the basis of BRB design in the AISC 341-10 standard. Applicability of that procedure has been verified by both experimental and numerical analyses in recent years [6]. The only European study on the topic [3] proposes a procedure with significant approximations in both design and validation. Furthermore, the proposed procedure does not fit easily in the current design concept of Eurocode 8. Several studies are interested in the evaluation of an optimal behaviour factor that shall be applied for BRBF [4]. Their recommendations show significant variation which is explained by the simplicity of their approach to numerical simulation. The performed numerical analyses are either detailed examinations of only a few structures, or they use crude models and fail to reproduce realistic behaviour with sufficient accuracy.
The authors are not aware of any publication on robust and reliable evaluation of BRBF design procedure performance.

1.2 Proposed EC8 conform design procedure

The primary interest during the development of the following proposal was the conformance to existing EC8 regulations on steel concentrically braced frames (CBF). Therefore, although the specifications in AISC 341-10 are considered, their direct application is not feasible. The authors propose to employ existing capacity design rules for CBF in EC8 6.7 and introduce a limited number of modifications to make them appropriate for BRBF design. BRB elements shall be introduced as a third bracing option besides diagonal and V bracings.
A complete list of suggested modifications is included in [7]; only a summary is presented here for brevity:

- *In elastic analysis braces shall be modelled using an increased, equivalent initial stiffness value.*
- *In pushover analysis the minimum requirement is the application of a bilinear material model to simulate brace behaviour. Inelastic hardening stiffness shall be the secant stiffness between the actual yield point and the point on the experimental backbone at the target BRB strain.*
- *Both tension and compression elements need to be modelled even in linear elastic analysis.*
- *The effect of asymmetric hardening on structural response needs to be taken into consideration.*
- *The appropriate flexural buckling resistance of BRBs needs to be verified.*
- *Braces shall fulfil the requirements of the EN 15129 standard [8] with the modifications suggested by the authors in [9] to improve the feasibility of nonlinear displacement dependent devices.*
- *Variation of brace overstrength (Ω) over the height of the structure shall be limited to 10% instead of the 25% currently applied at the standard to reduce the likelihood of soft story formation.*
- *The overstrength factor shall be calculated with the following expression:*

$$\gamma_{ov} = \gamma_{ov,m}\omega_{\epsilon_d}\beta_{\epsilon_d} \qquad (1)$$

where $\gamma_{ov,m}$ is the material overstrength; $\omega_{\varepsilon d}$ and $\beta_{\varepsilon d}$ are the strain hardening adjustment factor and the compression strength adjustment factor at the design strain level, respectively. The proposed value for $\gamma_{ov,m}$ shall be based on material test results from materials of actual BRBs and it is a manufacturer-specific number. In absence of tests, the recommended $\gamma_{ov,m}$ in EC8 6.2 (3) is supported. The value of adjustment factors corresponding to the design strain level (ε_d) shall be calculated using the design envelope response of the braces. The SAM approach [9] is recommended for this purpose.

- *A behaviour factor of 7.0 is proposed for concentrically braced frames with pinned BRBs in two-bay chevron configuration provided that braces are proven to have sufficient ductility.*
- *BRBF columns shall be made of Class 1 or 2 cross-sections to ensure sufficient deformation capacity under cyclic loading. This measure is necessary to avoid local plate buckling in columns under high drifts at large seismic intensities.*

A detailed application example of the proposed design procedure is presented in [7].

2 FRAMEWORK FOR DESIGN PROCEDURE EVALUATION

2.1 FEMA P695 for probabilistic seismic performance evaluation

The FEMA P695 procedure was recently developed by a group of experts to provide means for quantification of seismic performance of structures. The procedure describes structural performance through nonlinear collapse simulation on finite element models of archetype structures. The archetypes shall be designed with the procedure under evaluation and their set shall capture the variability of the performance characteristics of the structural system under consideration. FEMA P695 has been adopted by several researchers [10,11]. A modified version of the original framework has been applied in this research.

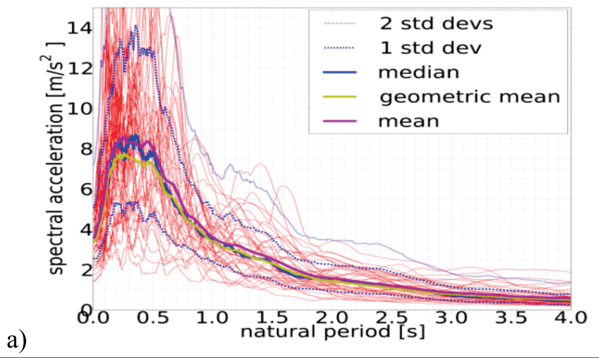
2.2 Design procedure evaluation methodology

The seismic hazard is described by a set of 22 pairs of pre-defined ground motion records that were selected to give a good approximation of the aleatoric uncertainty in the earthquake hazard (Fig. 1a). Engineering demand parameters (typically maximum interstory drift ratios) are evaluated by calculating structural response through nonlinear response history analysis with each of the 44 ground motion records scaled to several intensity levels. This evaluation procedure is often referred to as Incremental Dynamic Analysis (IDA) [12]. Note that the use of appropriate numerical models for these analyses is of critical importance for the reliability of the results. A new numerical BRB element that was developed for this research [7] in the OpenSees finite element code [13] to capture BRB behaviour with sufficient accuracy and special attention was paid to proper modelling of BRBF columns.

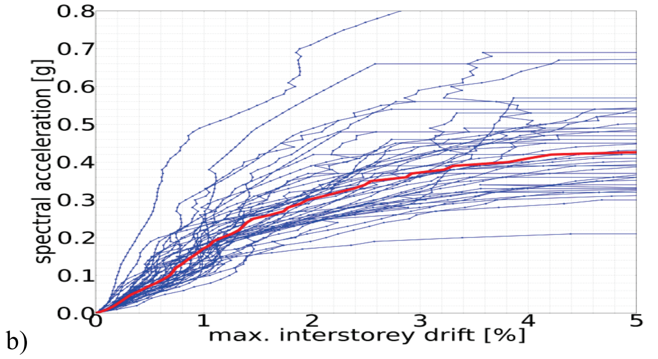
The result of such an analysis is a set of so-called IDA curves (Fig. 1b). Each continuous line represents the max interstory drift under the effect of one ground motion record at several seismic intensity levels. Seismic intensity is characterized by the spectral acceleration at the dominant structural period. The initial elastic behaviour is followed by a reduction in stiffness, period elongation and eventually structural failure because of sidesway collapse of the frame structure. The scatter of IDA curves describes the inherent uncertainty in the seismic hazard and encourages a probabilistic approach to damage assessment.

Collapse intensity (i.e. the spectral intensity at the dominant period of the structure that initiates global failure) is considered a random variable with lognormal distribution and it is described by a fragility curve (Fig. 2). The random collapse intensity is defined by collapse intensity or collapse probability samples from IDA analysis.

The raw fragility curve from IDA is modified to take additional sources of uncertainty (by increasing the standard deviation of the curve) and the effect of different spectral shape at high seismic intensities (by increasing the median spectral acceleration) into account (Fig. 2). Because the uncertainty assigned to external sources such as design requirements, test results and numerical models has a significant influence on collapse probability, the best and worst conceivable cases are evaluated independently. The resulting fragility curves are the maximal and minimal red and green curves on Fig. 2.



a)



b)

Fig. 1. a) 5% damped response spectra of the records in the Far Field set of FEMA P695; b) Typical IDA result

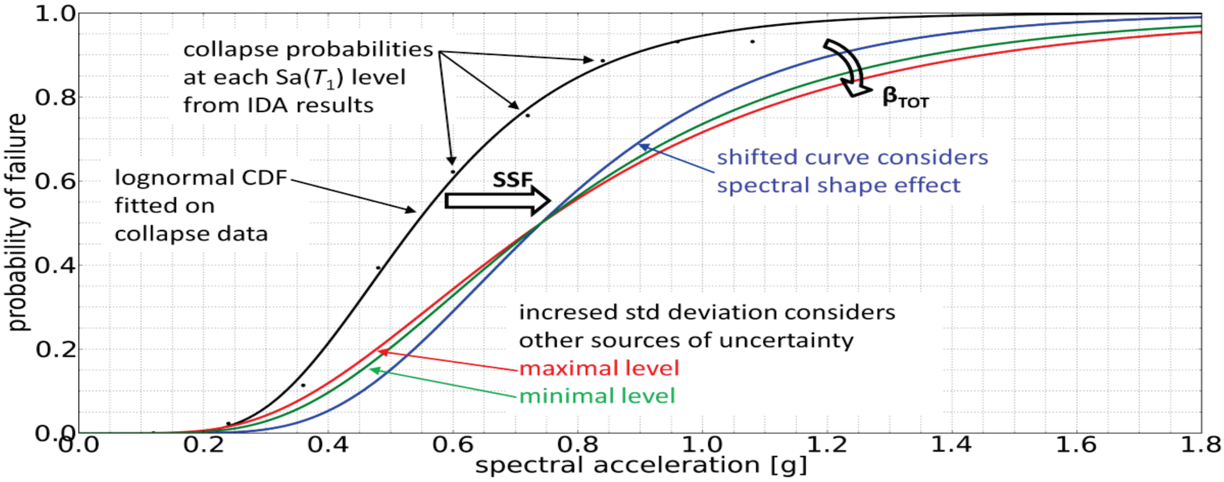


Fig. 2. Consideration of the spectral shape effect and additional sources of uncertainty in two post-processing steps

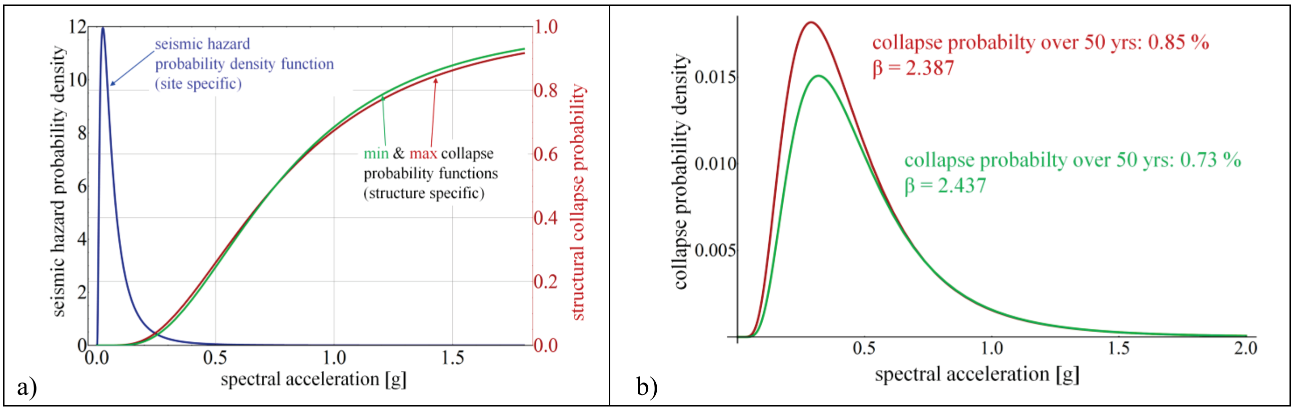


Fig. 3. a) Site-specific hazard curve and structure-specific fragility curves;
b) Estimated upper and lower bound collapse probability density functions

FEMA P695 defines limits on collapse probability at the design spectral acceleration level to evaluate the performance of a particular design procedure. The average collapse probability shall be below 10% under design seismic events. The authors are concerned about this approach, because it does not rate the performance of the structure at seismic intensities other than the design one. The basis of safety in the Eurocodes is reliability, namely the probability of failure over the lifetime of the structure. This approach is also suggested by researches in the United States as a more accurate alternative [14]. Therefore, this research also calculates reliability indices for the investigated BRBF.

Reliability analyses require characteristic hazard curves that describe the relationship between seismic intensity and the probability of occurrence of ground motions that result in such intensity at the site of the building. Such curves are provided by the European Facility for Earthquake Hazard and Risk (EFEHR) [15] for this research. Fig. 3 shows an example of the combination of occurrence probability from the seismic hazard curve (in blue) and collapse probability from fragility curves (in green and red) to arrive at a min and max collapse probability density function (pdf) for the given structure at the given site. The total probability of collapse is calculated by numerically integrating the collapse pdf over the entire spectral acceleration domain. The reliability index (β) expresses the distance of the probability of failure on the qunatile function from the mean of the standard normal distribution in units of variance.

3 EVALUATION OF BRBF DESIGNS

3.1 Structural archetypes

The scope of the presented results is limited to the performance of concentrically braced BRBFs with a two-bay chevron type brace configuration. Braced frames are at the perimeter of a steel frame structure made of pinned beams and continuous columns with hinged supports. BRBF

columns have fixed supports. Floors are made of reinforced concrete slabs and the walls are made of gypsum boards. A total of 24 archetype structures were designed and arranged into 8 Performance Groups (PG). Each group collects buildings with similar characteristics (Table 1).
The design gravity load level represents two extreme combinations of braced floor area (4x4 or 6x6 grid) for a BRBF system. High gravity loads are of particular interest because of their influence on second order effects. Regions of moderate and high seismicity are considered as two seismic hazard cases to verify the applicability of the proposed procedure on both ends of the seismic intensity spectrum. Corresponding peak ground accelerations are 0.15g and 0.4g. The short period domain set of the archetypes contains stiffer structures with 2-3 stories and dominant periods below T_C of the corresponding design spectrum. Design of these structures is typically force controlled and their performance is limited by the load bearing capacity of their members. The other group contains taller structures with 4-6 stories and longer natural periods that are more affected by displacement limits and second order effects.

Table 1. Performance Groups for BRBF design procedure evaluation and results of their seismic performance

Group No.	Characteristics			Number of Archetypes	$P_C\|S_a^D$ [%]		P_C [%]		β	
	Design Load Level		Period Domain							
	Gravity	Seismic			max	min	min	max	min	max
PG-1	Low	Low	Short	3	0.59	1.22	0.025	0.032	3.44	3.51
PG-2			Long	3	1.11	1.85	0.086	0.105	3.11	3.17
PG-3		High	Short	3	0.34	0.85	0.104	0.131	3.01	3.07
PG-4			Long	3	0.12	0.45	0.069	0.087	3.13	3.20
PG-5	High	Low	Short	3	1.72	2.84	0.044	0.056	3.29	3.36
PG-6			Long	3	1.17	2.12	0.102	0.125	3.02	3.09
PG-7		High	Short	3	0.47	1.12	0.116	0.144	2.98	3.04
PG-8			Long	3	0.09	0.38	0.071	0.089	3.14	3.21

Note: S_a^D is the design seismic intensity at the dominant period of the structure; $P_C|S_a^D$ is the probability of collapse conditioned on $S_a(T_1) = S_a^D$; P_C is the probability of collapse over 50 years; β is the reliability index

3.2 Automated BRBF design

The design procedure evaluation methodology presented in the previous section requires seismic performance assessment of a large number of structural archetypes. Therefore, feasibility necessitates automated braced frame design. This research employs an algorithm developed for automatic design of concentrically braced frames [16] with either conventional steel or buckling restrained braces. The algorithm uses heuristic search to find an optimal solution in the parameter space of design variables. On top of the capacity design specifications of EC8 it performs the design checks prescribed above and searches for the most economical set of column and BRB sections that fulfil all required conditions. Columns are selected from standard HE A, HE B and HE M sections. BRB cross-section sizes are selected from an array of candidates in the range of 500 mm^2 to 10,000 mm^2 with a step size of 10 mm^2. The resulting section sizes are used as inputs for the detailed nonlinear dynamic analyses required for design procedure evaluation.

3.3 BRBF performance assessment and design procedure evaluation

Table 1 summarizes the main results of performance evaluation for each PG. Detailed results for each archetype are available in [7]. All performance groups fulfilled the requirements of FEMA P695, namely that the conditional probability of failure of their structures at the design seismic intensity is less than 10%. The majority of individual collapse probabilities of the archetypes are below 3%. In spite of the good performance as per FEMA P695 the probabilities of collapse over the lifetime of structures and the corresponding reliability indices do not fulfil the limits in Eurocode 0 (EC0) for Ultimate Limit State design (P_C < 0.01%). Such a stringent regulation can only be fulfilled by structures that resist extremely rare ground motions. The authors believe that it is not economical to design a structure to resist such rare effects. This observation has been made by other researchers as well [17] and it draws attention to the need for further research on this topic and an assessment of relaxed EC0 limits for seismic performance evaluation.

4 CONCLUDING REMARKS

Buckling Restrained Braces in frames under seismic excitation have several advantages. Their good structural performance arises from their high ductility and energy dissipation capacity. Application of an appropriate design procedure is essential to utilize these characteristics of BRB elements. The conservative approach applied in both numerical modelling and uncertainty estimation provides high confidence in the collapse assessment results. Based on the performance of BRBF archetypes, the proposed design procedure is considered appropriate for BRBF design for frames that are within the scope of the presented research. Therefore, applicability currently is limited to concentrically braced frames with chevron-type brace topology, continuous columns with rigid supports and a maximum of 6 stories. (A detailed list of limits is included in [7].) Investigation of additional archetypes in the future will lead to a better understanding of BRBF behaviour and allow relaxation of the above limits.

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