DETERMINATION OF DISPLACEMENT AMPLIFICATION FACTORS FOR CHEVRON BRBFs

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1. ABSTRACT

Buckling restrained braced frames (BRBFs) can be used as a lateral load resisting system in high seismic regions. In the United States it is necessary to comply with the requirements defined in the Seismic Provisions for Structural Steel Buildings (AISC341-10) for the design and production of BRBs. In addition, response factors defined in Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) should be utilized. In this study, deflection amplification factor (C_d) is evaluated. Pursuant to this goal, three archetypes composed of chevron BRBFs were designed as per ASCE7-10 and AISC 341-10. Elastic and inelastic time history analyses of these archetypes were carried out. The archetypes were subjected to maximum considered earthquake ground motions by using 44 far-field records. C_d factor for BRBFs were obtained from analysis results and compared with the codified value. According to the results, the lower stories experience larger displacements and the upper stories experience lower displacements when compared with code estimates.

2. INTRODUCTION

Buckling restrained braced frames (BRBFs) are among various lateral load resisting systems used for steel structures under seismic loading. A typical steel BRBF is composed of beams, columns, and buckling restrained braces (BRBs). During a seismic event BRBs yield in tension and compression and contribute to energy dissipation. When compared with conventional steel braces, BRBs provide nearly equal tensile and compressive resistances.

In the United States design recommendations for BRBs have been incorporated into AISC 341-10 Seismic Provisions for Structural Steel Buildings [1]. According to AISC341-10[1] buckling restrained braces shall be designed, tested and detailed to accommodate expected deformations. Expected deformations are those corresponding to a story drift of at least 2% of the story height or two times the design story drift, whichever is larger. Qualifying cyclic tests are required for conformance demonstration. In general, uniaxial and sub-assemblage tests are performed according to the loading protocol recommended in AISC341-10 [1]. The loading protocol is based on the design story drift. Furthermore, the brace test specimen is required to achieve a cumulative axial deformation of at least 200 times the yield deformation under uniaxial testing.

The equivalent lateral force procedure can be used together with a set of seismic response factors to obtain the design story drift. This procedure enables elastic analysis and design which is based on reduced seismic forces. The idea here is that the amount of lateral forces is reduced by taking into account yielding and ductility of the lateral load resisting system. The general structural response shown in Fig. 1 can be considered to develop response factors. Their formulation according to Uang [2] is as follows:

$$\mu_s = \frac{\Delta_{\max}}{\Delta_y} \qquad R_\mu = \frac{V_e}{V_y} \qquad \Omega_o = \frac{V_y}{V_s} \qquad R = \frac{V_e}{V_s} = R_\mu \Omega_o \qquad C_d = \frac{\Delta_{\max}}{\Delta_s} = \mu_s \Omega_o \tag{1}$$

where, V_e is the ultimate elastic base shear, V_s is the base shear at the first significant yield, V_y is the base shear at the structural collapse level, Δ_s is the drift at the first significant yield, Δ_y is the drift at the structural collapse level, Δ_{max} is the maximum amount of drift, μ_s is the ductility factor, Ω_o is the overstrength factor, R_μ is the ductility reduction factor, R is the response modification factor, and C_d is the deflection amplification factor.

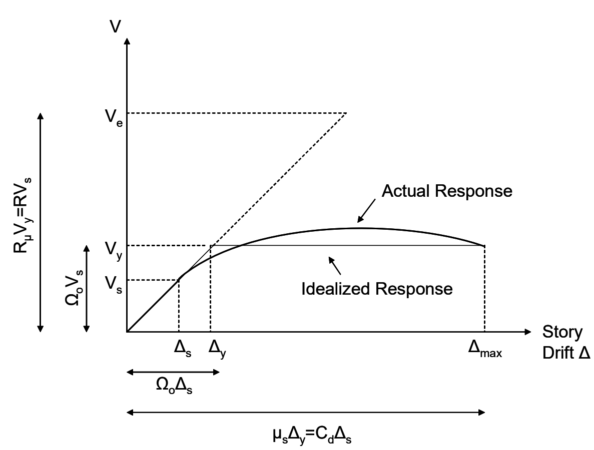

Fig. 1: General structural response

Seismic response factors were developed for various lateral load resisting systems based on observations from past earthquakes and engineering judgment. These factors vary from one specification to the other. In the United States, seismic response factors for BRBFs are given in Minimum Design Loads for Buildings and Other Structures [3] hereafter referred as ASCE7-10. The recommended values of the response modification factor (R), the overstrength factor (Ω_o), and the deflection amplification factor (C_d) are 8, 2.5, and 5, respectively.

Brace deformation demands must be accurately determined at the design stage for satisfactory performance of a BRBF. The design and detailing of a BRB is directly influenced by the design story drift which depends on the seismic response factors. A study has been undertaken to evaluate the deflection amplification factor for chevron BRBFs using the Methodology outlined in FEMA P695 [4]. Pursuant to this goal three archetype BRBFs utilizing chevron typed BRBs were designed and evaluated according to the Methodology. The details of the evaluation are presented herein.

3. OVERVIEW OF THE FEMA P695 METHODOLOGY

The Methodology requires nonlinear collapse simulation on the selected archetype models. Collapse simulation is conducted using a far field record set that consists of 22 pairs of ground motions that are defined in FEMA P695. All 44 ground motion records must be individually applied to an archetype in cases where a two dimensional analysis is performed. The ground motion records are scaled twice. The first scaling is required to anchor the median spectrum of the far field record set to the Maximum Considered Earthquake (MCE) response spectra at the fundamental period of the archetype. The second scaling is applied successively to all far field ground motions until 50 percent of the archetypes exhibit collapse. The amount of scaling that results in the collapse of 50 percent of the archetypes is compared with a variable named the Adjusted Collapse Margin Ratio (ACMR). The target ACMR values are tabulated in the FEMA P695 document and depend on the total system collapse uncertainty (β_{TOT}), and collapse probability. Two conditions must be satisfied for acceptable performance. The average value of ACMR for each performance group should meet the target ACMR for 10 percent collapse probability ($ACMR_{10\%}$). Furthermore, individual values of ACMR for each index archetype within a performance group should meet the target ACMR for 20 percent collapse probability ($ACMR_{20\%}$). While successive scaling approach can be adopted for new structural systems, scaling of all ground motions using a pre-calculated scaling factor is sufficient for evaluation of existing systems. Because individual archetypes are considered in this study, the 20 percent probability of collapse was adopted as a criterion for ACMR (i.e. $ACMR_{20\%}$).

The total system collapse uncertainty (β_{TOT}) depends on various factors such as record-to-record collapse uncertainty, design requirements-related collapse uncertainty, test data-related collapse uncertainty, and modelling-related collapse uncertainty. The methodology enables to use non-simulated collapse models for collapse failure modes that cannot be explicitly modelled. Non-simulated collapse modes can be indirectly evaluated using alternative limit state checks on structural response quantities measured in the analysis.

3. DESIGN AND SELECTION OF ARCHETYPES

Different Seismic Design Categories (SDC) can be adopted in the Methodology in order to represent the variation in seismic hazard. In the present study only one seismic design category namely SDC D_{max} was considered which represents the highest seismic hazard level. The MCE, 5 percent damped, spectral response acceleration parameter at short periods adjusted after site class effects (S_{MS}) was taken as 1.50g. The MCE, 5 percent damped, spectral response acceleration parameter at a period of 1 sec adjusted after site class effects (S_{M1}) was taken as 0.90g.

Two geometrical configurations can be adopted for BRBFs where the first one employs single diagonal braces and the second one employs chevron type braces. In the present study performance of only chevron type BRBFs was evaluated.

Only one type of floor plan shown in Fig. 2 was considered. The floor plan is rectangular with side dimensions of 36 meters and 22.8 meters. There are a total of four bays with single diagonal BRBs in the long direction of the floor plan which are indicated as BF-1 in Fig. 2. A total of two bays with chevron type BRBs are employed in the short direction of the floor plan which are indicated as BF-2 in Fig. 2. Only BF-2 type frames were designed as a part of this study. All beam-to-column connections of the BRBF were considered simple connections with no moment transfer. A dead load of 5 kN/m^2 and a live load of 2 kN/m^2 which are typical for steel office buildings were considered as loading. Story height was taken as 3.5 meters for all stories except the first story where the height was equal to 3.8 meters. In order to take into account variations in structural periods, 3, 6, and 9 story BRBFs were considered.

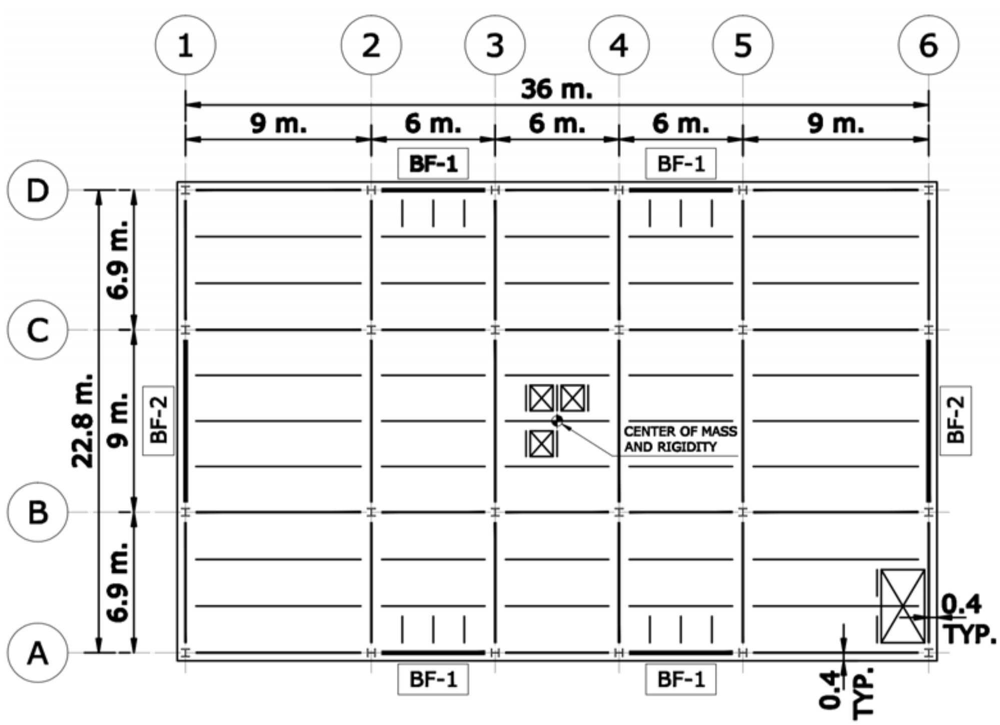

Fig. 2: Floor plan used for the study

AT	Story	Column	Beam	Brace	DISD %	DBAS %	MISD %	MBAS %
1	1	W14x68	W10x45	PL15x146	0.91	2.00	3.22	3.03
	2	W14x68	W10x45	PL15x123	0.87	1.94	2.74	2.82
	3	W14x38	W10x30	PL15x90.5	0.66	1.94	1.66	1.67
2	1	W14x132	W10x45	PL25x113	0.92	2.00	3.57	3.34
	2	W14x132	W10x45	PL25x97	1.04	2.01	3.14	3.21
	3	W14x68	W10x45	PL25x89	1.09	2.12	2.49	2.52
	4	W14x68	W10x45	PL25x79	1.12	2.17	1.67	1.62
	5	W14x48	W10x30	PL25x65	1.18	2.29	1.27	1.13
	6	W14x48	W10x30	PL25x47	0.92	1.94	1.03	0.90
3	1	W14x132	W10x45	PL25x112	1.03	2.06	3.66	3.44
	2	W14x132	W10x45	PL25x104	1.21	2.35	3.03	3.10
	3	W14x132	W10x45	PL25x103	1.33	2.59	2.48	2.42
	4	W14x68	W10x45	PL25x98	1.43	2.77	2.09	1.98
	5	W14x68	W10x45	PL25x92	1.48	2.88	1.56	1.58
	6	W14x53	W10x45	PL25x84	1.54	3.00	1.29	1.07
	7	W14x53	W10x45	PL25x73	1.55	3.01	1.18	0.96
	8	W14x53	W10x45	PL25x59.5	1.48	2.88	1.17	0.93
	9	W14x53	W10x45	PL25x43	1.27	2.46	0.97	0.76

DISD: Design interstory drift, DBAS: Design brace axial strain, MISD: Median interstory drift, MBAS: median brace axial strain.

Table 1: Member sizes of archetypes and response quantities

A992 grade steel with a yield strength of 345 MPa was considered for all framing members and the core plates of BRBs. It was assumed that the non-yielding portion of a BRB accounts for 50 percent of its total length. Designs were conducted according to ASCE 7-10 [3], AISC 341-05 [1], and AISC 360-05 [5]. Frame members of archetypes are given in Table 1. Archetype properties and scaling factors are given in Table 2.

AT	Ns	Mass (ton)	T (sec)	SF_1	μ_T	β_{RTR}	β_{TOT}	$ACMR_{20\%}$	SSF	CMR (SF_2)	SF
1	3	4.17	0.631	2.46	5.76	0.400	0.436	1.443	1.30	1.11	2.74
2	6	10.03	1.032	2.57	4.39	0.400	0.436	1.443	1.34	1.08	2.78
3	9	16.41	1.386	2.51	2.76	0.376	0.414	1.417	1.29	1.10	2.76

T: fundamental period of vibration, SF_1: First scaling factor for anchoring far-field record set to MCE spectral demand, μ_T: period-based ductility of an index archetype model, β_{RTR}: Record-to-record collapse uncertainty, β_{TOT}: total system collapse uncertainty, SSF: Spectral shape factor, CMR: Collapse margin ratio, SF: Ultimate scaling factor.

Table 2: Archetype properties and scaling factors

4. NUMERICAL MODELLING DETAILS AND ANALYSIS RESULTS

Performances of the designed archetypes were evaluated by making use of numerical analysis. The OPENSEES [6] computational framework was used for numerical simulations. Two-dimensional finite element models were used to model the archetypes. The beams and columns of the archetypes were modelled with nonlinear beam column elements and the braces were modelled with non-linear truss elements. In general, one of the BRBF bays was modelled and the tributary mass was added to two of the nodes at every story. Leaner columns carrying gravity loads were linked to the frame to simulate P-Δ effects.

The total system collapse uncertainty is dependent on four factors, three of which requires judgment. These factors depend on the knowledge level and modelling capabilities about the system of interest. BRBFs have been studied for over 15 years and have been implemented in the practice. Therefore, high quality level was assigned to design requirements-related collapse uncertainty (β_{DR}=0.1), test data-related collapse uncertainty (β_{TD}=0.1), and modelling-related collapse uncertainty (β_{MDL}=0.1). The fourth factor that needs to be considered is the record-to-record collapse uncertainty (β_{RTR}) which depends on the period based ductility (μ_T). The μ_T values were determined by conducting nonlinear static (pushover) analysis in accordance with ASCE41-13 [7] and are reported in Table 2. Resulting β_{TOT} and $ACMR_{20\%}$ are reported alongside Spectral Shape Factors (SSF) and ultimate scaling factors (SF) for each archetype in Table 2. The archetypes were subjected to 44 ground motion records and the records were scaled by the ultimate scaling factors. A two percent mass and stiffness proportional damping was used in time history analysis.

Evaluation of archetype performance was based on non-simulated collapse models. Buckling restrained braces generally exhibit stable behavior followed by fracture. Methodology allows for non-simulated collapse models where fracture in members is expected. Furthermore, when one brace fractures the force demand on the fractured brace has to be transferred to all the other braces which eventually results in overloading and fracture in those braces too. In addition, fracture of a brace in any one story triggers soft story mechanism which can potentially trigger collapse of the system.

Interstory drift ratios and brace axial strains were investigated in detail. Design demands and median demands obtained from time history analysis are indicated in Table 1. Variation of interstory drift ratios and brace axial strains along the height are given in Fig. 3. The median of response quantities were used for assessment purposes.

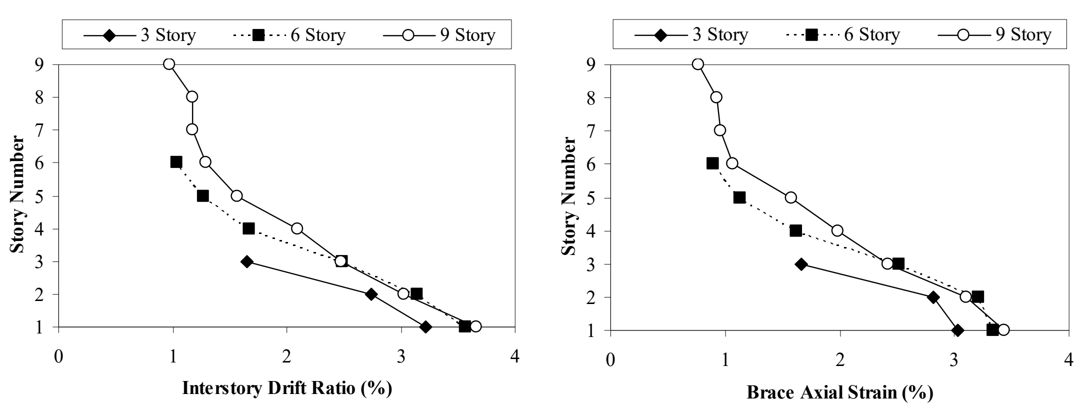

Fig. 3: Interstory drift ratio and brace axial strain of archetypes

5. EVALUATION OF DISPLACEMENT AMPLIFICATION FACTOR

The differences in response quantities observed at the lower stories can be attributable to the differences between the R and C_d factor adopted in design. According to Newmark's equal displacement rule the C_d factor should be taken equal to the R factor to be able to accurately estimate the inelastic demands. For BRBFs, however, the C_d factor is taken lower than the R factor. It should be emphasized that the designs of 3 BRBF archetypes were governed by strength limitations. In general, larger sections than required for strength are used at bottom stories to control the drifts at the top of the structure.

The procedure adopted to study displacement amplification factor differs from the Methodology outlined in FEMA P695. Instead of using the scaling procedure to arrive at the collapse level earthquake the Design Based Earthquake (DBE) level was considered. The three archetypes were analyzed under Design Based Earthquake (DBE) which is equal to the two-thirds of the Maximum Considered Earthquake (MCE). All 44 ground motions were used and separate elastic and inelastic analyses were conducted to obtain lateral drifts. The drift value from inelastic analysis was normalized with the one from elastic analysis to observe the applicability of the Newmark's rule for this type of a multi degree of freedom system. The ratios are presented in Fig. 4.

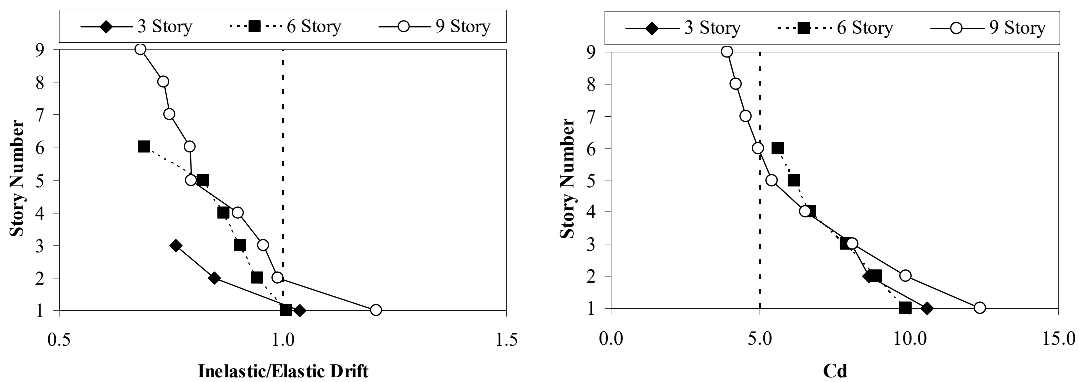

Fig. 4: Ratio of inelastic and elastic drift and variation of deflection amplification factor

6. CONCLUSIONS

A numerical study on seismic performance of BRBFs has been presented. The Methodology outlined in FEMA P695 was applied to BRBFs to evaluate the deflection amplification factor. Nonlinear time history analyses were conducted for 3 archetypes and the structures were subjected to a set of ground motions by taking into consideration the Design Base Earthquake (DBE) to assess deflection amplification factor and the Maximum Considered Earthquake (MCE) to assess story drifts, brace axial strains.

The analysis results indicate that there are marked differences between the calculated interstory drifts and design interstory drifts. These difference stem from the fact that different values are assigned to the deflection amplification factor (C_d) and response modification factor (R). Yielding of BRBs was observed to be non-uniform along the height and significantly higher demands are produced at the lower stories when compared with upper stories.

The deflection amplification factor was assessed as a part of this study. The results indicate that the current codified value of C_d underestimates the deflections of the lower stories while over-estimating the deflections of upper stories. In the past, deflection amplification factors that vary over the height of the structure were developed for other lateral load resisting systems [8] and a similar approach can be taken to arrive at BRBF behavior that results in more uniform yielding along the height. Future research should focus on developing relationships between response modification factor and deflection amplification factor for BRBFs that vary over the height of the structure. In addition, stiffness based design approaches [9] and dual system response [10] can be considered to reduce the amount of difference between the design and calculated deflections.

Brace strain demands are calculated using the interstory drifts at the design stage. Any underestimation of interstory drifts would result in an underestimation of the brace strain demands. In order to safeguard against underestimations the AISC341-10 Specification [1] provides a minimum brace strain demand that corresponds to 2 percent interstory drift. Analysis results showed that the maximum strain in the bottom story of 3 archetypes were higher than the design strains in spite of the minimum demand that corresponds to 2 percent interstory drift. The differences can exceed 50 percent which indicate a potential weakness in the design of buckling-restrained braces.

7. REFERENCES

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