SEISMIC PERFORMANCE OF CONCRETE TILT-UP BUILDINGS: CURRENT WALL-TO-SLAB CONNECTIONS

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ABSTRACT :

Low rise buildings are commonly constructed in Canada and the US by casting concrete walls on the ground and then tilting them upright. Solid tilt-up walls are inherently stiff and strong. The wall strength and inelastic response is controlled by the connections to these elements. The connection between wall panels and the base slab is the focus of the current study. In order to determine the interaction between vertical uplift on the connector and horizontal shear capacity of the connector, a series of 20 tests are being conducted. The detailed results from three specimens on monotonic uplift, cyclic uplift and reverse cyclic shear with 50 mm uplift are presented. The current procedure used to weld together the plate embedded in the wall to the steel angle embedded in the concrete slab was found to result in a weld failure after only a few cycles of uplift. A revised welding detail is being developed and will be tested. Failure of the wall-to-slab connection may lead to sliding or rocking of the wall on the foundation. To explore the nonlinear system performance of sliding and rocking tilt-up buildings after connection failure, three-dimensional nonlinear response history analyses were conducted using Perform-3D. The wall rocking mechanism generally results in much larger (e.g., four times larger) maximum roof drifts compared to the wall sliding mechanism. About 80% of the energy dissipated in the rocking mechanism is due to movement of building mass, while in the sliding mechanism, about 70% of dissipated energy is due to sliding. Despite the larger displacement demands, the rocking mechanism is felt to be more practical because the complex geometry of real buildings may not allow sliding, and unlike the sliding mechanism, rocking never results in significant residual displacements.

KEYWORDS: Concrete tilt-up buildings, embedded connectors, nonlinear dynamic analysis, testing.

1. INTRODUCTION

The technique of casting concrete panels on the ground and then lifting (tilting) them upright to form walls originated in California about 50 years ago as a method of constructing solid reinforced concrete walls for industrial buildings. Today, tilt-up concrete walls are commonly used throughout Canada and the US to construct warehouses, shopping centers, office buildings, and schools.

Many tilt-up buildings in California have timber roofs that dissipate energy when the structure is subjected to large seismic demands. Experience from recent California earthquakes has shown that the weak link in these structures is often the out-of-plane connection between the concrete tilt-up walls and the timber roof diaphragm (Hamburger and McCormick, 1994). In Canada, and more recently in the western US, steel deck diaphragms are commonly used, and these are constructed in such a way that the diaphragms will likely remain elastic during the design earthquake. When the stiff concrete tilt-up walls reach their lateral load capacity, the flexible elastic diaphragms cause a magnification of the drift demands (Adebar et al. 2004).

There are essentially two types of concrete tilt-up walls. Many modern tilt-up buildings have very large openings along an entire side of the building, or on more than one side. This occurs, for example, when there are offices along the front of warehouses, and with store fronts in shopping centers. The small strips of concrete walls around and between these openings resist in-plane seismic forces like a cast-in-place reinforced concrete frame; however concrete tilt-up wall panels with large openings are constructed with minimal ductile detailing in the “beams,” “columns” and “beam-column joints.” Thus tilt-up frames have much less inelastic drift capacity than typical cast-in-place frames. Tilt-up frames are also subjected to larger drift demands than typical cast-in-place frames because of the very flexible elastic roof diaphragms as described above. Further discussion on the seismic design of tilt-up wall panels with large openings is given by Adebar et al. (2004).

The other type of tilt-up wall is the original concept of mostly solid walls as shown in Fig. 1. Solid wall panels and wall panels with small openings are inherently stiff and strong. The strength and inelastic response of wall constructed of these elements is controlled by the connections to these elements. There are connections between two or more walls panels to increase the rocking resistance of the walls, and there are connections between wall panels and the roof diaphragm. Finally, there are connections between wall panels and the base slab. The research results presented in the current paper relates to the wall-to-slab connection.

Fig. 1. Concrete tilt-up walls (solid and with small openings) ready for casting of concrete.

2. EXPERIMENTAL STUDY

A survey of current construction practice in Canada and the US was done in order to establish the typical connection details that are used, and to establish the characteristics of typical buildings to be used in the nonlinear analysis of the building. The connection between wall panel and base slab typically involves an embedded plate (with shear studs) cast into wall panels, and an EM5 connector cast into the base slab. Fig. 2 shows the details of an EM5 connector, which has a bent 20M bar welded to the underside of a steel angle.

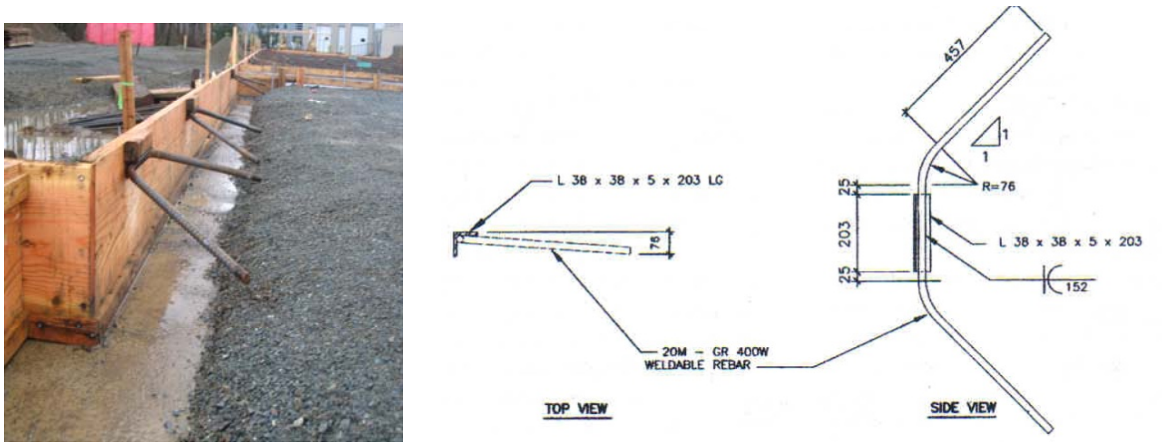


Fig. 2 EM5 Connector commonly used to connect tilt-up wall panels to base slabs: photo of connectors in base slab form (left), drawing of connector (right).

Previous testing at the University of British Columbia (Lemieux et al. 1998) demonstrated that the embedded plate in the wall panel, called EM2 or EM3, remains fairly rigid, while the strength and inelastic response of the connection is controlled entirely by the EM5 portion in the base slab. This permitted the wall panel and embedded steel plate to be simulated by a large steel plate in the current study.

Fig. 3 shows the test set-up for the current wall-to-slab connection tests. The test setup has a long horizontal steel loading beam which is used to load the connector in pure shear in both the horizontal and vertical directions. In an actual tilt-up wall panel, the base of the panel is free to move away from the slab and this was simulated in the test set-up. Vertical movement of the simulated wall panel is accomplished by two manual independently controlled hydraulic jacks connected to the horizontal loading beam. The horizontal movement is accomplished by a servo-controlled hydraulic jack (see left side of Fig. 3).

The purpose of the tests was to determine the interaction between shear response in the vertical and horizontal directions of the EM5 connector in the slab. In order to determine this interaction, 20 tests are be conducted with various levels of cyclic vertical uplift causing varying degrees of damage. Once cycled to a predetermined vertical displacement level, the vertical displacement level is held and then cyclic horizontal loading is applied so that the horizontal response for a given level of vertical damage can be determined. Tests will range from specimens with no vertical damage to damage levels approaching failure due to uplift alone.

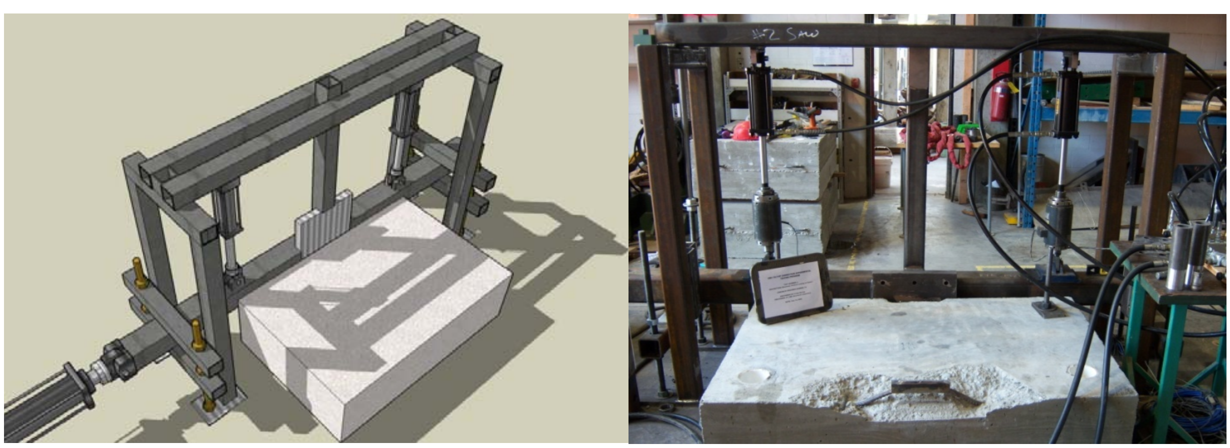


Fig. 3 Test set-up for wall-to-slab connection tests.

The first specimen was tested in monotonic uplift. As shown by the load – displacement plot shown in Fig. 4(a), the initial stiffness changed at about 2 mm vertical displacement and 19kN force. This occurred due to initial cracking of the concrete. A drop in the load was noticed at about 40 mm displacement which was caused by bending of the 20M reinforcing bar at the point of intersection with the internal slab reinforcement. The load then increased as tension force developed in the bent reinforcing bar. The final failure occurred at a vertical displacement of 130 mm and a vertical uplift force of 164kN. Failure was due to failure of the steel angle component of the EM5 connector (see Fig. 4a).

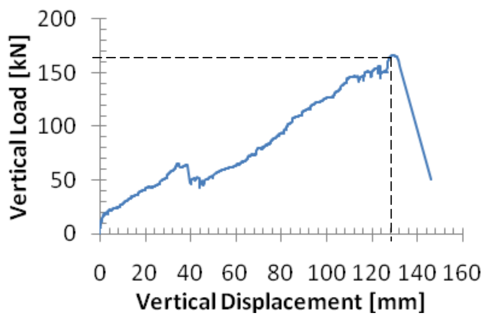
The second specimen was tested under cyclic uplift. The first three cycles were to a vertical displacement of 50 mm, which required a maximum vertical force of 83 kN as shown in Fig. 4(b). During the second cycle to 75 mm displacement, the weld between the EM5 and wall plate failed as shown in Fig. 4(b). The failed weld in the second specimen was repaired and testing was continued as shown in Fig. 4(c). The weld failed and then was repaired two more times. After loading to the third cycle at 100mm vertical displacement, the reinforcing bar in the EM5 connector fractured.

The third specimen was first subjected to three cycles to 50 mm vertical displacement. After being restrained in the vertical direction at 50mm displacement, the reverse cyclic horizontal force was applied. The first three cycles were +/- 25mm horizontal displacement. This required a maximum horizontal force of 199kN. The reinforcing bar fractured during the second cycle to +/- 37.5 mm horizontal displacement.

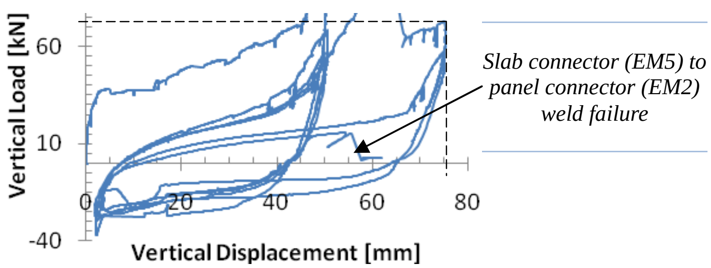
3. NONLINEAR DYNAMIC ANALYSIS

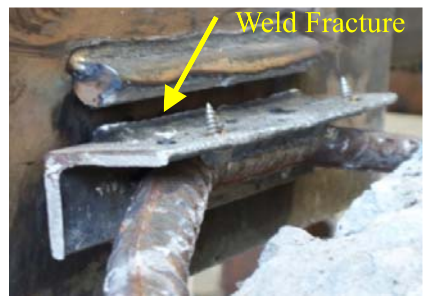
Based on the assessment of the wall-to-slab connections discussed above, strong ground shaking may result in failure of the wall-to-slab connections with limited energy dissipation at the connection. Failure of the connection may subsequently lead to either sliding or rocking of the wall on the foundation. In an effort to explore the nonlinear response of tilt-up buildings after failure of the slab-wall connection, and to investigate the preferred mode of response for the design of new tilt-up buildings, a nonlinear dynamic analysis study was undertaken considering two nonlinear models: (a) Sliding model where the walls are allowed to slide relative to the foundation after exceeding a friction force based on the dead load at the base of the wall panel; and (b) Rocking model where the walls are restrained from sliding but allowed to uplift as the overturning resistance provided by the weight of the panel and applied loads is exceeded. An extensive nonlinear analysis study applied the methodology of ATC-63 (2008) to evaluate the nonlinear system performance of tilt-up buildings (Olund 2008). Due to space limitations, the following will focus on the comparison of the sliding and rocking modes of response.

In order to establish a typical building design upon which to base the analytical models, a survey was conducted with various firms practicing in the tilt-up industry to assess ranges for various parameters of a single story tilt-up building. A building layout with components and dimensions most commonly used in current practice was adopted. A simple rectangular building plan [30.48m (100ft) x 60.96m (200ft)] was assumed for the nonlinear model. Due to the focus on the rocking and sliding modes of response, for the study presented here all walls were assumed to be solid panels 184 mm (7.25 in.) thick, 7.62m (25ft) wide and 9.144m (30ft) high. Walls with openings were considered in the comprehensive nonlinear study (Olund 2008). The roof deck, using 16ga and 18ga corrugated steel decking with No. 14 screws at side and end laps and Hilti pins to connect deck sheets to underlying steel members, was assumed to remain linear elastic. Previous analyses (Adebar, et al. 2004) have shown that if two components are acting in series, as is the case with a steel deck diaphragm and concrete wall panels in a tilt-up building, and one component yields with limited strain hardening, then the other component will be protected from yielding. As the focus of the current study is on the response of tilt-up buildings with nonlinear response concentrated in the wall panels and connections, the linear-elastic assumption for the steel deck diaphragm is justified. The design of the wall panels and connections was modified to ensure the building response displayed the desired mechanism; either sliding or rocking. For example, in order to ensure the desired mechanism could develop, no connections were provided between adjacent corner panels, such that the response of the model in two directions is decoupled.

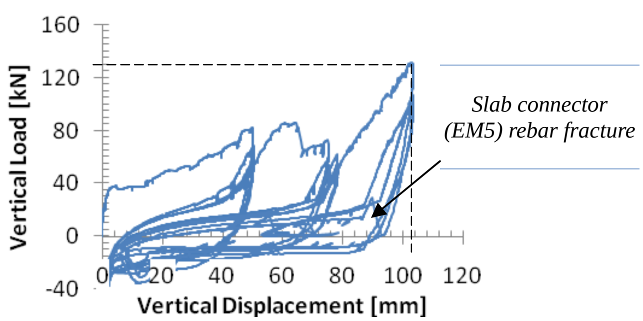


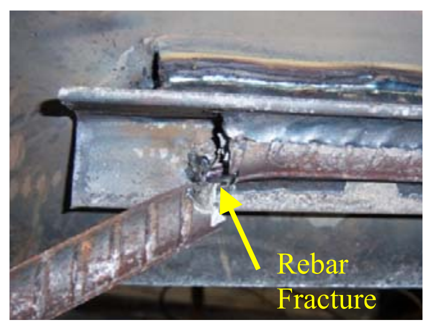
Specimen 1: Monotonic vertical uplift



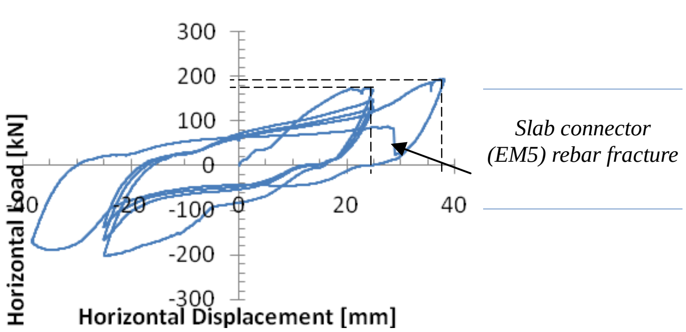


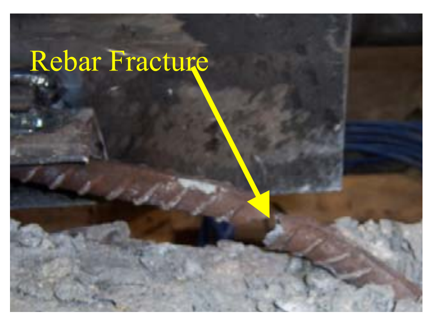
Specimen 2: Cyclic vertical uplift





Specimen 2 (after weld repair): Cyclic vertical uplift continued





Specimen 3: Reverse cyclic horizontal shear with 50 mm uplift displacement

Fig. 4 Summary of experimental results from first three wall-to-slab connection specimens.

Three-dimensional nonlinear models of the sliding and rocking buildings were developed using Perform-3D (CSI 2006). For the sliding model, it was assumed that all panels were connected to adjacent panels with contact elements at the base of the walls to model the friction between the wall panels and the footings. The properties for these contact elements include a large stiffness in compression, a very small stiffness in tension, and a large stiffness in shear. The shear resistance is proportional to the applied vertical compression load. A friction coefficient of 0.42 was selected to ensure the lateral force required to initiate wall sliding was the same as the lateral force required to initiate wall rocking in the rocking model. This was done to allow comparison between the two mechanisms. For the rocking model, the connection layout was modified. It was assumed that panels were not connected to adjacent panels and thus able to rock independently. The panels were also assumed to be connected at the base with connections that would stop sliding from occurring but would allow rocking of the panels to occur.

Rayleigh damping equivalent to 3% of critical was used at vibration periods of 0.2 and 1.5 times the first mode period. The first mode period from Perform-3D for both models was 0.58 seconds. The first mode period of the structure was also estimated using the following equation recommended by ASCE/SEI 41 (2007):

$$T1 = (0.1\Delta_w + 0.78\Delta_d)^{0.5}$$

where Δ_w and Δ_d are in-plane wall and diaphragm displacements in inches, due to a lateral load in the direction under consideration, equal to the weight of the diaphragm. Using the above equation, the first mode period was calculated to be 0.41 seconds, considerably shorter than the first mode period determined from the dynamic model. However, when the ASCE 41-06 estimate was modified by considering half the weight of the out-of-plane walls, the resulting period was 0.63 seconds, only 9% longer than the period determined from the dynamic model. The inclusion of half the weight of the out-of-plane walls is felt to be reasonable, since it is included in the seismic demands in standard design practice.

To observe the distinct characteristics of each mechanism, the results from a time history analysis for one earthquake applied in the direction of the short axis of the building was considered for each model. The record considered was from the Northridge earthquake and was recorded at the Beverly Hills - Mulhol recording station (MUL009). The record was scaled such that the 3% damped spectral acceleration at the first mode period was equal to 1.0g (i.e. $S_a(T_1) = 1.0$g) to initiate inelastic response. The breakdown of energy dissipation mechanisms was investigated for each model. The observations described below were typical of other ground motions used in the comprehensive nonlinear study (Olund 2008)

The response of the sliding model for the record described above can be illustrated by considering the drift at the roof mid-span (displacement at mid-span divided by the height of the roof) and the drift at the top of the end wall panels (displacement at top of end walls divided by the height of the end walls) (Fig. 5a). Approximately seven seconds into the earthquake, the end walls begin to slide on the foundation. The maximum roof drift is approximately 0.8% (73mm). A residual drift of approximately 0.2% (18mm) is observed. The roof diaphragm essentially oscillates about the position of the end walls. An investigation into the breakdown of the dissipated energy reveals that the majority of the energy dissipation (70%) consists of inelastic energy dissipated by the sliding action of the walls. The remaining energy dissipated is due mainly to damping resulting from movement of the building mass and deformation of the roof diaphragm (Alpha-M and Beta-K energy respectively).

Fig. 5(b) shows the response of top of wall and mid-span of roof for the rocking model when subjected to the same ground motion. Similar to the sliding model, approximately seven seconds into the earthquake, nonlinear response is observed as the end walls begin to rock on the foundation. After rocking is initiated, the walls and roof maintain nearly the same displacement for a significant duration of ground motion. The maximum roof drift is approximately 3.2% (290mm). There is no residual drift; the wall panels always rock back to their original position. It is important to note that the maximum roof drift is approximately four times the maximum roof drift observed for the sliding model. This is due to the fact that the rocking mechanism does not dissipate as much energy as the sliding mechanism. A review of a breakdown of the dissipated energy reveals that approximately 80% of the total energy dissipated consists of Alpha-M viscous energy dissipated by movement of the building mass. About 15% of the total energy dissipated consists of Beta-K viscous energy dissipated by deformation of roof bracing and wall panels.

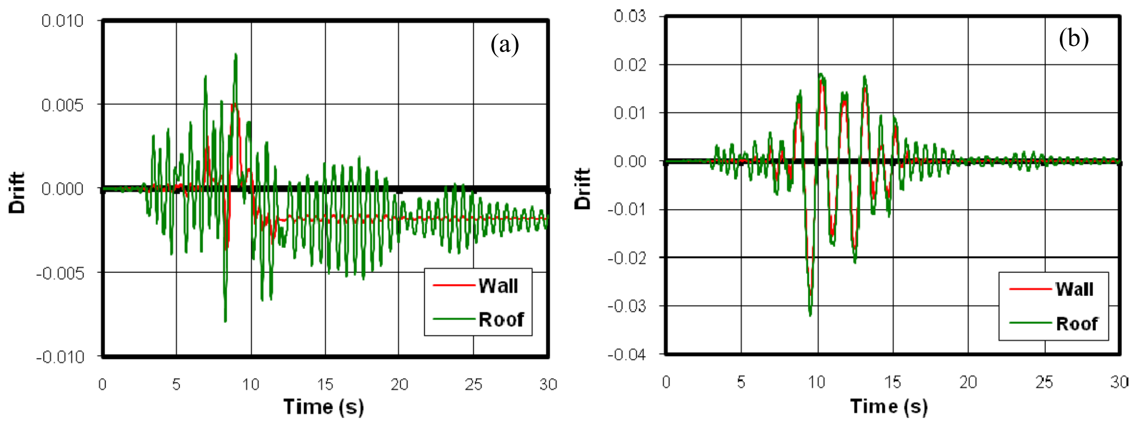


Fig. 5 Wall and mid-span roof drifts for ground motion MUL009 scaled to $S_a(T_1)$=1.0g: (a) Sliding model; (b) Rocking model

To further illustrate the behaviour of the sliding and rocking models, eight earthquake records from those recommended by ATC-63 were selected to perform Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002). The records were applied in the direction of the short axis of the building only. To illustrate the IDA results, the first mode spectral acceleration was plotted against median peak roof drift and median peak wall drift for the selected ground motions in Fig. 6.

From Fig. 6(a) it can be observed that the sliding model remains elastic until the ground motions are scaled up to a first mode spectral acceleration of approximately 0.25g, at which point the wall panels begin to slide. This is consistent with the pushover curve for this model, in which the total base shear required to cause the panels to slide was 0.25g. The median IDA curve appears to have a flattening trend,

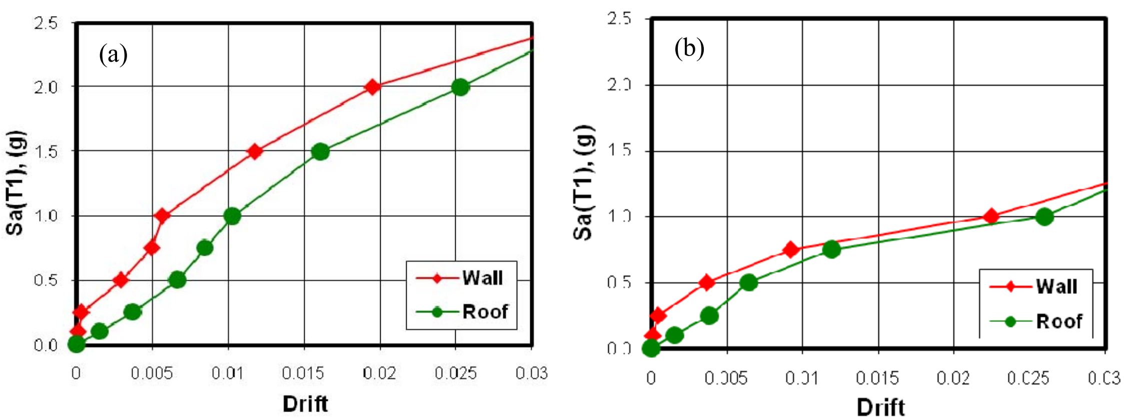


Fig. 6 Median IDA response for eight ground motions: (a) Sliding model; (b) Rocking model

but does not completely flatten out for the range of drifts shown above since the pushover curve for the sliding model is essentially elastic perfectly plastic and does not have a negative post-yielding slope. Similar to the sliding model, the rocking model also remains elastic until the ground motions are scaled up to a first mode spectral acceleration of approximately 0.25g, at which point the walls begin to rock (Fig. 6b).

Prior to sliding or rocking the roof diaphragm must accommodate nearly all of the displacement demands.

After sliding or rocking is initiated, the walls account for a larger portion of the displacement demands. This is particularly evident for the rocking model in Fig. 6(b), where after $S_a(T_1)$=0.75g the walls account for over 75% of the peak displacement demand. These results support previous studies by the authors (Adebar, et al. 2004) where simple nonlinear spring models were used to demonstrate that as the strength of a wall system decreases the wall displacement demands increase rapidly after the walls are permitted to yield, even though the total displacement demands do not change significantly.

The building layout used to form a basis for the analytical models is a very simple system. In reality, buildings are not perfectly rectangular and can have re-entrant corners, or some walls may not be parallel to each other. When these complications in building geometry are considered, the practicability of the sliding mechanism becomes questionable. Another problem is that the actual coefficient of friction between the wall panels and the foundation may be difficult to predict. In addition, due to the residual displacement inherent in the sliding mechanism, earthquake damage to a building designed to fail in this manner would be difficult to repair. Despite the larger drift demands observed in Figs. 5 and 6 for the rocking model, the rocking mechanism is more practicable since it inherently does not result in residual displacement; rocking panels always return to their original position. One aspect that must be given special consideration in the design of a rocking tilt-up system, however, is the deformation imposed on the roof perimeter angle due to rocking of the panels.

4. CONCLUDING REMARKS:

This paper has presented ongoing research at the University of British Columbia on seismic behaviour of typical wall-to-slab connectors used in one-story tilt-up buildings and the resulting nonlinear response of the building system after failure of connectors. Wall-to-slab connectors are expected to experience both shear and uplift demands when a tilt-up building is subjected to strong ground motion. Experimental results have shown that the weld connecting the embedded stud plate in the tilt-up wall panel to the EM-5 connector in the slab is vulnerable to fracture at approximately 75 mm of vertical uplift. A revised weld detail will be developed, and the experimental results will be used to develop a nonlinear analytical model for wall-to-slab connectors such that nonlinear dynamic analysis can be used to assess the deformation demands during strong ground shaking.

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