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SEISMIC PERFORMANCE OF SPECIAL TRUSS MOMENT FRAMES WITH X-BRACING TYPE: INFLUENCE OF BUILDING HEIGHT

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Abstract

This study investigated the implementation of Special Truss Moment Frames (STMF) X-braced type in accordance with the recommendations of the National Earthquake Hazards Reduction Program (NEHRP), 1997:2000 and Indonesian National Standards (SNI) 1726 where it is stated that the STMF may only be used up to a maximum building height of 48 m (160 feet) for Seismic Design Category of D. The analyses compared the seismic response of apartment buildings using STMF system with different height: 44 m (11 stories), 48 m (12 stories), and 52 m (13 stories). The nonlinear behaviors were investigated through a series of nonlinear time history and pushover analyses, including the inelastic response of the structural components, top-roof displacement, drift ratio, and ductility. The structural modeling and the selection of steel sections were performed in SAP 2000 based on SNI 1726 code, considering that the structure was built on medium soil in Aceh region. Seismic performance was examined through a series of pushover and non-linear time history analysis (NLTHA) using three scaled accelerogram: Elcentro 1940, Northridge 1994 and Kobe 1995 records. The results of the study show that in case of STMF structure with the height of 52 m (higher than the maximum height of STMF structure stated in the codes), the seismic performance of the structure was excellent and met the requirements. Axial plasticization occurred only in X-braces, not followed by the formation of plastic hinges at the ends of the ductile segments. Meanwhile, the other structural components remained elastic. This indicates that the structure can still be used after the earthquake, by replacing only the X-braced components. The drift ratio met the requirements and the roof displacements behavior was asymptotic, indicating that there is no residual horizontal displacement of the building after the seismic loads occurred.

Keywords:

STMF X-braced type;
Seismic performance;
Building heights;
NEHRP;
Infrastructure.

1 Introduction

Long span beam in a moment resisting frame-steel structure is often designed as a truss system. The truss beam system is meant both to carry gravitational loads and to act as a moment resisting frame member against lateral loads.

When compared to a solid beam, a conventional truss beam system has several advantages: simpler connection detail, less in weight to support longer span, and the empty space between the truss members can be used for utility installation.

On the other hand, such a system often results in a plasticisation of the column and the diagonal member during strong earthquakes. It is caused by the large depth of the truss beam, resulting in a stiffer beam when compared to the supporting column [1]. It is well known as a “Strong Beam Weak Column” design, as seen in Fig. 1a, which should be avoided in seismic resistant buildings.

To improve the seismic performance of the conventional truss beam, a modification was proposed by involving additional X-bracing, placed at the ductile panels at the beam mid span [2]. The X-bracing section can be either in the form of a single angle section or a compact rectangular thin

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plate. In such a system, the energy dissipation is initiated by the axial inelastic behaviour of the X-bracing before the generation of plastic hinge at the ends of ductile segments, as shown in Fig. 1b. The other structural members outside the ductile segments should remain elastic. The aforementioned ductile truss system was adopted in AISC Seismic Provisions, 2010 and SNI 1726 as a Special Truss Moment Frames [3, 4].

The following study proposed a Vierendeel panel at the mid span to act as a ductile segment, as seen in Fig. 1c [5]. This segment works as e fuse that dissipates the seismic energy by means of inelastic flexural deformation. During an earthquake, the lateral load generates shear force on the ductile segment, which is carried by the bottom and upper chords. It then results in four plastic hinges and a plastic mechanism of the truss beam is formed. The seismic performance of the Vierendeel system is considered less when compared to STMF X-bracing system.

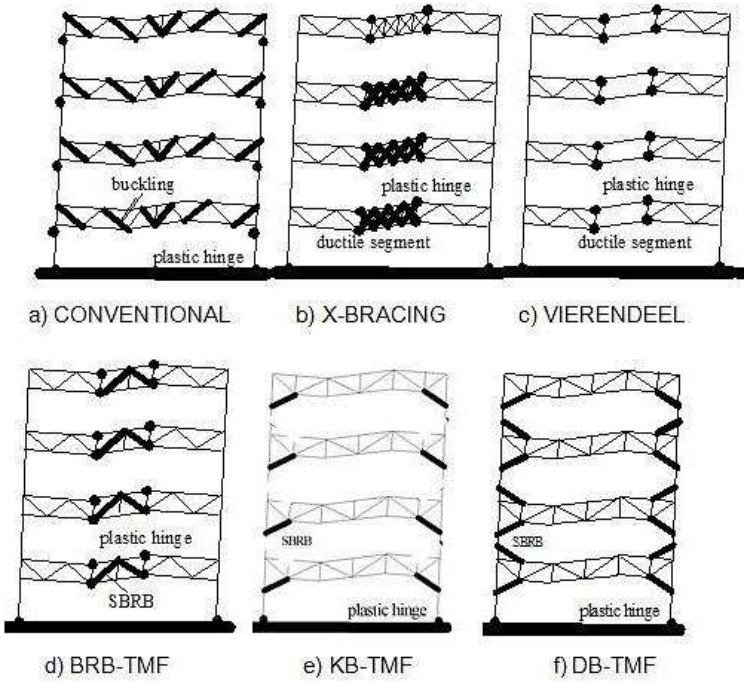


Fig. 1: Development of truss moment resisting frame systems.

A small buckling-restrained braces (SBRBs) was then proposed as a dissipative component, replacing the X-bracing, as shown in Fig. 1d. Analytical and experimental investigation of such a system was reported in [6, 7]. The hysteretic curve shows that in a truss system using SBRB, the cyclic behaviour was fairly stable, even though there was a stiffness degradation at larger cyclic loads. In the analytical study, using a bilinear model for the SBRB, the total energy dissipation was in agreement with the experimental result. An investigation on the performance of STMF-Vierendeel type was conducted, considering different vertical load distribution [8]. It was found that the formula to compute the shear force on the STMF girder as stated in the AISC Seismic Provisions was too conservative. Therefore, a new formula was proposed to obtain more effective design.

A knee braced truss moment frame (KB-TMF) using SBRBs with configuration as shown in Fig. 1e was proposed [9]. The systems showed good seismic performance, which the SBRBs could strain up to five percent. The capacity of STMF system when one of girder-supporting columns experienced progressive collapse was investigated in [10]. The results showed that the STMF structure designed according to AISC Seismic Provisions experienced failure due to the formation of plastic hinges at the ends of ductiled segment. Based on the balanced energy concept, such a failure should be avoided. Another study was conducted to improve the seismic performance pf STMF by adding buckling restrained braces (BRB) at the bottom and upper chords on the beam-column joints [11]. Based on the damage and energy concepts, it was found that such a strategy improved the seismic capacity of the STMF. In order to validate the hysteretic behavior of the SBRB as a fuse in a ductile truss system, an analytic verification was done using Menegotto-Pinto material model [12]. The result showed that its hysteretic curve was in agreement with the experimentation. It also showed a stable behaviour up to 2 % of strain. The cumulative ductility value satisfied the requirement for hysteretic component.

Another configuration of ductile truss system is the use of double SBRB located at both ends of the truss beam as ductile components, as seen in Fig. 1f. Based on an analytical study, such a system could improve the seismic performance of a structure in medium seismicity zone. The dissipation energy was increased, and the drift ratio was reported decreasing. In addition, the roof displacement showed an asymptotic behaviour. The cumulative ductility also satisfied the requirement of hysteretic structure even though there was a trend of pinching in the hysteretic curve [13]. A recent development on the STMF is the application of single and multi-Vierendeel in buildings with various heights [14]. Based on the fragility concept, it was found that the application of multi Vierendeel was beneficial to reduce the collapse probability in case of structures with 3 - 6 stories, while in for structures with 9 - 15 stories, the application of single Vierendeel was more effective.

The present study investigates the implementation of Special Truss Moment Frames X-braced type, considering the recommendations of the National Earthquake Hazards Reduction Program NEHRP, 1997:2000 [15] and Indonesian National Standards SNI 1726, where it is stated that the STMF X-Bracing may only be used up to a maximum building height of 48 m (160 feet) for Seismic Design Category of D. As a matter of fact, in a previous study, it was found that the application of single Vierendeel in STMF system was still effective in the case of 15-story structure (more than 48 m of height) [14]. Therefore, this study investigates the nonlinear performances of a building with the height more than 48 m that uses STMF X-bracing system, in which the ductility may exceed the requirement and the lateral stiffness may be too low. A series of nonlinear time history (NLTH) analyses were carried out in this study using two additional ground motions (Northridge 1994 and Kobe 1995) that contain different frequencies with the ones in the previous study.

2 Structural design and modeling

The study started with the design and analyses of steel structures having height of 44, 48, and 58 m using SAP 2000 software [16]. Special steel truss system with X-bracing type was used in this study and the considered structures are located in medium soil in Aceh region. The next step was to verify the strength capacity of the truss members. Once the required nominal strength of the truss members was obtained, nonlinear time history analyses were performed using three acceleration records: El Centro 1940, Northridge 1984 and Kobe 1985. The analyses were done in DRAIN-2DX software [17]. The structural elements were modelled as nonlinear trusses and beam-columns. After the analyses, the nonlinear behaviors were observed including ductility, drift ratio, and roof displacement.

2.1 Design of STMF system using X-bracing type

The ultimate shear capacity of the ductile segment in Fig. 1b is the contribution of the inelastic behavior of X-bracing and four plastic hinges formed at both ends of the ductile segment. If P_y - tensile yield strength of X-bracing; φ - ratio of post yield buckling to critical buckling of X-bracing; P_{cr} - critical buckling of X-bracing; α - slope of X-bracing to horizontal; M_p - plastic moment capacity of chords; L_s - length of ductile segment, the ultimate shear force of the ductile segment V_p is expressed by equation (1).

$$V_p = \left(P_y + \phi P_{cr}\right)\sin\alpha + \frac{4M_p}{L_s} . \quad (1)$$

The components outside the ductile segment should be always in elastic condition during an earthquake. Thus, an overstrength factor is involved to assure the strength of those elastic components. If R_y - expected yield strength; M_{nc} - nominal bending strength of chords of ductile segment (kN-m); EI - elastic bending stiffness of chords of ductile segment (kNm^2); L - length of girder (m); P_{nt} - nominal tensile strength of X-bracing (kN); P_{nc} - nominal compression strength of X-bracing (kN), the nominal shear strength of the component outside the ductile segment V_{ne} is expressed by equation (2). The other elements such as diagonal chords, vertical chords, and columns can be designed as recommended in [2].

$$V_{ne} = \frac{16.68R_yM_{nc}}{L_s} + \frac{333.6EI(L-L_s)}{L_s^3} + R_y(P_{nt} + 0.3P_{nc})\sin\alpha . \quad (2)$$

The elements in the ductile segment should have compact sections as recommended in AISC Seismic Provisions. In particular, the X-bracing chord built from a single L-shaped section should

satisfy equation (3) to assure the stable hysteretic behavior, where *b* - width of section; *t* - thickness of section; *E* - modulus of elasticity and f_y - yield stress.

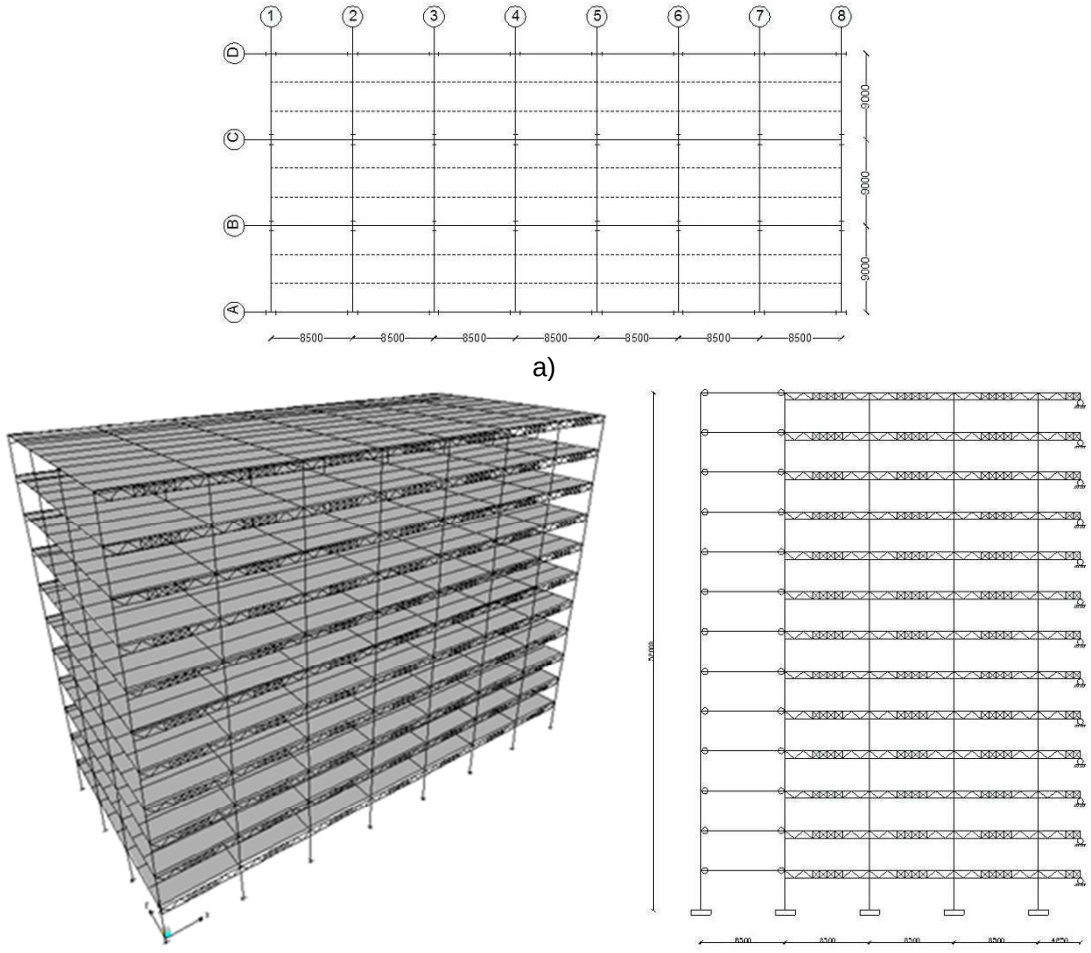
$$b/t \leq 0.18\sqrt{\frac{E}{f_y}}\,. \tag{3}$$

2.2 Numerical model of the structures

Fig. 2a shows the plan of the building under study. The 3D computer model used for response spectra analyses is presented in Fig. 2b. Meanwhile, for NLTH and the pushover analyses, the 2D model as shown in Fig. 2c was used. This 2D approached model is adopted from [2].

The longitudinal section of the building consists of eight columns with the distance between columns is 8500 mm. While, the transversal section consists of four columns with the distance between columns is 9000 mm. The height of each story is 4000 mm, the height of the truss beam is 750 mm, the length of the ductile segment is 3048 mm.

Three building models with different number of stories are considered in this study: 11 stories (44 m of height); 12 stories (48 m); and 13 stories (52 m). The applied live load for such apartment building is 2500 N/m^2 and 1000 N/m^2, for floors and roof, respectively. In order to observe the inelastic behavior of the truss elements, the ductile segment is assigned only in the longitudinal section of the building using four paired X-bracing built from single *L* section in each span.



a)

b) c)

Fig. 2: a) Building plan, b) Three dimensional model of structure with 44 m of height, and c) Analytical half-model of longitudinal frame of the structure with 42 m of height.

The truss elements outside the ductile segment were built from double L steel section, while the columns were built from wide flange (WF) steel section. All structural members were considered connected using weld. The nonlinear time history analyses were performed using spectral response of

Aceh region with medium soil as per SNI 1726, with modification factor of R = 7 and seismic design category of D.

2.3 Steel sections of structural members

The sections of the structural members were defined, based on the requirements expressed in equations (1-3). The results of the section design are presented in Table 1, for three building heights. The section of the ductile segments should be compact to assure the plasticization, which results in thick angle sections for the ductile segments.

Table 1: Steel sections of structural members.

Element	Height of building [m]		
	44	48	52
Upper - and bottom chord	2L 150 × 150 x 19	2L 150 × 150 x 19	2L 150 × 150 x 19
Inner vertical	2L 40 × 40 x 6	2L 40 × 40 x 6	2L 40 × 40 x 8
Outer vertical	2L 50 × 50 x 9	2L 60 × 60 x 8	2L 60 × 60 x 10
Diagonal	2L 70 × 70 x 9	2L 70 × 70 x 11	2L 75 × 75 x 12
X-bracing	L 34 × 34 x 8	L 35 × 35 x 8	L 38 × 38 x 8
Column; storey 1 - 5	WF 400 × 400 × 45 ×70	WF 400 × 400 × 45 × 70	WF 400 × 400 × 45 × 70
Column; storey 6 - 9	WF 400 × 400 × 30 × 50	WF 400 × 400 × 30 × 50	WF 400 × 400 × 30 × 50
Column; storey 10 - 13	WF 400 × 400 × 20 × 35	WF 400 × 400 × 20 × 35	WF 400 × 400 × 20 × 35

2.4 Seismic loads

The nonlinear performances of the building models were investigated, including: inelastic behavior of the elements; inter-storey drift; roof displacement and ductility. The 3D models as shown in Fig. 2b were subjected to ground motion records of El Centro, Northridge, and Kobe which were scaled to 1.225, 0.478, and 0.415, respectively, in order to match the spectral response of Aceh region. The scaling process was done in software and the results are shown in Fig. 3 [18]. It is interesting in the next study to calculate scale factor based on the maximum velocity of ground motion instead of the maximum acceleration as suggested by [19]. For pushover analyses in SAP 2000, the 2D models as shown in Fig. 2c were subjected to incremental lateral displacement to investigate the ductility of the structures.

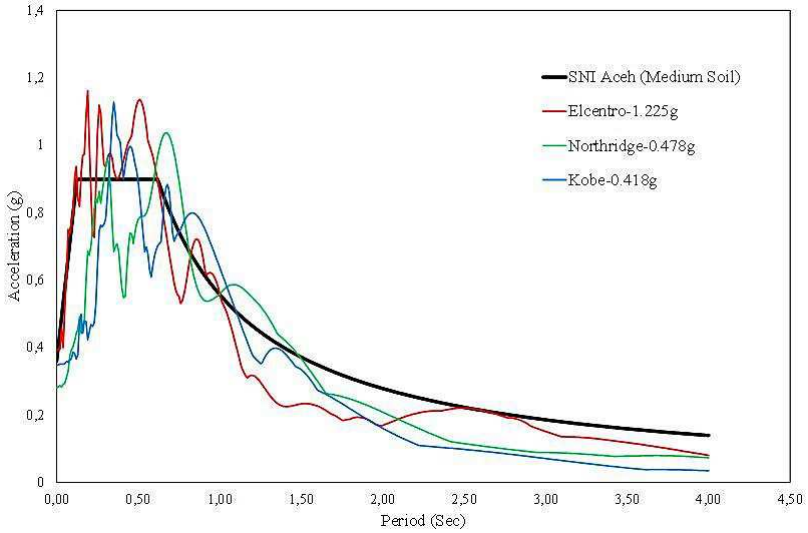


Fig. 3: The targeted spectral response of Aceh region based on SNI 1726 and the scaled spectral responses of the three ground motion.

3 Results

In the model of the structure with 52 m of height subjected to ground motions Elcentro, Northridge and Kobe, the plasticisation (axial yielding or critical buckling) took place only on the X-

bracing components, as shown in Fig. 4a, b and c, respectively. The black-bold lines indicate the yielding elements. In the 1st until 9th storeys, all X-bracings were yielded, while in the 10th until 13th storeys, many X-bracing components remained elastic and the rest yielded in different distribution. The number of X-bracings that remained elastic was caused by the use of identical sections for all X-bracing components along the height of the structures. Regarding the columns and the other components outside the ductile segments, all remained elastic in all earthquake cases. These results indicated that after earthquake, only X-bracing components that need to be replaced.

Regarding the drift ratio of the three structure models (i.e. 44 m, 48 m, and 52 m of height) subjected to three ground motions, all the storeys experienced drift ratio less than 2 % in all cases, as shown in Fig. 5. Such drift ratio limit is stated in the Indonesian code SNI-1726.

The pushover curves of the three building models obtained from the analyses in SAP 2000 are shown in Fig. 6. From those results, the ductility coefficients could be computed: 2.37, 2.74, and 2.44 for the model with 44 m, 48 m, and 52 m of height, respectively, as presented in Table 2. Such ductility coefficients were not directly proportional to the height of the structures due to the fact that all the models used identical sections for the bracing components. However, those ductility values are below the maximum requirement 4.00, as recommended in [4].

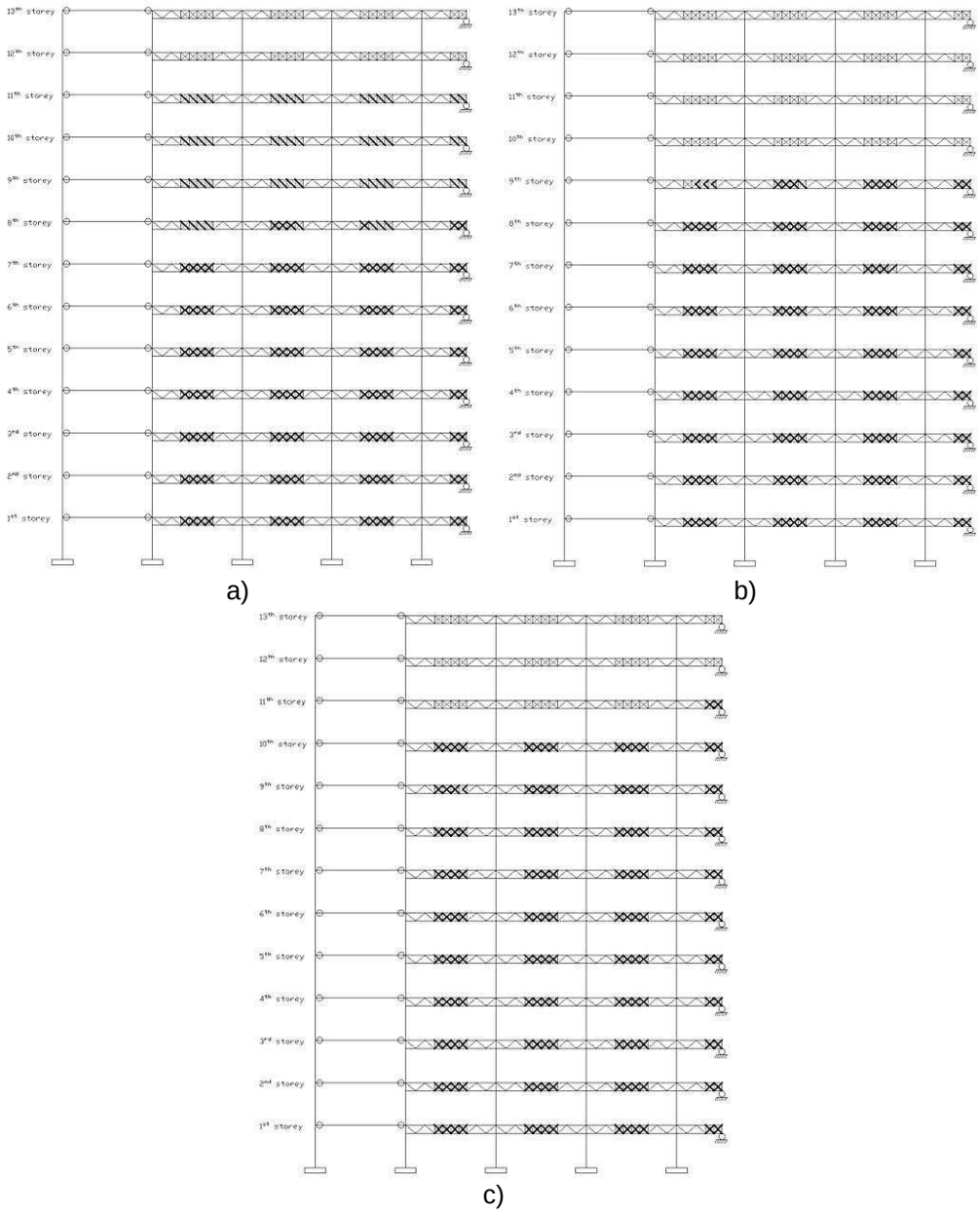
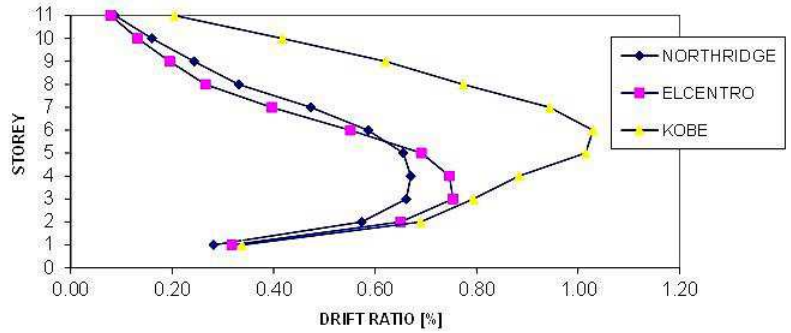
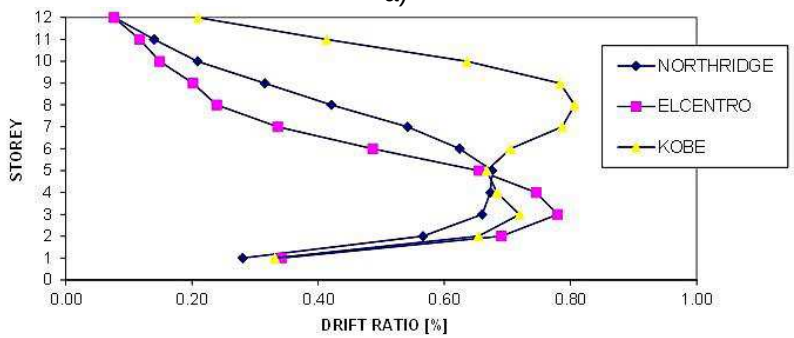


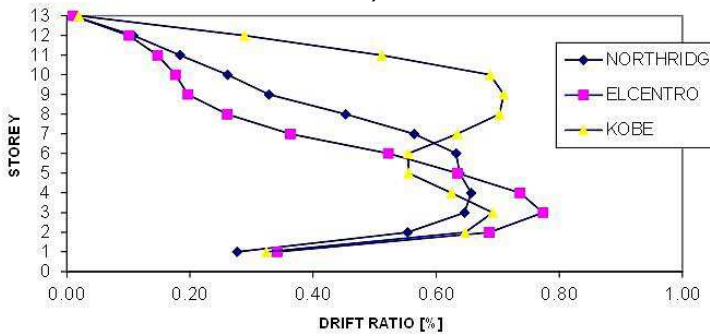
Fig. 4: Plasticisation (tensile yielding or compressive critical buckling) on X-bracings in the structure with 52 m of height due to: a) El Centro, b) Northridge, and c) Kobe ground motions.



a)



b)



c)

Fig. 5: Comparison of the drift ratio of three structures with the height of: a) 44 m, b) 48 m, and c) 52 m undergoing three ground motions.

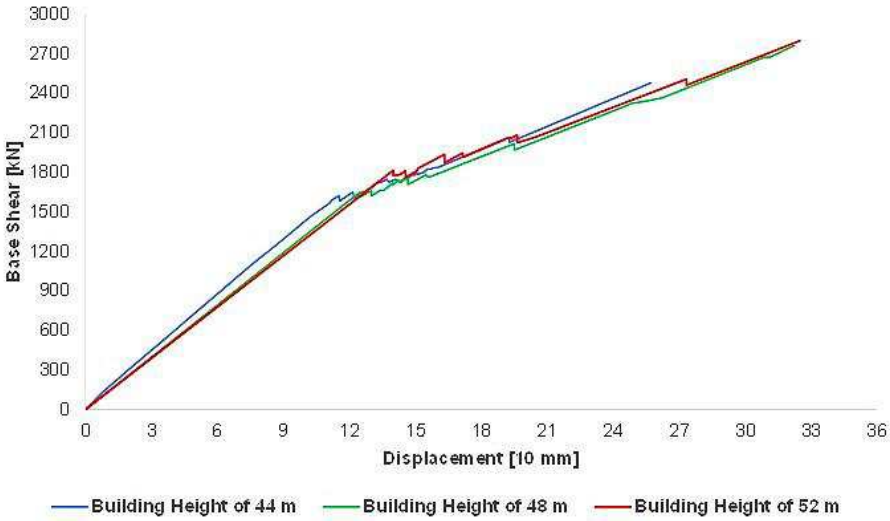


Fig. 6: Comparison of pushover curves of the three structures with the height of 44 m, 48 m, and 52 m.

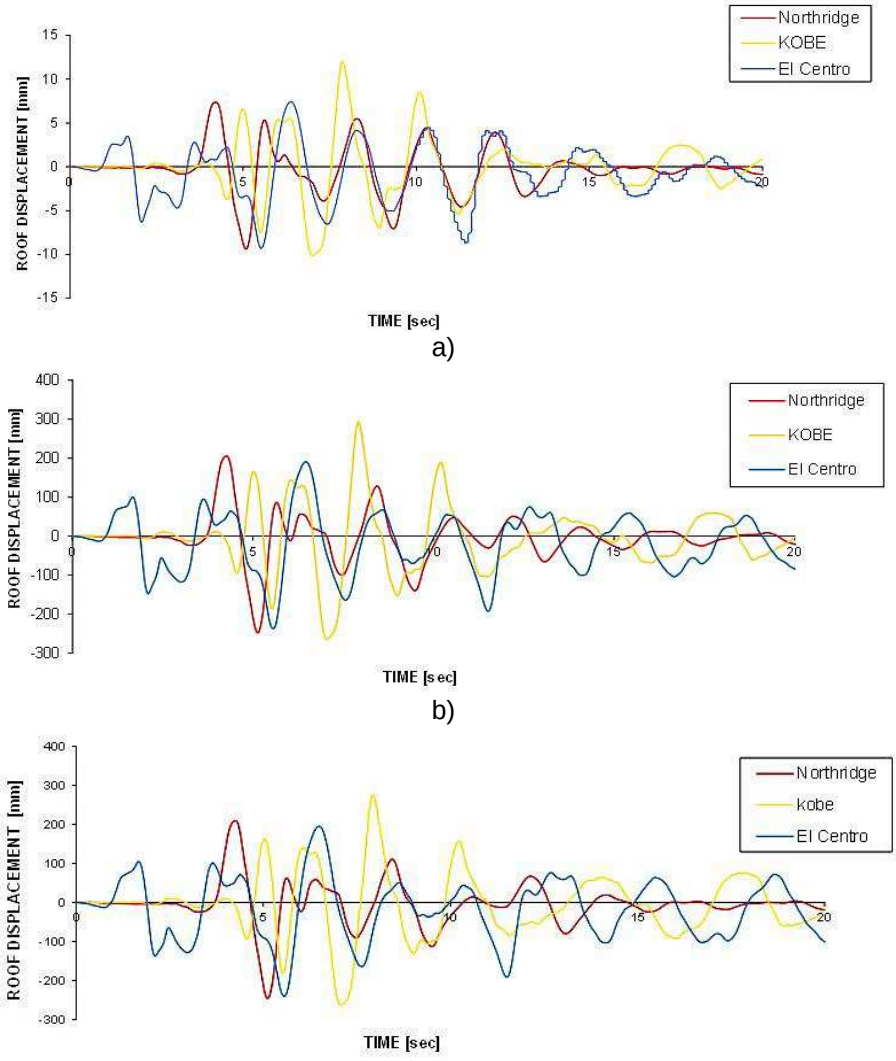


Fig. 7: Roof displacement-time history of the three structures with the height of: a) 44 m, b) 48 m, and c) 52 m undergoing three ground motions.

In Fig. 7, the top displacement-time history of three structure models subjected to the three ground motions are presented. The results indicated that under all selected ground motions, the top displacement showed asymptotic behaviour, in which the structures went back to the initial position after earthquakes. The difference of displacement responses between the three structure models were barely noticed due to the utilization of structural element sections which are proportional to the height of the structures. Fig. 8 presents the hysteretic curve of an X-bracing element in the 2nd storey of the three structure models due to El Centro earthquake. The axial plasticisation took places due to tensile yielding and compressive critical buckling.

Table 2: Ductility factor of the three structures obtained from pushover analyses.

Building height [m]	Ductility factor
44	2,37
48	2,73
52	2,44

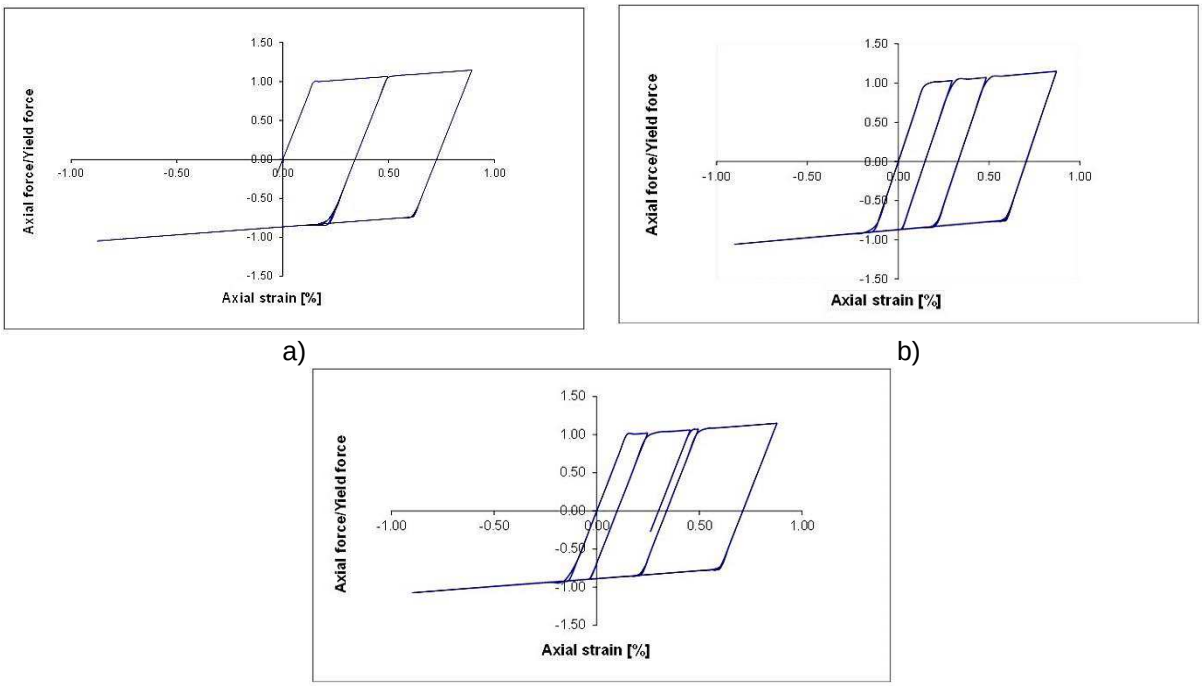


Fig. 8: Hysteretic curves of one of the X-bracing at second floor in the structures with the height of: a) 44 m, b) 48 m, and c) 52 m undergoing El Centro ground motion.

4 Conclusions

Nonlinear analyses on special truss moment frames STMF with X-bracing type have been conducted in this study. Building with different heights, i.e. 44, 45, and 52 m were considered and single angle *L* sections were used for the X-bracing members. Based on the results obtained, the following conclusions could be drawn:

• In the NEHRP 1997 : 2000 and SNI 1726 recommendation, the maximum building height with STMF system is 48 m (160 feet) for seismic design category D. However, based on this study, a 52 m building with STMF system showed a satisfactory performance against the considered earthquakes.

• The plasticization took place only on X-bracing components, without followed by the four plastic hinges at both ends of the ductile segment. The other structural elements outside the ductile segment remained elastic during earthquakes. It may result in an economical structural system, because only X-bracing members that needed to be replaced after the earthquakes.

• The inter-storey drifts due to all considered earthquakes showed a value that is still below the maximum requirement, 2 %.

• The ductility coefficients for buildings with 44, 48, and 52 m of height were found 2.36, 2.73, and 2.44, respectively. Those values satisfy the requirement in SNI 1726, in which the maximum ductility for STMF system is 4.00.

• The hysteretic curves of the X-bracing elements followed the nonlinear behavior of truss, yielding in tension and buckling in compression.

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