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An Investigation on Bracing Configuration Effects on Behavior of Concentrically Braced Steel Frames

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Abstract: Concentric bracing systems, because of their effectiveness in reducing seismic response, have been in practice for many years. Depending on concept, seismic design codes provide various response modification factors (R) for different types of lateral load bearing systems but configuration of these systems are often ignored in the proposed values. This study aims at considering the effect of different bracing configuration on the relative values of R. Six five-story three-bay braced frames with three different patterns and one-third scale have been made and experimentally evaluated. On the grounds of experimental results, 27 models were created and analyzed using Finite Element Analysis. The main variables of this study were the location of bracing, bay width and height of story. Results show that configuration of bracing system can affect the R values. Based on analyses and experiments, best results obtained when adjacent bays are braced. Furthermore a relation is presented to predict relative R value depending on bay to width ratio in each category.

Key words: Response reduction factor · Concentrically braced frame · Bracing configuration · Elastic strength demand

INTRODUCTION

The 'Response Modification Factor' (R) which is widely used in most of the seismic design codes all over world, is basically trying to capture the effects of ductility of the system to withstand earthquake induced load. The ultimate capacity of each structural system, such as a moment frame or a braced frame, depends on its structural configuration and specifications, e.g. type or size of bracing elements in case of braced frames [1]. Consequently, the codes give various values for R depending on the lateral load bearing system of the building. For example, some codes [2] suggest a value of 5 for the case of Ordinary Concentrically Braced Frame (OCBF) and a value of 6 for the case of Special Concentrically Braced Frame (SCBF). However, the R values in codes do not depend on the number of braced bays and their relative location, or even the overall pattern and form of bracing while the number of braced bays in a frame is important considering their effects on redundancy. Several analytical and experimental studies have been performed on braced frames since early 70s, of which some experimental works are briefly reviewed here. Shaishmelashvili and Edisherashvili [3] have done an experimental study on dynamic characteristics of large-scale models of multi-story steel frame buildings with different vertical bracings. They have tested some large-scale models of a 9-story building with 12 different bracing schemes in free and forced (resonance) vibration states. In the performed tests only linear behavior of the structure has been considered.

Suzuki *et al*. [4] performed an experimental study on the elasto-plastic behavior of tensile braced frames to obtain the restoring force characteristics of low-rise steel structures. Alternating horizontal force was applied at the second floor under a constant vertical load, paying attention to the behavior of two columns subjected to varying axial forces. Test specimens consisted of one-story, one-bay frames with wide flange sections and braces of round steel bars. The relation between shear

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force and displacement in each column was investigated by numerical analysis. According to the results, the elasto-plastic behavior of the two columns is obviously different; one is subjected to additional tensile force and the other to additional compressive force. From these results, it was found that the restoring force characteristics of braced frames were stable but that the hysteresis loops in each column became unstable because of the additional compressive force.

Inoue and Murakami [5] performed a study on the plastic design of braced multi-story steel frames by conducting some tests on the elasto-plastic behavior of 3-story 3-bay braced and un-braced steel frames under monotonic or alternating horizontal forces. Four specimens were tested, two braced frames and two un-braced frames and both were designed against the same factored horizontal forces. Test frames were subjected to horizontal forces proportional to design forces at each floor level. The force-deflection curves did not differ noticeably from test results reported by many investigators. In the case of the braced frames, the bracing members of the lowest story buckled and yielded at the outset, so that the relative story displacement of the lowest story increased before buckling and yielding developed in all other bracing members. However, the relative story displacement of each story increased uniformly. This result suggests that the bracing members should be designed so as to buckle or yield simultaneously against seismic force and that experimental force-deflection curves are well predicted by the generalized hardening hinge method.

Wakabayashi *et al*. [6] did some experimental studies on the elasto-plastic behavior of braced frames under repeated horizontal loading. In a part of those studies, experiments of one story-one bay braced frames were conducted to investigate the hysteretic behavior of this kind of steel frames whose braces were made of built-up H-shapes and whose columns and beams were made of rolled H-shapes. Hysteretic behavior and transition and change of load-carrying capacity of each component member of a frame, i.e., braces, columns and beams under repeated horizontal load, were examined individually, also the hysteretic behavior of a braced frame as a whole was investigated. Interaction behavior between the braces built into a frame and the components of the surrounding frame were also studied. It was found that the effective slenderness ratio for buckling of the braces built into a frame could be estimated by the slope-deflection method, taking the rotational rigidity of the members of the surrounding frame into account and that the effective slenderness ratio for the estimation of post-buckling and hysteretic behavior could be approximated by the assumption that the braces would be rigidly fixed at the ends. Those tests also showed that as the columns are subjected to repeated large axial load due to the deformation of the brace, the load-carrying capacity of the column decreases substantially and when the axial load is large, the behavior of the column is largely affected by the brace and that the load-carrying capacity and the ductility of the brace are reduced and exhausted when cracks are initiated as well.

Black and Popov [7, 8] extended the scope of these work to determine whether the effective length (kL) approach used for calculating the buckling capacity of elastic braces could be applied to braces cyclically loaded in to the inelastic ranges.

The research of Khatib and Mahin [9] demonstrated the sensitivity of the nonlinear response of a CBF to brace slenderness and stiffness; relatively minor changes in component stiffness can have a substantial impact on hysteretic response. given this sensitivity and obvious need to protect column that resist gravity load.

Red wood and Channagiri [10] studied the effect of nonlinear behavior of frames on ultimate forces which is transferred to column and braces. they stated that calculation of column loads by direct addition of the braces that could be delivered by the braces above the story under consideration is likely conservative for the medium to high rise CBF because maximum forces are generally not realized in all bracing simultaneously.

Shademan [11] analytically studied the effects of different X-braced steel frames and their effects on the structural performance using pushover analysis. It was concluded different bracing scheme can have significant effect on overall structural performance.

Lee and Bruneau [12, 13] studied the energy dissipation of compression members in concentrically braced frames. Design and detailing requirements of seismic provisions for CBFs were specified based on the premise that bracing members with low KL/r and b/t will have superior seismic performance, but they claim that relatively few tests have investigated the cyclic behavior of CBFs and hence, it is legitimate to question whether the compression member of a CBF plays as significant role as what has been typically assumed implicitly by the design provisions. In that study, the existing experimental data were reviewed to quantify the extent of hysteretic energy absorbed by bracing members in compression in past tests and the extent of degradation of the compression force upon repeated cycling loading. The focus of that

study was mostly on quantifying energy dissipation in compression and its effectiveness on seismic performance. Based on the experimental data reviewed from previous tests, they found that the normalized energy dissipation of braces having moderate KL/r (80-120) do not have significantly more normalized energy dissipation in compression than those having a slenderness in excess of 120 and that the normalized degradation of the compression force envelope depends on KL/r and is particularly severe for W-shaped braces.

Fahnestock *et al*. [14] performed an experimental study on a large-scale buckling-restrained braced frame using the pseudo-dynamic testing method. As part of an integrated analytical and experimental research program on buckling-restrained braced frames (BRBFs), a large-scale BRBF was subjected to multiple earthquake simulations using the explicit Newmark algorithm. A hybrid testing approach was implemented to account for the P-Delta effects associated with the gravity load carried by the prototype building's gravity frames. The test frame sustained significant drift demands with almost no damage. Story drifts of nearly 5% and buckling-restrained brace maximum ductility capacity of over 25 were observed in the maximum considered earthquake simulation. No stiffness or strength degradation was observed. Although residual drifts were large, the testing program demonstrated that the BRBF system can withstand significant seismic input and retain full lateral load-carrying capacity. Non-conventional brace-gusset and beam-column connections demonstrated excellent performance under very large drift demands.

Broderick [1] studied the response of concentrically braced sub-frames. Seismic codes of practice typically adopt the general philosophy of capacity design. In the case of concentrically-braced frames, capacity design normally implies the use of diagonal bracing members as the main dissipative elements to provide adequate overstrength factors for other frame components and to ensure compliance with the selected failure mode. Dissipative designs, such as Eurocode 8 rely on the capability of parts of the structure (dissipative zones) to resist earthquake actions beyond their elastic range. In the case of concentrically-braced frames, Eurocode 8 assumes that only tension diagonals participate in the lateral resistance of earthquake-induced loading and hence, in these structures dissipative zones are mainly located in the tension diagonals. This approach is different from that followed in other seismic codes. In their research they concluded that for harmonic shake table motion, time-history analysis produced a good, if slightly underestimated, prediction of the variation in lateral frame displacement throughout the test. The results also illustrated that the design approach adopted in European provisions whereby the lateral frame resistance is only based on the tension diagonals could provide a reasonable representation of the behavior within practical ranges of brace slenderness. However, in terms of satisfying the objectives of capacity design, it is important that additional checks are considered to account for possible adverse effects caused by the contribution of the braces in compression.

Jung-Han Yoo *et al*. [15] studied Analytical Performance Simulation of Special Concentrically Braced Frames. Detailed comparisons between experimental observations and computed results show that the analyses provided good correlation to actual behavior. In their research the fracture procedure was used to predict the expected deformation capacity of braced frames with other gusset-plate connection designs. The results indicated that the proposed elliptical clearance model provided good seismic performance and at a specific value could provide a good balance between the potential for weld fracture sustained for specimens with smaller offset distance and brace fracture sustained for specimens with larger offset distance. Thick gusset plates increase the potential for brace fracture, but in most cases reduce the potential for weld cracking. Thick gusset plates combined with heavier beam sections clearly increased the potential for brace fracture. Theoretical predictions used to estimate gusset plate weld cracking and brace fracture were verified by experimental results.

In this study it has been tried to also include the configuration of the braced bays as a parameter in evaluation of the structure overall performance. The role of configuration can be important in force distribution but is not explicitly addressed in the seismic design codes.

Considered Frames and Test Procedure: The experimental research provides a basis for the analytical research and calculation of R-values. It should be noted that in this research the computed R-values only account for tension as a part of total behavior of this kind of bracing but real behavior of concentrically braced frames is a function of tension, compression, buckling and post buckling behavior of bracing elements so here only comparative aspects of different configurations were of interest. Six five-story three-bay braced frames with three different bracing patterns (two identical samples of each pattern for more reliable results) and one-third scale of practical structures due to laboratory space limitation

Fig. 1: Second bracing scheme

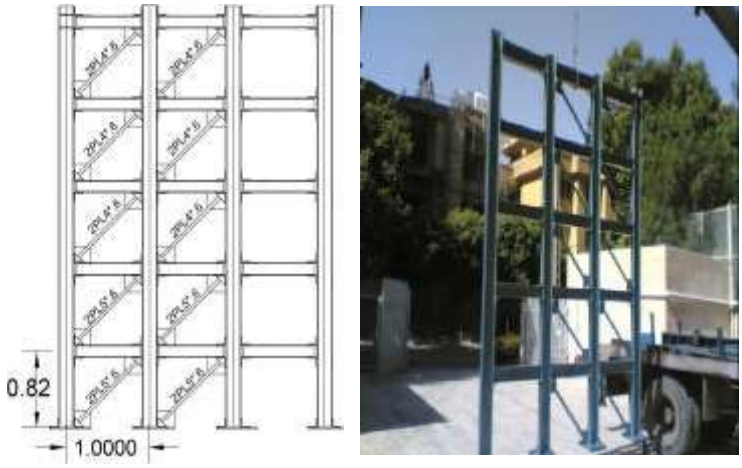


Fig. 2: Third bracing scheme

were designed and experimentally evaluated by authors. Beam to column connection was considered as pin type using welded seat angle connection type. Of course, the required quality control was applied during construction phase. In the first pattern all required bracing is concentrated in the middle bay, in the second pattern, first and third bays are braced and in the third pattern two adjacent bays are braced. Figures 1 and 2 show the schematic drawings of the second and third pattern, as it is evident from the drawings bay width is 1 meter and height equals 0.82 meter.

Test Frames were designed in tension according to UBC97-LRFD [16] approach. For columns IPE-180 sections and for beams except for highest level IPE-140 sections were used. Columns and beams of the scaled samples were chosen stronger than those in conventional steel CBFs, to make sure that all bracing element would yield before any beam or column. Oversized sections of beams and columns could result in some additional moment resistance in beam-to-column connections if they were constructed using the conventional construction method. Therefore, a triangular part of each connection plate was cut off to let the frames connections act as a hinged connection by bending the angles used at the top and bottom of beam profiles. Lab facilities consist of a strong floor based on strip footings which are located at 1500 mm center to center. On each row of the footing a continuous base plate is provided for locating support elements. Figure 3 shows schematic drawing of test configuration and placement of support elements, strain gauges, actuator and transition beam.

At the highest level due to the placement of actuator IPE-180 was used for beam sections, rectangular cross sections were used for bracings. It should be noted that tensile tests for determination of steel properties were also performed and results were provided in section 5 of this article. Displacement controlled loading was conducted using a 25Ton actuator at 0.1 $mm/\sec$ speed at the highest level of frames and a monotonic one-way loading was used.

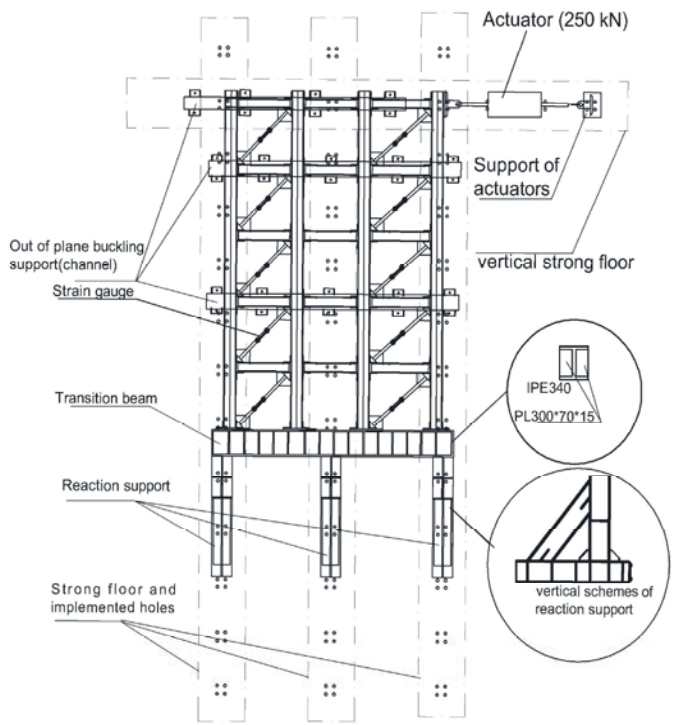


Fig. 3: Schematic drawing of test set up

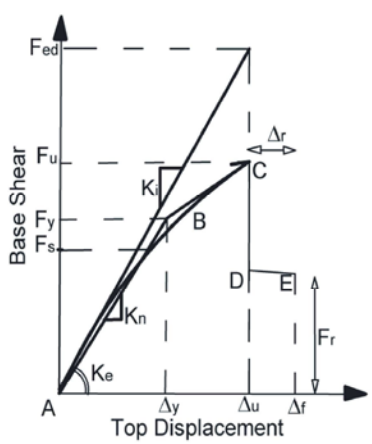


Fig. 4: Definition of parameters

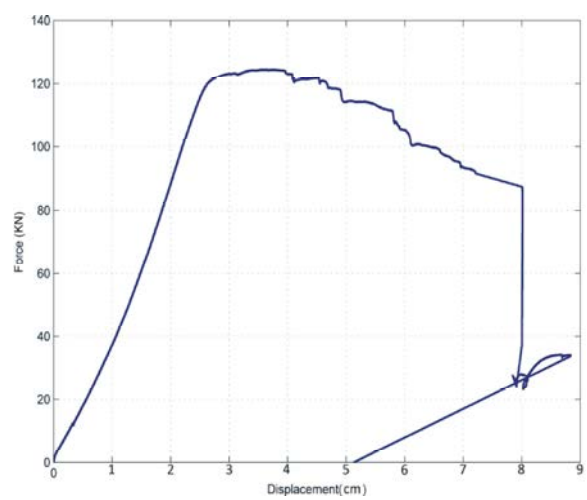


Fig. 5: Base shear vs. top displacement for test 2 of first pattern

Test Results: In this section summary of experimental results will be presented. For the purpose of discussion of test results, they will be divided into three groups and in each group, the results of each pattern will be presented then, the obtained results and computed R-Values for each group will be presented in the accompanying Table. Parameters are displayed in figure 4: Initial stiffness (K_i) (KN/cm), Yield displacement (Δ_y) (cm), Ultimate displacement (Δ_u) (cm), First significant yield (F_s) (KN), ultimate strength(F_u) (KN), elastic strength demand force (F_{ed}) (KN), response modification factor (R), residual strength (F_r) (KN). Figures 5 to 7 show the capacity curves of tested frames. Computed values are displayed in table 1. Regarding computed R-values, the third pattern of bracing scheme (pattern 3) provides better response modification factors. The minimum and maximum R-values computed in the third pattern are 5.31 and 5.56 whereas these values for the first pattern are 3.25 and 3.22 for the second pattern these values were computed as

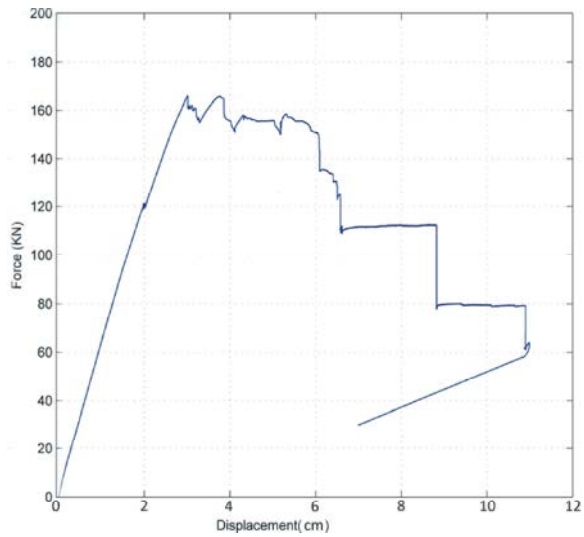


Fig. 6: Base shear vs. top displacement for test 2 of second pattern

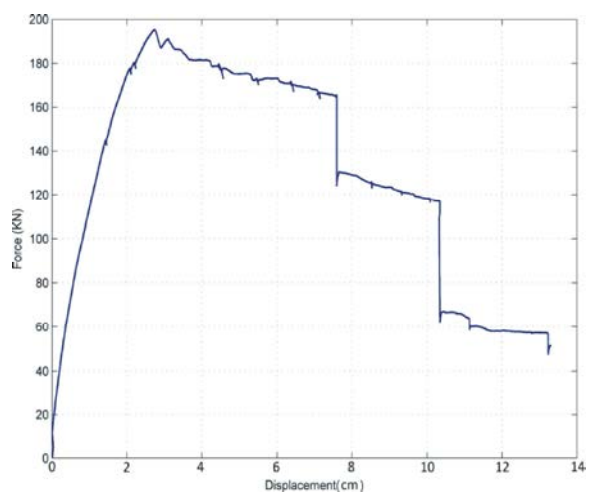


Fig. 7: Base shear vs. top displacement for test 2 of third pattern

3.47 and 3.32. Better performance of the third pattern can be due to truss action of two adjacent bays which can provide more nonlinear capacity. Furthermore; there are no considerable residual displacement and strength capacity in the first pattern where all the required bracing is concentrated at one bay.

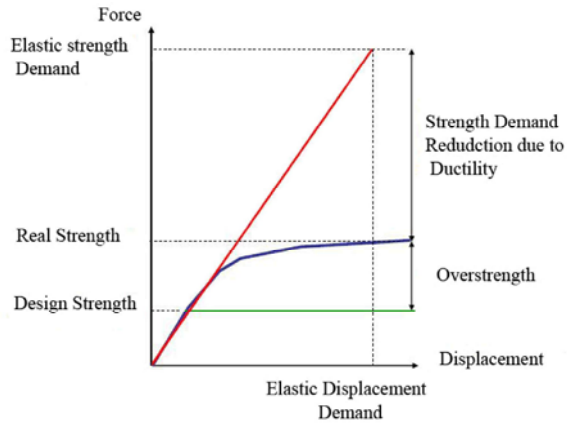


Fig. 8: Parameters used in R evaluation

Modeling Procedure

General Concept: As it was pointed out in the previous sections many researchers studied the behavior of braced frames from different points of view. But the effects of bracing configurations were not studies beyond elastic response as presented here. Computation of R-value can be carried out using the following method but there are some essential values which should be derived first. These values include: yield and ultimate displacements shown respectively by D_y and D_u; also yield force and elastic strength demand force which would be presented by F_y and F_{ed} notations respectively. R estimation can be performed by defining two factors: strength demand reduction factor due to ductility, R_d and overstrength factor, Ω. Figure 8 shows parameter derived for evaluation of R-values [17, 18].

$$R_d = \frac{Elastic\ Strength\ Demand}{Real\ Strength} \tag{1}$$

$$\Omega = \frac{Real\ Strength}{Design\ Strength} \tag{2}$$

and Then R can be computed as

$$R = R_d \Omega \tag{3}$$

Table 1: Computation of R factor and required parameters

Pattern	Test No.	K_i	Δ_y	Δ_u	Δ_r	F_s	F_u	F_{ed}	F_r	R
First	1	39.896	2.41	7.85	0	96.07	117	313.18	0	3.25
	2	46.194	2.48	8.01	0	114.7	124.8	369.55	0	3.22
Second	1	53.455	1.75	6.1	3.29	93.74	204.04	326.1	68.85	3.47
	2	66.338	1.82	6.06	4.84	120.9	166.2	402.	79.2	3.32
Third	1	103.487	1.45	7.79	2.45	145.4	192.3	806.16	58.2	5.56
	2	101.614	1.425	7.57	6.24	144.8	195.47	769.2	57.1	5.31

Units: KN, cm

Finite Element Modeling: ANSYS 5.4 [19] multi-purpose finite element modeling program was used to perform the numerical modeling of braced frames. FE models were created using the ANSYS parametric design language. The geometrical properties of the frame elements models were treated as parameters were bay-width and story-height. Numerical analyses of frame were performed including following considerations: Using eight-node first order SOLID45 elements and a mesh refinement study was conducted to determine the mesh size required to ensure convergence and accuracy of the FE solution and simultaneously minimizing the execution time.

ANSYS [19] can model contact problem using contact pair element: CONTA174 and TARGE 170 that work together so no penetration would occur during the loading process. The interaction between seat angle, column and beam flanges were modeled using mentioned contact element [20-22].

Boundary Condition and Applied Load: To satisfy boundary conditions of test model, in the analytical models some restraints were also applied. The most important restraints considered were: column, connections and beams out of plane buckling prevention which was provided by sandwich beams at story levels and a relatively rigid beam to act as the foundation of frames.

Material Properties: The material properties of these models had kinematic behavior with strain hardening in nonlinear phase to predict the reality of material precisely. The stress-strain relation for all connection components were represented using a tri-linear constitutive model. An isotropic hardening rule with a Von-Mises yielding criterion was applied to trace plastic deformations of the elements. The mechanical properties which were implemented in the analytical model were derived from coupon tests. The mechanical properties of both bracing and gusset plates were nearly similar to ST37 steel grade and for I-sections they were similar to ST52 steel grade. Modulus of elasticity is 1.88e6 kg/cm^2 for plates and 2.0e6 kg/cm^2 for chords. Average yield stress and ultimate stress for chords are 3210 kg/cm^2 and 4670 kg/cm^2 these values for plates are 2233 kg/cm^2 and 3274 kg/cm^2 respectively. The yield stress and ultimate stress of weld are assumed to be based on nominal properties of E7011. Figure 9 presents coupon test results.

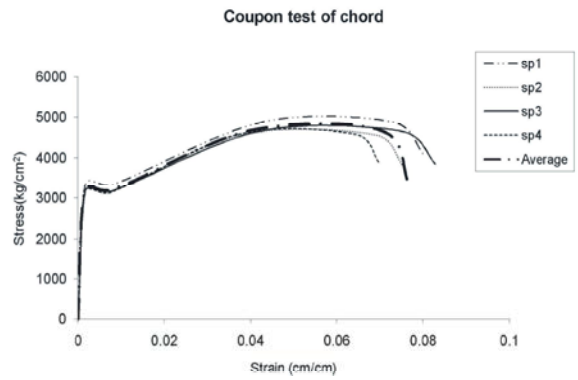


Fig. 9: Coupon test results chords

Fig. 10: Von-Mises stress contour for the first pattern bracing scheme

Verification of the Finite Element Model: To evaluate the accuracy of finite element modeling approach and to ensure the appropriate prediction of response by using FE, three finite element models were created and analyzed according to the actual test data which was mentioned earlier and results were compared with the test results. Figure 10 shows Von-Mises stress for the second pattern bracing configuration.

From Figure 11 it can be seen that for the first pattern bracing scheme, there are some differences between the test data and the finite element modeling. These differences are most likely rooted in test specimen defects like geometrical measurement or slippage in lateral bracing or supporting systems of test set up and also uncertainties in modeling and analytical tools available. Totally it can be seen that the analytical model has good agreement with test models and the results could be reliable for evaluating of frame behavior. The differences between the test data and the numerical models grow in nonlinear portion of curve.

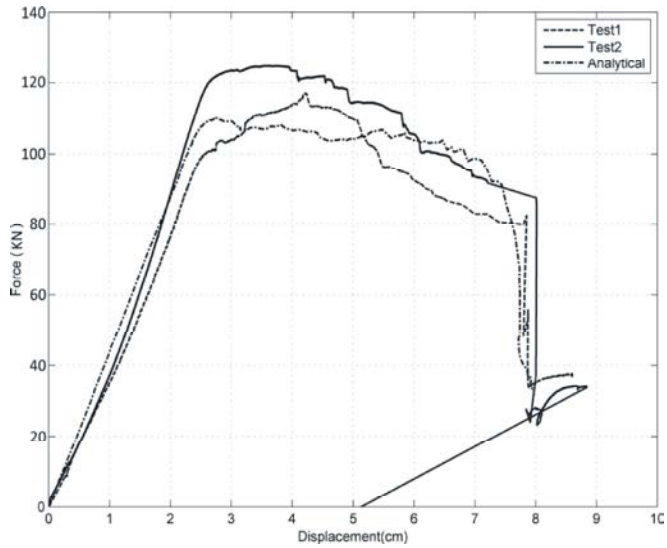
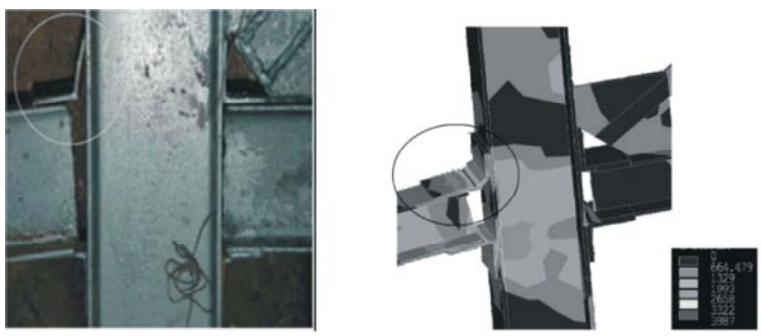


Fig. 11: Comparison between analytical and two experimental capacity curves for the first pattern bracing scheme



(a) (b)

Fig. 12: Comparison of connection behavior in test and FE model (a) test, (b) analytical deformation

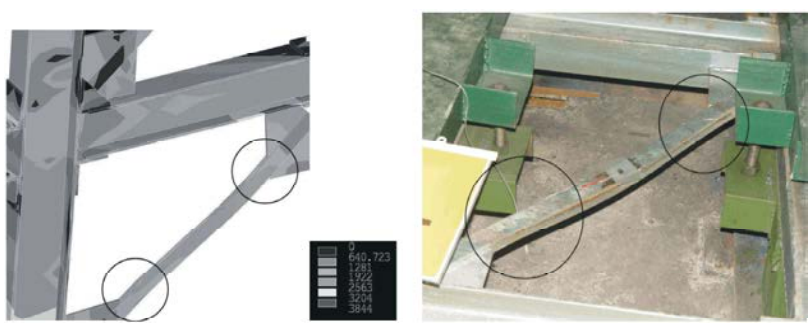


Fig. 13: Brace yielding in test and FE model

In the next step capacity curves for analytical models were extracted and compared to the test results. Figures 12 and 13 show samples of comparisons of FE models and test results for the first bracing pattern. This also has been done for the rest of the models and their respective test results. More details can be found in [11].

Figures 14(a to e) show comparison of finite element results and test data for strain in bracing elements vs. top displacement along with yield strain level.

Analytical Models: Three different concentric bracing schemes, as introduced earlier, were considered for calculating and comparison of R-values (and implicitly the behavior of the suggested configuration). Naming convention of analytical models can be found in tables 2 to 4. In these tables, 'b' refers to the width of each bay and 'h' shows story height of the model. Like the original test and verification models, design of these analytical models was in compliance with UBC-LRFD97 with seat

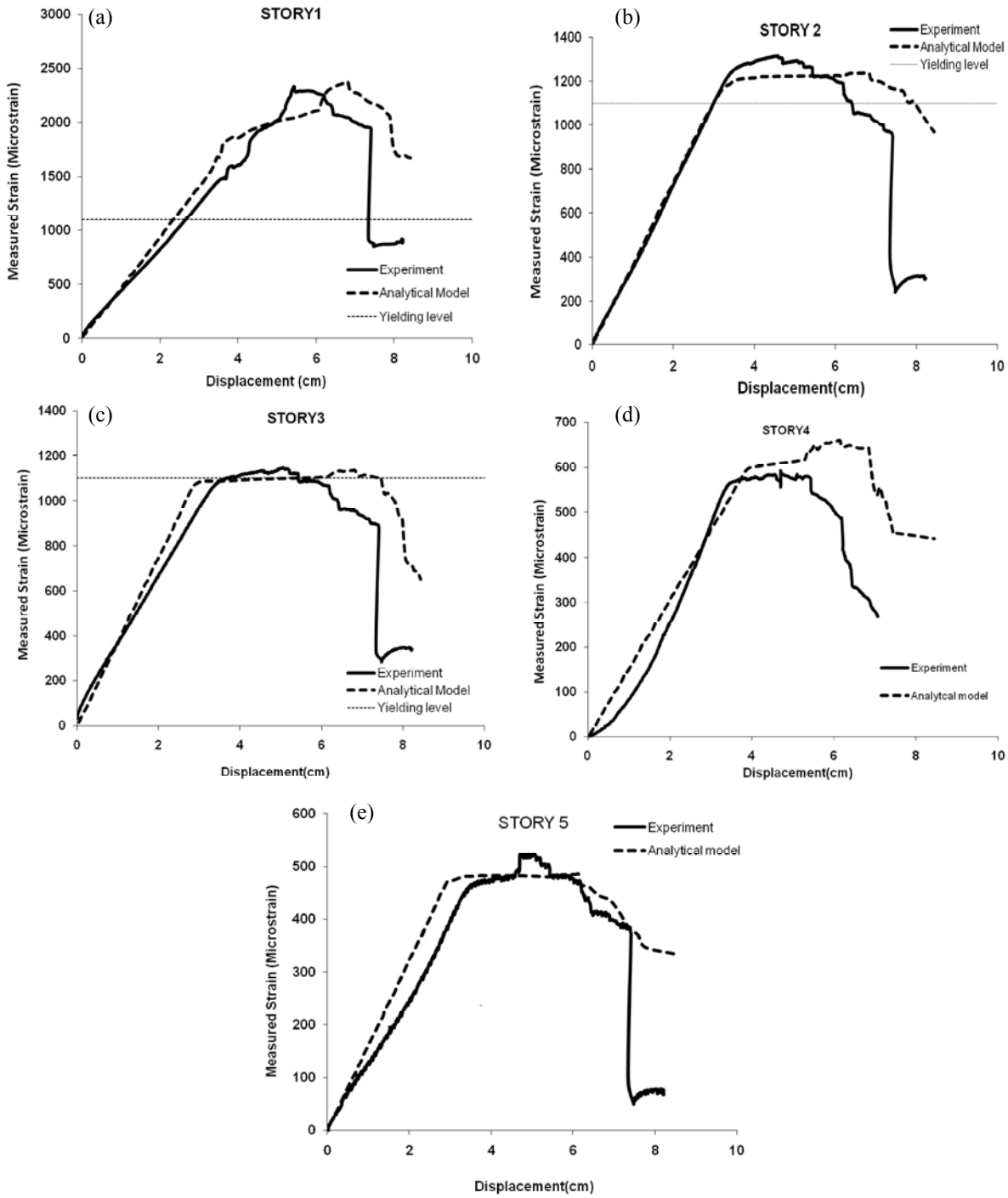


Fig. 14: (a to e) strain vs. top displacement for different stories

angle pin connections and one third scale. IPE180 and IPE140 were used for columns and beam sections and 30x30 cm base plates with 2cm thickness were used at the bottom of columns. Plate sections were used as bracing sections. All frames were designed for a 4200 kg lateral load $(25000/6 \approx 4200)$.

The material properties were the same as test models and verification models. Monotonic one-way loading was applied at the highest story in the test and stiffeners were used. To avoid stress concentration on the application point in the analytical models, monotonic loading was distributed

Table 2: First Pattern bracing naming convention

First Pattern	b= 80 cm	b= 100 cm	b= 120 cm
h= 68.5cm	A1	A2	A3
h=78.5 cm	A4	A5	A6
h=88.5 cm	A7	A8	A9

Table 3: Second Pattern bracing naming convention

Second Pattern	b= 80 cm	b= 100 cm	b= 120 cm
h= 68.5cm	B1	B2	B3
h=78.5 cm	B4	B5	B6
h=88.5 cm	B7	B8	B9

Table 4: Third Pattern bracing naming convention

Third Pattern	b= 80 cm	b= 100 cm	b= 120 cm
h= 68.5cm	C1	C2	C3
h=78.5 cm	C4	C5	C6
h=88.5 cm	C7	C8	C9

among some nodes. Out of plane buckling was restricted by restraining some internal nodes in the analytical models.

Numerical Results: Based on performed analyses, results can be presented as follows for each pattern considered. Figure 15 shows obtained capacity curves for the first pattern bracing scheme. It can be seen for different heights of stories (a, b or c in Figures 15 to 17) as the bay-width increases, D_y, D_u, F_y and F_u would also increase. The required parameters extracted from capacity curves are summarized in Tables 5 to 7.

Regression Analysis: According to the mentioned procedure, the R-value for each model was computed and regression analysis was performed to evaluate their relation.

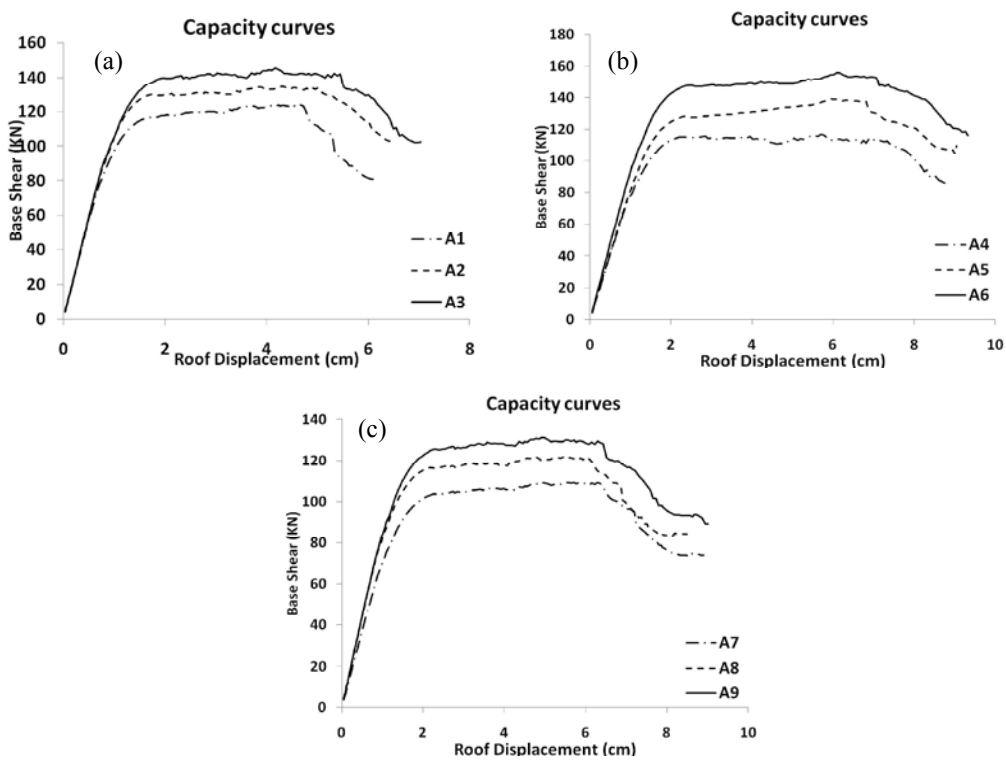


Fig. 15: Capacity curves for the first pattern bracing scheme with different story height and bay width

Looking at the results shown in Figure 18, despite geometrical variations, the third pattern provides better results in comparison with other patterns studied here. There is not a definite model for the case of first or second pattern, for some cases second pattern is better than first pattern and in most of the cases the difference is not too large.

Figure 19 shows computed R values and the regression line for the results of the first pattern. It seems that for this configuration, values of R have no meaningful dependency on the geometrical properties of the frame since the slope of the regression line is relatively small and for this kind of configuration computed R value are relatively low.

Table 5: Computed parameters and R values for the First pattern

Obtained Parameters	Analytical Model A1	A2	A3	A4	A5	A6	A7	A8	A9
F_{ed}	406.7	406.5	432.7	356.7	335.1	369.9	327.1	388.4	379.3
Δ_F	6.1	6.5	7.0	7.6	7.6	8.3	8.9	8.4	9.0
F_u	122.2	118.9	122.2	108.4	114.3	120.1	92.6	107.6	115.9
Δ_u	4.6	5.0	5.2	5.6	5.6	5.9	6.3	6.1	6.4
F_s	98.9	103.7	102.3	88.9	99.5	102.8	76.9	93.2	101.3
R_d	3.328	3.419	3.541	3.290	2.933	3.081	3.533	3.610	3.273
Ω	1.236	1.146	1.195	1.220	1.149	1.168	1.204	1.154	1.144
R	4.114	3.919	4.230	4.013	3.369	3.599	4.254	4.167	3.744

Units KN, cm

Table 6: Computed parameters and R values for the Second pattern

Obtained Parameters	Analytical Model B1	B2	B3	B4	B5	B6	B7	B8	B9
F_{ed}	483.95	424.61	428.84	387.05	393.37	458.71	315.60	360.03	520.66
Δ_F	8.7	12.7	13.6	11.0	12.5	13.4	8.8	8.4	8.9
F_u	144.90	145.20	145.30	118.10	138.40	157.00	91.00	107.00	138.00
Δ_u	6.4	6.7	7.1	6.9	6.7	7.0	6.4	6.1	6.3
F_s	135.82	125.25	119.37	97.67	114.02	137.86	74.27	92.18	144.45
R_d	3.34	2.92	2.95	3.28	2.84	2.92	3.47	3.36	3.77
Ω	1.07	1.16	1.22	1.21	1.21	1.14	1.23	1.16	0.96
R	3.56	3.39	3.59	3.96	3.45	3.33	4.25	3.91	3.60

Units KN, cm

Table 7: Computed parameters and R values for the Third pattern

Obtained Parameters	Analytical Model C1	C2	C3	C4	C5	C6	C7	C8	C9
F_{ed}	414.97	351.39	396.60	187.08	349.54	406.79	209.36	303.70	410.80
Δ_F	9.1	12.5	13.7	12.1	13.4	15.5	12.0	11.9	12.3
F_u	101.70	115.00	131.30	82.00	118.00	141.00	82.90	99.80	117.30
Δ_u	5.6	6.6	7.2	7.4	7.7	8.1	8.6	8.5	8.7
F_s	59.89	70.87	86.78	48.24	72.90	94.09	42.58	54.89	70.59
R_d	4.08	3.06	3.02	2.28	2.96	2.89	2.53	3.02	3.49
Ω	1.52	1.76	1.58	2.51	1.83	1.65	2.81	2.16	1.75
R	6.20	5.37	4.78	5.72	5.43	4.76	7.10	6.51	6.10

Units KN, cm

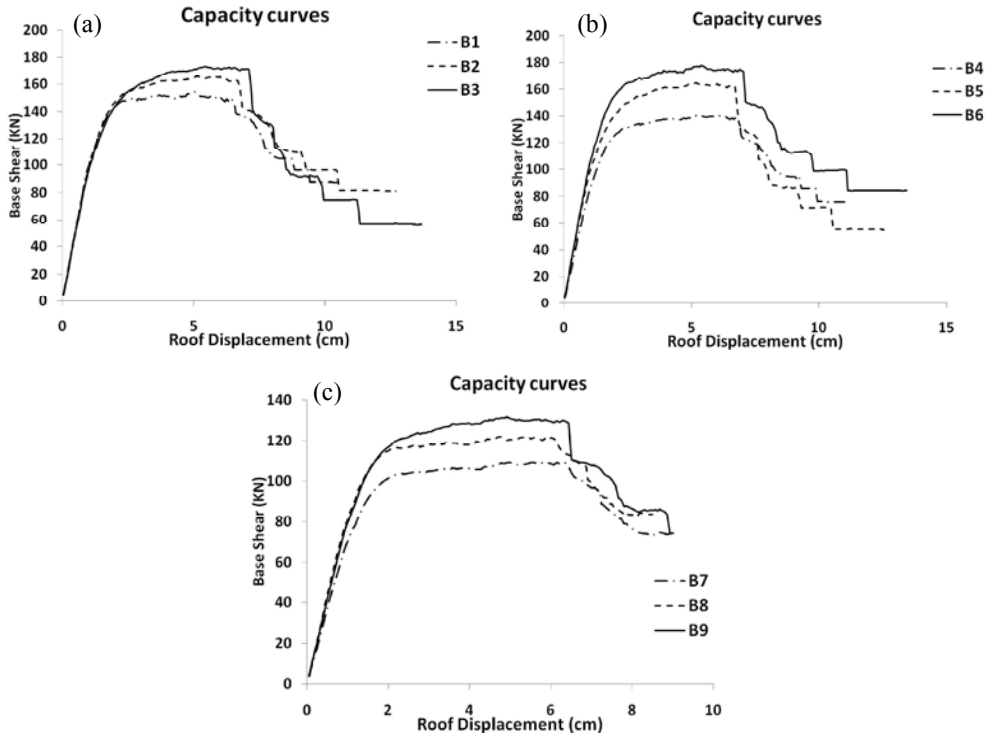


Fig. 16: Capacity curves for the second pattern bracing scheme with different story height and bay width

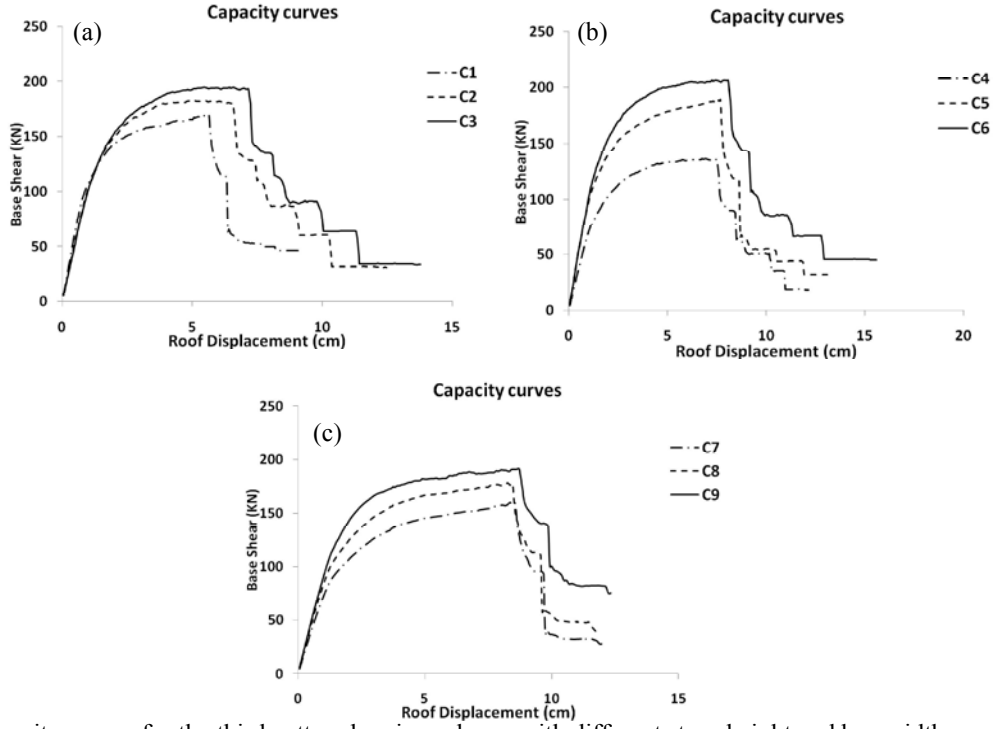
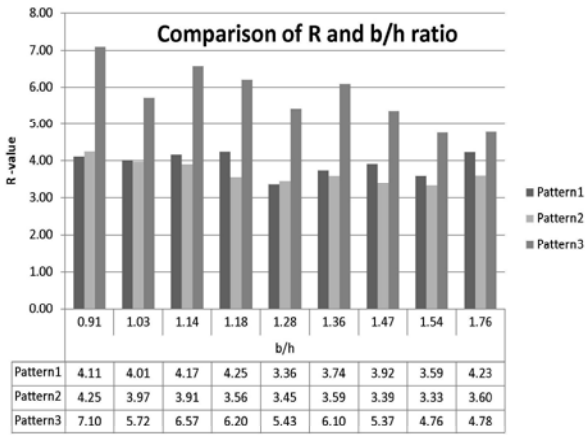


Fig. 17: Capacity curves for the third pattern bracing scheme with different story height and bay width

Figure 20 shows the results of the second pattern. Figure 21 shows the regression analysis of the third pattern studied here. In case of the third pattern, results are in the range provided by IBC2003-OCBF and UBC97-SCBF [16] and provide more promising results overall.



	0.91	1.03	1.14	1.18	1.28	1.36	1.47	1.54	1.76
Pattern1	4.11	4.01	4.17	4.25	3.36	3.74	3.92	3.59	4.23
Pattern2	4.25	3.97	3.91	3.56	3.45	3.59	3.39	3.33	3.60
Pattern3	7.10	5.72	6.57	6.20	5.43	6.10	5.37	4.76	4.78

Fig. 18: Comparison of R-Values depending on b/h ratio

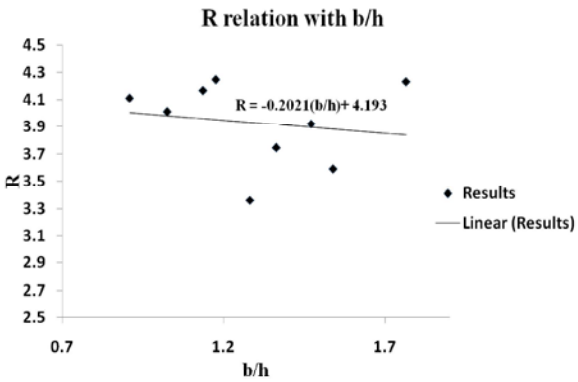


Fig. 19: Linear relation between R and (b/h) for pattern1 bracing scheme

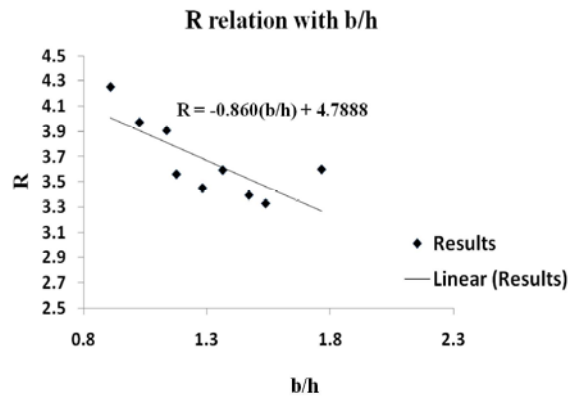


Fig. 20: Linear relation between R and (b/h) for pattern2 bracing scheme

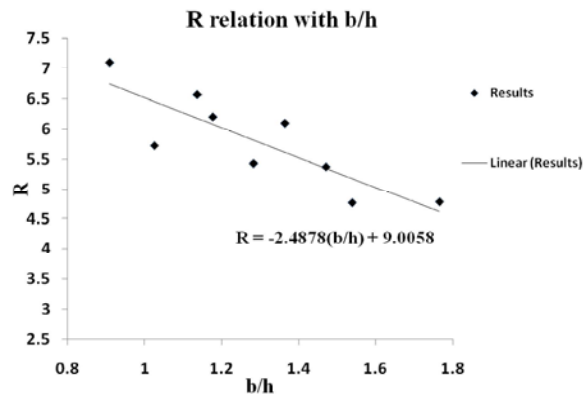


Fig. 21: Relation between R and (b/h) for pattern3 bracing scheme

CONCLUSION

It should be noted that in this research the computed R-values only account for tension as a part of total behavior of this kind of bracing but real behavior of concentrically braced frames is a function of tension, compression, buckling and post buckling behavior of bracing elements. From the obtained results according to tension behavior of the braced frames following observations can be made:

- According to the conducted tests and performed analyses, the third pattern concentric bracing scheme has better behavior with regard to other bracing schemes.
- Initial stiffness of the third pattern is greater than the two other patterns because of joint action of two adjacent braced bays. This can be justified using structural analysis concepts. And also third pattern due to more contribution of bracing elements generally provides more ultimate strength.
- There is no residual capacity in the first pattern studied. This observation backs up the general understanding of earthquake resistant design which recommends the distribution of resistant systems and avoidance of their concentration.
- As the b/h ratio increases in all of the models R decreases. This may be due to larger level of force carried only by bracing elements rather than combination of brace and column.
- The first pattern shows little R dependency on the b/h ratio and the obtained R values are nearly constant and can be chosen as 4.
- This study suggests that not only lateral load resisting system should be considered in definition of R-values but also configuration plays an important role in the inelastic capacity of the structural system.

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