International Journal of Civil Engineering

Structure-Stell

Effect of vierendeel panel width and vertical truss spacing ratio in staggered truss framing system under earthquake loads

E. Wahyuni[1,]*, Y. Tethool[2]

Received: August 2013, Revised: January 2014, Accepted: June 2014

Abstract

The purpose of this study is to determine the effect of vierendeel panel width and vertical truss spacing ratio in an inelastic behavior of the STF system due to earthquake loads. The STF system is applied to a six-storey building that serves as apartments [2]. The STF system is used in the building in the transverse direction (N-S direction), while in the longitudinal direction (W-E direction) the building system uses the special moment resisting frame. The structural behavior was evaluated using nonlinear pushover and time history analyses. The results showed that by increasing the ratio of vierendeel panel width and vertical truss spacing, the ductility of the structure was increased. Based on the performance evaluation, the ratio of the vierendeel panel width and vertical truss spacing on the STF buildings that provided satisfactory performance was more or equal to 1.6. The ultimate drift obtained from non-linear time history analysis was smaller than the pushover analysis. This result showed that the static nonlinear pushover analysis was quite conservative in predicting the behavior of the six-storey building in an inelastic condition.

Keywords: *staggered truss framing, Static nonlinear pushover analysis, Nonlinear time history analysis, Earthquake.*

1. Introduction

A Staggered Truss Framing (STF) system consists of a series of structured trusses, with an opening in the middle of truss span (vierendeel panel) that serves as a corridor on the floor [1]. For more information about STF system can be seen in Figure 1. The STF system was efficiently used for mid-height buildings such as apartments, hotels, flats, hospitals and other structural systems that required low height between the floors. The staggered-truss framing system is one of the only framing system that can be used to allow column-free areas sized 18 to 21 meters, thus the structural system provided the freedom for architects to set the floor function [1, 2]. Furthermore, this system is normally economical, simple to fabricate and erect, and as a result, is often cheaper than other framing systems. Truss elements of the STF system require an opening space in the middle span (vierendeel panel) that serves as a corridor with the sufficient ratio of the width and height. According to Tethool and Wahyuni [2] the vierendeel panels has an important role in the plasticization process of STF system that influences the collapse process.

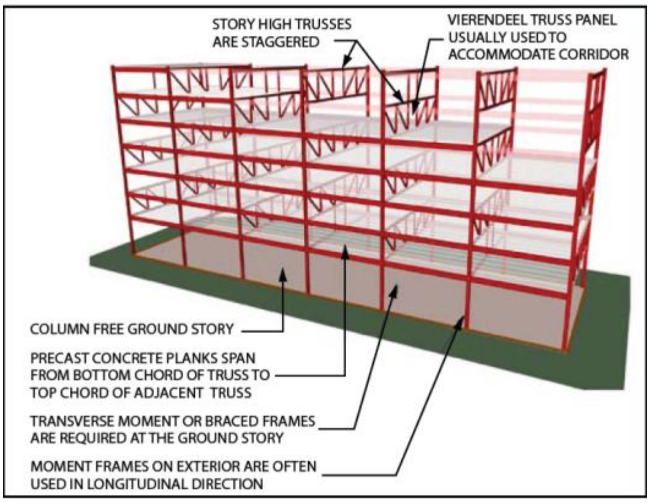


Fig. 1 Staggered Truss Framing System [1, 7]

Kim and Lee [3] studied the behaviour of the 4-, 10-, and 30-storey STF by pushover analysis and are compared with concentric braced frame and moment-resisting frames. The STF system showed superior or at least equivalent seismic load-resisting capacity to convensional ordinary concentric braced frames. For low-rise structures with STF turned out to have enough seismic load-resisting capacity, however, in mid- to high-rise structures, localization of plastic damage in vierendeel panel caused week storey and resulted in brittle failure of structure. To explore the vierendeel panel behaviour, the purpose of this study is to determine the effect of ratio vierendeel

* *Corresponding author: endah@ce.its.ac.id*
1 PhD Sepuluh Nopember Institute of Technology, Civil Engineering Department, Campus ITS Sukolilo, Surabaya 60111, East Java, Indonesia
2 MSc Sepuluh Nopember Institute of Technology, Civil Engineering Department, Campus ITS Sukolilo, Surabaya 60111, East Java, Indonesia

[DOI: 10.22068/IJCE.13.2.213]

panel width and vertical truss spacing in the inelastic behaviour of the STF system due to earthquake loads.

2. Nonlinear Pushover and Time History Analyses

Performance evaluation of a structure can be solved by four different analyzes namely a linear static, dynamic, nonlinear static and dynamic analyses [4]. Nonlinear analysis (push over analysis) is used to accommodate the post-yielding behavior of the structure, where the applied loads are gradually increased by a factor until one lateral displacement target of the reference point is reached. The result of the nonlinear static pushover analysis is a curve that describes the relationship between the base shear forces and the displacement at reference point on the roof [2, 3]. The nonlinear time history analysis is a way to determine the dynamic responses of structures that have nonlinear behavior caused by earthquake ground motion. The earthquake ground motion is used as an input data, which is the dynamic response in each time interval calculated by gradual integration method [2]. This study used the El Centro earthquake record (May 18, 1940), Denpasar earthquake record (July 14, 1976) and the Kern County earthquake record (July 21, 1952).

Nonlinear analysis requires plastic hinges modeling to define the nonlinear behavior of the structural elements including beams, columns and bracing. Plastic hinges are assumed to occur at both ends of the beams and columns, and for bracing elements the assumed plastic hinges are occurred in the middle span. The plastic hinge models are used in this study based on FEMA 356 [6] this is built in the SAP2000 program [9].

3. Building Performance Level

Determination of the building performance level is based on the safety level for residents in the building during and after an earthquake toward the damage of the building. The performance level of a building is set as follows [5, 6]:

1. Operational

Building can operate normally. This target Building Performance Level (BPL) are expected to sustain minimal or no damage to their structural and non-structural components. The building is suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with power, water, and other required utilities provided from emergency sources, and possibly with some nonessential systems not functioning. Buildings meeting this target BPL pose an extremely low risk to life safety.

2. Immediate Occupancy (IO)

This target BPL is expected to sustain minimal or no damage to the structural elements and only minor damage to the non-structural components. While it would be safe to reoccupy a building meeting this target BPL immediately following a major earthquake, non-structural systems may not function, either because of the lack of electrical power or internal damage to equipment. Therefore, although immediate re-occupancy of the building is possible, it may be necessary to perform some cleanup and repair and await the restoration of utility service before the building can function in a normal mode. The risk to life safety at this target BPL is very low.

3. Life Safety (LS)

Buildings meeting this level may experience extensive damage to structural and nonstructural components. Repairs may be required before reoccupancy of the building, and may be deemed economically impractical. The risk to life safety in buildings meeting this target BPL is low.

4. Collapse Prevention (CP)

Little residual stiffness and strength, but load bearing columns and walls function. Large permanent drifts occurred. Some exits were blocked and the building nearly collapses. Buildings meeting this target BPL may pose a significant hazard to life safety resulting from failure of nonstructural components. However, because the building itself does not collapse, gross loss of life may well be avoided. Many buildings meeting this level will be complete economic losses.

4. Response Spectra Design

According to Indonesian Standard [7], the response spectrum of the earthquake plans must be done in advance according to the desired location. Based on the standard [7] the bedrock acceleration of Manokwari city, Indonesia is Ss = 1.446 and S1 = 0.553. The classified site of SC is selected based on the hard soil type. Figure 2 displayed the spectral response design for the Manokwari city, Indonesia that is used in this study.

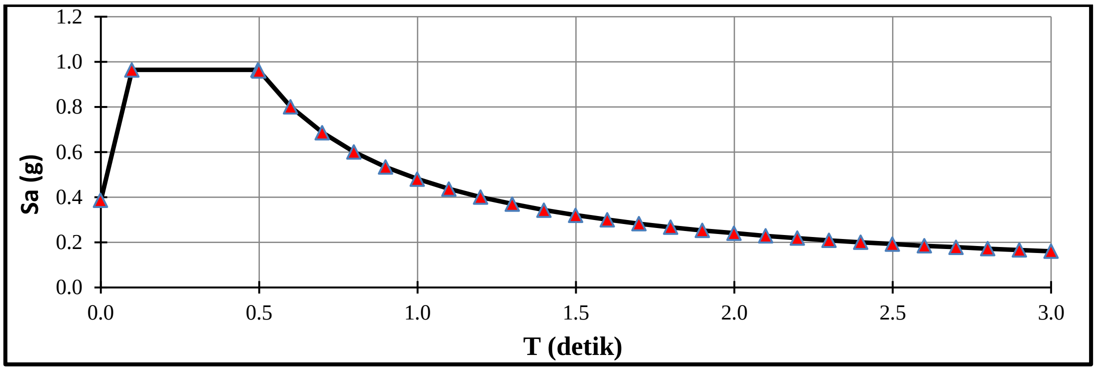


Fig. 2 Design Response Spectra Manokwari According RSNI 03-1726-2010 [6]

[DOI: 10.22068/IJCE.13.2.213]

Based on the Indonesian standard [7], the minimum of three earthquake accelerations should be analysed, thus the earthquake accelerations of El Centro, Denpasar and Kent County were analyzed in this study. The calculations of spectral acceleration and spectral velocity are made by SREL program based on the acceleration data. The earthquake-scale plans of the El Centro, Denpasar and Kent County will be scaled to the spectral response of Manokwari according to the standard [7]. The scale factors of the El Centro, Denpasar and Kent County are 1.133, 2.535 and 2,672 respectively.

5. Staggered Truss Framing System (STF) Models

The building plan with the applied STF system can be seen in Figure 3. The technical data of the building as follows:

- Building width : 21.00 meters
- Building length : 48.00 meters
- Total building height : 20.25 meters (6-storey)
- Quality steel profile : BJ41 (f_y=250 MPa, f_u=410 MPa)
- The profile is used as shown in Table 1 based on Indonesia standard [10, 11].

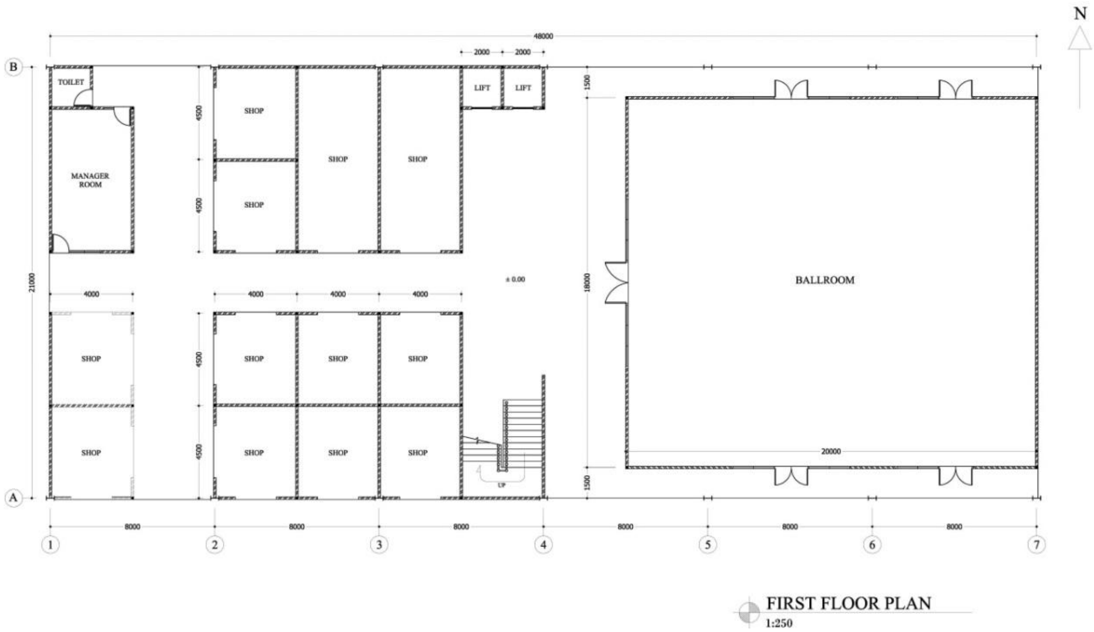
Table 1 The profil specification of the building

Profile Specification	
Names	**Structural Elements**
H 400x400x30x50	Column
WF 600x200x11x17	Spandrel Beam
WF 300x200x8x12	Truss Chord, Vierendeel Panel
HSS 200x200x12	Diagonal Truss, Hanger Truss, Knee brace Truss, Post Truss, Vertical Truss

The 3D building model with the STF system and the longitudinal cross-section in W-E direction are shown in Figures 4(a) and 4(b), while the cross-section in the transverse (N-S) direction for odd and even axes are shown in Figures 4(c) and 4(d). The seismic reduction factor (R) of the STF system is equal to 7 in the N-S direction due to the STF system, while in the longitudinal direction (W-E direction) the seismic reduction factor (R) is equal to 8 due to the special moment resisting frame (SMRF) system as mention in AISC [1]. In region high-seismic activity, researchers suggest that the behavior of STF be evaluated utilizing time history analysis enveloped with a spectrum for the site under consideration. The ductility demands on the chords can then be evaluated directly from the analysis. The response characteristics of STF that dissipates energy mainly through Vierendeel panels are similar to a ductile moment frame or an eccentrically braced frame. This would imply that an R factor of 7 or 8 could be used for the design in transverse direction of the building [1]. The STF building models as shown in Figure 4 were analyzed with the variations of ratio of the vierendeel panel width (=a) and the distance between the vertical truss (=b). Table 2 describes in detail the variations of the model that was analysed in this study.

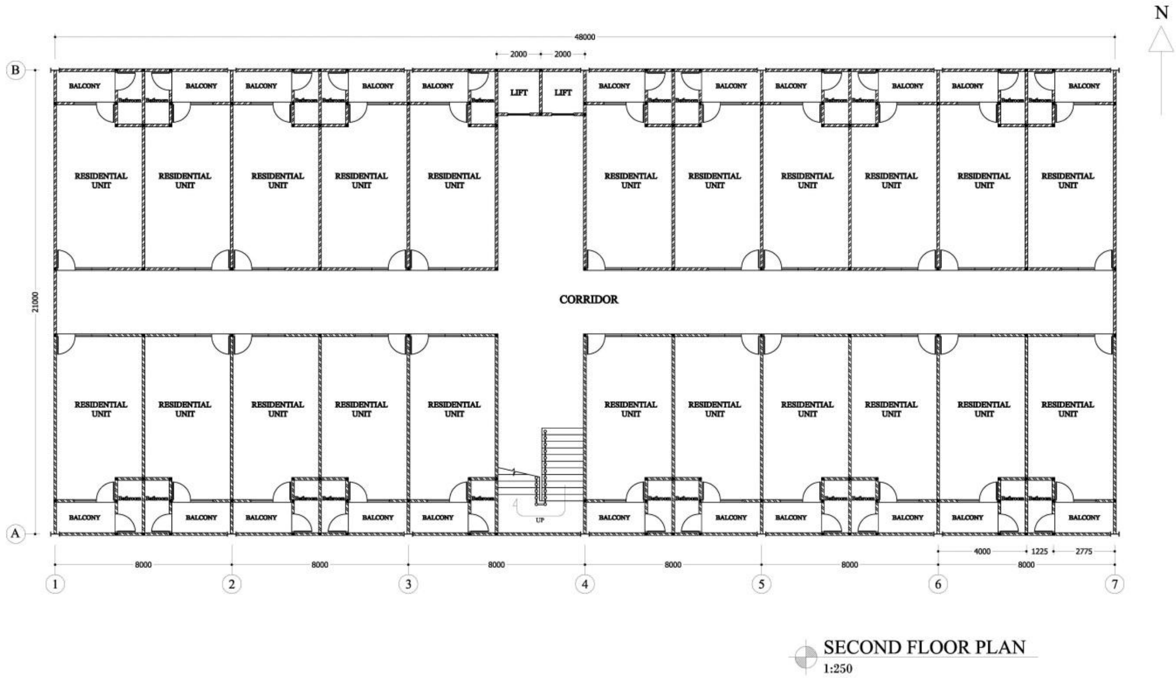
Table 2 Details of the STF Building Models

Models	Vierendeel Panel Width (a)	Vertical Truss Spacing (b)	Ratio (a/b)
STF1	2.0m	2.375m	0.842
STF2	2.5m	2.313m	1.081
STF3	3.0m	2.250m	1.333
STF4	3.5m	2.188m	1.600
STF5	4.0m	2.125m	1.882



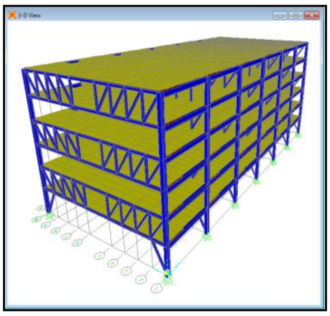
(a) Level 1

[DOI: 10.22068/IJCE.13.2.213]

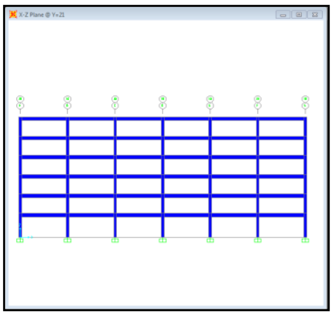


(b) Level 2 and so on

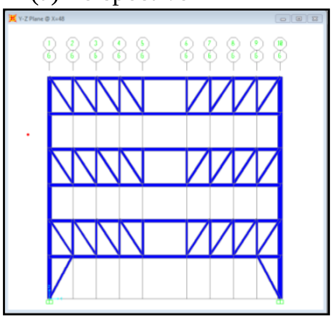
Fig. 3 Building Plan



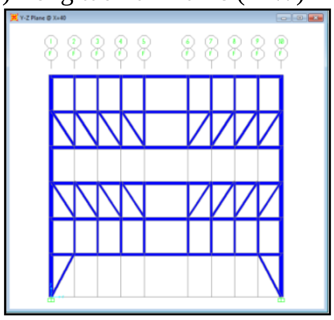
(a) Perspective



(b) Longitudinal Frame (E-W)



(c) Transverse Frame, Odd Axis (N-S)



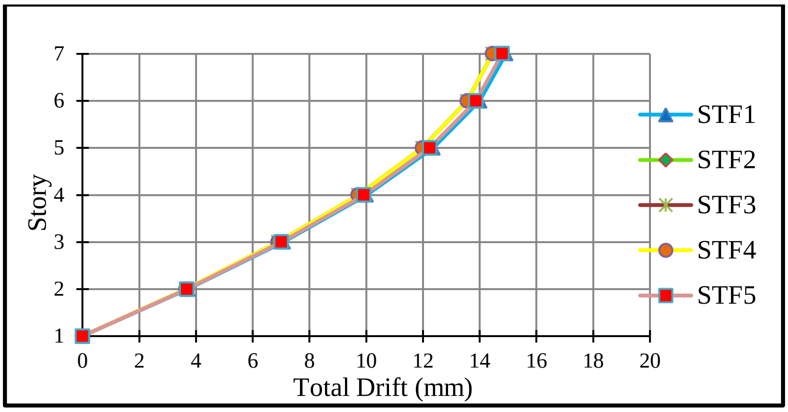
(d) Transverse Frame, Even Axis (N-S)

Fig. 4 Building Modeling With STF System

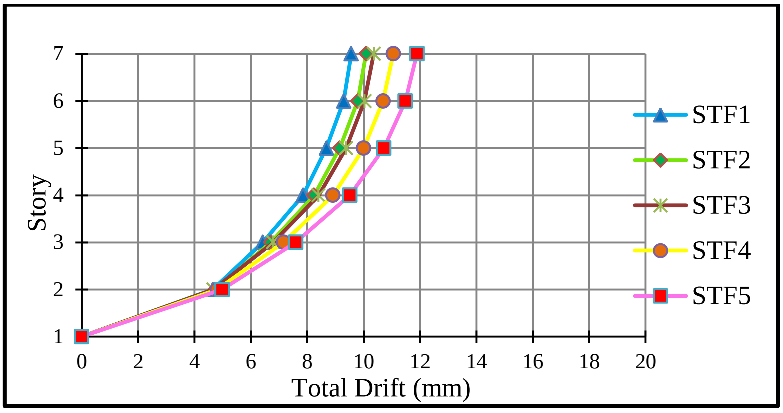
6. Linear Analysis Results

The results of the linear analysis of the building structure are presented in the inter-story and total drifts of each STF model as shown in Figures 5 and 6. Figures 5(a) and 6(a) showed the typical total drift in the W-E direction, there was no significant difference between the occurred total drifts of the five models. The total drifts of the five models for N-S direction are shown in Figure 5(b) and 6(b). There are differences among each of STF models, which showed that the STF1 model has smallest total drift while the STF5 model has the greatest total drift. It can be concluded that the total value of inter-story drift is increased by the increasing the vierendeel panel width and vertical trusses spacing ratio.

[DOI: 10.22068/IJCE.13.2.213]

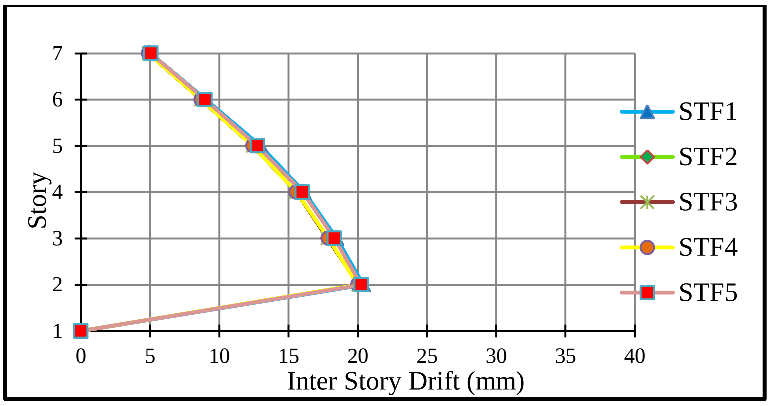


(a). W-E Direction

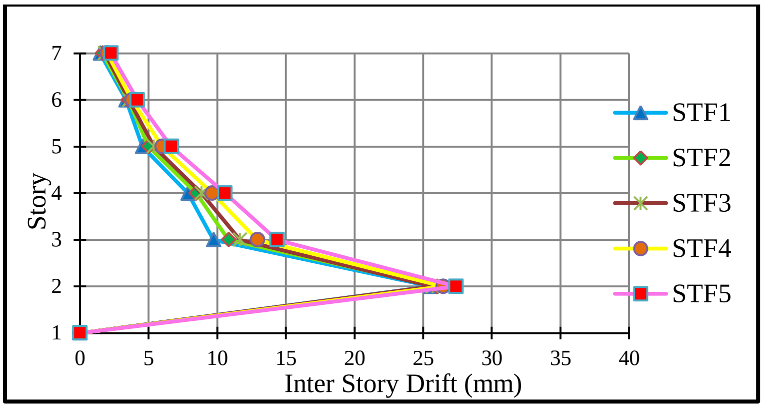


(b). N-S Direction

Fig. 5 Total drift comparison using linear analysis



(a). W-E Direction



(b). N-S Direction

Figure 6. Inter story drift comparison using linear analysis

[DOI: 10.22068/IJCE.13.2.213]

Table 3 shows the comparison of periods from the five models. The periods in the longitudinal direction for the models are similar due to use the same SMRF system. The STF1 to STF5 models in transversal direction increases the periods from 0.489 seconds to 0.623 seconds because of increasing the ratio. It is shown that the length of vierendeel influences the stiffness of the structure; the longer the vierendeel will make the weaker the structure.

Table 3 Periods of the STF Building Models

Models	Periods (seconds) of longitudinal direction (SMRF)	Periods (seconds) of transversal direction (STF)
STF1	0.785	0.489
STF2	0.775	0.518
STF3	0.775	0.550
STF4	0.775	0.585
STF5	0.785	0.623

7. Nonlinear Static Pushover Analysis Results

Equation (1) is used to calculate the ductility (μ_Δ) of a building model [7]:

$$\mu_\Delta = \frac{\delta_u}{\delta_y} \tag{1}$$

Where δy is the displacement at the first yield point and δu is the displacement at the ultimate point. Table 4 and Table 5 summarized the results of the nonlinear pushover analyses. The tables show the roof displacement and the base shear at the first yield and at the ultimate point, thus the value of building ductility can be obtained.

The nonlinear static pushover analyses showed the relationship between the roof displacements and the base forces in both directions of the building as shown in Figure 7 and Figure 8. The comparison of the capacity curve based on nonlinear static pushover analysis for W-E direction (SMRF system) is shown in Figure 8. There were no significant differences in the values of the base forces and roof displacements between the five models, where the STF5 model the collapsed roof displacement was slightly larger than in the other models. The comparison of the capacity curve for the N-S direction (STF) showed that in the first yielding point the base force value decreased, but the value of the roof displacement increased as shown in Figure 8.

Table 4 Ductility Calculation of W-E Direction

Models	Ratio	δy (mm)	Vy (kN)	δu (mm)	Vu (kN)	μΔ	Performance Level
STF1	0.842	51.56	6443.85	198.11	14633.25	3.842	Collapse
STF2	1.081	49.873	6394.762	206.930	14768.703	4.149	Collapse Prevention
STF3	1.333	49.807	6378.489	208.270	14730.290	4.182	Life Safety
STF4	1.6	49.716	6358.658	210.245	14670.292	4.229	Life Safety
STF5	1.882	51.282	6373.631	229.020	14811.868	4.466	Life Safety

Table 5 Ductility Calculation of N-S Direction

Models	Ratio	δy (mm)	Vy (kN)	δu (mm)	Vu (kN)	μΔ	Performance Level
STF1	0.842	32.19	11953.84	59.05	19516.31	1.834	Collapse
STF2	1.081	35.896	11845.716	65.940	19609.483	1.837	Collapse
STF3	1.333	37.151	10821.561	72.430	18657.227	1.950	Collapse Prevention
STF4	1.6	39.539	10132.224	82.349	18331.364	2.083	Life Safety
STF5	1.882	42.722	9608.751	94.982	18087.275	2.223	Life Safety

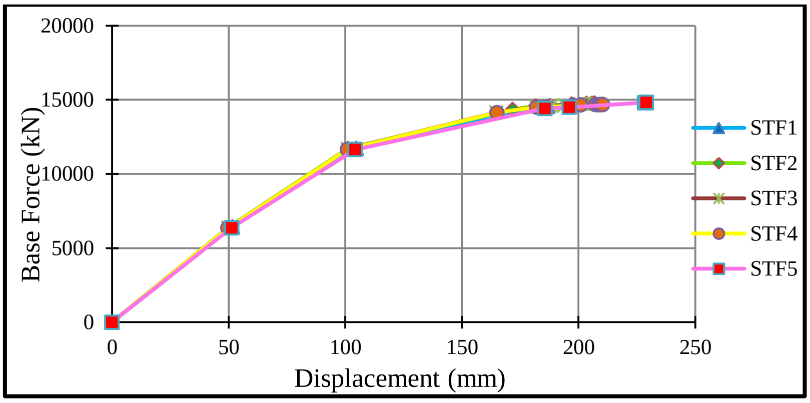


Fig. 7 Capacity Curve Comparison for W-E Direction using Pushover analysis

[DOI: 10.22068/IJCE.13.2.213]

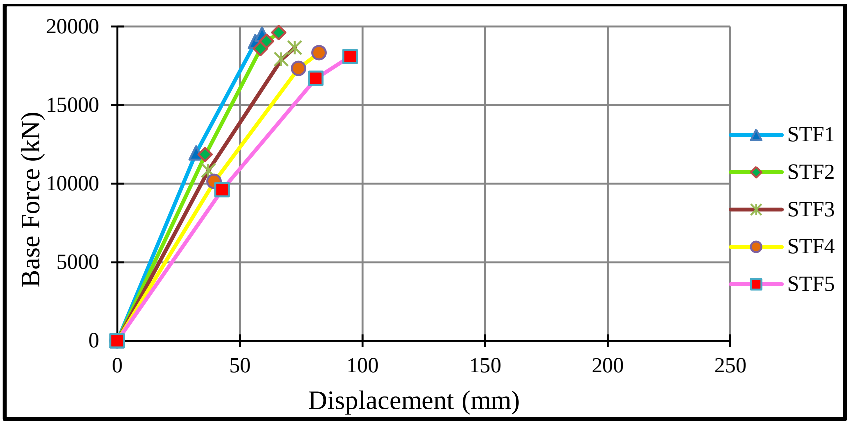


Fig. 8 Capacity curve comparison for N-S Direction using Pushover analysis

Increasing the vierendeel panel width and the vertical trusses spacing ratio can be clearly seen in Figure 9 causes the increased structural ductility. The increased in ductility occurred in both directions of the building, but for the W-E direction (SMRF) it has a higher ductility values compared to the N-S direction (STF). The reason was due to the used truss elements in the N-S direction caused the stiffness in that direction was increased. Based on the performance evaluation results as shown in Table 4 and Table 5, it is recommended to use a ratio of the vierendeel vertical width and the vertical truss spacing of more than 1. The ratio gave the building performance level of Life Safety to Collapse Prevention. When the ratio was used more or equal to 1.6, it provided an ideal behavior of the building with STF system, because the building performance was in Life Safety level.

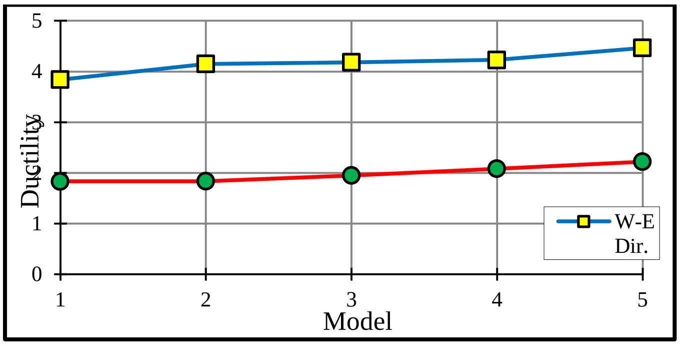
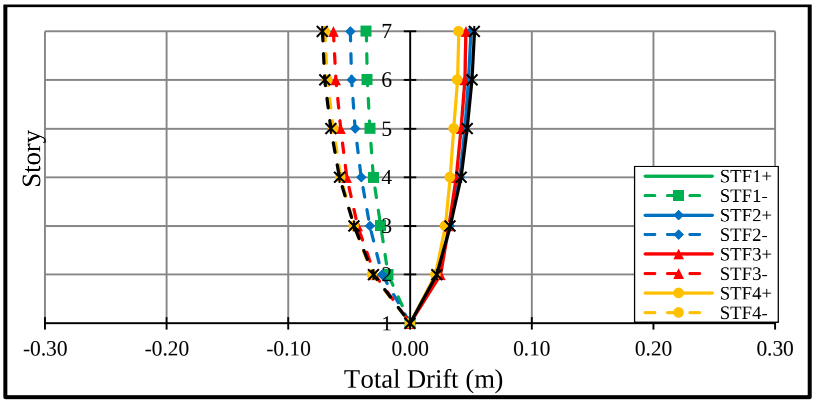


Fig. 9 Ductility comparisons using Pushover analysis

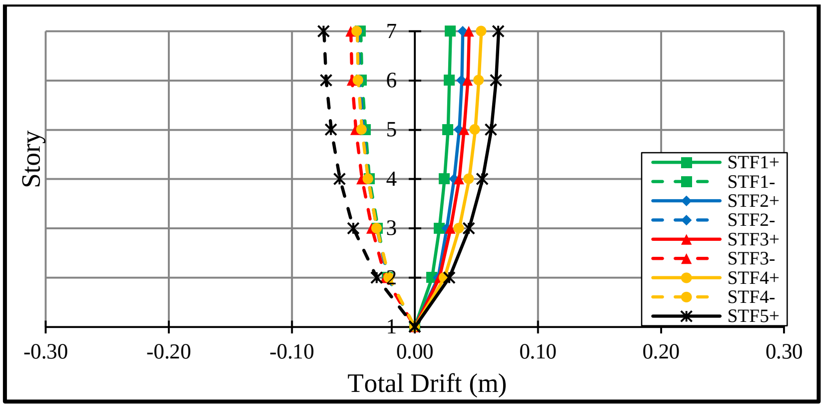
8. Nonlinear Time History Analysis Results

Time history analyses were conducted to investigate the performance of the models. All models used the El Centro, the Denpasar and the Kern County earthquake records. The comparisons of deformation and maximum drift at each floor of the five models in the ultimate conditions obtained by nonlinear time history analyses in the N-S direction (STF system) have relatively the same values as shown in Figure 10. The STF5 model, which has the biggest ratio of the vierendeel panel width and vertical truss spacing, has the bigger drift to be compared to the other models. It was clear as shown in Figure 11 that the effect of adding ratios of the vierendeel panel width and spacing vertical trusses caused increasing the deformation and maximum drift at each floor of the building. The ultimate drift obtained from non-linear time history analysis was smaller than the pushover analysis. This result showed that the static nonlinear pushover analysis was quite conservative in predicting the behavior of the six-storey building in an inelastic condition.

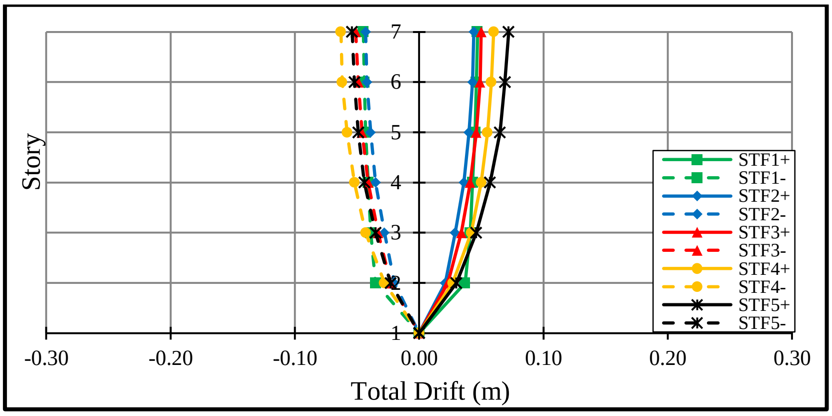


(a). Time History of Elcentro

[DOI: 10.22068/IJCE.13.2.213]

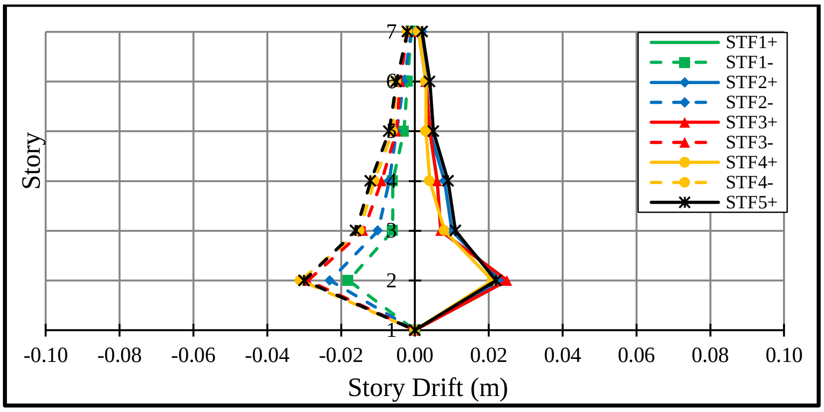


(b). Time History of Denpasar

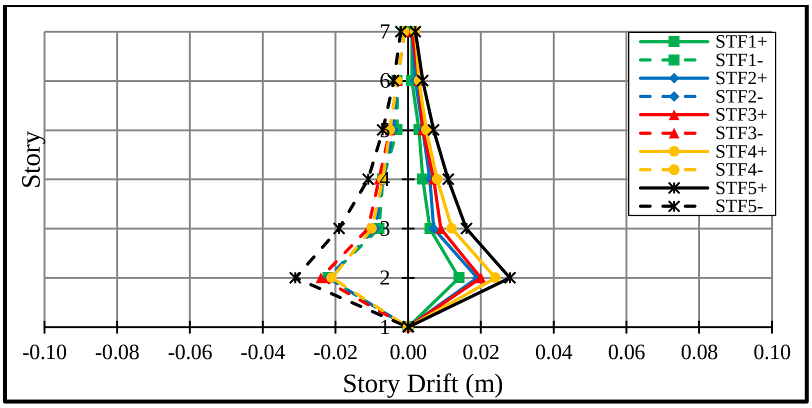


(c). Time History of Kern County

Fig. 10 Deformation in N-S Directions using Nonlinier Time History Analysis

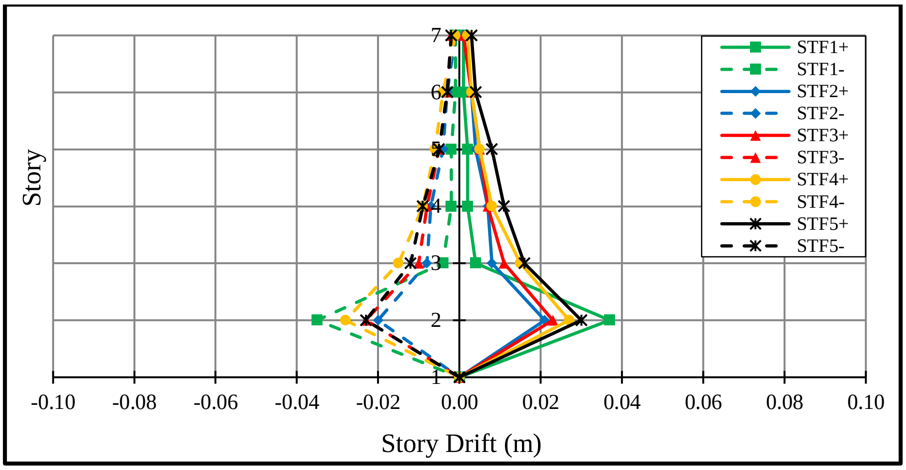


(a). Time History of Elcentro



(b). Time History of Denpasar

[DOI: 10.22068/IJCE.13.2.213]



(c). Time History of Kern County

Fig. 11 Maximum Drift in N-S Directions using Nonlinier Time History Analysis

9. Conclusions

The effect of increasing the ratios of the vierendeel panel width and the vertical truss spacing in the staggered truss framing system increased the ductilities in both directions of the building. Increasing the ratio also linearly increased the displacement of the building that occurred in the N-S direction (STF), but in the W-E direction (SMRF) there was no significant change in the displacements of the five models. The results of the analysis also showed that the ductility and the displacement in the W-E direction of the building were larger than the N-S direction, because of the increasing stiffness due to the truss elements in the building.

Based on the results of the performance evaluation it was shown that the model STF 1, STF 2 and STF 3 (ratios of 0.842, 1.081 and 1.333) provided a poor level of performance because in the critical condition it was in the Collapse Prevention to Collapse levels. While the model STF 4 and STF 5 (ratios of 1.6 and 1882) showed a good performance because the critical condition of the building was still at the Life Safety level.

The nonlinear time history analysis using Elcentro, Denpasar and Kern County earthquake record showed that the deformation from every floor and a maximum drift of the STF building is smaller than the nonlinear static pushover analysis. Thus, the nonlinear static pushover analysis was quite conservative when used in design, especially in evaluating the staggered truss framing structural building.

References

[1] Wexler N, Feng-Bao L. Steel Design Guide Series: Staggered Truss Framing Systems, USA, American Institute of Steel Construction, Inc, 2003.

[2] Tethool Yoga CV, Wahyuni E. Behavior evaluation of staggered truss framing system (STF) using nonlinear pushover and time history analysis, Proceedings of the National Seminar on Civil Engineering ITS Surabaya IX, 2013.

[3] Kim Jinkoo, Joonho Lee. Seismic behavior of staggered truss system, First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland, 2006.

[4] Dewobroto Wiryanto. Performance evaluation of earthquake resistant for steel structures with pushover analysis, Civil Engineering National Conference, Semarang, 2005.

[5] Pranata Yosafat Aji. Ductility assessment of reinforced concrete building structures with time history analysis and pushover analysis, Civil Engineering ITS Journal, 2008, No. 3, Vol. 8.

[6] ASCE. FEMA 356 - Prestandard and Comentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington DC, 2000.

[7] National Standardization Committee. Design Guideline for Earthquake Resistance of Structural and Non Structural Buildings (SNI 03-1726-2010), Bandung, BSN, 2010.

[8] Seismology committee structural engineering association of california. seismic design of steel staggered truss systems, California, Structure Magazine, 2008.

[9] SAP2000 Manual. Computer and Structure Inc, Barkeley, California, USA, 2010.

[10] National Standardization Committee. Design Guideline for Structural Steel Buildings (SNI 03-1729-2000), Bandung, BSN, 2000.

[11] National Standardization Committee. Loading Calculation Procedure for Hauses and Buildings (RSNI 03-1727-1989), Bandung, BSN, 2005.

[DOI: 10.22068/IJCE.13.2.213]