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EFFECTIVE DESIGN MEASURES AGAINST SOFT STORY DEVELOPMENT IN BUCKLING RESTRAINED BRACED FRAMES

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Abstract

Buckling Restrained Braced Frames (BRBF) are concentrically braced steel frames with Buckling Restrained Braces (BRB) in their diagonals. BRBs are special displacement dependent anti-seismic devices. Buckling of the internal steel core of a BRB is prevented by the continuous lateral support of its casing. This configuration ensures balanced hysteretic behavior and large energy dissipation capacity. The advantageous dissipative properties of BRBs can only be utilized with an appropriate design procedure.

The objective of the presented paper is to draw attention to the high probability of soft story formation in concentrically braced BRBF under seismic excitation beyond the design level of the frame. Although BRBF had been shown to have sufficiently high performance, a significant reduction in collapse probability could be achieved if premature failure due to soft stories was prevented by appropriate design measures.

The issue of soft story formation is tackled using three BRBF archetypes. Structural design is performed with several versions of the BRBF design procedure proposed by the authors for European application. Two dimensional finite element models are created for each scenario (i.e. archetype + design) with the OpenSees finite element code. A custom BRB element developed by the authors and calibrated to experimental results is used to improve the description of nonlinear cyclic hardening behavior of BRBs. An extended version of the probabilistic seismic performance assessment framework presented in FEMA P695 is used for this study. Performance for each scenario is evaluated with nonlinear dynamic analyses in a multi-stripe framework and described by fragility curves after taking several sources of uncertainty into consideration.

Results of a sensitivity analyses on hundreds of versions of a two-story BRBF suggest that soft story development can be effectively mitigated by either strengthening the columns of the braced frame or shifting the distribution of lateral stiffness along the height of the BRBF from a modal towards a triangular shape. Detailed probabilistic seismic performance assessment on a few selected structures confirms the advantages of the modifications. These results also highlight that increased interstory drifts shall be expected below the design seismic intensity level due to shifting the BRBF stiffness distribution.

Keywords: Buckling Restrained Braced Frame design; soft story; probabilistic seismic performance assessment

1. Introduction

A Buckling Restrained Braced Frame (BRBF) is typically a steel frame structure with Buckling Restrained Braces (BRB) in its diagonals. This paper focuses on the behavior of Concentrically Braced Frames (CBF) with BRB elements. The primary advantage of a BRB over a conventional steel brace is its stable behavior under cyclic loading. Because of their high slenderness, conventional steel braces buckle before reaching their yield force under compression. BRB buckling is prevented by the hollow casing that provides continuous lateral support to the steel core.

After their introduction in Japan [1], Southeast Asia [2] and the United States [3] numerous experimental tests have focused in the past three decades on understanding and improving the behavior of BRB elements. Besides the overall cyclic performance of the element [4], investigators often pay special attention to a particular issue, such as design of the casing [5], low cycle fatigue performance [6], connection and gusset plate [7] or subassemblage [8] behavior. The large number of test results facilitate numerical model development for BRB elements and their connections to beam-column joints.

Although abundant experimental results are available for BRB element behavior, the response of BRBF – especially beyond their design load – is rarely investigated [9, 10]. The high cost of full frame tests makes it necessary to complement them with numerical analyses if we want to explore frame response under highly uncertain seismic loading. Recent high quality numerical BRB models [11, 12] and the rapid increase in computational resources allows estimation of BRBF performance under seismic excitation in a robust, probabilistic manner [13]. A framework was proposed in FEMA-P695 [14] that provides a common set of tools and methodologies for Probabilistic Seismic Performance Assessment (PSPA). That framework had been employed to verify the BRBF design procedure in the USA [15] and to evaluate particular design parameters such as the behavior factor [16].

The authors applied an extended version of the FEMA-P695 framework in a recent study to propose [17] and verify [18] a BRBF design procedure that conforms with the Eurocode 8 (EC8) standard [19]. We focused on CBF with pinned beam connections and diagonal BRB elements. Results in [18] confirm that BRBF archetypes designed with the proposed design procedure have sufficiently low collapse probability. Judging by their high performance, one would expect that the archetypes developed an advantageous global mechanism before collapse. This expectation is also supported by the design rules that are supposed to ensure simultaneous yielding of BRB elements and elastic behavior of non-dissipative elements of the braced frame. However, the advantageous global mechanism seldom developed in our numerical analyses under high intensity ground motions. Numerical results suggest that archetypes achieve high performance in spite of soft story formation.

The observation of frequent soft story development in CBFs beyond their design seismic intensity is not new and not BRB-exclusive [20]. BRBF are used in this paper as a vehicle to investigate soft story formation in CBF and results are possibly applicable to a wider range of steel CBF. We explore the reasons behind the phenomenon and propose design changes that can lead to more predictable frame behavior under high intensity seismic loading.

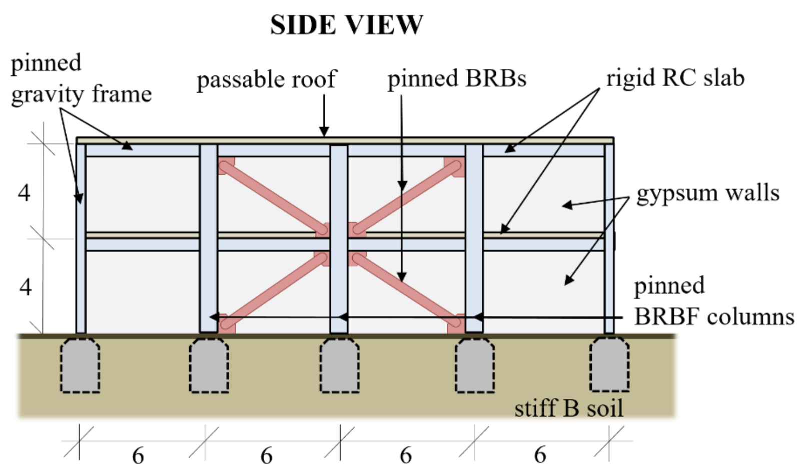
2. Problem statement and objectives

The problem of soft story formation is illustrated through a simple BRBF with only two stories. Vibration of the example frame is governed by the first two modes. This is considered an advantage, because it is easier to understand the correlation between design variables and the frame response. The structure is a regular office building with steel gravity columns, perimeter BRBFs for lateral load resistance and reinforced concrete slabs (Fig.1). All columns are continuous with pinned connections at their base. All stories are 4 m tall and all bays are 6 m wide. Each BRBF has to support a tributary area of 600 m^2 at each story. Braces are installed in a two-bay X-brace configuration (i.e. back-to-back, adjacent bays of "zigzag" diagonal bracing). The frame is made of S235 grade steel.

The structure is at an area of high seismicity with 0.4 g Peak Ground Acceleration (PGA). The site is on relatively stiff, type B soil as per EC8. The frame is designed with the EC8 conforming BRBF design procedure proposed by the authors [17]. Main characteristics and design variables are shown in Fig.1.

The demand to capacity (D/C) ratio of the braces is practically identical and close to 100%. Thus, they fulfill the uniformity condition of EC8 and they are expected to yield simultaneously. The capacity curves of the structure from modal load distribution are shown in Fig.2a. Base shear values are normalized by $V_{base,d}$. Story drifts are in good agreement with roof drifts, there is no apparent soft story formation. The first yield occurs at the proximity of the design level. The additional capacity is due to material overstrength of BRB elements. Overstrength at the design drift level (δ_d = 1.36%) is conservatively estimated by γ_{Rd}.

Fig.2b and 2c show the results of PSPA (the evaluation methodology is explained briefly in section 3): the interstory drift response and the fragility curve of the structure. The horizontal axis is normalized by $S_{a,D}$, the design spectral acceleration. The red curve is the final result that includes all required modifications. The conditional probability of collapse at 1.5 $S_{a,D}$ is 5.73%. The total probability of collapse over the lifetime of the structure is 1.63%. Both values are within the limits typically applied in seismic design evaluation. PSPA results confirm the good performance of the designed structure and suggest that it behaves according to the assumptions taken during design.



CHARACTERISTICS

m_{tot} = 795 t T_1 = 0.61 s $\theta_{pl,max}$ = 0.10

	Buckling Restrained Braces			
story	A_y [mm^2]	N_{Ed} [kN]	η_{BRB}	Ω/Ω_{min}
2	1760	414	100.0%	1.000
1	2640	619	99.7%	1.003

BRBF column section: HEM120

γ_{Rd} = 1.613 $\delta_{DLC,d}$ = 0.68% $V_{base,d}$ = 977 kN

Fig. 1 – Layout and main characteristics of the sample BRBF with two-bay X-brace configuration.

Notation: m_{tot} – total braced mass of each BRBF; T_1 – dominant period of vibration; $\theta_{pl,max}$ – plastic stability index; A_y – cross-sectional area of the BRB steel core in its yielding zone; N_{Ed} – design axial load; η_{BRB} – BRB demand / capacity; Ω / Ω_{min} – BRB uniformity check result; γ_{Rd} – design overstrength; $\delta_{DLC,d}$ – design drift for damage limitation checks; $V_{base,d}$ – design base shear force

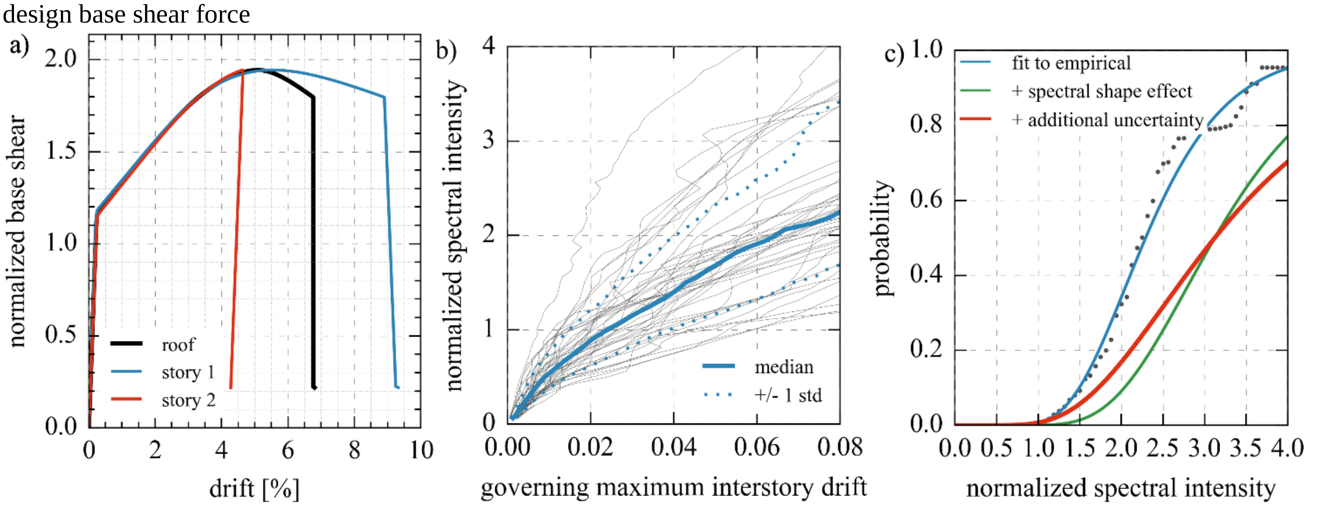


Fig. 2 – Pushover capacity curves (a), incremental dynamic analysis results (b) and the corresponding fragility curve (c) of the sample BRBF. Ordinates of the capacity curve plot are normalized by $V_{base,d}$. Ordinates of the IDA plot and abscissae of the fragility curve plot are normalized by $S_{a,D}$.

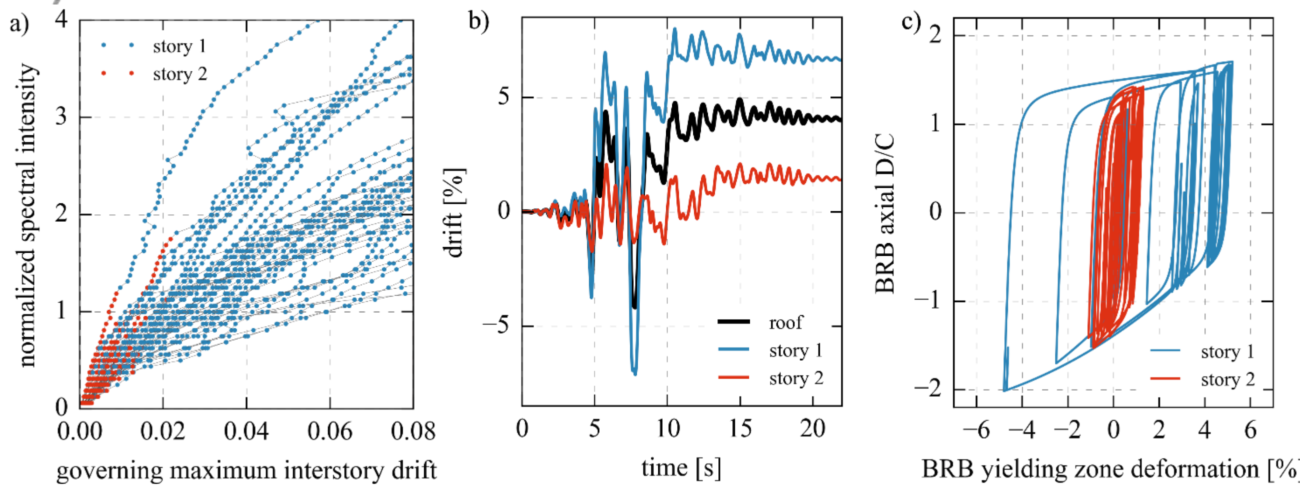


Fig. 3 – Incremental dynamic analysis results of the sample BRBF with governing stories highlighted (a). Drifts over time (b) and BRB force-deformation plots (c) from one particular dynamic analysis with 8% max. interstory drift. Ordinates at the first plot are normalized by $S_{\mathrm{a,D}}$.

The topic of this paper is introduced in Fig.3a. The figure is based on the results already presented in Fig.2b. Each dot in Fig.3a corresponds to one dynamic analysis. Connected dots correspond to analyses with the same ground motion record scaled to increasing levels of spectral acceleration. The color of a dot indicates if the maximum interstory drift (δ_{max}) is measured at the first (blue) or at the second (red) floor. Design drift at the second floor is larger, thus the second floor is expected to govern δ_{max} as long as BRB elements remain in the elastic domain. After a small intermediary zone with mixed results, there are only a few cases with the second floor being dominant beyond δ_d. Second story drifts hardly exceed 3% regardless of the spectral intensity level.

Fig.3b and Fig.3c show detailed response from a typical dynamic analysis in Fig.3a that corresponds to 8% governing δ_{max}. First story drifting becomes dominant after the first large amplitude movement of the structure and remain larger throughout the entire event. Residual drifts at the first story are also significant. The observed non-uniform story drifting is especially disadvantageous, because deformation of the diagonal braces is proportional to story drifts. While BRB elements at the first story experience high amplitude cyclic loading, second floor BRBs remain within a small ±1% strain domain. Consequently, the potential energy dissipation capability of second floor braces is barely utilized.

These analyses confirm that although the example structure has sufficiently small collapse probability, there is still room for improvement in its design. Changes are sought in the design procedure that can reduce the number of soft story scenarios and ensure development of a more advantageous global plastic mechanism in the majority of cases. To reach that goal, we have the following objectives within the scope of this paper:

- Analyze the influence of several design aspects on soft story formation through a sensitivity study on nonlinear braced frame response. Important aspects shall include relative story stiffness, column stiffness, column base rigidity and seismic demand intensity.
- Use the findings in the sensitivity analyses to develop a proposal of improvements in the BRBF design procedure.
- Compare the performance of structures designed with the improved and the original procedures and evaluate the merits of the proposed improvements based on the results.

3. Methodology

The problem of soft story formation is investigated with the following methodology:

- A large number of BRBF are designed using Modal Response Spectrum Analysis (MRSA) with an EC8 conforming design procedure [17]. A subset of the frames is designed with modified parameters and/or according to modified design rules.
- A high quality numerical model is created for each designed frame.
- Behavior of each numerical model is analyzed in a nonlinear static and a series of nonlinear dynamic analyses.
- Design procedures are qualified through statistical evaluation of the performance of corresponding structures.

The following subsections provide a brief explanation of each part of the methodology. Further details are available in [18].

3.1 Structural design

Structural design is performed with the design procedure proposed by the authors for the upcoming revision of EC8. The procedure is an extension of existing capacity design rules for CBF in EC8. The main differences in linear elastic MRSA of BRBF compared to that of CBF is in the modelling of the braces and the calculation of structural overstrength. The proposed extensions lead to a procedure that is similar to the one in the AISC 341 standard [21], but fits in a European design environment:

- Both tension and compression braces shall be modelled as truss elements using A_y, the cross-section of the so-called yielding zone of the brace.
- The influence of non-prismatic steel core geometry is taken into account through an increase of the initial stiffness of the BRB steel material by an f_{SM} stiffness modification factor. The value of f_{SM} is in the range of 1.2 – 1.4 for typical brace and bay sizes.
- Design loads shall be calculated with a behavior factor of q = 7.0.
- The maximum variation of D/C ratio of BRB elements (Ω) along the height of the frame is 10%. (25% variation is allowed for general CBF). This so-called uniformity condition is supposed to prevent soft story formation by forcing the braces to yield simultaneously. The reduction of allowable variation is justified by the custom design of BRBF.
- Non-dissipative elements (e.g. columns, foundation) of the braced frame are designed for an increased load level as per the capacity design rules of EC8. The amplification is controlled by the γ_{Rd} overstrength factor that is typically in the range of 1.5 – 2.5 for BRBF. Besides Ω and the material overstrength (γ_{ov}), an additional factor due to strain hardening shall be considered in the calculation of γ_{Rd}. Hardening under tension and the non-symmetric hardening component under compression are described by strain dependent ω and β factors, respectively.
- Columns are required to have high ductility capacity so that they will be able to support the braced frame even after plastic hinge formation. This is especially important in BRBF with fixed columns.

All frames presented in this paper are designed with the aforementioned procedure using MRSA in AxisVM, a commercial finite element environment that is used by practicing structural engineers in Europe [22]. Designs that serve as a basis of comparison are perfect in a sense that the frame is optimized up to the point where it fulfills all regulations of the Eurocodes as well as possible. Investigation of the influence of good, but not perfect designs is out of the scope of this paper. All braced frame columns are designed with pinned bases. Note that although columns with fixed bases are a less favorable design choice, it has been confirmed in [18] that such BRBF also have sufficiently high performance. This study uses pinned columns to avoid the additional complexity in frame response due to plastic hinge formation at base columns at moderate drift levels.

It is assumed that design of the braced frame is governed by seismic effects. Therefore, wind and snow load is not considered during design. The gravity frame is assumed to have no lateral stiffness, while the slabs are assumed to provide a rigid diaphragm for load transfer. Beams of the braced frame are not designed in detail, because their sizing is controlled by gravity loading. The high stiffness of the slabs minimizes the influence of beam axial rigidity on the structural response.

3.2 Numerical frame model

Numerical models for nonlinear analyses were developed with the OpenSees 2.4.6 finite element code [23]. A planar beam model of one braced frame is created for each BRBF design. Beams of the frame and members of the leaning truss are modelled with truss elements and a perfectly elastic material with a Young's modulus of 10 TPa. 2% Rayleigh damping is applied following the recommendations in [24]: Mass proportional damping is applied to all nodes with lumped masses, while stiffness proportional damping is applied to the linearly elastic elements.

Frame columns are modelled by continuous elastic beam elements (*elasticBeamColumn*). Geometric nonlinearity is considered through corotational geometric transformations. The modified Ibarra-Medina-Krawinkler deterioration model is used with the parameters of Lignos and Krawinkler [25] to model the inelastic moment-rotation response at column ends. Axial force (N_c) and bending moment interaction is taken into account by an approximate reduction of plastic hinge bending capacity using N_c dependent reduction formulae from Eurocode 3 (EC3) [26] that are based on the theory of plasticity. N_c for this calculation is defined as the axial load in the column at the design drift level during pushover analysis. This approach provides conservative estimates for N_c. Axial force of columns is consistently below their flexural buckling resistance, therefore, stability failure of columns is not expected.

Each BRB is modelled by a combination of a truss element (*corotTruss*) and an advanced nonlinear material model (*Steel4*). The material model was developed by the authors to provide sufficiently accurate description of BRB behavior under cyclic loading It can simulate kinematic and isotropic hardening, ultimate strength, non-symmetric behavior and the influence of load history. Low cycle fatigue is also considered by coupling Steel4 with the Fatigue material in OpenSees. Material parameters are defined using functions that were calibrated by results of 15 uniaxial cyclic load tests (e.g. [27]). Different parameter sets are applied for pushover and dynamic analyses, because the former requires the envelope of cyclic response. Therefore, BRB response in pushover analysis is set to follow the design BRB backbone curve. The superiority of this modelling approach for BRB elements over conventional bilinear models had been confirmed in [28] and the significant error that stems from the application of simplified BRB models is highlighted in [18].

3.3 Numerical analyses

Numerical analyses are performed in the OpenSees environment. All analyses include geometric and material nonlinearity. Three types of analyses are used in the following investigations:

- Pushover (Fig.2a): nonlinear static analysis with a constant vertical and a monotonically increasing horizontal loading. The horizontal load distribution is either modal or uniform.
- Dynamic (Fig.3b): a single response history analysis where a real ground motion record (GM) is used as acceleration input. The GM is selected from the Far Field Set of FEMA-P695. It is typically scaled to an appropriate spectral acceleration level, but otherwise not modified.
- Incremental Dynamic (IDA) (Fig.2b): all GMs from the Far Field Set are used at gradually increasing levels of spectral acceleration to get detailed information on the dynamic response and collapse probability of the structure. Responses with more than 12% governing δ_{max} are discarded, because they are deemed to be beyond the range of applicability of our models.

3.4 Performance evaluation

Three performance levels and corresponding performance limits are considered during evaluation. All three are determined from IDA results:

- Collapse prevention: performance is described by the probability of collapse over the lifetime of the structure and the conditional probability of collapse at a particular $S_a(T_1)$ spectral intensity. Because there is no experimental evidence of stable cyclic BRB response above 6% yielding zone strain, braces are conservatively assumed to fail above that limit. Consequently, collapse is defined as a response with governing $\delta_{max} > 8\%$. The distribution of governing δ_{max} at each examined $S_a(T_1)$ level is modeled with a censored two parameter Lognormal distribution, where collapsed cases are the censored samples. Parameters of the Lognormal distribution are fitted with maximum likelihood estimation. The empirical fragility curve is defined as the cumulative probability of exceeding δ_{max} as a function of $S_a(T_1)$. The empirical fragility curve is modified to take the spectral shape effect and additional sources of uncertainty into consideration. The modified fragility curve is used to quantify collapse probability.
- Life safety: performance is checked at $S_{a,D}$, the design spectral acceleration level. The structure shall not sustain significant damage at this level. Expected damage and structural performance is described by the magnitude of governing residual interstory drifts.
- Immediate occupancy: performance is checked at 0.5 $S_{a,D}$, because this corresponds to the Damage Limitation Criteria in EC8. Significant damage is not allowed at this small spectral intensity. According to current EC8 regulations, the deformation capacity of non-structural elements is depleted beyond 1% interstory drifting. Therefore, structural performance at this performance level is described by the governing δ_{max} value at 0.5 $S_{a,D}$ spectral acceleration.

4. Sensitivity analysis

The sample 2-story BRBF from the problem statement section is used as the basis of the following investigation. The presented sensitivity analysis focuses on two main components of the frame: (1) columns and (2) braces. Modification of each component is analyzed in its own subsection, followed by a proposal for improving the seismic performance of BRBF.

4.1 Column strengthening

When a multi-story CBF experiences seismic excitation, braces at one of its stories will inevitably yield sooner than the others. The stiffness of yielded braces decreases to a few percent of its elastic value and the participation of columns in the lateral stiffness of that story becomes more significant. If the columns have sufficient flexural rigidity available to effectively support the yielded braces and ensure that deformations at other stories continue to increase, then they can bridge the gap between yielding of BRB elements at different stories of the frame. Increasing the stiffness of columns increases the size of the gap they can bridge between yielding of BRBs.

Current CBF capacity design procedure in EC8 ensures that columns have sufficient capacity to resist the typically axial internal forces that develop at the design seismic scenario, but otherwise does not ask the designer to consider the role of columns in the global behavior of the frame. Consequently, the designer is not directly encouraged to choose a column with large flexural rigidity.

The results presented in Fig.4 describe the dynamic response of the sample BRBF as a function of horizontal braced frame stiffness due to its columns (K_{col}). 60 BRBF were created with each with a specific European wide flange (HEA, HEB, HEM) section as its column. Each structure was subjected to 44 GMs of the Far Field Set. Each GM was scaled so that it induced a governing $\delta_{max} = \delta_{target}$ at the sample BRBF with its original, small column sections. The scaling factor was kept constant for the other 59 structures. The change in δ_{max} under the same seismic excitation is assumed to be a good indicator of the change in structural performance. The δ_{max} values from the 44 GMs are considered samples of a random $\delta_{max,rnd}$ variable. Statistics of $\delta_{max,rnd}$ are calculated for every structure and plotted in Fig.4. Continuous, dashed and dotted lines show the mean, median and mean ± one standard deviation of the samples, respectively. The shaded areas correspond to the full range of available samples. Analyses were performed for several δ_{target} values (Fig.4b and 4c) to explore the dependence on drift magnitude. The stiffness of columns is expressed relative to the stiffness of BRB elements (K_{BRB}) at the first story.

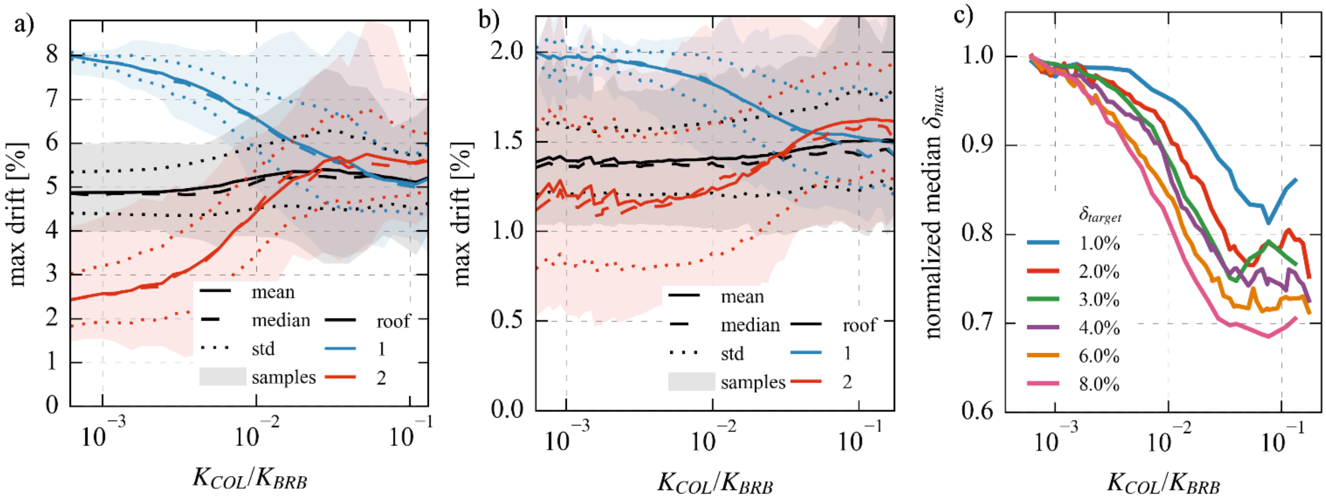


Fig. 4 – Performance of the sample BRBF as a function of braced frame column stiffness. Statistics of δ_{max} in dynamic analyses with δ_{target} = 8% (a), δ_{target} = 2% (b) and comparison of median δ_{max} at several δ_{target} levels (c).

Because the capacity design rules in EC8 do not prescribe a minimal flexural capacity for CBF columns, the original design of the sample BRBF can fulfill EC8 criteria with columns characterized by $K_{col} < 0.001K_{BRB}$. Fig.4 confirms that soft story formation at the first story seems inevitable for such frames. Drift of the second story does not exceed half of the first story drift in any of the 44 examined samples (red shaded area at the left side of Fig.4a). Increasing column stiffness beyond $K_{col} > 0.01K_{BRB}$ leads to a significant reduction in δ_{max} difference between the first and second stories under the same seismic excitation. Analysis of the same structure at smaller δ_{target} values reveals similar trends, but the beneficial effect of column strengthening gradually decreases with decreasing δ_{target}. This is explained by the smaller likelihood of soft story formation and column utilization at smaller drift levels.

4.2 Brace stiffness adjustment

If the uniformity condition is fulfilled during CBF design, the relative stiffness of its braces follow the distribution of story shear force along the height of the structure. Because seismic loads in MRSA are based on the shapes of elastic vibration modes, such a design approach inherently assume elastic behavior. Nonlinear response is practically considered in MRSA-based design by load reduction with the behavior factor and the assumption of the equal displacement rule. The behavior factor is defined as a sufficiently small value to avoid premature yielding and large deformations under frequent ground motions. This approach provides a good approximation for single degree of freedom systems, but it is only appropriate for those multiple degree of freedom systems that can maintain their elastic stiffness distribution during their inelastic response. The authors argue that BRBF is not such a system.

Let us assume that braces at a particular story yield in a BRBF during a seismic event and there is negligible lateral support available from the columns of the braced frame. The sudden reduction in story stiffness changes the vibrational properties of the frame. Stories above the yielded braces will move similarly to a rigid body on top of the soft story. This corresponds to a so-called uniform, mass-proportional load distribution. Such a distribution increases story shear at lower stories and decreases it at upper ones when compared to a typical modal load distribution of relatively short buildings. On the one hand, because they were designed for a modal load distribution, the braces at top floors will be unloaded and will no longer participate in energy dissipation. On the other hand, braces at the bottom floors will experience higher demands than their design capacity and this exacerbates the problem of premature brace yielding. This provides a possible explanation for the soft story formation at the first floor of the sample 2-story BRBF.

The argument above highlights an important design difficulty: Different brace stiffness distribution is optimal for low intensity, frequent earthquakes when the response remains in the elastic domain and for rare ground motions that cause nonlinear behavior. The sample BRBF is used here to demonstrate the difference. Keeping $A_{y,1}$, the first story BRB cross-section at a constant size, $A_{y,2}$, the area of the second story BRB cross section is gradually reduced. The original $A_{y,2,0} = 0.667A_{y,1}$ is the result of the standard design and as such, it corresponds to the modal load distribution. Such results are referred to as modal brace stiffness distributions below. The lower limit of $A_{y,2}$ is $0.50A_{y,1}$, which corresponds to the uniform load distribution. Such brace design is referred to as triangular brace stiffness distribution hereinafter. A total of 11 $A_{y,2}$ values are investigated. Each BRB configuration is combined with a column section that corresponds to one of the following K_{BRB} stiffnesses: 0.1%, 0.5%, 1.0%, 2.0%. K_{BRB} is calculated using $A_{y,1}$ and the same column sections are applied on both stories of the frame. Dynamic response of every structure is evaluated with the 44 GMs in the Far Field Set. Each GM for each structure is scaled up to the point where the governing $\delta_{max} = \delta_{target}$. Better structural solutions are expected to need higher scale factors for the GMs to reach δ_{target}. Similarly to the previous investigation, results are considered samples of a random variable. Fig.5a and 5b show statistics of the random drift variable. Note that the ordinates in Fig.5a and Fig.5b are normalized for each GM by the maximum intensity reached among all BRB configurations (horizontal axis) with the weakest columns ("-" case). This normalization facilitates the comparison of structural performance among different BRB configurations and column stiffnesses.

The median curves in Fig.5a clearly show that the triangular BRB stiffness distribution is advantageous at high drift levels, especially if the columns can provide only moderate support to the stories. Performance of the modal stiffness distribution is inferior with weak columns, but it is heavily affected by the increase in column stiffness. After the column stiffness reaches 2% of K_{BRB}, the modal option becomes superior at δ_{target} = 8%. Fig.5b focuses on the comparison of performance at different target drift levels. Results at $\delta_{max,}$ δ_d and δ_{DLC} are shown. Because the majority of structural response is elastic at small drifts, the modal BRB stiffness distribution clearly performs better at low drift levels. Fig.5c displays the results of an analysis similar to those in Fig.4, but for a BRBF designed with triangular stiffness distribution. It confirms that the difference between first and second story drifts reduced significantly and the increase of column stiffness has less pronounced benefits.

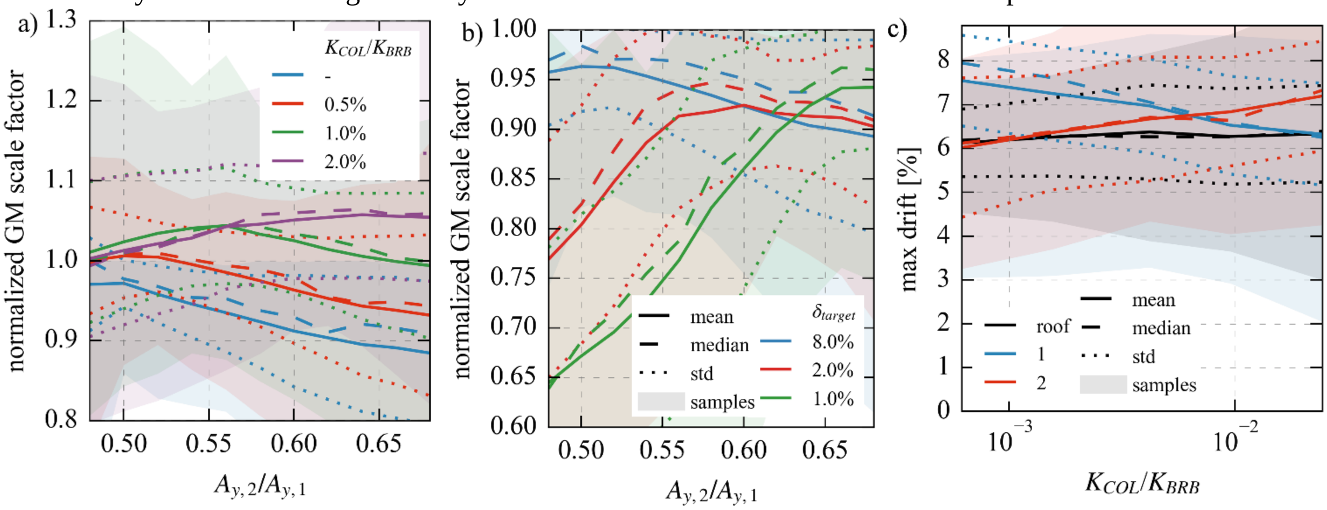


Fig. 5 – The influence of relative BRB stiffness on structural performance. Statistics of normalized GM scaling factors from dynamic analyses with various K_{COL} at δ_{target} = 8% (a), and at various δ_{target} levels with the lowest K_{COL} (b). Statistics of δ_{max} in dynamic analyses with δ_{target} = 8%.

4.3 Proposed design approach

On the one hand, results of the sensitivity analyses confirm the possibility of significant improvement in structural performance beyond the design seismic intensity. On the other hand, Fig.4 and Fig.5 also highlight that the required structural modifications are expected to increase drifts in the elastic domain. In a typical case when costs are to be minimized while the performance objectives shall be fulfilled at all performance levels, the optimal design configuration depends on the sensitivity of the structure to the damage limitation criteria. If drifts at the DLC level are close to the acceptance limit, the BRB sections shall not be reduced at top stories, but the columns shall be increased instead. If the DLC drift limit is less restrictive, the modification of BRB sections according to the triangular stiffness distribution is recommended. The suggested level of amplification of column sections decreases as the stiffness distribution is shifted from modal to triangular, because the performance of the frame with triangular distribution is less sensitive to soft story formation and consequently to column stiffness.

Each load distribution defines only relative sizes of BRB elements along the height, but the design procedure has to regulate the sizing of BRB A_y as well. It has been confirmed by several researchers that designing braces with modal stiffness distribution for axial loads with the behavior factor of 7.0 leads to BRBF with high performance. In the next comparison both stiffness distributions are applied with identical q = 7.0 behavior factors, but further, detailed analysis of a larger number of structures is required to evaluate if this behavior factor is appropriate for the triangular stiffness distribution in general.

5. Comparison of design procedure performance

Three types of BRBF are used to evaluate the merits of the proposed design improvements. Besides the sample 2-story building, frames with 4 and 6 stories are designed to explore how the observations in the sensitivity study can be applied to taller structures with more complex response. Each building is designed with several approaches including the original EC8 design. The design procedures are evaluated through the comparison of structural performance at the three pre-defined performance levels. Investigated scenarios and the corresponding results are summarized in Fig.6. The number of column stiffness options is reduced in the four and six story frames because the minimal column sizes from EC8 conforming capacity design already correspond to cross-sections with flexural stiffness exceeding the omitted stiffness levels. Results in black in Fig.6 refer to the modal brace stiffness distribution, while results with triangular stiffness distribution are printed in red color.

Collapse probability of the investigated BRBF significantly improved by the application of stiffer columns and by the modification of the brace stiffness distribution. Note that the former improvement corresponds to a minor cost increase, while the latter does not require any additional material or human labor. Results also corroborate expectations from the sensitivity study that BRBFs with modal stiffness distribution are more sensitive to the stiffness of their columns.

stories	K_{COL} / K_{BRB}	stiffness distr.	base column section	BRB $A_{y,1}$ [mm^2]	BRB $A_{y,top}$ [mm^2]	P_C | 1.5 $S_{a,D}$ [%]	δ_{DLC} [%]	δ_{rsd} [%]
2	-	M	HEM120	2640	1760	5.73	0.90	0.81
		T	weak axis	2840	1560	2.97	1.05	0.79
	0.5%	M	HEA220	2640	1760	4.42	0.89	0.73
		T		2840	1560	2.32	1.02	0.62
	1.0%	M	HEA260	2640	1760	3.42	0.87	0.67
		T		2840	1560	1.96	0.99	0.54
	2.0%	M	HEA320	2640	1760	2.22	0.85	0.58
		T		2840	1560	1.90	0.97	0.48
4	-	M	HEM200	3660	1670	8.32	1.12	1.04
		T	weak axis	4110	1330	2.77	1.42	0.98
	1.0%	M	HEA280	3660	1670	5.07	1.11	0.87
		T		4110	1330	1.36	1.37	0.71
	2.0%	M	HEA340	3660	1670	3.05	1.02	0.75
		T		4110	1330	0.98	1.28	0.62
6	-	M	HEM280	5150	1860	2.92	1.15	0.91
		T	weak axis	5880	1380	0.65	1.67	0.93
	2.0%	M	HEM280	5150	1860	1.29	1.00	0.67
		T		5880	1380	0.25	1.41	0.59

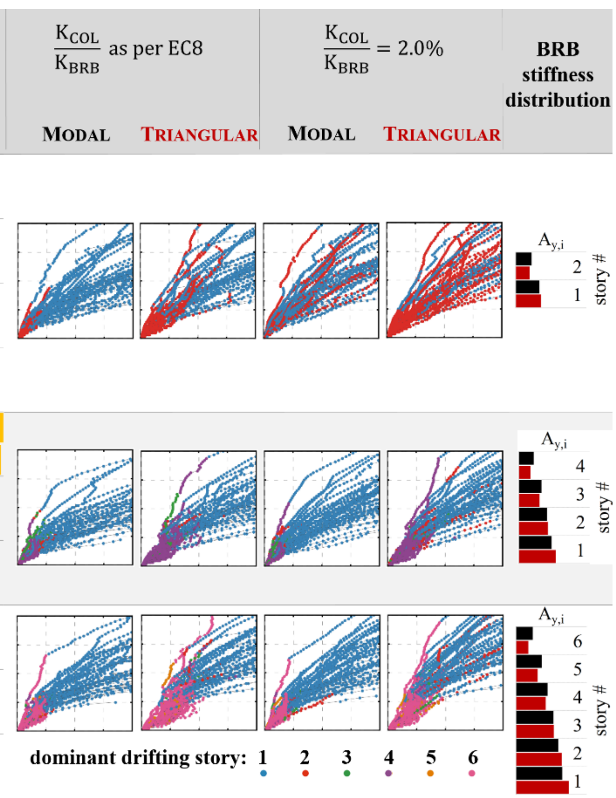


Fig. 6 – Summary of seismic performance and dominant drifting stories corresponding to the investigated BRBF designed with the original EC8 procedure and the proposed modifications in column and brace stiffnesses.

Interstory drifts corresponding to the immediate occupancy performance level are acceptable for all configurations of the two story frame. Taller structures – although designed with $\delta_{DLC,d}$<0.8% – experience interstory drifts beyond the 1% performance limit in all design scenarios. Designs with the triangular stiffness distribution deserve particular attention, because those frames experience significant increase in δ_{DLC} compared to conventional designs. Results suggest that δ_{DLC} cannot be effectively reduced by increasing column stiffness. Residual drifts at the design seismicity (δ_{rsd}) are considered acceptable in all frames. Increasing column stiffness is an effective method for residual drift reduction for all frame sizes and BRB stiffness distributions.

The detailed plots of dynamic results in the right side of Fig.6 are similar to the plot in Fig.3a. They show which floor governs the interstory drift at each dynamic analysis. They confirm that the proposed modifications lead to a more balanced response for the two story frame. While modification of the stiffness distribution does improve the response of taller archetypes, they are still dominated by first floor drifting beyond the design seismic intensity. These cases require further investigation to explain the increase in performance despite only moderate improvement is achieved in the mitigation of soft story formation at the first floor.

6. Concluding remarks

Capacity design rules in Eurocode 8 are expected to prevent soft stories or at least impede the progress of their development. This paper shows through several examples that the application of conventional MRSA to CBF design yields frames where soft stories are expected as soon as the seismic excitation surpasses its design level. The authors argue that the widespread application of performance based design and development of various expectations in terms of structural behavior at performance levels beyond the design seismic intensity increases the importance of mitigating soft story formation.

Results of the presented sensitivity analyses and probabilistic performance assessments confirm that it is possible to reduce the number of cases with soft stories. Two measures are presented: (1) shifting the lateral stiffness distribution along the height from a modal towards a triangular shape; (2) increasing the flexural rigidity of continuous columns of the braced frame. Results at lower performance levels draw attention to the considerable increase in interstory drifting at top stories due to the change in lateral stiffness distribution. The effectiveness of column strengthening in taller buildings is not unanimous, although it is supported by the quantitative results of PSPA. Future work in this direction shall provide sufficient data to improve our understanding and arrive at more advanced design procedures that can ensure advantageous structural response at all performance levels.

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