16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 459

Registration Code: S-B146473030

IMPROVED SEISMIC RESPONSE OF TILT-UP BUILDINGS

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Abstract

Tilt-up construction is a cost-effective technique for constructing low-rise and mid-rise buildings due to its lower construction costs and a shorter construction period. Though it is popular in the United States, these types of structures are extremely vulnerable to moderate and severe earthquakes. Significant structural and nonstructural damage of tilt-up buildings has been reported since the 1964 Alaska earthquake.

In order to estimate the potential vulnerability of different types of tilt-up constructions, a critical comparison is made of the response characteristics of three types of one story tilt-up buildings. These include an existing instrumented tilt-up building with continuous concrete walls and timber roof that is representative of the construction prior to the 1980's

Additional buildings include a tilt-up building designed with segmented walls and timber roof and a designed tilt-up building with segmented walls and steel roof. These two buildings are more representative of current construction practice. The three models are subjected to individual, moderate and severe earthquakes.

These three dimensional FEM models represent the three different types of tilt-up buildings considered in this study. Nonlinearities of different structural components used in the tilt-up buildings are incorporated in the models. The overall performance will be presented in terms of modal periods, base shears, story drifts, displacement time histories and the distributions of forces on critical structural components. Since the capacity and ductility of the connections significantly influence the overall performance of the whole structure, the strength and ductility of these individual components will be noted in detail. Based on these results, recommendations are made for evaluating and improving the seismic performance of these types of structures located in seismically active areas.

Keywords: tilt-up construction, seismic response, modern design

1. Introduction

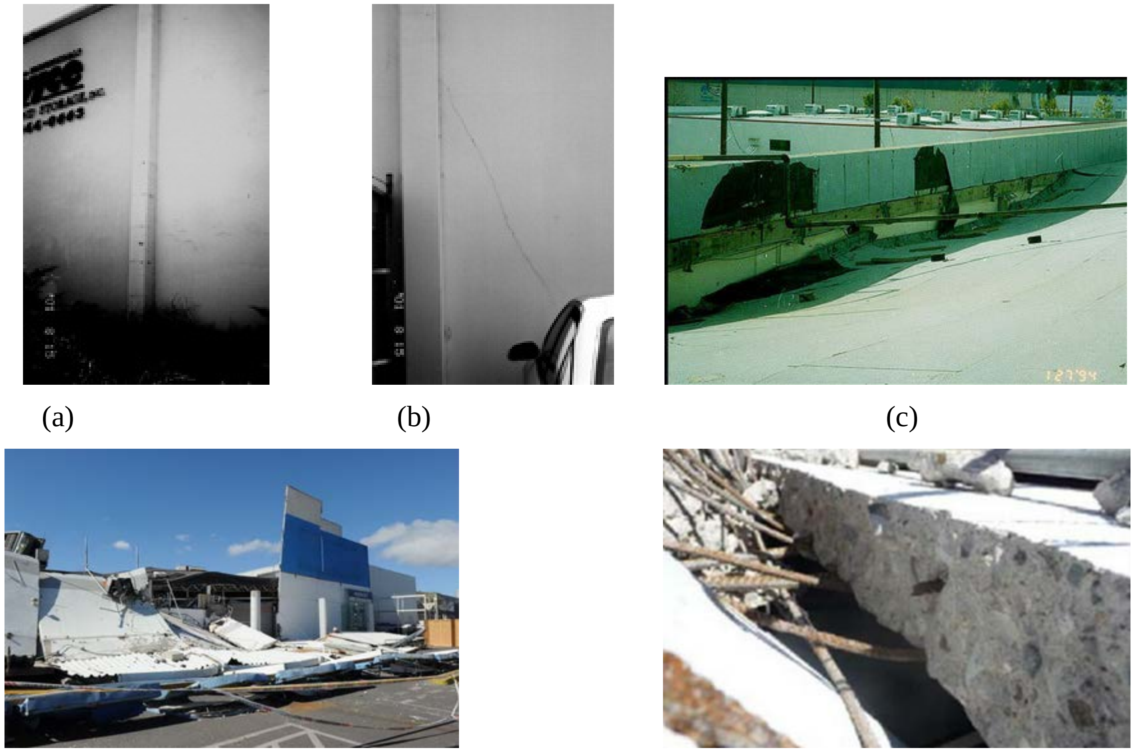
Tilt-up buildings are widely used in the United States for low-rise industrial buildings. Although, tilt-up buildings are popular, they are very vulnerable to strong ground motions. During the past sixty years, significant seismic damage of tilt-up buildings was observed after some moderate and severe earthquakes, including: Alaska (1964), San Fernando, California (1971), Whittier Narrows, California (1987), Morgan Hill, California (1987), Loma Preita, California (1989), Northridge, California (1994) and Chile (2010).

The purpose of this this paper is to evaluate the effects of moderate and strong earthquakes which have different peak ground accelerations and dynamic characteristics on three types of one-story tilt-up buildings which are widely used in the United States. They are: (1) a tilt-up building with continuous concrete walls and timber roof which represents the traditional construction; (2) tilt-up buildings with segmented concrete walls and plywood panels; and (3) tilt-up buildings with segmented concrete walls and metal decks. In order to reasonably assess the seismic response, a series of three dimensional models proposed according to available design information, construction details and experimental data. Elastic linear analysis and nonlinear analysis are performed in SAP 2000. Analytical results are summarized and compared in terms of mode shapes, periods, base shear forces, displacements, forces in connections, behavior of perimeter walls and behavior of the roofs.

2. Seismic Vulnerability of Tilt-up Buildings

Tilt-up buildings are vulnerable to moderate and severe ground motions, seismic damage of tilt-up buildings were observed in the past sixty years. In the 1964 Alaska earthquake, the anchor bolts between the pilasters and the roof were pulled out, causing the collapse of three out of five bays of a tilt-up building. Brittle failure of the steel reinforcement in the concrete columns and of the welds connecting the cross bracing in the firewalls to the steel roof framing members were also observed [1]. During the 1971 San Fernando earthquake, significant structural damage was observed in tilt-up warehouses. After this earthquake, investigators identified that the possible cause that resulted in the partial collapse of the roof was the inadequate wall anchorage and continuity ties [2]. Poor seismic performance led to stricter code requirements in the following years. The 1987 Whittier Narrows earthquake, the 1987 Morgan Hill earthquake and the 1989 Loma Prieta earthquake tested the revised tilt-up design requirements. The number of the roof and the wall panels that collapsed in these three earthquakes was markedly reduced compared with those during the 1971 San Fernando earthquake due to relatively moderate ground shaking and short durations [3], [4]. However, partial collapse of roof diaphragms and wall panels were still observed. In the 1994 Northridge earthquake many of tilt-up structures were severely damaged. It was estimated that one third of tilt-up structures in the San Fernando Valley had significant structural damage. Failure of the roof-to-wall connections resulted in the separation of the wall panels from the roof, and led to partial collapse of the roof and wall panels. Steel ties were pulled out of the wall between beams and concrete panels [2]. After the 1994 Northridge earthquake, almost no strong and severe earthquakes occurred in California so that no further seismic damage was reported until now in California.

Typical failure modes of tilt-up buildings are: (1) Cracks on the perimeter walls; (2) Failure of the roof-to-wall connections; (3) Failure of the wall-to-slab connections. Cracks on the tilt-up walls are one type of common damage, including horizontal cracks and diagonal cracks, as shown in Fig. 1 (a) and Fig. 1. (b). The most critical structural elements of tilt-up buildings are connections, since the roof diaphragm and the perimeter concrete walls are tied together by different types connections. Significant damage of tilt-up buildings resulting from the failure of these connections were commonly observed during the past sixty years. Failure of these connections, especially the roof-to-wall connections may lead to partial collapse of the roof diaphragm and overturn of the perimeter walls, as shown in Fig. 1 (c) to Fig (e).



(a) (b) (c)

(d) (e)

Fig 1: a) Horizontal cracks [2], b) Diagonal cracks [2], c) Failure of the roof-to-wall connections [2], d) Partial collapse of wall and roof [5], e) Damage to the wall-to-slab connections [6]

3. Case Study Building and Proposed Models

In order to simulate tilt-up buildings with different construction details, three types of models are developed in SAP 2000 version 15 [7], they are listed as follows:

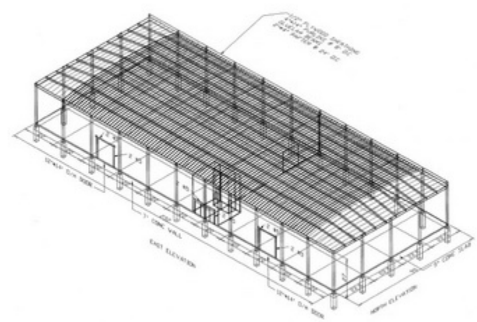
- Tilt-up structures with continuous walls and timber roof which is defined as CT models.
- Tilt-up structures with segmented walls and timber roof which is defined as ST models.
- Tilt-up structures with segmented walls and steel roof which is defined as SS models.

CT models are developed to simulate an existing instrumented one-story tilt-up building which is located in Redlands, California. This building is selected from the database of California Strong Motion Instrumentation Program (CSMIP). It represents the pre-1971 design and construction details. ST models and SS models are designed one-story tilt-up buildings which represent the modern design and construction practice.

3.1 Case study building: one-story tilt-up building in Redlands, CA

The first type of model is an instrumented one-story tilt-up building which is located in Redlands, California. The general view of this building is shown in Fig 2.a. This one-story tilt-up industrial warehouse was designed in 1971 after the San Fernando earthquake. It has a rectangular plan which is 232 feet by 90 feet. The building has four identical spans with 22.5 feet length in the E-W direction whereas in the N-S direction has six long spans with 22 feet length and five short spans with 20 feet length. The walls of the Redlands warehouse are 7" tilt-up concrete panels which contain one #4 bar spaced at 12" horizontally and one #5 bar spaced at 12" vertically. The roof diaphragm consists of 4' x 8' x 0.5" thick structural I plywood nailed to 4" x 14" purlins running in the N-S (longitudinal) direction and 4"x4" sub-purlins running in the E-W (transverse) direction. The 30" x 10.75" glulam beams running in the E-W (transverse) direction, which supports the purlins and sub-purlins, were mounted on the top of pilasters along the perimeter walls. Wood ledgers run along the perimeter walls to support the edge of the plywood structural panels. The details are shown in Fig 2.b. Since 1986, dynamic response has been recorded in the Redlands Warehouse as part of the California Strong Motion Instrumentation Program (CSMIP). Twelve sensors are placed on two levels in the building. Three sensors are located on the slab of the structure, three sensors record the response in the longitudinal direction at the roof level, five sensors record transverse direction response at the roof level, one sensor records the out-of-plane response of the east (longitudinal) wall at the mid-height of longitudinal walls.

(a)



(b)

Fig 2: a) General view of Redlands Warehouse [8], b) Design Details of Redlands Warehouse [9]

3.2 Proposed Models

In order to evaluate the linear and nonlinear behavior of different designs, elastic and inelastic models are developed separately for each type of tilt-up structure. For tilt-up structures with continuous walls and timber roof (CT models), one elastic model (CT 1) and three inelastic models (CT 2, CT 3 and CT 4) are developed. The differences among three nonlinear CT models are: (1) CT 2 model is the second model developed based on the elastic model (CT 1) by incorporating the nonlinear behavior of the roof to wall connections (glulam beam to wall connections and purlin to wall connections); (2) CT 3 model is a nonlinear model with inelastic roof-to-wall connections and inelastic plywood panels with sparse nailing; (3) CT 4 model is developed by considering nonlinear behavior of roof to wall connections and plywood panels with dense nailing. Similarly, for tilt-up structures with segmented walls and timber roof (ST models), one elastic model and three inelastic models with different nonlinear properties are developed. For tilt-up structures with segmented walls and steel roof (**SS** models), one elastic model and one inelastic model are developed. The classifications of the proposed models are listed in Figure 3.

4. Selected Earthquake Records

In this study, three types of tilt-up buildings are subjected to selected individual earthquakes which represent moderate and severe ground excitations in California. Each record consists of two orthogonal horizontal components. These earthquakes are: Whittier, California (1987); Loma Prieta, California (1989); Landers, California (1992); Big Bear, California (1992); Northridge, California (1994); Parkfield, California (2004). The selection criteria are: (1) The selected earthquakes occurred in California where tilt-up buildings are widely used. The selected stations are located in the populated areas of California; (2) The magnitudes of the selected earthquakes are greater than 5.5; (3) The selected ground motions should cover the frequencies of interest, varying from 0.1 second to 0.5 second. (4) The absolute peak ground accelerations of the recordings are between 0.2g and 1.0g to induce different levels of nonlinearity in the proposed models. Table 1 summarizes the descriptions of the selected ground motions in terms of magnitudes, locations of stations, and absolute peak ground accelerations.

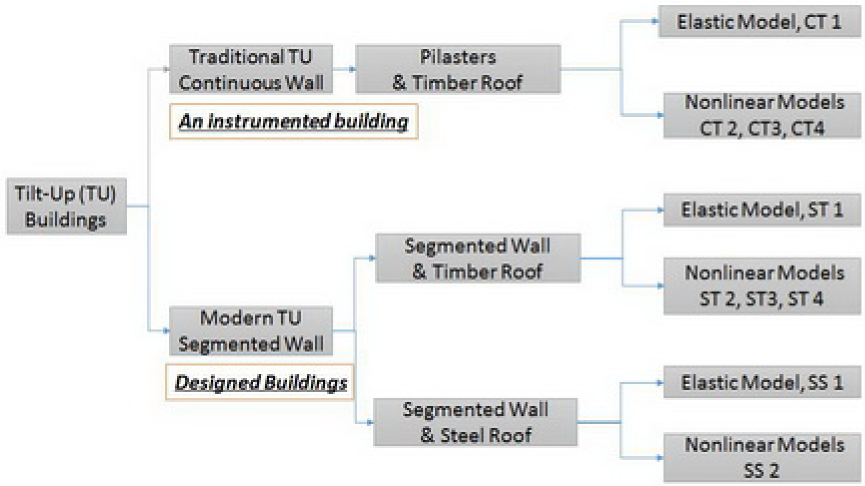


Fig.3 Classification of proposed models

Table 1 Summary of selected earthquakes

Selected Earthquake	Magnitude	Station	Component	Absolute PGA (g)
Landers	7.3	Joshua Tree Fire Station	90	0.28
			0	0.27
Whittier	5.9	Alhambra Fremont School	270	0.39
			180	0.29
Loma Prieta	6.9	Muncip San Jose de Maipo	90	0.44
			0	0.44
Big Bear	6.5	Big Bear-Civic Center Grounds	270	0.48
			360	0.55
Parkfield	6.0	Parkfield Gold Hill 3W	90	0.68
			360	0.42
Northridge	6.7	Sylmar Country Hospital	90	0.60
			360	0.84

5. Discussion of Analytical Results

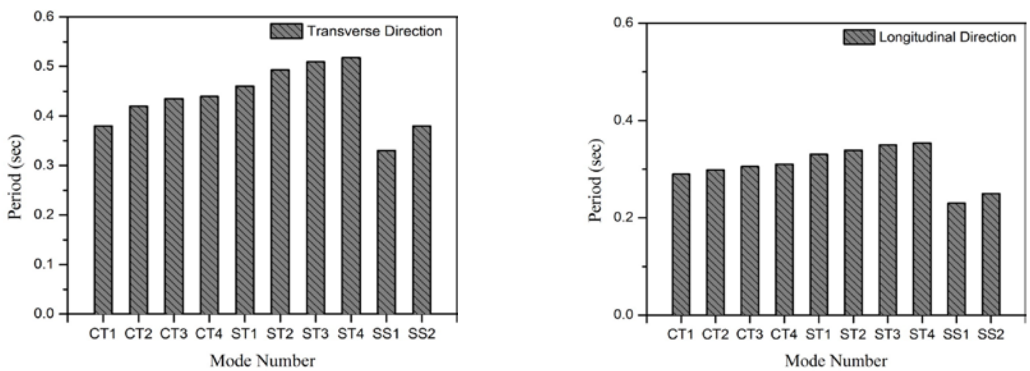
5.1 Periods

Modal analysis of ten proposed models is conducted to determine the fundamental periods and mode shapes. Table 2 summarizes the results for the fundamental periods in two principal directions. To give a more visual and clear understanding of the changes in periods which resulted from different designs and nonlinearity of structural members, the comparisons of periods of ten proposed models in both directions are listed in Fig. 4. It is observed that the fundamental periods of tilt-up buildings with segmented walls and timber roof (ST-1, ST-2, ST-3, and ST-4) are greater than other two types, ranging from 0.46 second to 0.52 second in the transverse direction and 0.33 second to 0.36 second in the longitudinal direction. It indicates that this type of design is the more flexible type among the three. For tilt-up structures with continuous walls and timber roof (CT-1, CT-2, CT-3 and CT 4), the elastic analysis indicates that the fundamental periods are 0.38 second in the transverse direction and 0.29 second in the longitudinal direction. By incorporating the nonlinearity, the periods increase by

1% to 18% respectively in the transverse direction and 3% to 7% in the longitudinal direction. Apparently, this trend indicates that the development of inelastic behavior has more significant influence on the lateral stiffness of the structure in the transverse direction. For tilt-up buildings with segmented walls and steel roof (SS-1 and SS-2), the periods vary from 0.33 second to 0.38 second in the transverse direction and 0.23 second to 0.25 second in the longitudinal direction. Application of steel roof significantly increases the lateral stiffness of this type of design.

Table 2 Summary of Periods

Principal Direction	Model Number									
	CT1	CT2	CT3	CT4	ST1	ST2	ST3	ST4	SS1	SS2
Transverse (E-W)	0.38	0.42	0.44	0.45	0.46	0.50	0.51	0.52	0.33	0.38
Longitudinal (N-S)	0.29	0.30	0.31	0.31	0.33	0.34	0.36	0.36	0.23	0.25

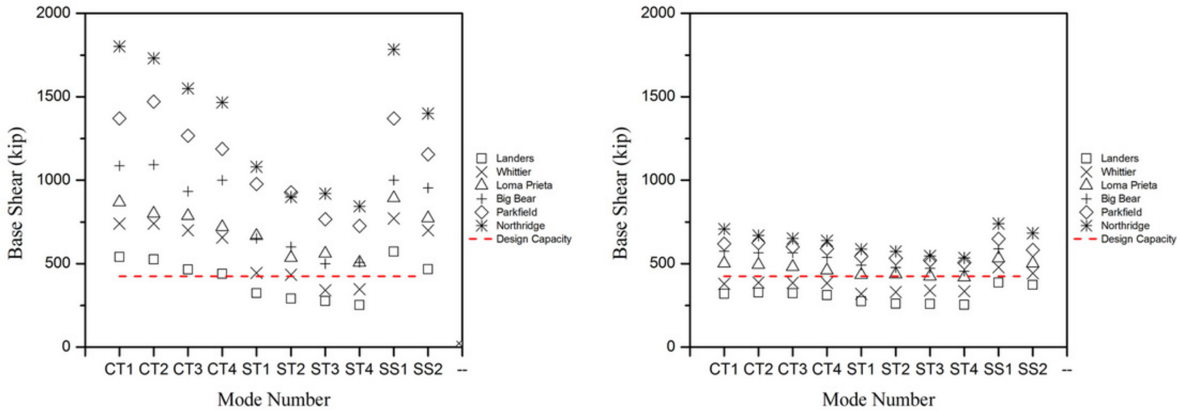


(a) Transverse Direction (b) Longitudinal Direction

Fig. 4 Comparisons of periods

5.2 Base Shears

Predicted base shears in the transverse and longitudinal directions during the selected ground motions are evaluated and compared with the design capacities. Comparisons of base shears in both directions are summarized in Fig. 5. The base shears of the proposed models when subjected to seven different earthquake inputs are compared to the design base shear force which is obtained from UBC code. Fig. 5 (a) compares the base shears in the transverse direction with the design capacity of 426 kips. For the tilt-up building with continuous walls and timber roof, the base shear of the elastic model (CT 1 model) is noticeably larger than other three models. The base shear forces for most of the selected earthquakes are above the design capacity, indicating the inadequacy of the design force in the current codes. The base shears of the ST models are smaller than other two types. Segmented walls and timber connections increase the flexibility of this design type. Fig. 5 (b) compares the base shears in the longitudinal direction with the design capacity. For all the cases, the base shears of the proposed models in the longitudinal direction are smaller than those in the transverse direction. When the peak ground accelerations are greater than 0.44 g, the base shear forces in this direction (longitudinal) exceed the design capacity.



(a) Transverse Direction (b) Longitudinal Direction

Fig. 5 Comparisons of base shears

5.3 Forces in Connections

Connections are important factors that influence the overall seismic behavior of the tilt-up buildings. Due to the complexities of the construction details of this type building, the designs of the roof to side wall connections and the roof to end wall connections are different, which lead to different capacities and ductility of these connections. For the side wall to roof connections, comparisons of the forces in the GLB-to-wall connections are summarized in Fig. 6 (a). The axial forces in the GLB-to-wall connections for eight models (CT models and ST models) when subjected to the selected earthquakes are compared with the capacity of 26 kips. In the two elastic models (CT 1 and ST 1), the axial forces are higher than the capacity when the peak ground accelerations of the inputs are greater than 0.5g. It indicates that the connections are pulled into the inelastic range when subjected to the moderate and severe earthquakes. Generally, the axial forces in CT models are greater than those in ST models. Different types of walls influence the distributions of axial forces in the GLB-to-wall connections. The comparisons of horizontal shear forces in the GLB-to-wall connection in Fig.6 (b). The horizontal shear forces for elastic models are much greater than the capacity, indicating the development of nonlinearity. Similarly, the horizontal shear forces of the GLB-to-wall connections in the ST models are greater than those in the CT models. Fig. 6 (c) summarizes the vertical shear forces of the GLB-to-wall connections, when the nonlinear behavior of structural components is taken into consideration, for all the proposed models, the vertical shear forces are less than the capacity of the connections. Additionally, the applications of different types of roof panels slightly influence the axial, horizontal shear and vertical shear forces.

Comparisons of the axial, horizontal shear and vertical shear forces in the roof-to-side-wall connections are summarized in Fig. 6 (d) to (f). As shown, the capacity for the purlin-to-wall connections is 16 kips. If nonlinearity is taken into consideration, the maximum axial forces of in the connections for all proposed models are smaller than the capacity when subjected a moderate earthquakes like the selected Big Bear earthquake. Comparisons of the horizontal shear forces in Fig. 6 (e) indicate that for all the nonlinear models the horizontal shear forces are below the capacity line when subjected to an earthquake with PGA less than 0.7 g. Fig. 6 (f) shows that the vertical shear forces are less than the capacity, indicating that the forces in this direction is not a crucial factor that influences the behavior of the purlin-to-wall connections.

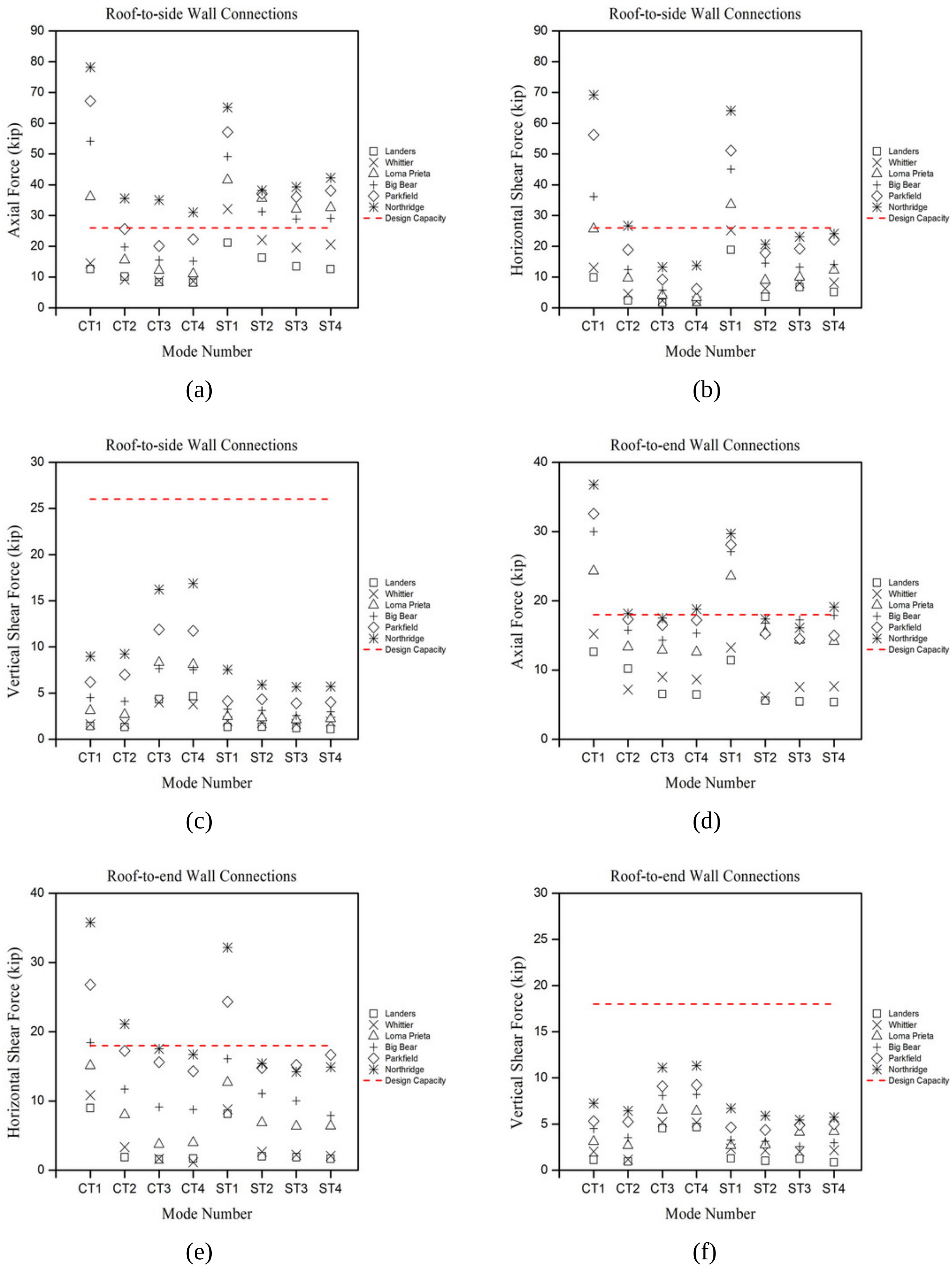
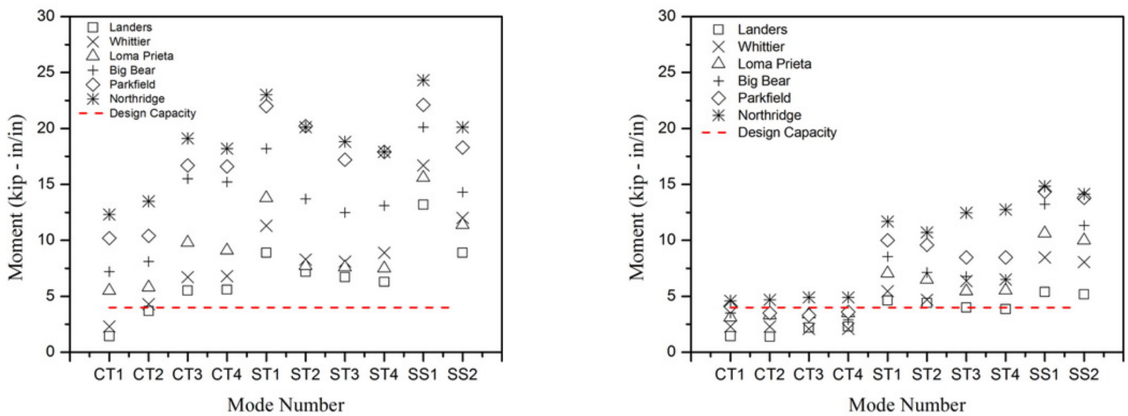


Fig. 6 Summary of maximum forces in roof-to-wall connections

5.4 Wall Behavior

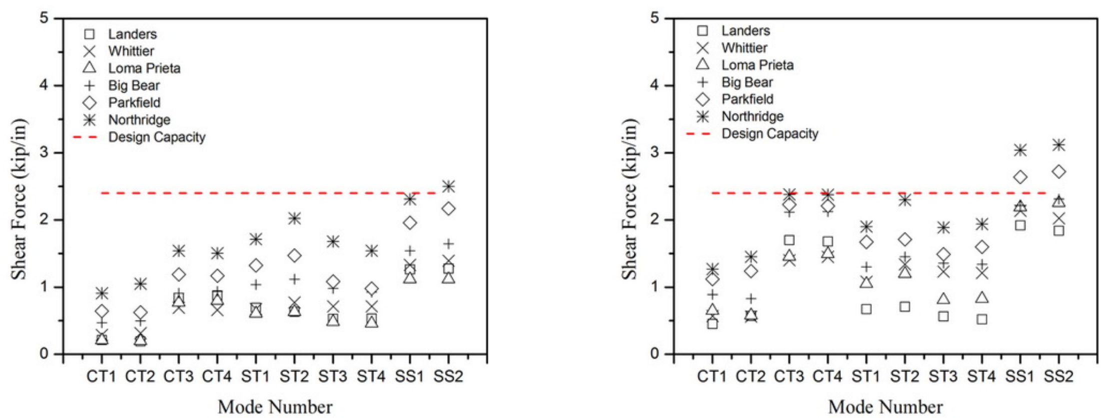
The response of the perimeter walls is studied in terms of the out-of-plane bending moments and the in-plane shear forces. Comparisons of the maximum out-of-plane bending demands are summarized in Fig. 7. In the transverse direction (on the side walls), almost for all the proposed models, the demands exceed the capacity of 4.1 kip-in/in. However, in the longitudinal direction (on the end walls), it is observed that the demand for all the CT models are smaller than capacity except the results obtained from the Northridge inputs which is also close to the design capacity.

The in-plane shear demands of the walls are also compared to the design capacity. Comparisons of the maximum in-plane shear demands are presented in Fig. 8. In both directions the in-plane shear demands for almost all the proposed models are smaller than the capacity value of 2.4 kip/in.



(a) Side wall (b) End wall

Fig. 7 Maximum out-of-plane bending demand



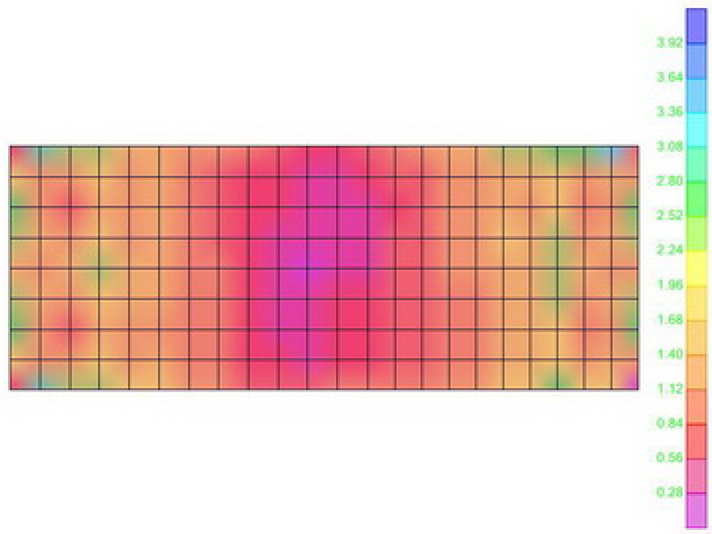
(a) Side wall (b) End wall

Fig. 8 Maximum in-plane shear demand

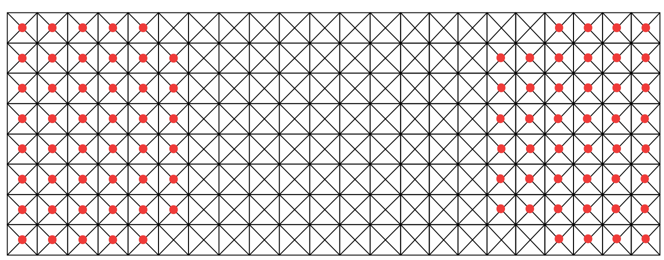
5.5 Roof Behavior

The elastic and inelastic behaviors of the roof panels are also evaluated. Due to the limitation of space, only the samples of the in-plane shear contour of the roof panels and development of nonlinear elements are presented in Fig. 9. Fig. 9 (a) shows the in-plane shear contour of CT 1 model when it is subjected to the worst scenario, the

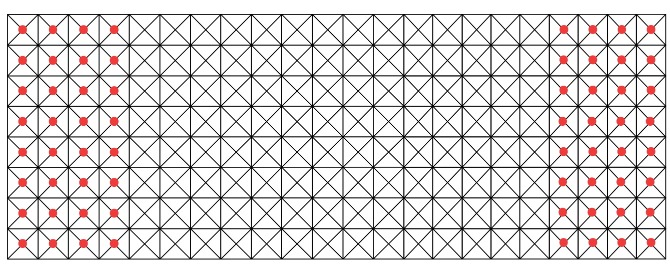
Northridge earthquake. The maximum value is observed in the area which is close to the end walls, reaching 3.36 kip/ft. If compared with the design value of 1.152 kip/ft, the demands are over the design values, indicating that nonlinear behavior should be taken into consideration. According to the distribution of in-plane shear forces, the roof areas can be divided into three different zones which have different amplitudes of average mean shear force, including two end zones which are close to both ends and one middle zone. Development of nonlinear elements in different types of tilt-up structures is also evaluated when subjected to the Northridge earthquake. Three types of models are listed in Fig. 9 (b) to Fig. 9 (d). Red dots are used to represent the portion of elements which are push into the inelastic stage. It is observed that nonlinear elements are located in two end zones which are close the end walls. The roof panel elements in the CT 3 model experience higher nonlinearity than ST 3 and SS 2 model, indicating that the in-plane shear forces are larger in the traditional construction.



(a)

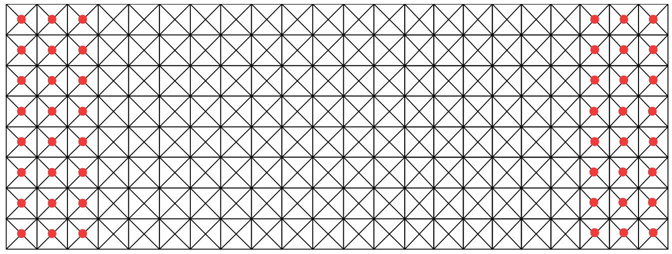


(b) CT 3



(c) ST 3

Fig. 9 Roof behavior of the proposed models (Continued)



(d) SS 2

Fig. 9 Roof behavior of the proposed models

6. Conclusions

Based on the analytical results discussed above, some conclusions can be drawn that: Compared to the traditional continuous walls, the application of segmente noticeably reduces the forces in different structural components when subjected to moderate and severe earthquakes. Compared to the plywood roof deck, the use of metal decking increases the overall lateral stiffness and decreases the story drift. Meanwhile, the seismic performance of tilt-up buildings with metal deck is more critical than those with plywood panels. The forces in the structural components and connections exceed the design forces when the structures are subjected to moderate and severe earthquakes. Hence, the design forces in the codes need to be revised in order to improve the earthquake-resistant capacity. In-plane shear stresses in the concrete walls are less than the capacity of current design recommendation. While the out-of-plane bending stresses exceed the design forces when subjected to severe earthquakes, which may induce the horizontal cracks in the concrete walls. For all the proposed tilt-up structures, the roof-to-wall connections need to be strengthened to resist axial and horizontal shear forces.

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