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Seismic Evaluation of Concentric Braced Frames using Nonlinear Static and Dynamic Time History analysis

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***Abstract:*--** **These Concentric braced frames (CBFs) are frequently used in steel structures as an efficient structural system to resist earthquake loads and control the structural drift. The braced system designed as per code provisions will be subjected to yielding and post buckling failures when acted upon by strong ground motions resulting in excessive interstorey drifts. Eurocode 8 (EC8) presents various guidelines for the design of dissipative elements along with requirements of non-dissipative parts to ensure a predetermined failure mode. The paper presents the results of a study on the seismic evaluation of four most commonly used CBFs namely diagonal, X, Chevron and zipper designed as per seismic provisions of EC8. A nonlinear static (pushover) analysis is first performed to obtain the capacity curve, over strength and the sequence of plastic hinge formation. Nonlinear dynamic time history analysis is then performed using preselected ground motion records to confirm the findings of pushover analysis and assessment of performance levels. The results show that all CBFs met the expected performance levels and ensured intended yield mechanisms. The assessment of behavior factors indicate that zipper configuration is the most efficient followed by the diagonal & X bracings. The Chevron bracings is characterized by the least overstrength.**

Index Terms:- Concentric braced frames, nonlinear static pushover, capacity curve, dynamic time history analysis, interstorey drift, hysteresis, ductility

I. INTRODUCTION

Braced steel structures consist either concentric braced frames (CBFs) or eccentrically braced frames (EBFs). In CBFs horizontal earthquake forces are mainly resisted by axially loaded members whereas in EBFs by axially loaded members, but where the eccentricity of the layout is such that energy can be dissipated in seismic links by means of either cyclic bending or cyclic shear [1]. CBFs represent the most widely used seismic lateral load resisting system used in steel structures. Based on their configuration CBFs are categorized as diagonal-braced, X-braced, Chevron-braced or zipper-braced, the choice of a particular type being governed by their architectural functionality and structural efficiency. While steel structures provided with CBFs normally satisfy the code specified drift limitations, much attention shall be given to various design provisions to ensure achievement of a ductile global failure mechanism.

Eurocode 8(EC8) [1] require that structures in seismic regions be designed and constructed to meet both the "No-Collapse requirement" and "Damage limitation requirement" essentially referring to ultimate and serviceability limit states respectively. The code identifies the predetermined parts where the dissipative capabilities are mainly located (dissipative zones) and non-dissipative zones. While the dissipative elements yield during seismic events, the non-dissipative structural elements such as beams, columns and connections shall remain elastic and therefore be designed to have an adequate over-strength with respect to the dissipative elements. Therefore, the entire CBF systems shall be "capacity designed". CBFs may consist of dissipative zones in tension diagonals only (diagonal & X bracings) or in both tension and compression diagonals (Chevron bracings).

The CBFs in general provide the required strength and ductility through yielding of tension braces and post buckling of compression braces, their behaviour and design being largely influenced by the brace configuration. The diagonal and X configurations are characterized by the tensile yielding of brace members. The chevron configurations are associated with an undesirable post buckling behaviour in which the unbalanced vertical components of the compression and tension brace forces results into a connecting beam flexure mode of failure rather than a truss action. The flexure mode of beam failure shall be avoided following relevant code provisions leading to strong connecting beams. The zipper configuration employs a vertical strut connecting all braces of a chevron bracing system, the behaviour being identified by the bucking failure of all braces on one side of the frame and non-occurrence of tension yielding.

Several studies are available in literature regarding the capacity evaluation and performance assessment of CBFs. The design provisions with reference to EC8 for CBFs with X-bracings are critically reviewed [2]. It is recommended that the code include a comprehensive force-displacement model for brace members and define ultimate conditions for the complete definition of behaviour factor(q). The influence of the brace-intercepted beam stiffness on the overall performance of Chevron CBFs is investigated [3]. A design methodology for zipper-braced frames with an objective of achieving a ductile behaviour is proposed [4]. The study highlighted the importance of zipper struts in achieving a uniform damage over the height of the structure. Seismic performance assessment of CBFs to improve understanding their behaviour and identify more efficient analysis procedures is carried out [5]. The modelling aspects that govern the seismic response of steel CBFs are studied in detail [6]. The study was based on experimental investigation along with numerical simulations.

The paper presents seismic evaluation of an eight-storey steel structure provided with four configurations of CBF namely diagonal, X, chevron and zipper. The configuration models being identified by the markings 3-D, 3-X, 3-V & 3-Z respectively. The member design calculations are performed using Eurocode 3 (EC3) [7] with the seismic demand and overstrength requirements adopted from EC8. A nonlinear static (pushover) analysis is first performed to estimate the capacity and ductility associated with each configuration. Nonlinear dynamic time-history analysis is then carried out to assess the inter-storey drifts and associated performance levels. Based on the findings, the salient aspects in the seismic behaviour of the adopted configurations are discussed.

II. DESCRIPTION OF THE BUILDING MODEL

The building model used for the present study is an eight-storeyed steel structure with plan dimensions of 15mx15m. The storey height is 4.0m for the first floor and 3.0 m for other floors. The structure uses moment resistant frames (MRF) in one direction and CBFs in the other direction provided along the periphery. Plan bracings were provided at all floor levels to ensure a rigid floor diaphragm action. Fig.1 shows the typical building floor plan. Four configurations of CBFs used for the present study are indicated in elevations taken along the braced bay and shown Fig. 2.

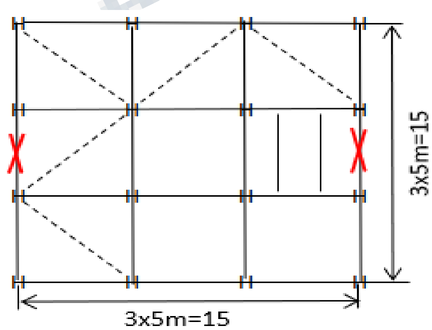

Fig. 1 Building floor plan

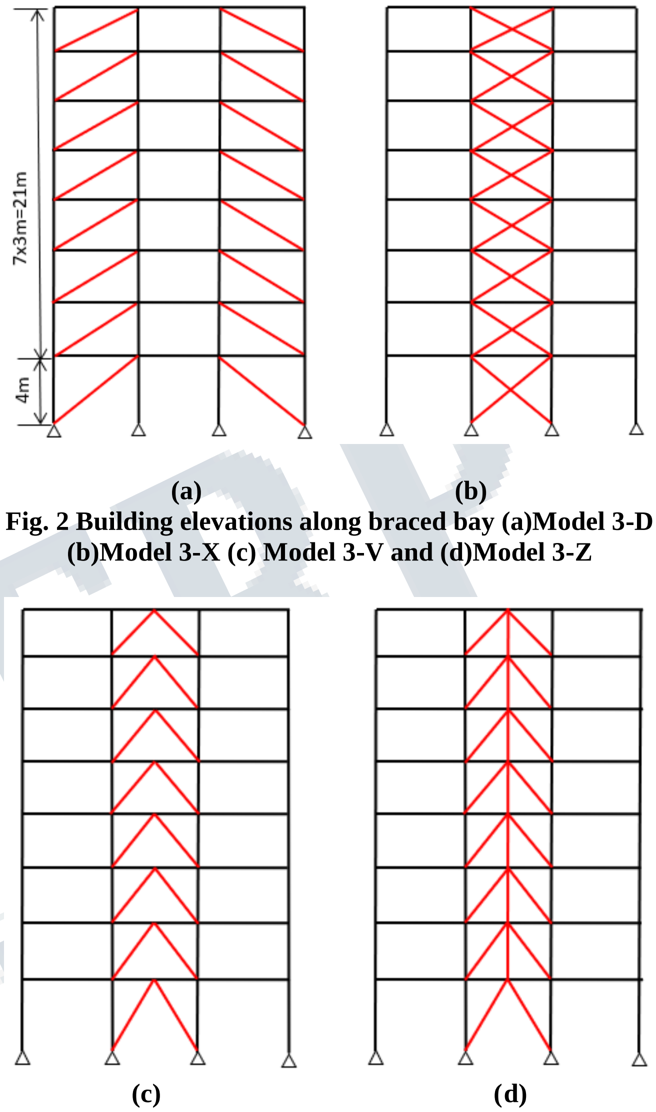

Fig. 2 Building elevations along braced bay (a)Model 3-D (b)Model 3-X (c) Model 3-V and (d)Model 3-Z

III. SEISMIC DEMAND CALCULATIONS

Seismic demand on the structure is calculated as per the regulations given in EC8. Ultimate limit state corresponding to "No-collapse requirement" is checked using seismic action associated with a 10% probability of exceedance in 50 years (return period=475 years). Ground type 'B' is considered. Assuming Type-1 elastic response spectra & soil factor S=1.20, the corresponding spectral periods are TB=0.15s, TC=0.50s & TD=2.0s. The reference peak ground acceleration on type A ground is taken as agR=0.25g. The importance factor is assumed as I=1.0. Viscous damping of 5% is considered for the spectra computations. The values of

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behaviour factor (q) are taken as 4.0 for models 3-D & 3-X and 2.0 for model 3-V considering medium ductile class (DCM) in all models. In the case of zipper arrangement, the code do not provide specific values for behaviour factor(q). Therefore, a q-value of 4.0 is assumed for the model 3-Z to be verified during the assessment stage. The corresponding elastic and design spectra are shown in Fig. 3.

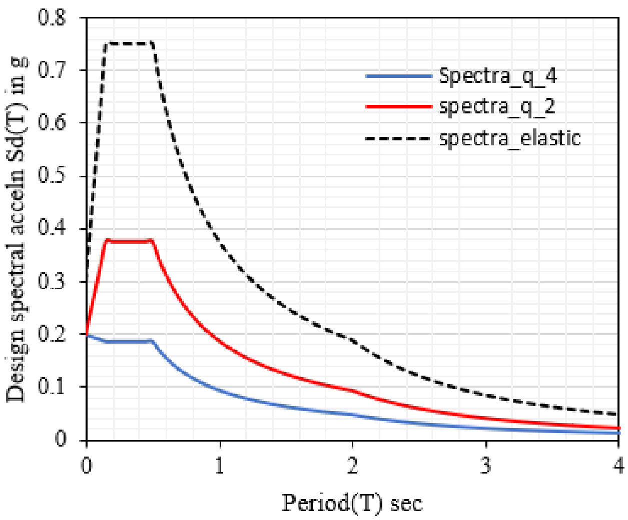

Fig. 3 Elastic and design spectra (EC8)

All brace members are assumed to be pin connected to the framing members and column ends hinged. The European member profiles are used in the calculations with steel of grade S275. The braced frames are idelized as 2-D frames. For all calculations, the finite element program SAP2000 [8] with capabilities of nonlinear static and dynamic time hiostory analysis is used. A dynamic response spectrum analysis is performed to obtain the natural periods of vibration,mass participation ratios and the base shears. The summary of results is presented in Table 1. The dynamic characteristics of all braced configurations are observed to be almost the same. The CBF arrangement 3-V shall be deisgned for a larger base shear force due to an associated limited ductility.

Table 1:Results of dynamic spectrum analysis

Model	Mode	Period(s)	Mass Participation(%)	Base shear(kN)
3-D	1,2,3	0.74,0.27,0.14	73,20,5	271.9
3-X	1.2.3	0.85,0.27,0.14	69,24,5	251.1
3-V	1,2,3	0.83,0.31,0.15	72,23,4	523.8
3-Z	1,2,3	0.76,0.27,0.14	72,23,4	282.2

IV. DESIGN CRITERIA FOR DISSIPATIVE STRUCTURES AS PER EC8

The CBFs in the present study are designed as per the requirements laid out in chapter 6 of EC8 for dissipative structures. This requires identifying, designing the dissipative zones & ensuring their occurrence at predefined locations and avoidance of elastic instabilities at non-dissipative zones. Some important design criteria used in the present study are briefly mentioned.

The code permits the designer to use either medium or high ductility classes (DCM or DCH) along with the reference behaviour factors (q). For DCM considered in the study, all ductile members in the dissipative zones should be of class 1 or 2, section class defined in EC3. In the analysis of any type of CBFs, both the tension and compression diagonals can be considered provided a nonlinear static (pushover) or dynamic time history analysis is used and that both the pre-buckling and post buckling situations are considered in the modelling behavior of diagonals.

In frames with X diagonal bracings, the non-dimensional slenderness □□should be limited to 1.3<□□2.0 and for V bracings □□<2.0, the upper limit controls the deformations due to post buckling of braces in compression and the lower limit the internal forces in columns to which the braces are connected. The code also puts limitations on the variations in the values of overstrength ratio defined as ratio of brace design force to its plastic resistance. In order to ensure a homogeneous dissipative behaviour of brace members, the ratio of maximum and minimum overstrength (□max/ □min) should not be more than 1.25. Having designed the braces and computed the overstrength factors □□the capacity design criteria requires that beams and columns be sized for a force given by NplRd > NEdG+1.1□ov□□EdE , where NplRd is the design buckling resistance of beam or column calculated in accordance with EC3, NEdG axial force in the column or beam due to non-seismic action, NEdE axial force in the column or beam due to seismic action,□□ov is overstrength factor and □ is minimum value of □i computed of all diagonals of braced system.

In frames with V bracings, the code stipulates that the beams be designed to resist all non-seismic actions without considering the intermediate support given by the diagonals. The beams shall also be designed to resist the unbalanced vertical seismic action effect after buckling of the compression diagonal. This action effect shall be calculated using plastic resistance NplRd for the braces in tension and 30% of NplRd for braces in compression.

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V. NONLINEAR STATIC (PUSHOVER) ANALYSIS

In nonlinear static analysis popularly knowns as pushover analysis, a detailed mathematical model representing the nonlinear load-deformation characteristics of the structure is subjected to a progressive step by step increase in the lateral forces till a target displacement is reached. Such an analysis generates a capacity curve that represents the relationship between the applied lateral force and displacement at the specified control point. The calculated internal forces and displacements in structural members will be reasonable approximations of those expected during the design earthquake. The pushover analysis is capable of providing the sequence of yielding in various elements and the final mode of global structural failure.
In the present study all the four CBFs model are subjected to a conventional pushover analysis using finite element software SAP2000 using the displacement-based method. The generalized component force - deformation relations for modelling the inelastic behaviour are taken from ASCE standard ASCE/SEI 41-17 [9] and shown in Fig. 4. For braces in tension, the yield strength and corresponding deformations are used, whereas in compression the calculated buckling strength along with respective deformations are used. The load pattern used for the pushover analysis corresponds to the precalculated earthquake forces obtained by dynamic response spectrum method.

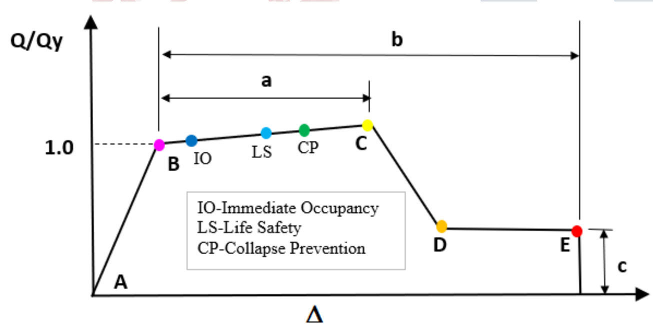

Fig. 4 Generalized force-deformation relation (ASCE 41-17)

The pushover analysis indicates that all four models of CBFs behaved as envisaged during design in terms of sequence of plastic hinge formation and the global yield behaviour. The status of plastic hinges observed during the last step are given in Fig. 5 and the pushover curves in Fig 6. Salient points are marked such as point-1 indicating the formation of first plastic hinge, point-2 corresponding to an interstorey drift of 2% and point -3 representing ultimate capacity of the system. Where a storey drift of 2% could not be reached, point-2 is not shown. The colour- code attached with the hinges corresponds to various performance levels shown in Fig. 4. Models 3-D & 3-X shows a similar behaviour with compression diagonals first buckling progressively from the base towards the top followed by yielding of tension braces. The model 3-V is designed for a larger seismic demand and thus resulting in larger strength and stiffness. The behaviour shows the progressive buckling of compression members followed by yielding of tension members. No hinges are observed in brace intersecting beams which are designed for unbalanced vertical force resulting from buckling and yielding of braces. The addition of zipper columns as shown in model 3-Z is found to improve the overstrength and deformation capacity as compared to model 3-V. A uniform pattern of hinges is observed in buckled members on one side of zipper column.

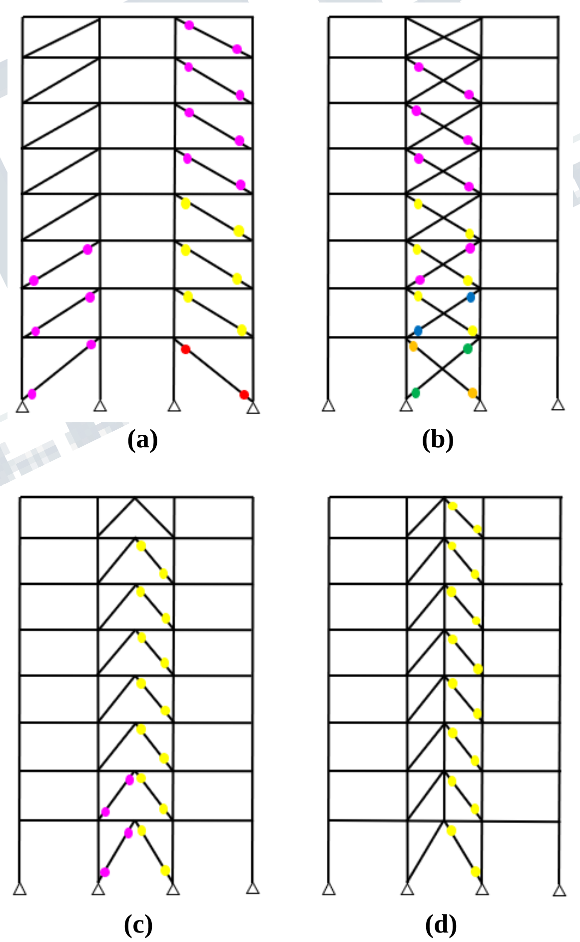

Fig. 5 Pattern of plastic hinge formation(a)Model 3-D (b)Model 3-X (c) Model 3-V and (d)Model 3-Z

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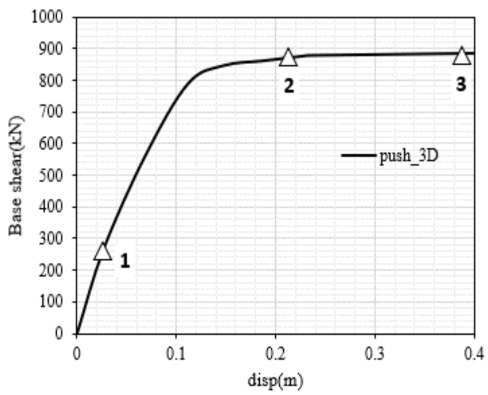
A review of pushover curves show that the models 3-D,3-X and 3-Z though possess the displacement ductility, the occurrence of 2% storey drift at relatively smaller displacements provides limited useful ductility to models 3-D and 3-X compared to 3-Z. However, the models 3-D & 3-X possess higher overstrength as compared to 3-Z. The model 3-V provides least overstrength.



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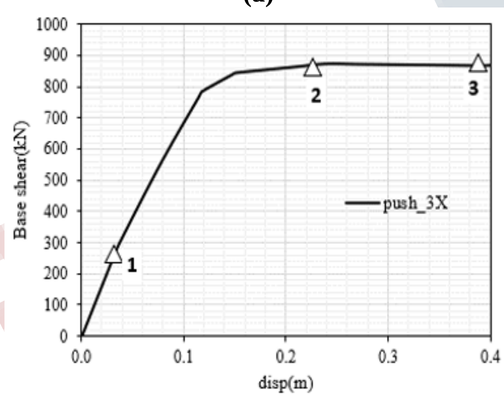

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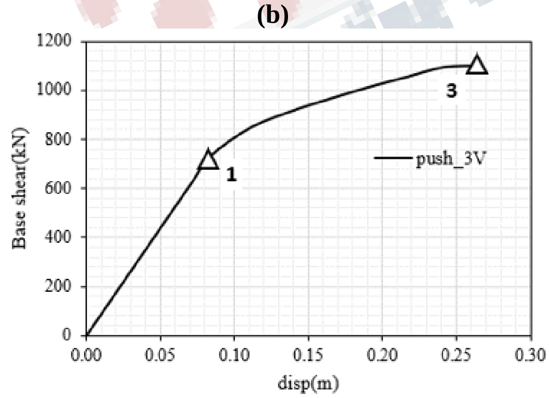

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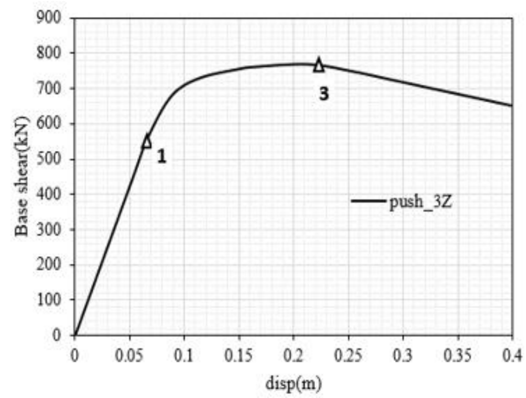

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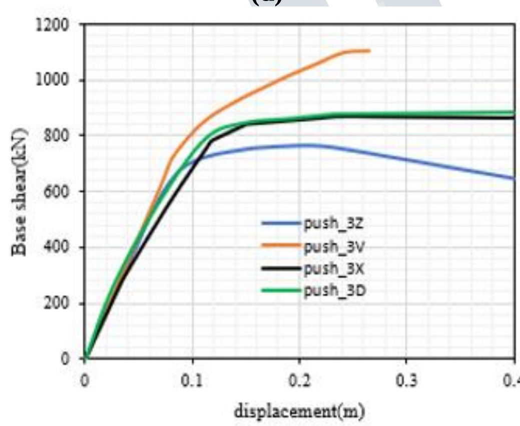

(e)

Fig. 6 Pushover curves (a)Model 3-D (b)Model 3-X (c) Model 3-V and (d)Model 3-Z(e)Combined

VI. COMPUTATION OF BEHAVIOUR FACTOR

The behaviour factor(q) represents the factor used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures [1]. In the present study, the procedure given in ATC-19[10] is used for the calculation of "q" factor using the parameters derived from already performed nonlinear static analysis. The behaviour factor q (R factor in ATC) is represented as

$$R = R_{\Omega} R_{\mu} R_{\rho} \qquad (1)$$

The design reserve strength factor R_{Ω} is defined as the ratio of first yielding strength (V_y) to the design strength (V_d) of the structure. The redundancy factor R_{ρ} can be calculated as the ratio of the ultimate strength (V_u) to V_y. The ductility factor R_{μ} can be readily obtained from the displacement

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ductility (μ) defined as the ratio of ultimate displacement (Δ_u) to yield displacement of the structure (Δ_y). Using the simple version of N2 method, a bi-linear spectrum for the factor R_μ can be obtained as given below.

$$R_\mu = (\mu - 1)\left(\frac{T}{T_c}\right) + 1 \qquad T < T_c \qquad (2)$$

$$R_\mu = \mu \qquad T \geq T_c$$

Where T_C is the characteristic period of ground motion.

The EC8 do not specify the failure mode in terms of drift criteria corresponding to which the values of behaviour factor-q are recommended for different structural systems. For the present study an inter-storey drift of 2% or that corresponding to the global ultimate capacity is used as the criteria for the evaluation of q-values. The summary of calculations is given in Table-2. The calculated q-values are always higher than code specified values. For the model 3-V, EC8 grossly underestimates the q-values despite the stringent criteria provided for their design. Of the four CBFs investigated, the zipper configuration 3-Z with higher q-values represents an efficient CBF and chevron configuration 3-V with the lowest q-values provides a lesser dissipative structure. Component calculations for q-values indicate that the models 3-X and 3-D possess a higher overstrength, whereas the zipper 3-Z is characterized by a larger useful displacement ductility

Table 2: Computed values of behaviour factor(q) for different CBFs

Model	Code specified q-values(DCM)	q-values used	Computed q-values	Percentage difference (%)
3-D	4	4	6.67	66.7
3-X	4	4	7.02	75.5
3-V	2	2	5.06	153
3-Z	-	4	9.49	137

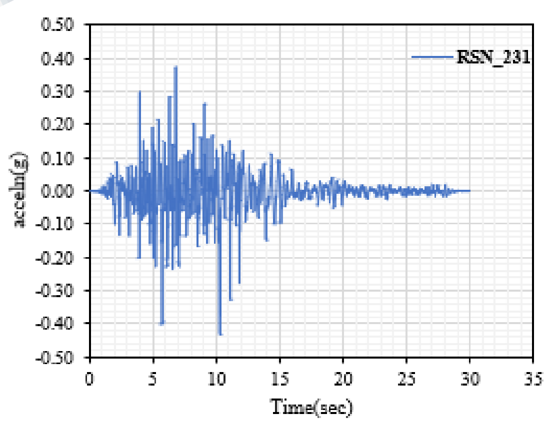
VII. NONLINEAR DYNAMIC TIME HISTORY ANALYSIS

A nonlinear dynamic time history analysis is performed to verify the capacity curves obtained from pushover analysis and to arrive at the performance levels. EC8 provides two options for performing time history analyses. In one option, a set of minimum seven ground motion records shall be used and the average of the response quantities from all of these analyses shall be taken as the design value. As an alternate to the above, a set of minimum three ground motion records can also be used in which case the maximum of the response quantities from all of these analyses shall be taken as the design value. For the present study, a set of four ground motion records with small mean squared error with respect to the elastic target spectrum is selected from PEER (Pacific Earthquake Engineering Research Centre) Strong Ground Motion Database. The selected ground motions are scaled such that in the range of periods between 0.2T1 and 2T1 (T1 is fundamental period of the structure in the direction of ground motion application), no value of the mean 5% damped elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damped elastic response spectrum [1]. The most unfavourable value of the response quantities among the analyses are chosen for calculation of earthquake action effects. Table 3 provides the detailed information of selected ground motion records. The ground motion records are shown in Fig. 7. The elastic response spectra of selected ground motions along with their mean and the target spectrum as per EC8 are shown in Fig. 8.

The seismic performance of CBFs is evaluated in terms of inter-storey drift ratio (IDR). The peak base shear demand and maximum IDR obtained from non-linear dynamic time history analysis is summarized in Table 4. The variation of IDR along the structure height for various configurations of CBFs for the considered ground motions is given in Fig. 9.

Table 3: Groundmpotion details

Record seq.no.	Scale factor	Event	Year	Mag	Duration(s)	PGA(g)
231	1.3503	Mammoth Lakes-01	1980	6.06	29.995	0.4303
289	1.6725	Inpinia,Italy-01	1980	6.9	35.2128	0.1262
735	1.9474	Loma Prieta	1989	6.93	39.99	0.157
739	1.319	Anderson dam	1989	6.93	29.995	0.2465



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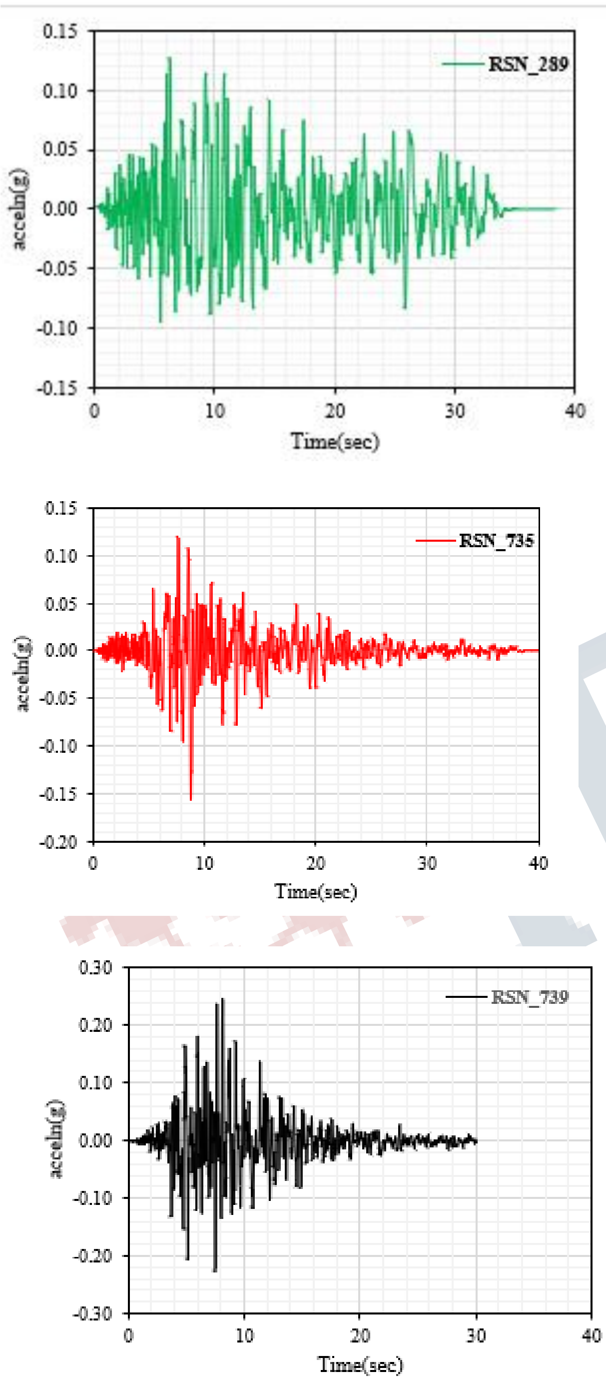


Fig. 7 Ground motion records

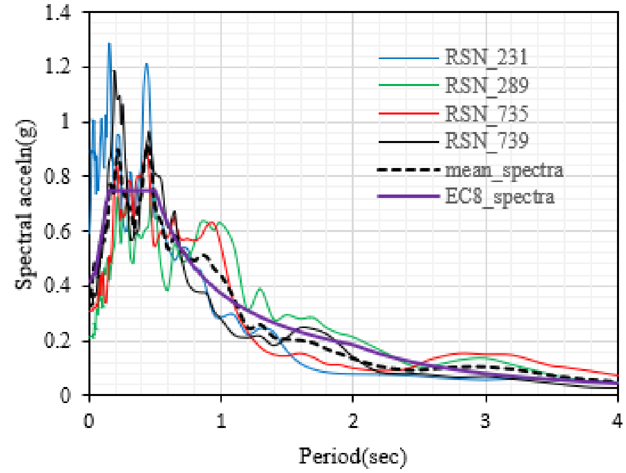
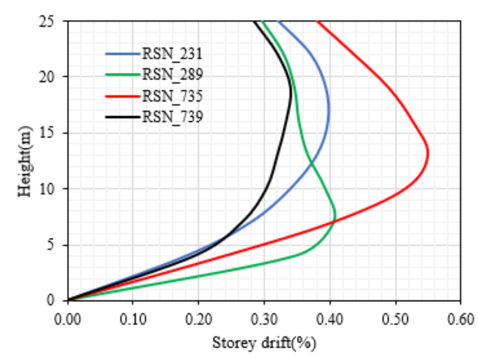


Fig. 8 Response spectra for scaled ground motions

Table 4: Results of nonlinear dynamic analysis

CBF	Record Seq.no.	Base shear(kN)	Peak disp.(mm)	IDR(%)	Dyn.base Shear(kN)	Dyn. IDR(%)
3-D	231	706.7	81.75	0.397	711.8	0.55
	289	705	88.8	0.407		
	735	711.8	108.9	0.550		
	739	706	72.8	0.340		
3-X	231	707.1	103.4	0.473	711.5	0.65
	289	708.4	110	0.500		
	735	711.5	121.3	0.650		
	739	682.2	86.7	0.427		
3-V	231	918.4	89.8	0.510	918.4	0.68
	289	902.2	95.3	0.497		
	735	902.2	120.2	0.680		
	739	794.8	82.76	0.493		
3-Z	231	865.9	92.1	0.497	941.9	0.693
	289	918.1	97.3	0.493		
	735	941.9	120.9	0.693		
	739	739.1	83.9	0.473		



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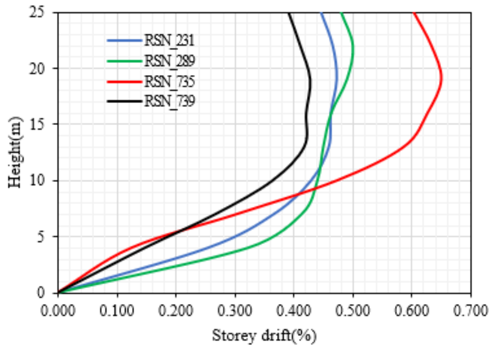

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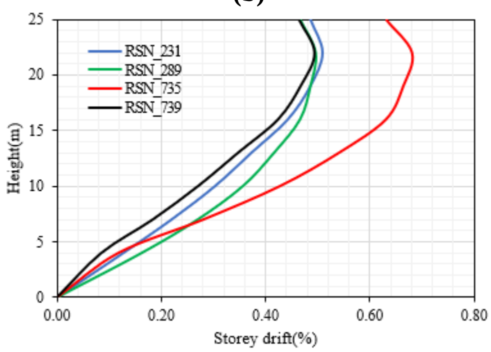

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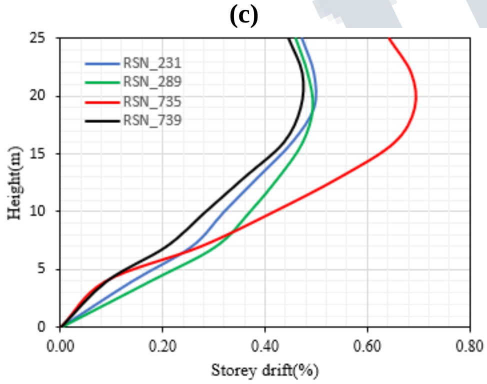

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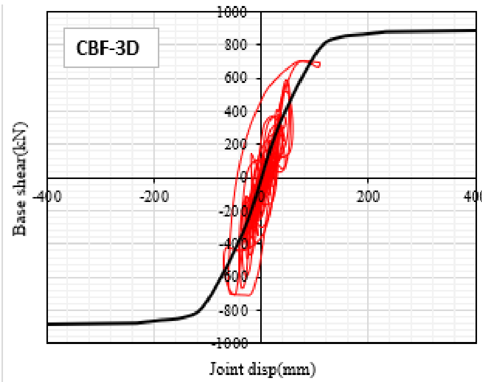
Fig. 9 Interstorey drift curves for CBFs (a)Model 3-D (b)Model 3-X (c) Model 3-V and (d)Model 3-Z

EC8 permits the calculation of inter-storey drifts using the same seismic action intended for ultimate limit state requirements. However, the calculated drifts shall be reduced considering the lower return period of the seismic action associated with the damage limitation requirement. A reduction factor of 0.50 is used in the present study. Accordingly, as per EC8 the storey drifts shall be limited to 1.0 % assuming that the building consists of non-structural elements of brittle material attached to the structure. The



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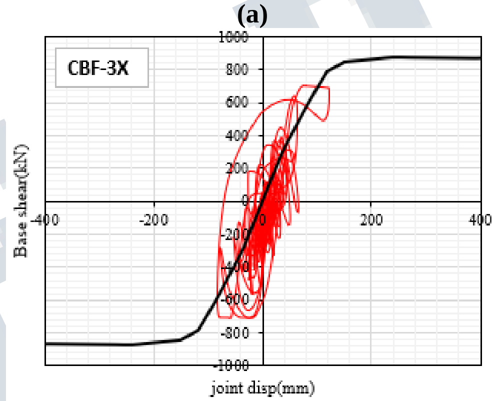

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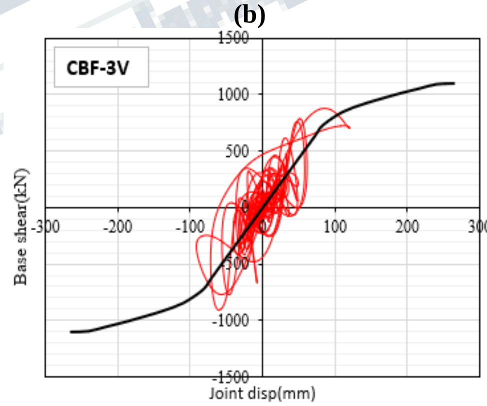

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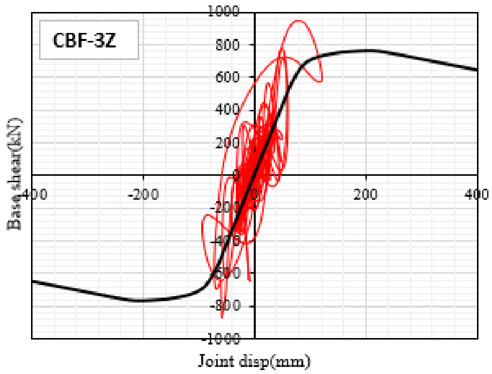

(d)

Fig.10 Hysteretic Vs pushover curve for CBFs subjected to record RSN_735 (a)Model 3-D (b)Model 3-X (c) Model 3-V and (d)Model 3-Z

maximum calculated drifts are found to be within the permissible limit for all the CBFs studied.

For a global performance assessment of the CBFs under the governing earthquake record namely RSN_735, the hysteretic curve for each CBF is plotted against the corresponding capacity curve obtained from pushover analysis. These curves are shown in Fig. 10. It can be seen that for all CBFs, the behaviour is essentially contained in the elastic range.

VIII. CONCLUSIONS

Siesmic assessments are perofrmed on four configurations of CBFs using the nonlinear static (pushover) analysis followed by nonlinear dynamic time history analysis. The CBFs are assessed for a ductility class DCM.All the CBFs are found to have similar dynamic characteristics such as the period and modal paraticipation factors. The assessments are done using EC8 provisions.

A review of the puhover curves along with the behaviour factor calculations indicate that the diagonal and X- braced CBFs are superior in terms of their overstrength whereas the zipper arrangement provides better ductility characteristics.The Cheveron configuration possess minimum overstrength. In terms of q-values, which takes into account both the overstrength and ductility characteristics, the zipper configuration is the most preferred followed by diagonal and X-braced.The chevron configuration results in heavy member sizes as they are designed for a higher seismic force to account for limited ductility.Further the requirement of brace connecting beams to withatand the unbalanced forces from brace members make them stocky. In all CBFs, the yielding is observed only in brace members. In the case of zipper configuration, only the brace members on one side of the zipper undergoes progressive yielding . No yielding is observed in zipper columns. In all cases, it is observed that the limits imposed by EC8 on non-dimensional slenderness ratio improve the reserve strength of the structure.

The study emphasizes the importance of adhering to the code requirements for both the dissipative and non-dissipative components. A properly designed steel CBF with a preconceived hierarchy of hinge formation shall be subjected to both nonlinear static and time history analysis to ensure that both the local yielding and global failure mode are in accordance with the design objectives.

Calculations of behaviour factor from the pushover analysis indicate that q-values given in EC8 for all CBFs studied are highly underestimated. The calculations show that q-factors range from 5.0-9.5 with the lowest for Chevron and the highest for zipper. In particular, the study highlights the necessity to review the low q-values in the code assigned to the chevron system in spite of stringent provisions relevant to its design. Currently the code does not specify q-value for zipper system which is found to exhibit a better performance among all CBFs studied. Therefore, a recalibration of q-values provided in EC8 is required to arrive at a better balance between design and economy.

A nonlinear dynamic time history analysis for the recorded ground motions selected in accordance with the code requirements shows that the calculated inter-storey drift demands are well within the code specified values. The base shear-displacement hysteresis plot indicates that all CBFs are essentially in the elastic zone under spectrum compatible recorded ground motions

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