PERFORMANCE OF BUCKLING-RESTRAINED BRACED FRAMES IN DUAL SYSTEMS FOR ASSESSING GLOBAL SEISMIC PERFORMANCE FACTORS

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ABSTARCT

Buckling Restrained Braced Frame (BRBF) is an emerging seismic force-resisting system that ASCE currently permits to be used either as a single seismic force-resisting system or in combination with other seismic force resisting systems. In conventional practice, ASCE suggests that when BRBF is used in conjunction with other lateral force resisting systems in a dual configuration, the lowest Response Modification Factor (R) pertaining to the softer system shall be used. This may result in a significant overdesigning of structures as higher contribution from the BRBF system often remains unutilized. This research aims at developing a methodology for calculating modified Response Modification Factor (R) for structures where dual system occurs horizontally and investigates the effect of using the newly suggested Modified Response Modification Factor (R) for dual systems, where a BRBF system is combined with an Intermediate Moment Frame (IMF). The study aims at proposing an innovative way of calculating Response Modification Coefficient (R), Over-strength Factor (Ω) and Deflection Amplification Factor pertaining to the dual system. A variety of archetype sets are designed following FEMA guidelines with modified R as trial values for Seismic Design Category D. Nonlinear 3D static (pushover) analyses were performed to validate the archetype models and to calculate over-strength factors. The nonlinear models directly simulate essential deterioration modes that contribute to collapse behavior. Subsequently, nonlinear incremental dynamic analyses are conducted for collapse assessment.

INTRODUCTION

Dual seismic force-resisting systems are comprised of individual lateral force-resisting systems in complementary abilities. These systems' design requirements were first written into code in the 1959 SEAOC Blue Book and later in the 1961 UBC. Presently, most of the steel dual systems suggested by ASCE/SEI 7 (2005) are combinations of primary steel Braced Frames (BFs) and secondary Moment Frames (MFs). The combined systems provide many alternative loading paths after member failures, and therefore, are more resistant to seismic perturbations. Design of dual systems (combination of BRBFs with MFs) are challenging but offers significant benefits. Architectural openness, flexibility in interior design and appropriate building facade are architectural advantages. From structural stand point, BRBFs can control inter-story drifts at lower levels that are critical for MFs; conversely, the latter can be a more effective system in controlling inter-story drifts in higher levels of structure. Providing MFs to remain elastic until Buckling Restrained Braces (BRBs) yield, help the redistribution of forces from BRBFs to MFs and prevent the concentration of damage that BRBs can sustain due to their low post-yield stiffness (Maley et al., 2010). Combination of BRBFs and MFs results in an intriguing dual system without incipient shortage of respective systems.

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Presently American Society of Civil Engineers (ASCE) outlines directions for designing dual seismic force-resisting system comprising of BRBFs and SMFs. However, ASCE/SEI 7 (2010) does not provide any conclusive suggestions regarding coupling of BRBFs with any other type of steel moment frames such as Intermediate Moment Frames (IMFs). This study develops global seismic performance factors for dual seismic force-resisting systems consisting of Buckling-Restrained Braced Frames (BRBFs) with ordinary beam-to-column moment connections and Intermediate Moment Frames (IMFs) with prequalified Reduced Beam Section (RBS) moment connections capable of resisting at least 25% of seismic force. Current ASCE recommendation for horizontal combination of different structural systems is that the designer should use the more conservative approach in selecting the seismic response coefficients. For example, the Response Modification Factor (R) for BRBF system and IMF systems individually are 8.0 and 4.5 respectively. ASCE does not have any recommendation when BRBFs are used in combination with IMFs in a dual system. Since the Response Modification factor (R) is not listed by ASCE, we will quantify the values for the seismic response parameters for the BRBF/IMF dual system and compare the results with current ASCE code of practice.

GLOBAL SEISMIC PERFORMANCE FACTORS

The Response Modification Coefficients, R factors, were first presented in the ATC 3-06 report (FEMA P695, 2009). Most of the building codes allow a reduction in seismic design loads by amount of R taking advantage of the fact that structures have considerable reserve strength (over-strength) and capacity to dissipate energy (ductility). In fact, seismic performance factors are used to design seismic force-resisting systems that are designed using linear approaches of analyses, but are responding in the nonlinear range. Values of the response modification coefficient, R, the over-strength factor, Ω_0, and the deflection amplification factor C_d, greatly depend on structural seismic force-resisting system and structural material. Figure 1 [FEMA P695, Figures C4.2-1 and C4.2-3 from the commentary of the NEHRP Recommended Provisions (FEMA 450, 2004)] shows an idealized pushover curve of a seismic force-resisting system.

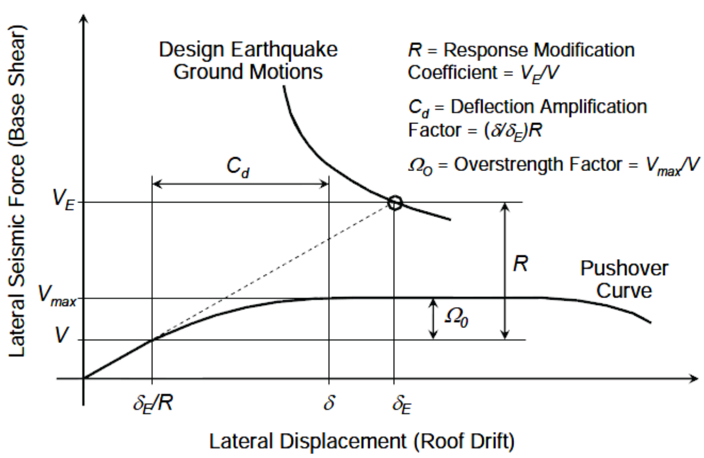

Figure 1. Idealized pushover curve and seismic performance factor definitions (Courtesy to FEMA P695, 2009)

In Figure 1, V_E is the maximum base shear that develops in the structure's seismic force-resisting system, if the system, under severe earthquakes, remains in elastic range and none of the components experience inelastic behavior. V_{max} and V represent maximum strength of a fully yielded system and design base shear, respectively. R factor as shown in Eq.(1) is the ratio of maximum base shear considering elastic behavior to design base shear. Structural over-strength which is due to redistribution of internal forces, higher material strength than those specified in design, strain hardening, various load combination, member oversize because of member grouping and so forth is called over-strength factor (Uang, 1991). Over-strength factor, Ω_0, is defined in Eq.(2).

$$R = \frac{V_E}{V} \tag{1}$$

$$\Omega_0 = \frac{V_{\max}}{V} \tag{2}$$

In Figure 1, the term δ_E/R represents roof drift corresponding to design base shear and the term δ is roof drift of yielded seismic force-resisting system corresponding to design earthquake ground motion. The deflection amplification factor, C_d, is used to calculate the expected maximum inelastic displacement from elastic displacement induced by the design earthquake. Deflection amplification factor is a fraction of R factor (less than 1.0) and is highly dependent on the inherent damping of the system. In this study, Global Seismic Performance Factors will be developed for dual systems considering the Maximum Considered Earthquake (MCE) ground motion and collapse level ground motion concepts. MCEs are 1.5 times the design level ground motions which are defined as mapped acceleration parameters based NEHRP Recommended Provisions. Collapse level ground motions, as depicted in Figure 2, are defined “as the intensity that would result in median collapse of seismic force-resisting system” (FEMA P695, 2009). Figure 2 which is derived from FEMA P695 (2009) illustrates the concept of Collapse Margin Ratio (*CMR*).

In Figure 2 an idealized pushover curve, MCE ground motion and collapse level ground motion are shown using spectral coordinates. S_{MT} is the maximum considered earthquake (MCE) spectral acceleration at the fundamental period (*T)* of the system and $\hat{S}_{CT}$ is the median 5% damped spectral acceleration of the collapse level ground motion. Terms SD_{MT} and SD_{CT} are spectral displacements relevant to S_{MT} and $\hat{S}_{CT}$, respectively. As expressed in Eq.(3) the *CMR* is defined:

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} = \frac{SD_{CT}}{SD_{MT}} \tag{3}$$

The *CMR* calculated by preceding equation, is the prominent parameter to characterize the collapse safety of structures.

FRAME WORK

The Methodology proposed in Quantification of Building Seismic Performance Factors FEMA P695 (2009) was utilized to develop global seismic performance factors, recognizing that it is not necessarily in full compliance with every requirement of FEMA P695 (2009). The process involves five main steps: (1) system information; (2) archetype development; (3) model development; (4) nonlinear analysis and results; and (5) discussion (FEMA P695, 2009). Preceding steps outline the key elements for developing essential design information with enough details to find out the allowable range of application for proposed seismic force-resisting system, to simulate nonlinear response and evaluate the collapse risk of the system.

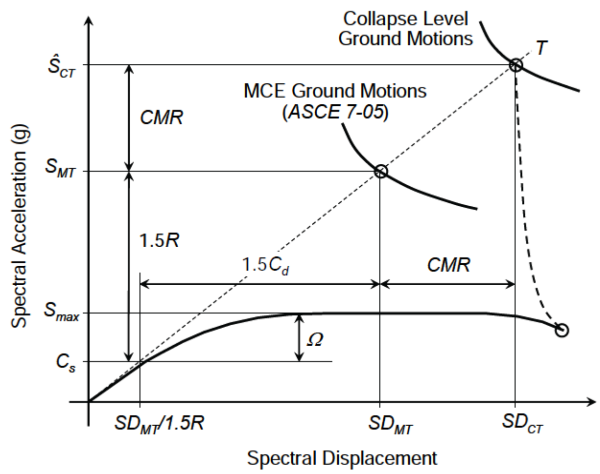

Figure 2. Collapse margin ratio and seismic performance factors (Courtesy to FEMA P695, 2009)

SYSTEM INFORMATION

A prerequisite to reliable assessment of structural response and development of linear and nonlinear model are detailed system design requirements and comprehensive test data. The former are indispensable provisions and criteria that engineers use to proportion and detail various members and analyze the structural response (FEMA P695, 2009). Novelty and uniqueness of a newly proposed system requires establishment of new design criteria. Since the BRBF/IMF dual system is comprised of two individual systems that are already established, the design requirements pertaining to each system are utilized. Minimum Design Loads for Buildings and Other Structures ASCE/SEI 7 (2010), Specification for Structural Steel Buildings AISC (2010), and Seismic Provisions for Structural Steel Buildings ANSI/AISC 341 (2005) are well-vetted design requirements that were used in this study.

Experimental results were employed to provide essential data for nonlinear modeling of components. The data provided by StarSeismic® LLC for Powercat™ BRBs were applied to both linear and nonlinear analyses (Rutherford, 2011). Empirical equations that take into account combinations of both geometric and material parameters and suggested by PEER/ATC-72-1(2010) were used for lateral beams and columns.

ARCHETYPE DEVELOPMENT

The dual BRBF/IMF system used for evaluation comprises of non-perimeter BRBFs with ordinary beam-to-column moment connections and perimeter IMFs with prequalified Reduced Beam Sections (RBSs) (ANSI/AISC358, 2005) as shown in IMFs are designed so that they are capable of resisting at least 25% of prescribed seismic forces. The building has 5 bays in each direction and all bays span 30 feet. The story height is 13 feet except for the first story, which is 18 feet high. Chevron type BRBs are used and all floor diaphragms are assumed to be rigid. The building is used as an office building with an Occupancy Category II per ASCE/SEI 7 (2005). The intended range of application is for upper bound of Seismic Design Category D (SDC D_{max}) and Site Class D (stiff soil). The mapped MCE spectral response acceleration at short period (S_S) and at 1-second period (S_1) were taken as S_S = 1.5g and S_1 = 0.59g, respectively. The 1-second value of MCE spectral response acceleration was intentionally defined less than 0.6g to avoid Equation 12.8-6 of ASCE/SEI 7 (2005) and to assess the minimum base shear design requirements. Floor and roof dead loads (exclude frame elements self weight) are taken 80 and 66 psf, respectively. For the sake of simplicity, all live loads were deemed to be non-reducible by 50 and 20 psf intensities for floor and roof, respectively. It is important to mention that generic seismic design criteria such as structural irregularities, redundancy, and soil-structure interaction that are equally effective for all seismic force-resisting systems are not taken into account.

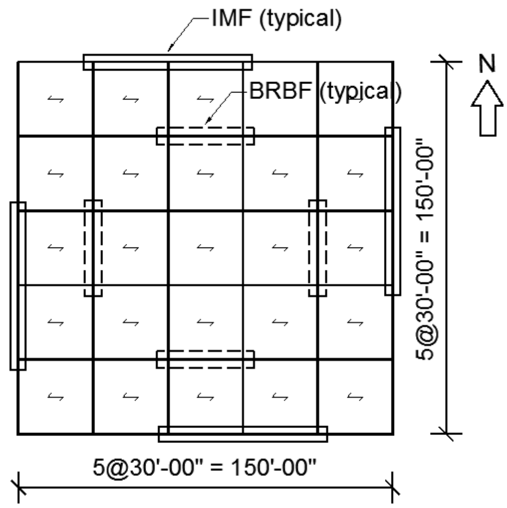

Figure 3. Plan View of Typical Archetype Building

MODEL DEVELOPMENT

To prepare archetype models, initial trial values of the response modification coefficient, R, deflection amplification factor, C_d, and over-strength factor, Ω_0, are required. In this study, three 8-story dual BRBF/IMF structures have been designed based on three different series of seismic performance factors. Table 1 depicts three 8-story archetype IDs, pertinent seismic performance factors, and secondary moment frames (IMFs) seismic force capacity. It is important to note that IMF portion of archetype 206 is capable of resisting at least 35% of seismic forces. On the other hand, the other two are capable to resist at least 25% of seismic forces. Hereafter in the discussion and figures archetype IDs are denoted as they are shown in Table 1.

Table 1. Archetype Seismic Design Criteria

Archetype ID	R	Ω_0	C_d	IMF Seismic Force Capacity
Archetype 106	6.25	3	6	25% of Prescribed Seismic Force
Archetype 206	7	2.5	6	35% of Prescribed Seismic Force
Archetype 306	10	2.5	7	25% of Prescribed Seismic Force

Gravity and lateral beams, girders, and columns are A992 Grade 50 steel with wide flange sections. Braces are made of StarSeismic® BRB sections with minimum yield strength of 39 ksi. The Response Spectrum Analysis (RSA) method of ASCE/SEI 7 (2005) and FEMA 451 (2006) was used to analyze the structures. P-delta effects of both gravity and seismic force-resisting system were considered. The important exception in analysis procedure is period determination. Although ASCE/SEI 7 (2005) permits to use approximate period, T_a, to perform the analysis, the fundamental period, T, as shown in Eq.(4) is used within the study (FEMA P695, 2009). The calculated fundamental period of the 8-story archetype buildings is 0.944 seconds.

$$T = C_a T_a = C_u C_t h_n^x \tag{4}$$

The direct analysis method of design, offered by AISC (2010), was used to design structures. By utilizing this comprehensive approach, the effects of member initial imperfections were taken into account. The seismic designs of BRBFs and IMFs were based on Seismic Provisions for Structural Steel Buildings ANSI/AISC 341 (2005) and Seismic Design of Buckling-Restrained Braced Frames by Lopez and Sabelli (2004). Table 2 and Table 3 indicate BRBF and IMF element sectional properties, respectively.

Table 2. BRBFs Sectional Properties for Each Archetype Structure in N-S Direction

Story	Archetype 106			Archetype 206			Archetype 306		
	BRBF Col.	BRBF Beam	BRB (in^2)	BRBF Col.	BRBF Beam	BRB (in^2)	BRBF Col.	BRBF Beam	BRB (in^2)
1	W14x426	W21x93	17	W14x370	W21x83	14.5	W14x311	W21x73	13
2	W14x342	W21x93	16	W14x283	W21x83	14	W14x233	W21x73	12
3	W14x257	W21x83	13.5	W14x233	W21x68	11.5	W14x193	W18x65	10
4	W14x193	W21x73	12	W14x176	W21x68	10.5	W14x145	W18x60	9
5	W14x145	W18x65	10.5	W14x145	W18x60	9	W14x132	W18x55	7.5
6	W14x132	W18x60	9.5	W14x132	W18x55	8.5	W14x82	W18x50	7
7	W14x68	W18x55	8	W14x53	W18x50	7	W14x48	W16x50	6
8	W14x48	W16x40	5.5	W14x48	W16x40	5	W14x38	W14x38	4

Table 3. IMFs Sectional Properties for Each Archetype Structure in N-S Direction

Story	Archetype 106			Archetype 206			Archetype 306		
	IMF Interior Col.	IMF Exterior Col.	IMF Beam (RBS)	IMF Interior Col.	IMF Exterior Col.	IMF Beam (RBS)	IMF Interior Col.	IMF Exterior Col.	IMF Beam (RBS)
1	W14x211	W14x211	W27x102	W14x233	W14x257	W27x129	W14x193	W14x193	W24x84
2	W14x159	W14x132	W27x102	W14x176	W14x145	W27x114	W14x132	W14x109	W24x84
3	W14x145	W14x132	W27x94	W14x159	W14x145	W27x114	W14x132	W14x109	W24x84
4	W14x132	W14x109	W24x94	W14x145	W14x109	W27x102	W14x109	W14x109	W21x83
5	W14x120	W14x109	W24x84	W14x132	W14x109	W24x94	W14x109	W14x82	W21x73
6	W14x109	W14x68	W21x73	W14x109	W14x74	W21x83	W14x82	W14x61	W21x62
7	W14x68	W14x61	W21x57	W14x82	W14x61	W21x62	W14x53	W14x43	W21x50
8	W14x48	W14x43	W16x36	W14x48	W14x43	W16x40	W14x43	W14x43	W14x38

The computer program PERFORM-3D was used to develop models of the archetype buildings. All archetype structures' gravity and seismic force-resisting elements were modeled in 3D space. The inclusion of gravity members was only to account for P-delta effects. Concentrated nonlinear springs or nonlinear hinges (lumped plasticity) were utilized to model BRBFs' and IMFs' beams and columns. The exact locations of the RBS nonlinear hinges were incorporated in the IMFs' beams model. For the BRBFs' beams, the nonlinear hinges were considered at 0.5 of a beam depth from columns' face (FEMA 350, 2000). BRBF and IMF's columns consist of elastic column elements with concentrated nonlinear hinges at their ends. The well-enhanced Ibarra-Krawinkler backbone curve model was used to develop seismic force-resisting system's columns and beams behavior (PEER/ATC-72-1, 2010). The backbone curve's prominent feature is the effective post-capping negative stiffness, which has crucial impact on collapse capacity assessment. Values pertaining to Ibarra-Krawinkler backbone curve were determined by empirical equations based on multivariate regression analysis that account for geometric and material parameters (PEER/ATC-72-1, 2010). Because the cyclic deterioration was not explicitly incorporated in the models, the initial (monotonic) backbone curves were modified to properly account for strength and stiffness cyclic deterioration. Fulfillment of preceding purpose was achieved by applying numerical values of modification factors recommended in section 2.2.5 of PEER/ATC-72-1 (2010). Rigid offsets were used at the ends of the lateral beams and columns to account for rigidity of the panel zones. Due to the possibility of nonlinear behavior at the lateral beam and column ends during an earthquake, panel zones were modeled at those locations. The panel zone model proposed by Krawinkler (1978) and presented in PEER/ATC-72-1 (2010) was used to explicitly simulate the panel zones shear distortion (PEER/ATC-72-1, 2010).

To model BRBs the built-in compound components of Perform-3D were employed. It is a compound bar type element that resists axial forces only and cannot withstand any bending or torsional forces. BRB compound component consists of a BRB basic component, an elastic bar basic component and a stiff end zone It was assumed as two bars in series: a linear (non-yielding) portion and a nonlinear (yielding) portion (Moehle et al., 2011). In this study, 45% of node-to-node length was considered non-yielding region, and 55% of node-to-node length was deemed to be yielding region. The former was composed of elastic bar, stiff end zone and the portion of the column, and the latter was only BRB basic nonlinear component. It was presumed that the elastic portion is very stiff compare to the yielding portion and it would not fail under large displacements.

An important feature of BRBs is their deformation hardening behavior, which includes two type of hardening behavior: kinematic hardening and isotropic hardening (Fahnestock et al., 2003). In this research, both types of hardening were explicitly taken into account. A report by Rutherford and Chekene (2011), on nine tested StarSeismic® Powercat™ BRBs shows “Maximum Deformation Only” option provides the best fit. 3% viscous damping and 0.2% Rayleigh damping were incorporated in the analysis. A small amount of Rayleigh damping is applied in order to ensure that higher mode displacements are damped.

NONLINEAR ANALYSIS AND RESULTS

Nonlinear static (pushover) analyses were conducted with a combination of 105% dead load and 25% of live load (expected gravity load), and static lateral forces (FEMA P695, 2009). The vertical distribution of the lateral story forces was proportional to the fundamental mode shapes of the archetype buildings (ASCE/SEI 41, 2006). To determine the elastic natural periods and mode shapes of the archetype structures, RSA were conducted using the Perform-3D program.

Figure 4 shows pushover curves of archetype structures in N-S and E-W directions. In order to quantify over-strength factor, Ω_0, (see Eq.(2)) the maximum base shear correspond to each archetype’s pushover curve was found out. The period-based ductility, μ_T, as shown in Eq.(5), is defined as the ratio of ultimate roof displacement, δ_u, to the effective yield roof drift displacement, $\delta_{y,eff}$ (FEMA P695, 2009):

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{5}$$

The ultimate roof displacement, δ_u, that is taken as the roof displacement at the point of 20% strength loss was gleaned from pushover curve for each structure (see Table 5). The effective yield roof displacement corresponding to each archetype was calculated based on equation (6-7) of FEMA P695 (2009). The final values for over-strength factor and period-based ductility were then calculated by averaging the values from each of the principal directions (see Table 5).

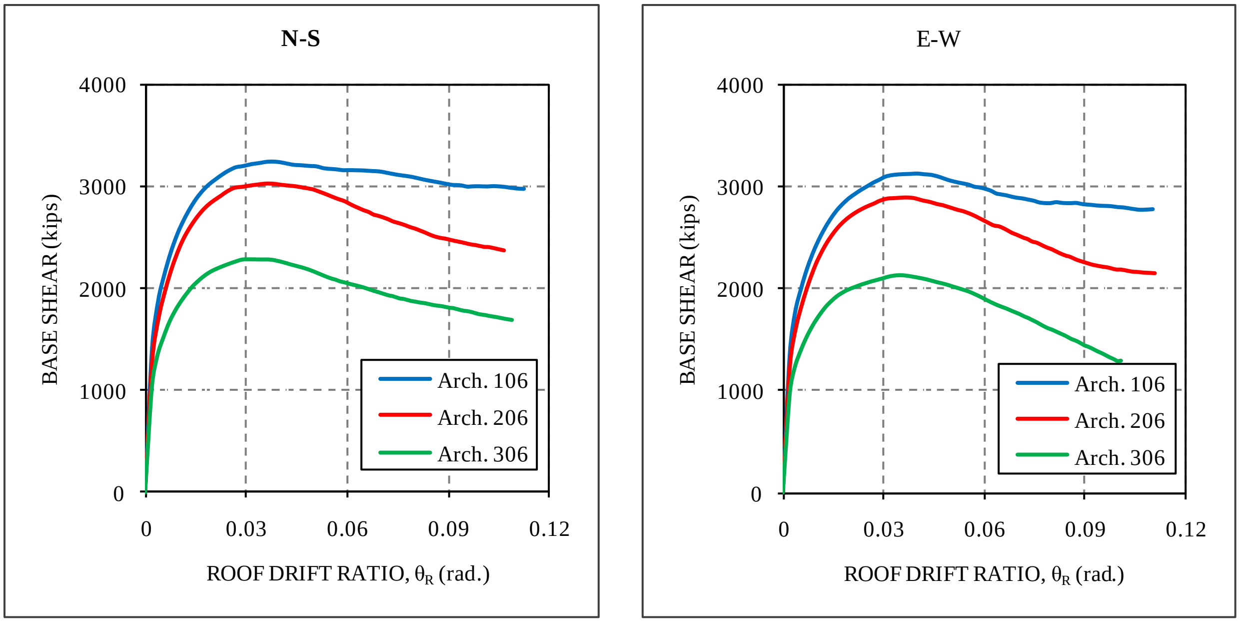

Figure 4. Pushover curve of archetype structures in N-S and E-W direction

Figure 5 shows plots of the tangent stiffness history versus roof drift for archetype buildings. These plots, which represent the slope of the pushover curve at each roof drift value, are more effective than pushover plots in identifying when yielding happens. A comparison between archetype 106 and 306 in E-W direction indicates that in both structures first yield occurs at the same roof displacement of approximately 3 in. However, the tangent stiffness at the first yield displacement are 359 and 503 kips/in. for archetype 306 and 106, respectively. Both structure's original stiffness were used up by the time the roofs displacement reach 18 in., but archetype 106 has 2% more tangent stiffness than archetype 306 at that point. Archetype 306 reaches the negative residual stiffness at a displacement of approximately 45 in., and archetype 106 attains that point at a displacement of approximately 54 in.

To compute the median collapse capacity of each archetype building the Incremental Dynamic Analysis (IDA) method was utilized. In this study, only the sidesway mechanism was considered for collapse assessment and it was defined as excessive lateral displacement (lateral dynamic instability) or where the IDA's curve reaches a flat line (Vamvatsikos, 2002). To plot an IDA curve, the maximum inter-story drift ratio, θ_{max}, and the spectral acceleration at the structure's first-mode period, S_{T1}, were deemed to be damage measure and ground motion intensity measure, respectively.

Ten pairs of natural ground motions (Table 4) were chosen, from the twenty-two pairs of horizontal ground motions offered by FEMA P695 (2009). The selected ground motions include far-field record sets from the sites located 10 km or more from fault rupture. They were downloaded from the Pacific Earthquake Engineering Research Center database (PEER, 2000). To avoid possible event-based bias in record sets, not more than one record was chosen from any earthquake event. Table 4 lists characteristic information of selected set of ground motion records including Magnitude (M), fault type, site condition and distance to the epicenter (R_E).

All individual records in each set were normalized by their corresponding peak ground velocities, to remove unwarranted variability between records due to intrinsic variations in event magnitude, distance to source, fault type and site type without eliminating record-to-record variability (FEMA P695, 2009). Afterwards, the normalized ground motions were collectively scaled upward to the point that makes 50% of ground motions to collapse the archetype structure. Figure 4 shows how the IDA approach was utilized to compute collapse capacity of each archetype structure. Due to application of three-dimensional analyses, the ten record pairs were applied twice to each archetype. Once each ground motion pair applied along each principal direction (N-S and E-W), and then again they were rotated 90 degrees and reapplied.

The spectral acceleration at collapse (S_{CT}) due to the 20 ground motions of the far-field set was computed. The median collapse level ($\hat{S}_{CT}$), as it is shown in Figure 6, was computed for each individual archetype building. The CMR , defined in Eq.(4) as the ratio of median 5%-damped spectral acceleration of the collapse level ground motions, $\hat{S}_{CT}$, to the 5%-damped spectral acceleration of the MCE ground motions, S_{MT}, are 9.44, 8.73 and 5.48 for archetype 106, 206 and 306, respectively.

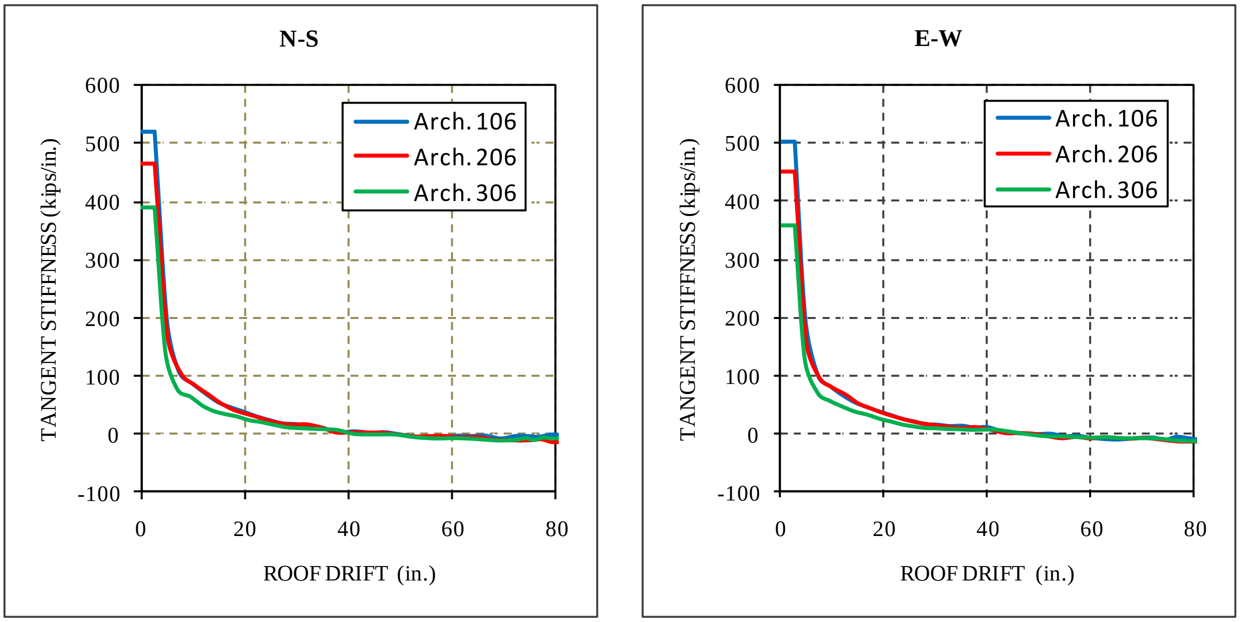

Figure 5. Tangent stiffness history of archetype structures for N-S and E-W direction

Table 4. Summary of PEER NGA Database Information

Ground motion No.	Earthquake Name	M	Year	Fault Type	Site Class	Component 1	Component 2	R_E (km)
1	Northrdige	6.7	1994	Thrust	D	NORTHR/LOS000	NORTHR/LOS270	26.5
2	Duzce, Turkey	7.1	1999	Strik-slip	D	DUZCE/BOL000	DUZCE/BOL090	41.3
3	Hector Mine	7.1	1999	Strik-slip	C	HECTOR/HEC000	HECTOR/HEC090	26.5
4	Kobe, Japan	6.9	1995	Strik-slip	C	KOBE/NIS000	KOBE/NIS090	8.7
5	Kocaeli, Turkey	7.5	1999	Strik-slip	D	KOCAELI/DZC180	KOCAELI/DZC270	98.2
6	Landers	7.3	1992	Strik-slip	D	LANDERS/YER270	LANDERS/YER360	86
7	Loma Prieta	6.9	1989	Strik-slip	D	LOMAP/CAP000	LOMAP/CAP090	9.8
8	Cape Mendocino	7.0	1992	Thrust	D	CAPEMEND/RIO270	CAPEMEND/RIO270	22.7
9	Chi-Chi, Taiwan	7.6	1999	Thrust	C	CHICHI/TCU045-E	CHICHI/TCU045-N	77.5
10	Friuli, Italy	6.5	1976	Thrust	C	FRIULI/A-TMZ000	FRIULI/A-TMZ270	20.2

DISCUSSION

For the proposed dual system, the quality of design requirements, test data and archetype models were rated Good, Good and Fair per FEMA P695 (2009) Table 3-1, 3-2 and 5-3, respectively. Total system collapse uncertainty was calculated based on preceding sentence statements and corresponding uncertainty values, and Record-to-Record (RTR) uncertainty. RTR uncertainty, $\beta_{RTR,}$ was accounted for variability in response of each archetype model in IDA to different ground motions. It was considered β_{RTR} = 0.4 for systems with $\mu_T \geq 3$. The total system collapse uncertainty for each archetype, β_{TOT}, as shown in Table 5, was quantified per Table 7-2a of FEMA P695 (2009). Acceptable Adjusted Collapse Margin Ratio, *ACMR*, are calculated based on total system collapse uncertainty, β_{TOT}, and established values of acceptable probabilities of collapse (FEMA P695, 2009). Relevant values to 20% probability of collapse for MCE ground motion, $ACMR_{20\%}$, was selected for each archetype structure from Table 7-3 of FEMA P695 (2009) (see Table 5). The Adjusted Collapse Margin Ratio, *ACMR*, for each model was computed as the multiple of the Spectral Shape Factor, *SSF*, (Table 7-1b for SDC D), *CMR* and 1.2 (effect of 3-D nonlinear dynamic analysis).

Table 5. Summary of Final Collapse Margins and Comparison to Acceptance Criteria

Arch. ID	Design Configuration		Computed Over-strength and Collapse Margin Parameters								Acceptance Check
	No. of Stories	R	δ_U	$\delta_{y,eff}$	β_{TOT}	μ_T	*SSF*	Static Ω	*CMR*	*ACMR*	$ACMR_{20\%}$
106	8	6.25	248.5	5.23	0.6	47.0	1.45	1.95	9.44	16.4	1.66
206	8	7	120.2	5.36	0.6	22.1	1.45	2.20	8.73	15.1	1.66
306	8	10	105.4	5.16	0.6	20.5	1.45	2.05	5.48	9.5	1.66

CONCLUSIONS

This paper presents BRBF/IMF dual system assessment to develop global seismic performance factors. One of the major objectives of this research is to quantify the seismic performance factors (R, Ω_o and C_d) for dual systems, which are not described by the available codes or listed in any standards. For this study we took BRBF/IMF combination as our dual system, which is not listed in ASCE. We ascertained the values for the seismic performance factors for the proposed dual system. The performance assessment is based on nonlinear static (pushover) analysis and IDA approach. The collapse capacities of different archetype buildings were gleaned from IDA results. The analytical models exploited in this study are 3-dimensional models of 8-story archetype buildings designed based on three different R values (6.25, 7 and 10). The major observations of this study are summarized as follows:

- The *CMR* computed from IDA results is not sensitive to fractional differences in response modification coefficient, R, and leads to nearly equal margins.
- All three archetype structures being evaluated fulfill the requirement of collapse performance, but the one with the lowest *CMR*, archetype 306 with R = 10, would be the best option for further assessment.
- Comparison of *CMR*s obtained from three different R factors, could be a crude indication that R factors pertaining to stiffer system (BRBF) has more impact on overall dual system behavior and great reliance would be on this system. Although proposed system is not an explicit model representing a horizontal combination of two different seismic force-resisting systems, it indicates that ASCE suggestion to utilize the least value of R for horizontal combination of different seismic force-resisting systems could be deficient of realistic approach.
- The values obtained for over-strength factor have fractional differences, therefore a value between 2.2 and 2.5 could be deemed to be adequate.

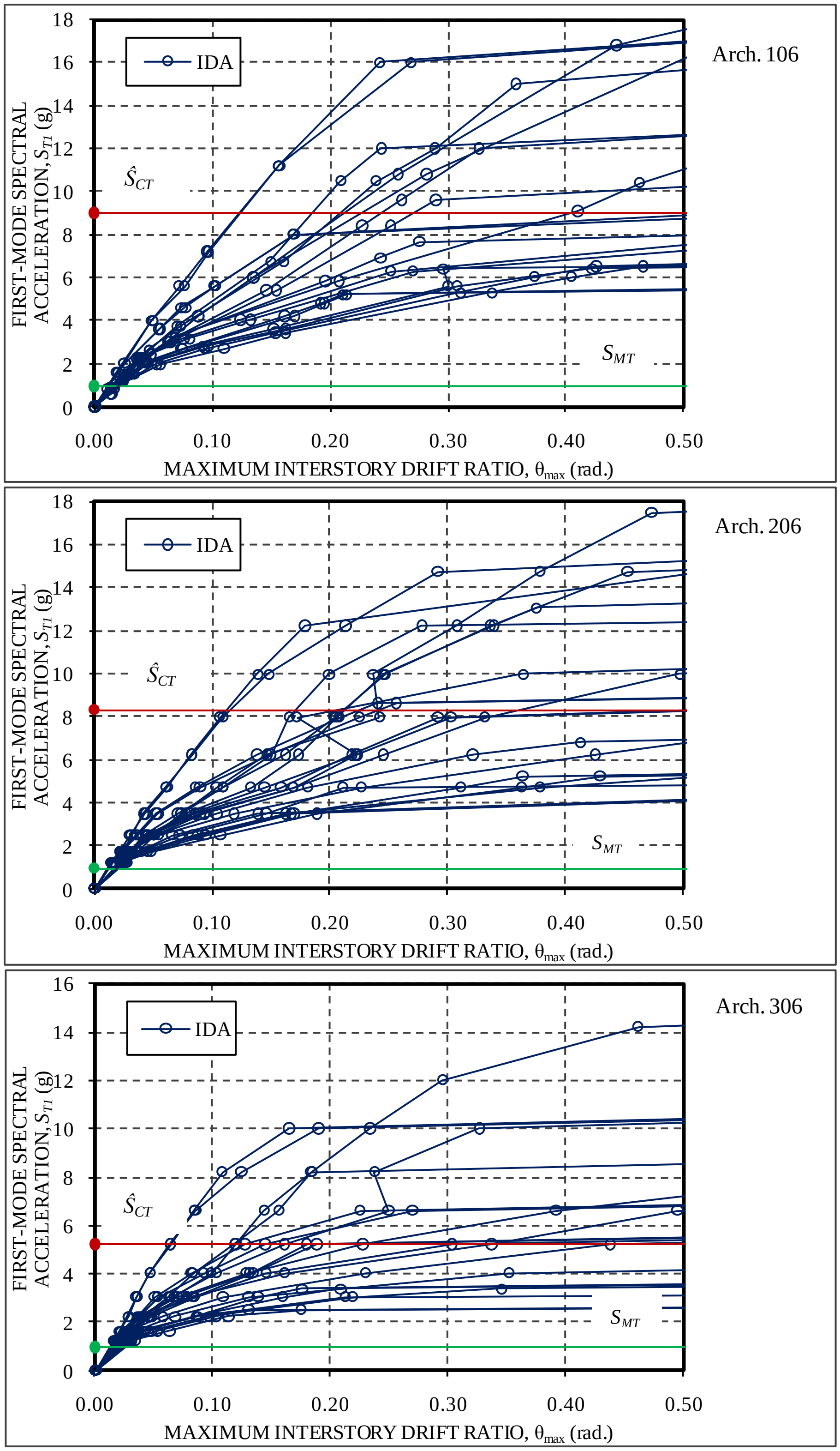

Figure 6. IDA to collapse, showing the MCE ground motion intensity (S_{MT}) and median collapse capacity ($\hat{S}_{CT}$) for each archetype structure.

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