IJMRSET

e-ISSN:2582 - 7219

INTERNATIONAL JOURNAL OF MULTIDISCIPLINARY RESEARCH

IN SCIENCE, ENGINEERING AND TECHNOLOGY

Volume 4, Issue 7, July 2021

INTERNATIONAL STANDARD SERIAL NUMBER INDIA

Impact Factor: 5.928

| ISSN: 2582-7219 | www.ijmrset.com | Impact Factor: 5.928|

| Volume 4, Issue 7, July 2021|

| DOI:10.15680/IJMRSET.2021.0701014 |

Analysis of Zipper Strut to the Seismic Rehabilitation of Structure

Mr. Umaid Patel[*1], Prof. Dilip L. Budhlani[*2]

[*1] Research Scholars, Dept. of Civil Engineering, Guru Nanak Institute of Technology, Nagpur Maharashtra, India

[*2] Assistant Professor, Dept. of Civil Engineering, Guru Nanak Institute of Technology, Nagpur Maharashtra, India

ABSTRACT: The scope of this thesis is to refine the design method for the Zipper Braced Frame System which was initially proposed by Tremblay and Tirca (2003) and to study the system's behavior under seismic loads by means of accurate inelastic time-history analysis. The main objective of this research project is three-fold: To develop accurate computer brace models by using Drain2DX and OpenSees and to validate the accuracy of computations with experimental test results forslender, intermediate and stocky braces;To refine the existing design method for CBFs with strong zipper columns; To validate the refined design method by studying the performance of CBF systems with strong zipper columns in Drain2DX and OpenSees environment forlow-, middle- and high-rise buildings. The design procedure has been divided into two phases: design of braces, columns and beams according to NBC 2005 and CSA-S16-09 and design of zipper columns. A spreadsheet was developed for a 4-, 8- and 12-storey buildings and six different patternloads related to the distribution of internal brace forces over the structure height were proposed. Based on this study, the best suited pattern load distribution is selected and considered for zipper column design. In conclusion, good seismic performance was found for all studied buildings. Then forresin the zippers were equal to or lower than predicted in the design method. All zipper columns performed in elastic range while buckling of braces propagated upward ordownward within seconds. It was clearly demonstrated that by using CBF's with zipper columns the storey mechanism was mitigated and in almost all cases the inter storey drift was uniformly distributed over the structure height. In addition the median estimations of the interstorey drifts were below than 2.5% hs limit prescribed in the NBC-05 code for buildings of normal importance. The outcomes of this research project will be further used as input data for a future experimental test planned to be conducted on an 8-storey braced frame with zipper columns sample. Keyword: zipper, 12-storey, CBF, Storey drift

I.INTRODUCTION

1.1. Concentrically braced frames (CBF) and Zipper:

This system is considered as being the most stiffness efficient when braces behave in elastic range. Once the inelastic response is initiated, the lateral stiffness starts degrading and an asymmetrical response is developed. The popularity of this system is attributed to the reduced cost, supervised fabrication process and speed of erection.

Past studies have shown that braced frame structures exhibit a limited redundancy due to the tendency of earthquake loads to concentrate in a specific floor where large storey forces and inter storey drifts are developed. Consequently, this specific floor becomes vulnerable and prone to storey mechanism formation (plastic hinges in CBF columns) while the structure is driven toward a dynamic sideway collapse. In the case of concentrically braced frames with a chevron configuration, the

Stability of the system is enhanced when strong floor beams are employed. These beams are designed to resist the postbuckling unbalanced vertical load transferred from braces in combination with the corresponding gravity load. When the floor beams are not designed to carry the vertical unbalanced force that develops after braces buckle, the storey shear resistance diminishes and forces are redistributed into the structural system. Even if a chevron bracing system with larger floor beams is designed, it is relatively inefficient to redistribute the lateral loads over the building height. In light of this, the 1995 edition of the National Building Code of Canada (NBCC'95) has imposed a limitation the number of storey for CBF structures in function of ductility and seismicity zone. Later on, in the 2005 edition of the National Building Code (NBCC'05) the limits were changed from the number of storey to the height of the building expressed in meters. Although these limits are considered, the CBF

System is still prone to storey mechanism formation under earthquake excitations characterized by different frequency content. In order to mitigate the formation of storey mechanism and to achieve a stable inelastic seismic response, Khatib et

| ISSN: 2582-7219 | www.ijmrset.com | Impact Factor: 5.928|

| Volume 4, Issue 7, July 2021|

| DOI:10.15680/IJMRSET.2021.0701014 |

al (1988) proposed to add a zipper column to link together all brace-to-beam intersecting points, with the aim being to force all compression braces to buckle and tensile braces to yield, such that a large amount of energy will be dissipated.

1.2 OBJECTIVE

The aim of this research project is three-fold:

1. To develop accurate computer brace models by using the inelastic time-history software Drain2DX and OpenSees and to validate the accuracy of computations with experimental test results for slender, intermediate and stocky braces;
2. To refine the existing design method for CBFs with strong zipper columns;
3. To validate the refined design method by studying the performance of CBF systems with strong zipper columns in Drain2DX and OpenSees computer environment for low-, middle- and high-rise buildings.

Through this research, the overall understanding of the CBF system with strong zipper columns is improved by means of accurate numerical predictions. The outcome of this study will be further used as input data for experimental tests.

II. METHODOLOGY

Proposed Methodology during the tenure of research work.

For attaining the aforementioned objectives the following steps will be carried out:

Results from experimental tests were selected to emphasize the difference in behaviour of slender, intermediate and stocky tubular braces subjected to quasi-static cyclic loads. Based on these test results, analytical brace model were developed and two computer programs such as Drain2DX and OpenSees were selected for numerical simulations. To study the influence of loading type on brace response, a forth sample (intermediate brace) was selected for investigation. All selected braces are tubular, compact cross-sections belonging to class 1 of section. This selection was made to analyse the inelastic brace response which depends on the size of the brace cross-section and type of loading. To bring refinement to the design method of CBFs with strong zipper columns and to assure that zipper columns behave elastically, additional lateral load distribution patterns of internal brace forces are developed herein and different brace buckling scenarios are considered. In this regard, beside the sequential triangular load distribution employed in the previous study, the added patters are the following: triangular; parabolic; sequential parabolic; uniform; and sequential uniform. The maximum tension and compressive force developed in zippers under each one of the aforementioned scenarios was considered for design. Therefore zipper columns are designed to withstand the probable tensile and compressive force developed in braces.

To improve the overall understanding of the CBF with strong zipper columns and to validate the design method, a 4-, 8- and 12-storey building were analysed under three ensembles of ground motions typical for Victoria, British Columbia. The first ensemble is labelled “ordinary ground motion” and is composed of eight simulated and historical accelerograms.

III. MODELING AND ANALYSIS

MODELING OF TUBULAR BRACING MEMBERS UNDER QUASI-STATIC LOADING

Braces are the most critical elements in typical braced frames. Thus, an accurate nonlinear brace model is in demand to simulate the seismic response of the zipper braced frame. Several researchers (Archambault, 1995; Walpole, 1996; Tremblay et al., 2001; Shaback, B., and Brown, T., 2003; Broderick et al., 2008; Haddad et al., 2009) have conducted experimental tests on the cyclic behaviour of tubular brace members in order to investigate the nonlinear brace response under cyclic loading. A comparative study of analytical brace response obtained in Drain2DX and OpenSees against experimental results under quasi-static cyclic loading is carried out in this chapter.

3.1General characteristics of the refined brace model implemented in Drain2DX

The brace model, termed Element 05, was implemented in Drain2DX by Ikeda and Mahin (1984). This nonlinear brace element is defined by the refined physical theory model which consists of two elastic beam segments joined with a plastic hinge at midspan (Figure 3.1 a). The beam segments allow elastic axial and flexural deformations while the state of the plastic hinge is defined by a P-M interaction curve. Empirical parameters for defining the P-M interaction curve and the tangent modulus of elasticity are included in the Ikeda and Mahin refined physical theory model (1984).

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| Volume 4, Issue 7, July 2021|

| DOI:10.15680/IJMRSET.2021.0701014 |

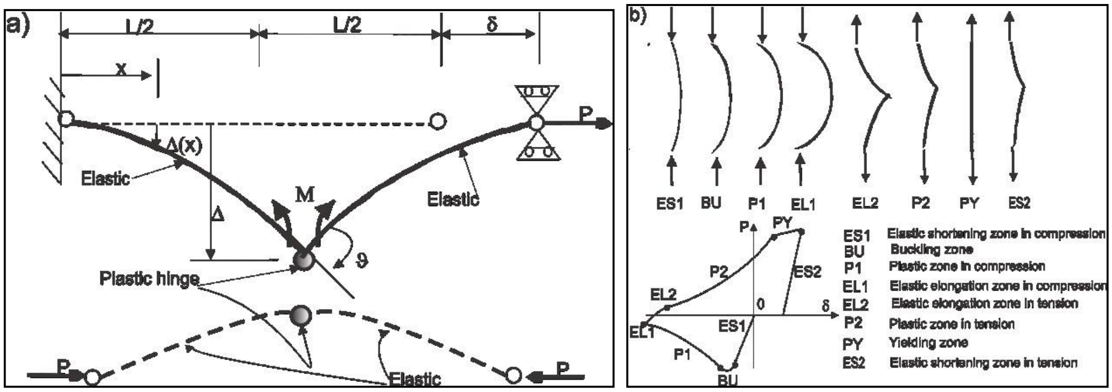

Figure 3.1 Refined physical theory model of brace: a) Refined physical theory brace models; b) Basic behaviour of a brace associated with each zone (Ikeda and Mahin, 1984)

When the brace member is loaded with axial compressive force P, a mid-span deflection is raised. This deflection increases with the axial shortening deformation until the plastic hinge rotation is formed under the internal bending moment M. At this point, the force starts decreasing following the implemented P-M interaction curve and the rotation simulates the buckling of the brace.

The brace's hysteresis cycle can be divided in four zones: elastic zone, plastic zone, yielding zone and buckling zone, as shown in Figure 1 b). The elastic zone is divided into shortening (ES2 and ES1) and lengthening zones (EL2 and EL1) both in tension and compression and the plastic zone is divided into two zones in compression (P1) and tension (P2). The plastic hinge rotation is assumed to occur only in plastic zones and is defined as a function of axial force P and loading history. In Element 05, each zone is divided into a finite number of sections with constant tangent stiffness.

In Drain2DX, the tangent modulus E_t implemented in Element 05 (Ikeda and Mahin, 1984) influences the inelastic cyclic stretching and shortening of braces. For an elasto-perfectly plastic material assigned to the hinge zone, elongation increases under constant tensile load. In this model, two linear empirical curves are implemented as shown in Figure 3.2. These curves are defined as a function of the normalized axial force, $p = P/P_y$, to define the ascending and descending patterns when the axial force decreases or increases. Sets of four parameters e1, e2, e3 and e4 are selected and calibrated based on available experimental data on the tangent modulus of elasticity. It is assumed that tangent modulus is constant until the specimen starts buckling or yielding; then it increases bilinearly when the axial force reverses. However, the difference between the deteriorations of the tangent modulus from cycle to cycle is ignored.

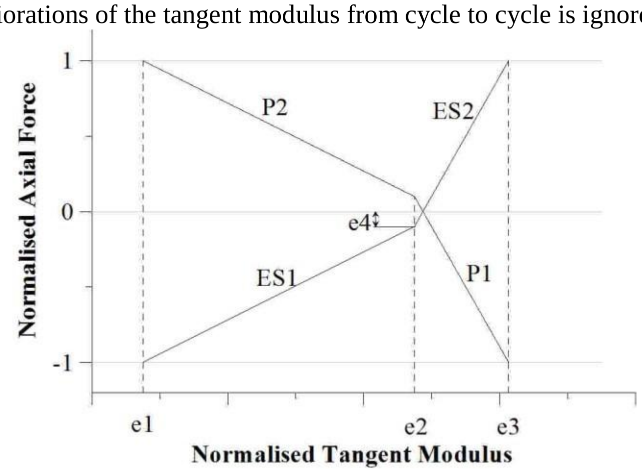

Figure 3.2 Linear Idealization Curves for Tangent Modulus History (Ikeda & Mahin, 1984)

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Thus, in the nonlinear dynamic analysis, the Element 05 implemented in Drain2DX is capable of simulating the inelastic behaviour of braces with various types of cross sections. The accuracy of brace response depends on the defined P-M interaction curve, empirical parameters used to define the tangent modulus variation and the magnification factors in tension and compression. In conclusion, the behavioural characteristics implemented in Element 05 are identified as: i) the material non-linearity per cycle, expressed by the tangent modulus of elasticity in place of the elastic modulus; ii) the deterioration of the cyclic plastic hinge rotation; iii) the consideration of residual displacement once the strength of the material starts degrading. However, local buckling and Bauschinger effect, the progressive degradation of tangent modulus during cycles and the spread of plastification along the brace's length are not considered in this model.

3.2General characteristics of the brace model in OpenSees

In order to overcome the aforementioned limitations of Drain2DX brace model, the Nonlinear Beam-column element with fibre section was selected from the OpenSees's library to simulate the inelastic brace response. This Nonlinear Beam-column element allows plasticity to be spread along the member length. A corotational transformation method is selected to account for the large displacement and a bilinear material law known as Menegotto-Pinto material with isotropic strain hardening is used.

Uriz and Mahin (2004) have underlined that more accurate inelastic brace behaviour can be simulated with nonlinear beam-column elements combined with fibre section model, by applying an initial camber at the member mid-length and attributing the uniaxial Material Steel02, also known as Giuffre-Menegotto-Pinto model with isotropic strain hardening. The Menegotto-Pinto functions (1973) express stresses as a function of strain and the material model is defined based on the following equation:

Equation 3.1 $$\sigma^* = b\varepsilon^* + \frac{(1-b)\varepsilon^*}{(1+\varepsilon^{*R})^{\frac{1}{R}}}$$

where σ* and ε* are the effective stress and strain depending on the unload/reload interval, b is the ratio of the final to initial tangent stiffness and R is a material parameter which defines the shape of the unload curve. The Menegotto-Pinto function for the strain stress curve is able to describe the response of the highly nonlinear model accurately. It is stated that the initial tangent stiffness E_0 is equal to the elastic stiffness E, the stress-strain relation is linear in the elastic range and under the yielding plateau, the strain increases from yielding strain to strain hardening while the stress is constant. The Menegotto-Pinto model accounts for the accumulated plastic deformation at each point of load reversal. Thus, the hysteresis loop follows the previous loading path for a new reloading curve while the deformation is cumulated.

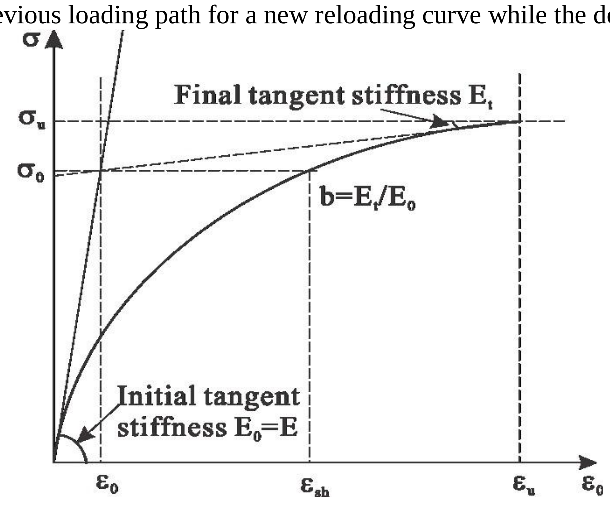

Figure 3.3 Menegotto-Pinto model for steel material

In OpenSees environment, the brace model consists of a number of force-based elements with distributed inelasticity over the length of the member. The steel fibres of the elements are defined with Menegotto-Pinto stress-strain

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relationship. Thus, both Bauschinger effect and P-M interactions are considered. The force displacement relation in the standard force-based element formulation is established on the basis of local coordinates, which has been transformed to global reference system following the concept of the Corotational geometric transformation, in other words, large displacement geometry is also considered in the model. With this approach, two elements for each brace are sufficient to simulate the buckling zone. Even though the local buckling is not considered in the model, according to Uriz and Mahin (2008), the nonlinear response of hollow cross-sectional braces does not seem to be substantially affected.

3.3 SEISMIC ANALYSIS AND STRUCTURAL RESPONSE OF MULTI-STOREY ZIPPER BRACED FRAMES

The 4-, 8-, and 12-storey zipper braced frames designed in Chapter 4 based on the proposed methodology developed on the Excel spreadsheet under the considered 2 scenarios: zippers in tension and zippers in compression, is analyzed herein with Drain2DX and OpenSees. The purpose of these analyses is to validate the design method as well as to assess the performance of this innovative structural system. A detailed description of analytical models is provided in the first part of the chapter, and a comparative discussion related to the time-history responses as obtained in Drain2DX and OpenSees is conducted in the second part.

3.3.1Zipper braced frame modeling

In order to validate the proposed design method, results from numerical analyses performed with ETABS (elastic analysis) and two finite element programs: Drain2DX and OpenSees are considered. As presented in Chapter 2, most of the zipper braced frame analyses conducted by following researchers: Sabelli (2003), Tirca and Tremblay (2003, 2004), Leon and Yang (2003)), were performed by using Drain2DX. Thus, for a consistent discussion related to the previous researches, the Drain2DX program, developed at UC Berkeley, was selected as being the first analytical tool. In addition, the second computer program selected to overcome the limitations of Drain2DX was the most popular earthquake engineering simulation platform, OpenSees.

Prior to numerical modeling of zipper braced frames, the following assumptions have been made:

- For simplicity, the building sample has a symmetrical layout and the accidental in plan torsion was omitted.
- In Drain2DX, the zipper braced frame is modeled in 2 Dimension. Therefore, the out-of-plane buckling of brace elements was neglected.
- To take into account the effect of gravity columns, all the gravity columns were considered along with the braced frame in a 2D layout. The gravity columns were connected to the brace frame through rigid links to simulate their behaviour in the structure. The lateral shear forces were transferred to the braced frame through these links.
- All the connections within the structures are assumed to be pin connections, which include the brace end connections, beam to column connections, and the column ends connections.
- Gusset plates are modeled as rigid extensions. The yielding and buckling effect of gusset plates is neglected.

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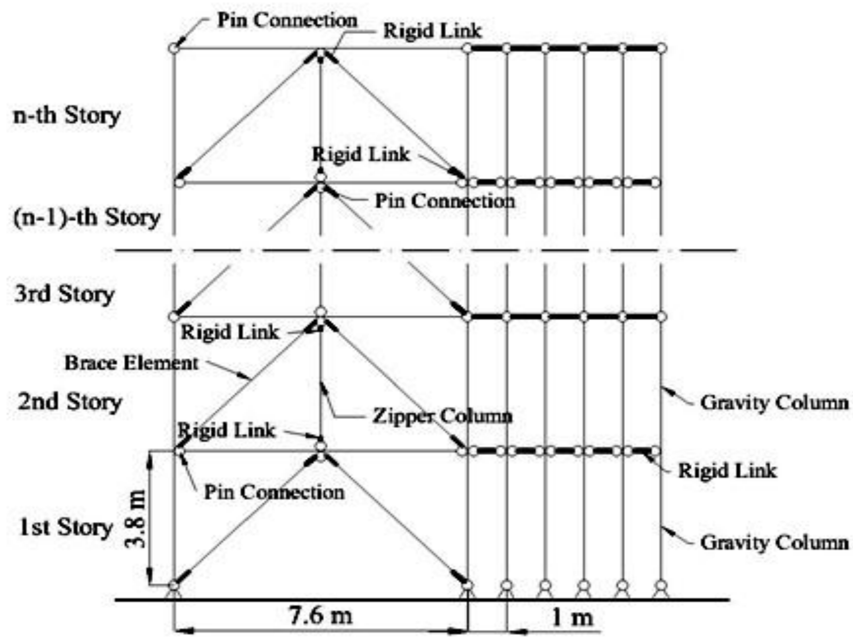

Figure 3.3.1 Computer model of CBF with zipper columns

IV. RESULTS AND DISCUSSION

In this study a 3% Rayleigh damping was assigned to the model. All the zipper columns and braces are pin connected to a gusset. P-delta effect has been considered for both frame columns and gravity columns. A typical analytical model

4.1Drain2DX results

The response of the studied structures depends on the frequency content, the ratio peak ground acceleration over the peak ground velocity and the duration of the selected ground motions. For example, under the regular ground motion excitations, the largest tensile forces developed in the zipper columns occur at the lower part of the structure (levels 3rd, 4th), while the maximum compressive forces occur at the upper part. However, during Cascadia ground motion, larger seismic demand is required at the bottom part of the structure forcing zippers to act mostly in tension, while the Near-field ground motions excite the upper modes and drive the largest demand towards the upper part of the structure. For the studied buildings, the maximum and the mean + standard deviation magnitude of axial tension and compressive forces developed under the three considered ground motion ensembles are illustrated in Figure 5.3. As explained in Chapter 4, the braces, beams and columns of the braced frame structure with zipper columns were designed in agreement with S16-2009 seismic design requirements for moderately ductile CBF with a chevron bracing scheme. Several distribution patterns of internal forces generated by the unbalanced brace force propagated upward or downward were considered in order to capture the maximum demand in zippers. However, the demand in zippers varies from one pattern load to the other as is shown in Figure 5.3. By analyzing the compression side, the demand coming from both pattern loads: sequential triangular and sequential parabolic differs about 10% for the upper part of the structure. In this respect, the sizes of zippers were chosen to cover the demand resulting from the sequential triangular (LP-ST) distribution, which is in agreement with the method proposed by Tremblay and Tirca (2003).

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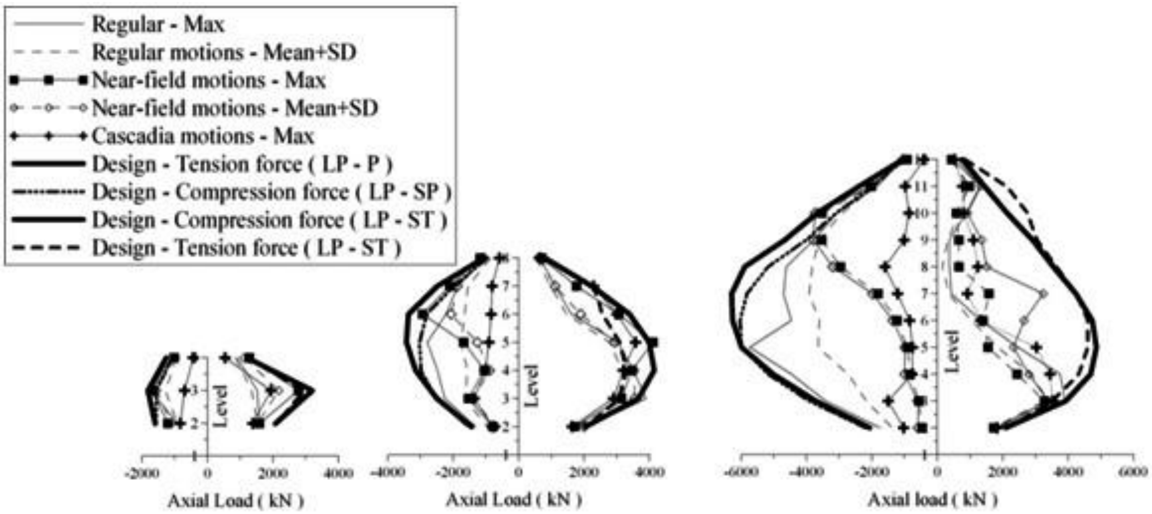

Figure 4.1Axial force in zipper columns obtained from nonlinear dynamic time-history analyses of: a) 4-storey building; b) 8-storey building; c) 12-storey building

This exercise is able to demonstrate that by considering a parabolic distribution versus the sequential triangular distribution pattern, a slightly larger tensile demand is obtained in zippers. Therefore the concern raised by Tremblay & Tirca (2003) in their study as shown in Figure 4.2 is overcome by adopting a different lateral load distribution pattern LP-P.

Therefore, the LP-ST load distribution pattern was retained to compute the compressive demand of zipper columns in order to size the zipper cross-sections and the LP-P load distribution pattern was retained to compute the tensile demand and to verify the selected cross-sections.

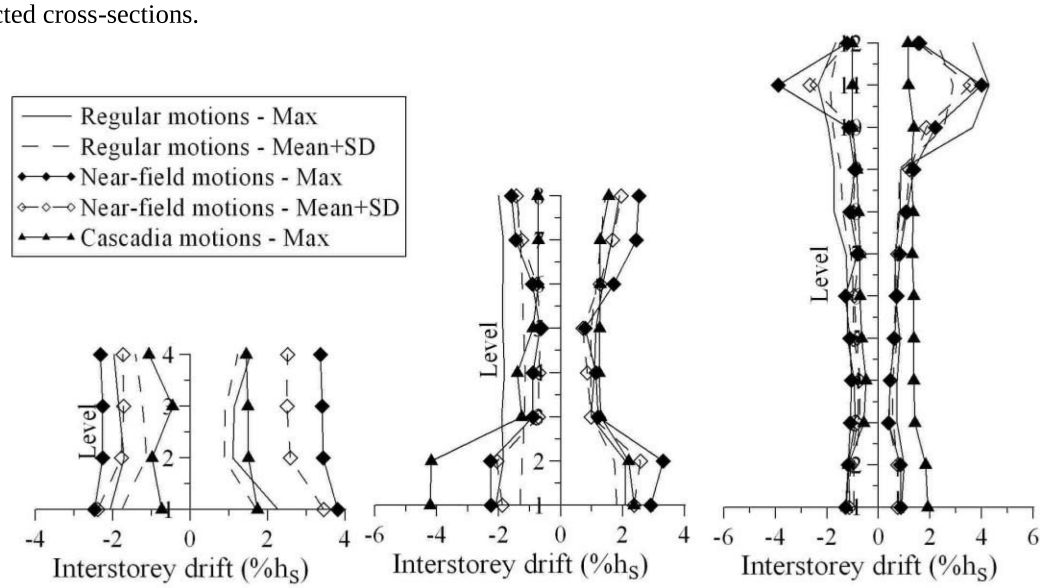

Figure 4.2 Computed interstorey drift: a) 4-storey building; b) 8-storey building; c) 12storey building

For the studied buildings, the maximum and the Mean+SD (standard derivation) interstorey drifts have been selected as seismic response parameters. As shown in Figure 5.4, these structures show almost uniform distribution of the interstorey drift along the height of the building. During regular ground motions, the maximum interstorey drifts for the 4- and 8-storey buildings are around 2% h_s, where h_s is the storey height. The 12storey building showed a different behaviour influenced by

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the higher modes effect. Thus, the top three stories are prone to larger deformation. Under the Near-field and Cascadia ground excitations, the 8-storey structure undergoes a larger demand at the lower storeys. When the bottom braces buckle, and beams lose their brace support, the zippers are activated in tension and transfer the load to the upper undamaged floors. When Mean+SD values are considered instead of the maximum interstorey drift values, upper limit

4.2.Performance assessment of the 4-storey building

The 4-storey building generally deflects into the first mode of vibration. the buckling of braces initiates at the first storey, and then propagates upward. It is interesting to note that the buckling of braces and hinging of beams happen in different stages. In general, braces on the compressive side buckle first, then, in the subsequent cycle, the braces on the other half of the CBF reach the buckling force. Once the stiffness degrades, beams start hinging usually in the same sequence. The behaviour of the 4-storey building follows the prescribed zipper mechanism. The effect of higher modes is hardly noticeable due to the relatively short period of the structure. However, because of different characteristics of the selected ground motions, cases in which the brace buckling initiates at the top floor are also observed, under the R5, R7 and R8 ground motions. Under all considered Near-field ground motions but one (N5), the buckling of braces were initiated at the base. However, under both Cascadia ground motions the first brace buckled almost simultaneously at the first and at the second floor. the peak axial tensile forces computed in the zipper columns under the regular and the Cascadia ensembles were lower, especially at the 3rd floor, than those values estimated in design. The larger values of both tensile and compressive forces were obtained in zippers under the Near-field ensemble. Under all regular and Cascadia considered ground motions, the maximum interstorey drift was remaining within the code limit.

Another interesting phenomenon which has to be noted is related to the sequences of braces buckling, which occurs within 1 second as illustrated.This typical response of zipper braced frames proves the efficiency of adding zippers to CBF systems. The zippers transfer the unbalance forces from the damaged floor to the adjacent undamaged floor. The capability of zippers to control the redistribution of lateral forces after braces have buckled has been demonstrated.

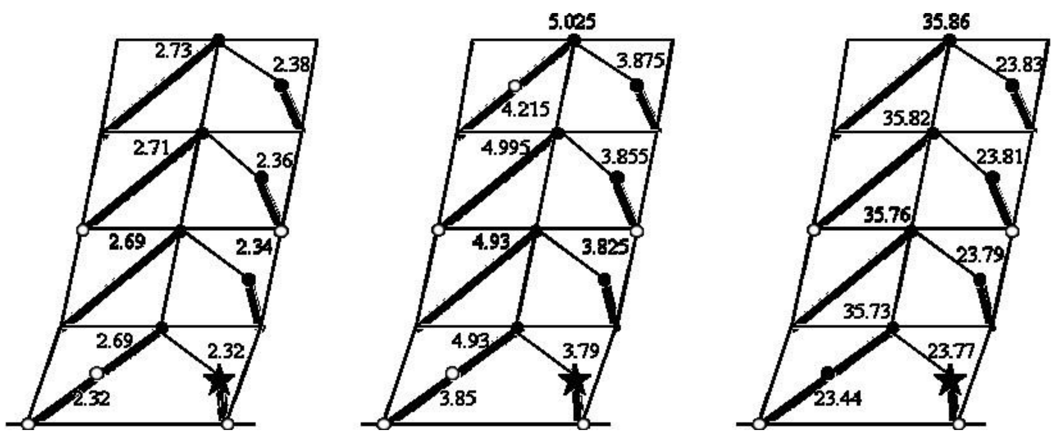

Figure 4.3Time-history response of brace buckling and beam hinging for the 4-storey building under: a) R1 regular ground motion; b) N6, Near-field ground motion; c) C2, Cascadia simulated ground motion

4.3Performance assessment of the 8-storey building

Considering interstorey drift as being the main parameter for assessing the structural performance of the middle-rise building, it is noted that the maximum response of the structure under the eight selected regular ground motion excitations, and the Mean+

Standard deviation values of the Near-field records are within the 2.5% limit (Figure 5.6). In general, under the six out of the eight regular ground motion excitations, the first buckling occurs at the bottom floor and the buckling is then propagated upwards. Thus, the structural response under the R2 and R6 excitation is characterised by a large demand concentrated at the upper part which forces the top floor brace to buckle. Contrary, under the Cascadia subduction ground motions, a larger demand is observed to occur at the bottom of the building. For example, the mechanism of braces buckling and beam hinges is illustrated in Figure 5.6a under the ground motion record R1. Herein, the first bottom floor

| ISSN: 2582-7219 | www.ijmrset.com | Impact Factor: 5.928|

| Volume 4, Issue 7, July 2021|

| DOI:10.15680/IJMRSET.2021.0701014 |

brace buckles at the 2.28th second, and the unbalanced force is transferred to the upper floor through the zipper column. Due to this redistribution of forces, the brace located at the second storey on the verge of buckling reaches its probable compressive capacity at the 2.32nd second. The buckling of braces is propagated upward within 0.35 seconds. After all braces belonging to the same half-span of the framed bay buckled, the unbalance forces in braces produced hinging of the beams at their mid-span. As illustrated in the aforementioned figure, all the beam hinges developed within the time interval 2.79 seconds to 3.11 seconds.

In addition, Figure 5.6b and c shows the behaviour of the same 8-storey building under the N6 (Near-field) ground motion and Cascadia record C2. Although in these cases the first buckled brace is located at the first floor, the building behaviour is different. Under the Near-field time history acceleration the building behaves mostly after the second vibration mode with larger demand at the bottom and the top parts.

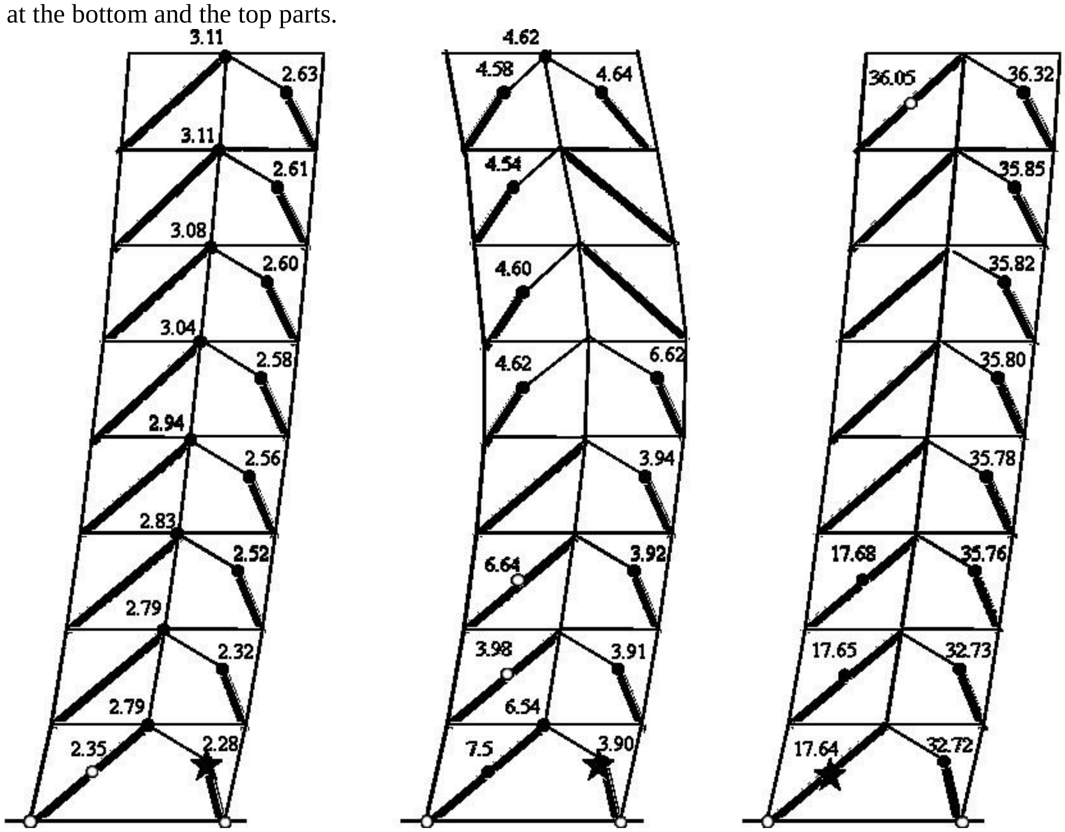

Figure 4.4Time-history response of brace buckling and beam hinging for 8-storey building under: a) R1 regular ground motion; b) N6 Near-field ground motion; c) C2 Cascadia simulated ground motion. .

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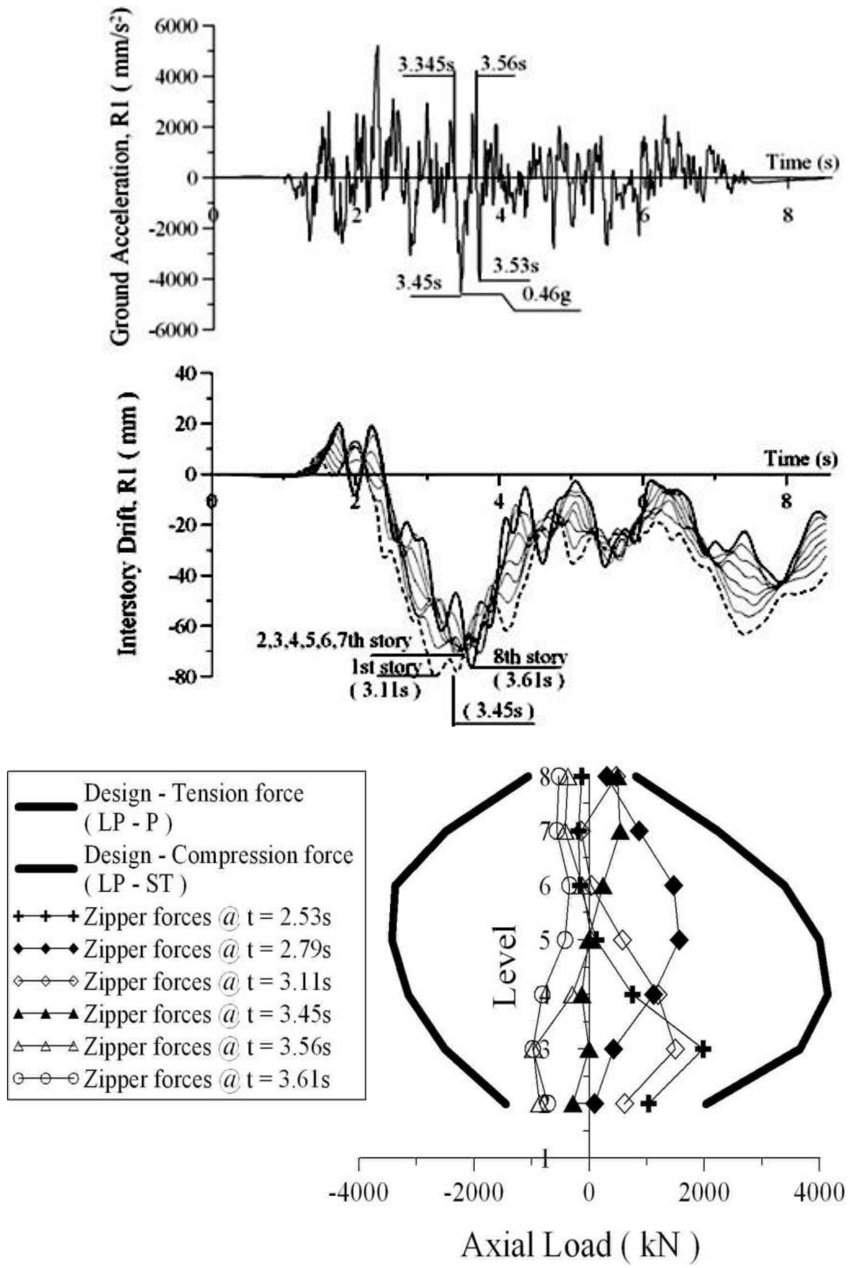

The history of axial forces developed in zipper columns over the building height are shown in Fifor the following time steps: t = 2.28s; t=2.53s; t=2.79s; t=3.11s; t=3.45s and t=3.56s. The axial force in the zipper columns corresponding to the maximum displacement is lower. The maximum interstorey drift corresponding to 2.2% storey height and 2.1% storey height, occurred at t=3.345s at the first floor and at 3.56s at the roof respectivelyCascadia subduction ground motions cause a larger seismic demand at the bottom of the building rather than at the upper floors.

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4.4Performance assessment of the 12-storey building

Under the regular ground motion ensemble, the Mean+SD interstorey drift values of the 12-storey building are below the code limit (2.5%hs) and are equally distributed over the building height. However, under the R4 regular ground motions, the interstorey drift of the top 3 storeys have reached 3.5%hs. During Near-field excitations, the top stories are always influenced by the higher modes effect and experienced large interstorey drift demand in the interstorey drifts. The building response under two out of four Near-field ground motions showed a greater demand at the 11-th storey. Contrary to the behaviour of the 8-storey, building under Cascadia ground motions, the 12-storey structure shows a uniform interstorey drift distribution over the structure height with peaks below the code limit

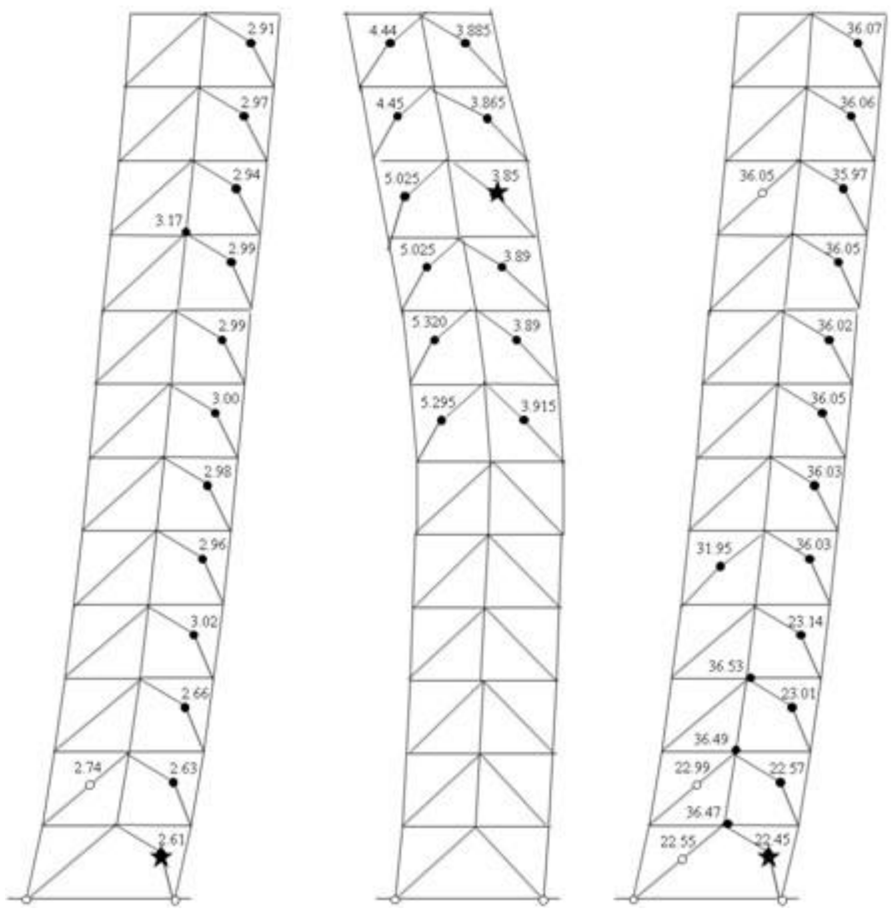

Figure 4.5 Time-history response of brace buckling and beam hinging for 8-storey building under: a) R1 regular ground motion; b) N6 Near-field ground motion; c) C2

Although Cascadia ground excitations did not show a great demand in terms of interstorey drifts, brace buckling and beam hinging were still observed, which suggests that significant amount of energy had been dissipated through the plastic deformations. Therefore, the effect of zipper columns clearly demonstrates the spreading of inelasticity all braces.

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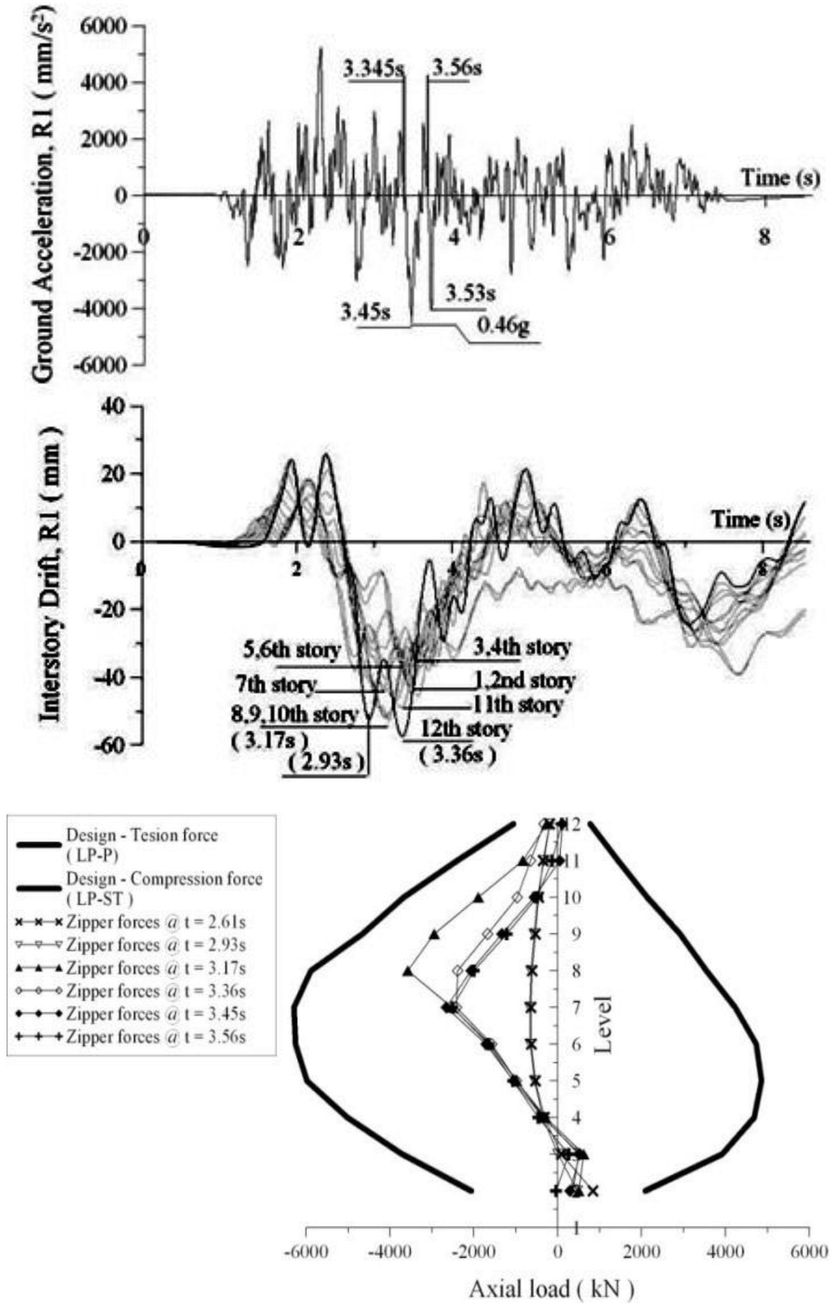

Figure4.6.Inelastic response of the 12-storey building under the R1 ground motion: a) Simulated accelerograms, R1; b) Time-history of interstorey drift; c) Axial forces in zipper columns

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| DOI:10.15680/IJMRSET.2021.0701014 |

4.5 Open Sees results

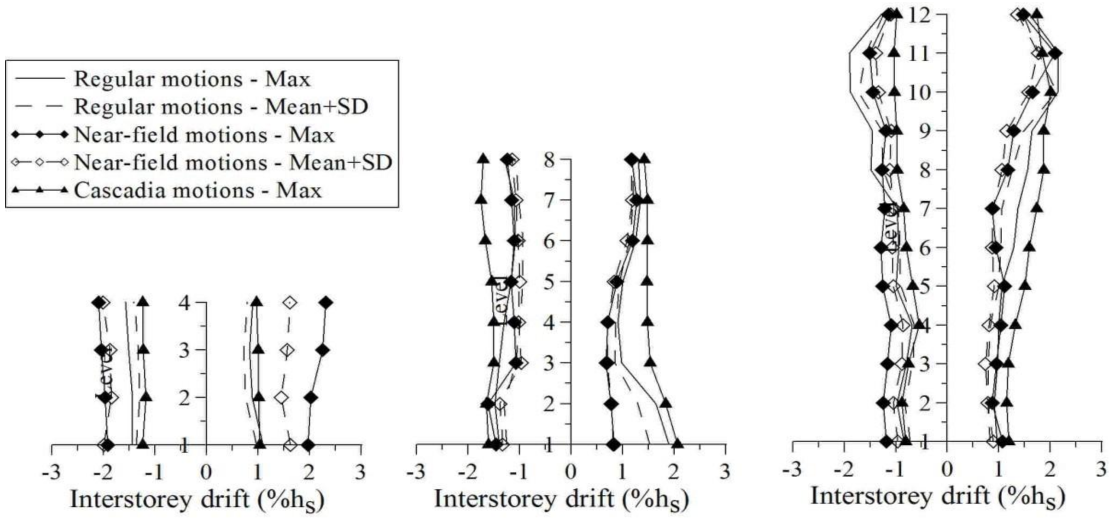


Figure 4.8 Computed interstorey drift: a) 4-storey building; b) 8-storey building c) 12storey building

In general, the 4-storey structure showed a first-mode based deformed shape and a similar behaviour was found when the structure was subjected to Near-field ground motions. The seismic response of the 4-storey building under the ground motion excitation N3 is shown in Figure 5.14, while the sequence of brace buckling is shown in Table II-1 of Appendix II. The first brace buckled at the bottom floor at 2.36s. Then, when the ground motion reverses direction, the brace on the other side of the same floor began to buckle and the buckling of braces propagated upwards successively.

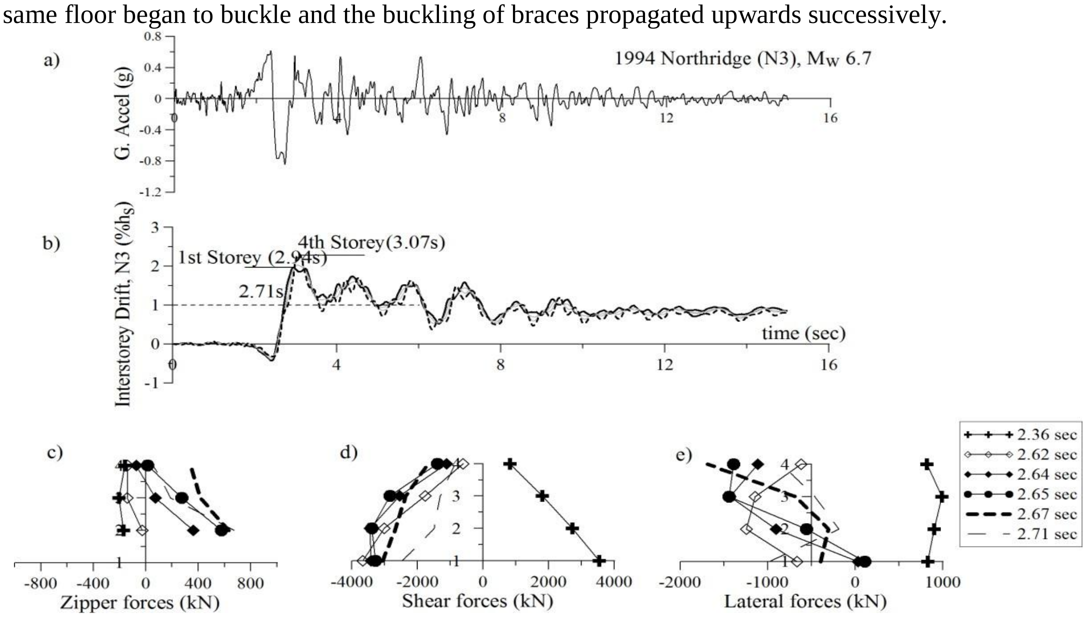


Figure 4.9. Structural response of 4-storey zipper braced frame under ground motion N3:

a) Regular ground motion excitation N3; b) Interstorey time-history record under ground motion excitation N3; c) Axial forces in zipper columns at specified times; d) Shear forces distribution along the building height at specified times; e) Storey forces induced into structure at specified times.

| ISSN: 2582-7219 | www.ijmrset.com | Impact Factor: 5.928|

| Volume 4, Issue 7, July 2021|

| DOI:10.15680/IJMRSET.2021.0701014 |

4.6. Discussion of results

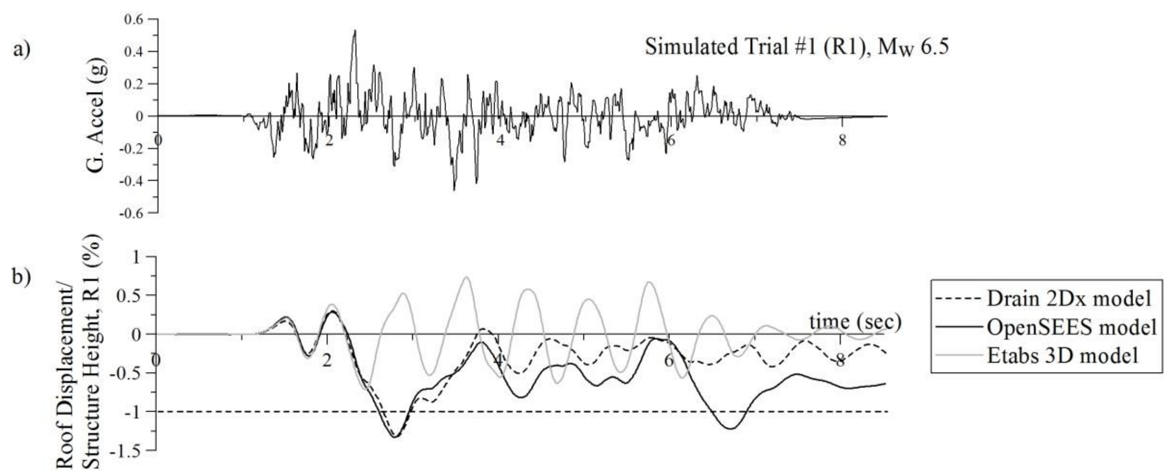
The analyses conducted with ETABS, Drain2DX and OpenSees have shown almost the same fundamental period for the three studied structures in the elastic range (Table 5.3).

Table 4.1 Analytical fundamental periods of vibration of the 4-, 8- and 12-structures

Story	Height [m]	Ta [s]	ETABS T_1 [s]	ETABS T_2 [s]	ETABS T_1/T_a	Drain2DX T_1 [s]	Drain2DX T_2 [s]	Drain2DX T_1/T_a	OpenSEES T_1 [s]	OpenSEES T_2 [s]	OpenSEES T_1/T_a
4	15.3	0.76	0.72	0.28	0.95	0.70	0.27	0.92	0.75	0.29	0.99
8	30.4	1.52	1.74	0.59	1.14	1.70	0.56	1.12	1.75	0.59	1.15
12	45.7	2.28	2.74	0.86	1.20	2.71	0.83	1.19	2.76	0.88	1.21

In the given table, T_a is the fundamental period calculated as per the current edition of NBCC times two ($T_a = 2\times(0.025h)$, where h is the height of the building), which in fact is the allowable upper limit. For the low-rise (4-storey) building, a very good match was found, while for the 12-storey building this difference has slightly increased. By obtaining almost the same dynamic properties with the three computer programs, the accuracy of the computation is validated.

Figure 4.9 shows the time-history roof displacement of the 4-story zipper braced frame obtained from Drain2DX, OpenSees and ETABS.



As can be observed from the figure, when the structure behaves elastically, the results obtained from Drain and OpenSees well agreed with that from the ETABS. However, after the first brace buckled (at t = 2.32s) and inelasticity initiated, the response of the structure models in OpenSees and Drain2Dx were driven as expected. It is noted that no plastic characteristics were assigned to the ETABS model. The difference between the results obtained in Drain2DX and OpenSees models are explained by the limitation of the Drain2DX brace model which has been. Another difference consists in the damping formulation in both programs. Implemented in OpenSees, the Rayleigh damping command allows users.

V. CONCLUSION

1. Chevron braced frames have been widely used in North America as a structural configuration against earthquake excitations. Due its limitation in redistributing the internal brace forces once braces buckle, the structure is exposed to

| ISSN: 2582-7219 | www.ijmrset.com | Impact Factor: 5.928|

| Volume 4, Issue 7, July 2021|

| DOI:10.15680/IJMRSET.2021.0701014 |

storey mechanism formation and reduced energy dissipation capacity. In light of this, zipper columns are introduced to overcome the CBF limits. In this study, zipper columns are designed to remain in elastic range throughout the entire ground motion excitations while transferring the unbalance brace forces resulted from buckling.

2. The first objective pursued in this study was to refine the method proposed by Tremblay and Tirca (2003), who have considered only the LP-ST (Sequential triangular) load redistribution pattern for zipper column design. Thus, in this study, in order to capture the maximum compressive and tensile forces in zipper columns, the following force redistribution patterns are considered, such as: LP-T (Triangular), LP-ST, LP-P (Parabolic), LP-SP (Sequential parabolic), LP-U (Uniform) and LP-SU (Sequential uniform).

3. In the compression side, a small difference exists between the zipper force envelope defined by the LP-ST and LP-SP patterns. The larger values were estimated when the LP-ST pattern was considered, which was then selected for the zipper column design.

4. In the tension side, the maximum force envelope was captured by the LP-P pattern, followed by the LP-ST and LP-SP patterns. Accordingly, the LP-P pattern is recommended and adopted in design.

5. The second objective of this study was to validate the proposed design method under different circumstances, and to evaluate the behaviour of zipper braced frames. To analyze the inelastic behaviour of a zipper braced frame structure, two finite element computer programs: Drain2DX and OpenSees were compared. The accuracy of modeling brace inelastic behaviour with both programs was validated in Chapter 3 against experimental test results. In this validation, the effects of local buckling, residual stress and low-cycle fatigue have been neglected.

6. A new P-M interaction curve was proposed and implemented in the Drain2DX program to define the yielding surface of a HSS profile, such that a better match between computation and experimental test would be obtained. Better results were identified for stocky and intermediate braces when the cumulative energy dissipation parameter had been employed to measure the modeling accuracy.

7. Parameter studies of the construction of an OpenSees brace model have been carried out on a general basis, and recommendations for brace modeling in the OpenSees environment have been given. Brace models consisted of a minimum of 4 nonlinear beam-column elements with finely meshed fiber sections, 4 integration points per element, and an initial sinusoidal out-of-straightness with amplitude corresponding to $1/500^{th}$ brace length was found to give a satisfactory buckling force. An equation for determining the value of the out-of-straightness has also been verified. In terms of cumulative energy dissipation, OpenSees offers a better match than Drain2DX due to its omission of the Bauschinger effect and assumption of concentrated plasticity.

8. On this basis, numerical models of zipper braced frames of a 4-, 8- and 12-storey buildings were designed according to the proposed method and analyzed with the Drain2DX and OpenSees programs. Three ground motion ensembles (regular, Cascadia and Near-field) consisting of 16 ground motions, were selected and scaled to match the design spectrum of a specified site location.

9. A two-step ground motion scaling method has been proposed in this study, which requires the match of spectrums over the periods of interest: $0.2T_1$ - $1.5T_1$.

10. The results of the nonlinear dynamic analyses examined in Chapter 5 have shown uniformly distributed interstorey drifts over the structure height and the tendency of expected zipper mechanism formation. The maximum forces induced into the zippers were well-predicted by the proposed design method. Detailed investigation reveals that the presence of zipper columns efficiently transferred the unbalanced brace forces from the floor where brace buckled to adjacent non-damaged floors. Adding zipper columns in chevron braced frame successfully triggered the zipper mechanism which leads to uniformly distributed damage to the structure.

11. For the 4-storey building, under a large number of ground motion excitations, the first brace buckles at the bottom floor. Buckling has initiated when the base shear reaches its maximum value simultaneously with a larger storey force. When the first brace buckles, the structure deflects in the first-mode shape. Buckling of brace normally starts when the interstorey drift of corresponding stories is around 1%hs.

12. For the 8-storey building, buckling initiates either at the first floor and propagates upward, or at the roof level and progresses downward. Different earthquake characteristics lead the building to behave differently. The higher modes effect has been observed, as well as the occurrence of the whipping effect. In general, when the first brace has buckled, the structure deflects either in the 2^{nd} or the 3^{rd} vibration mode shape. It is observed that buckling of braces also initiates when the interstorey drift reaches 1%hs.

| ISSN: 2582-7219 | www.ijmrset.com | Impact Factor: 5.928|

| Volume 4, Issue 7, July 2021|

| DOI:10.15680/IJMRSET.2021.0701014 |

13. The behaviour of the 12-storey building is similar with that of the 8-storey building. However, the subsequent buckling of braces cannot be developed under a singular ground motion cycle and is divided in tiers of braces buckling. The effect of higher mode effect is further emphasized.
14. Theses analyses were performed at the design level, while the structures still have remaining strength until failure is initiated. To discuss the behaviour of the structure at the near-collapse state, the incremental dynamic analysis method has to be employed in future studies.

VI. **ACKNOWLEDGEMENT**

I would like to thank firstly my project guide Prof. Dilip L. Budhlani and my HOD Prof. Sushant M. Gajbhiye for their guidance and invaluable words of advice, support, and help given to me to do this review paper. Secondly, I would like to thank my dear parents and my dear friends for their appreciation and belief in me.

REFERENCES

1. American Society of Civil Engineers (ASCE), (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings, prepared for the SAC Joint Venture, published by the Federal Emergency Management Agency, FEMA-356, Washington, D.C.
2. Aguero, A., Izvernari, C. and Tremblay, R., (2006). Modelling of the Seismic Response of Concentrically Braced Steel Steel Frames using the OpenSees Analysis Environment. International Journal of Advanced Steel Construction, 2, 3, pp 242-274.
3. Agureo, A., Izvernari, C. and Tremblay, R., (2005). Numerical Comparison and
4. Optimization of Force and Displacement Based Elements for the Analysis of the Inelastic Cyclic Response of Steel Bracing Members. Advances in Steel Structure, Vol. II, 12351240.
5. Applied Technology Council, (1992). Guidelines for Cyclic Seismic Testing of Components of Steel. Structures, ATC-24, Redwood City, CA.
6. Archambault, M.H., (1995). Etude du comportement seismique des contreventements ductile en X avec profiles tubulaires en acier. Rapport No. EPM/GCS-1995-09 Ecole Polytechnique, Montreal.
7. Astaneh-Asl, A., Goel, S.C. and Hanson, R.D., (1985). Cyclic Out-ofPlane Buckling of Double-Angle Bracing. Journal of Structural Engineers, ASCE, 111, pp. 1135-1153.
8. Baker, J. W., (2009). The conditional mean spectrum: A tool for ground motion selection, ASCE Journal of Structural Engineering (in press.)
9. Broderick B.M., Elghazouli, AY, Goggins, J., (2008). Earthquake testing and response analysis of concentrically-braced sub-frames, Journal of Constructional Steel Research, Vol 64, Issus9.
10. Bruneau, M., Engelhardt, M., Filiatrault, A., Goel, S. C., Itani, A., Hajjar, J., Leon, R., Ricles, J., Stojadinovic, B. and Uang, C.-M., (2005). Review of selected recent research on US seismic design and retrofit strategies for steel structures. Progress in Structural Engineering and Materials, 7:103–114. doi:10.1002/pse.192.
11. Canadian Standard Association. (2009), CAN/CSA-S16-09 Limit States Design of Steel Structures. Canadian Standard Association, Toronto, ON.
12. De Sousa, R. M., (2000). Force-Based Finite Element for Large Displacement Inelastic Analysis of Frames PhD Thesis, University of California, Berkeley.
13.Dicleli, M. and Mehta, A., (2007). Simulation of Inelastic Cyclic Buckling Behaviour of Steel Box Sections. Computer & Structures Journal pp. 446-457.
14. Federal Emergency Management Agency (FEMA), (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings, FEMA-356, Washington, D.C.