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INTERACTION BETWEEN STRUCTURAL FRAME AND WALL CLADDING IN A SHAKE TABLE TEST OF A FULL SCALE FOUR STOREY BUILDING

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ABSTRACT

A full scale four storey steel frame building designed following current Japanese specifications was tested by the E-Defense Hyogo Earthquake Engineering Research Center in Japan. The model was subjected to three consecutive earthquakes with increasing amplitude simulating different intensity seismic demands. The tested building had Autoclaved Lightweight Concrete (ALC) panel external wall cladding installed to the structural frame, using the locking method.
This paper presents results of nonlinear dynamic analyses performed on a structural model calibrated to measured properties of the steel building prior to testing. The variations in the expected performance levels with and without accounting for the presence of the ALC panels is investigated. The results of the study show that the contribution in stiffness of the ALC panels significantly reduce the nonlinear deformation demands in the structural elements for the tested building and should be considered in the analysis as part of a performance based design.

Introduction

A shake-table test of total collapse of a four-story moment frame was carried out in September 2007 at the E-Defense shake-table facility. This test is a part of the experimental project on steel buildings being conducted at the E-Defense shake-table facility (Kasai et al, 2008). The building specimen was designed following current Japanese specifications and practices. This test allowed observing the seismic performance of structural and non-structural components in the building.

Along with this test program, a blind analysis contest was carried out with the task of presenting a prediction of the response before and after the test. This paper presents a numerical model of the building carried out using the structural analysis software PERFORM 3D (Powell, 2007) and the results of a sensitivity analysis to determine the effects of various parameters on the analysis results.

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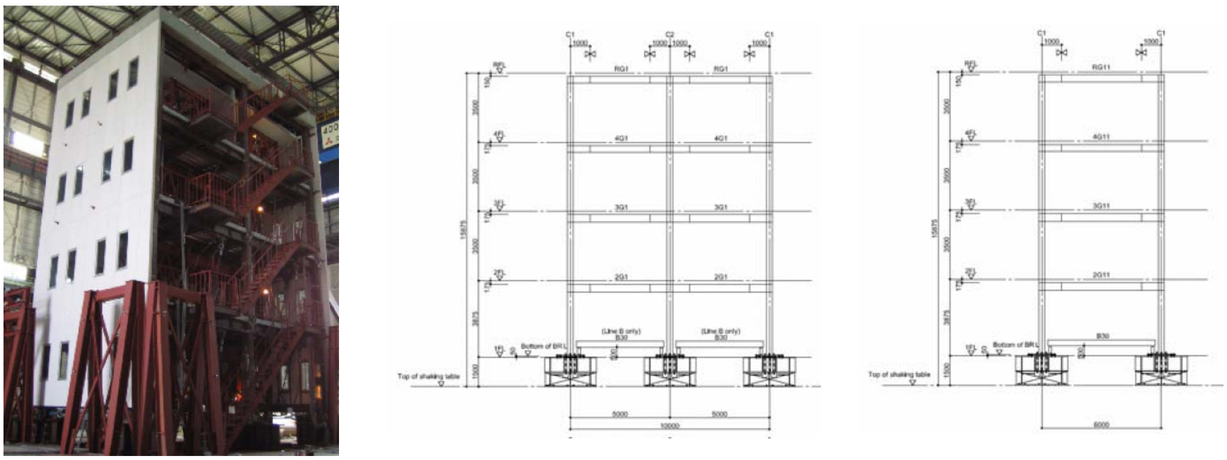
Description of Test Specimen

The test structure was a steel moment resisting frame with concrete slab and Autoclavated Lightweight Concrete (ALC) panels for exterior walls, as shown in Figure 1a. The structure consisted of two frames composed by two bays 5m long in the NS-direction and three frames with one bay 6m long in the EW-direction. The height of the first floor columns was 3.85m, other floors were 3.50m and at the top of the building there was a parapet 0.9m height from the net height of the roof slab, for a total height of the building equal to 15.25m. Figure 1b & 1c shows the structures dimensions.



a) Specimen Building

b) Side View
NS-Y direction

c) Side View
EW-X direction

Figure 1. Building description and structural system detail (reproduced from Kasai et al. 2008)

All the columns in the lateral resisting frame had a square cold-formed hollow structural section of 300mm by 300mm with 9mm thickness. All the girders in the lateral resisting frame were made of hot-rolled wide-flanges. For the design of the steel frame building the yield stress of 300 MPa was assumed for beams and 380 MPa for columns. Improved details and fabrication practice developed following the 1995 Hyogoken-Nanbu (Kobe) earthquake were used for all beam-column connections.

Concrete slab was constructed using design strength concrete of f'c=21 MPa and a steel form deck that would remain permanent as a composite floor deck. The total depth of the deck varied between 75mm and 175 mm, and the bottom of the floor deck was placed on top of the upper flange of the girders.

The specimen building had exterior walls using autoclaved aereated (ALC) concrete panels and dry partition for interior walls. The material properties were not specified for the panels or the connection elements. Other non-structural components included sash windows, ceilings, parapets, and the anti collapse system.

Analytical Model by UBC

Prior to the shake table tests, practicing engineers and researchers were presented with the challenge of modeling the four storey steel frame building and predicting the response of the structure for different levels of consecutive shaking using the 1995 Kobe Takatori acceleration record. The blind prediction was to include the envelopes of displacement, acceleration, interstorey force and drift response values of the tested structure for the level of 60% of the amplitude of the ground motion. All the necessary structural properties as well as the dead loads were presented by the organizers (E-Defense 2007).

The authors of this paper prepared an analytical model at the University of British Columbia (UBC) that consisted of 3d frame element model of the four storey frame and that from here on in will be referred to as the "UBC Model #1". Each structural component was modeled as a bending element with linear elastic material properties along the length and accounting for nonlinear behavior only at both ends of the member. Nonlinear behavior for columns was modeled using fiber elements and for beams using nonlinear rotational springs. Concrete slabs were modeled as fully elastic and the interaction with beams elements was accounted for in the latter's positive moment capacity. The structural joints or panel zones, based on their tested lateral strength and stiffness (E-Defense 2007), were assumed as rigid elastic elements. The ALC panels were not included in the model.

The fiber element springs in the UBC Model #1 for the steel hollow square sections followed the corresponding material stress strain properties. The plastic hinge length was allowed to have a 200 mm length, separated into two segments for simulation of the distribution of plastic strains. The model was then compared to the results of the cyclic static loading of columns with the same section and material properties(E-Defense 2007). The comparison allowed observing close agreement in the estimation of the columns elastic stiffness. The column cross section was found to be a class 3 following the canadian steel standard (CISC, 2004) and would allow attainment of the yield moment, without significant ductility capacity.

A modal analysis was performed on the UBC Model #1 prior to the nonlinear time history analysis. The modal analysis showed that the directions X and Y had decoupled modes of vibration. The periods of vibration for the lateral modes shown in table 1, were consistent with the initial analytical model prepared for E-Defense (Kasai et al 2007).

Table 1.0 Periods of Vibration for Blind Prediction Structural Mode

Mode	Lateral X	Lateral Y	Torsional	Up
First	1.00sec	0.96sec	0.81sec	0.16sec
Second	0.32sec	0.31sec	0.27sec	0.11sec
Third	0.17sec	0.16sec	0.14sec	0.06sec

Shake Table Experiment at E-Defense

The objectives of this experiment were to evaluate structural and service performance of the steel moment frame under design-level ground motions, and to determine the safety margin against collapse under exceedingly large ground motions. A detailed report on the observed results of the shake table tests were presented in Kasai et al 2008.

The testing program consisted of consecutive shake table tests running all three directional components of the Kobe Takatori acceleration record. The ground motion was run scaled at amplitudes of 5%, 20%, 40%, 60% and 100%.

Prior to applying the first level of excitation, a free vibration test was carried out to examine the fundamental natural period and damping factor of the specimen building. Fundamental natural period of X and Y direction were 0.80 sec and 0.76 sec, and damping factor of the X and Y directions were 2.1% and 2.3%, respectively.

In the response under 20% amplitude no yielding was observed in any structural member except for minimal local deformation of partition walls at the connection with frames of doors and sashes. The peak story drift angle was less than 0.5%.

For the 40% amplitude, all columns of the first story were slightly yielded at the base and remained elastic at the top. All beams behaved elastically with larger yield strength than column members. The primary inelastic deformations were observed in the panel zones at the 2nd floor and also yielding in the lower portion of the center columns.

For the 60% amplitude, the structure reached a maximum reported lateral load value of 1410 kN in the Y direction and 1160 kN in the X direction. More plastic hinges formed at the top portion of the center columns and at the beam column joints of the 3rd floor. The maximum measured interstorey drift for the first floor was of 1.9% drift in the Y direction and 1.3% in the X direction.

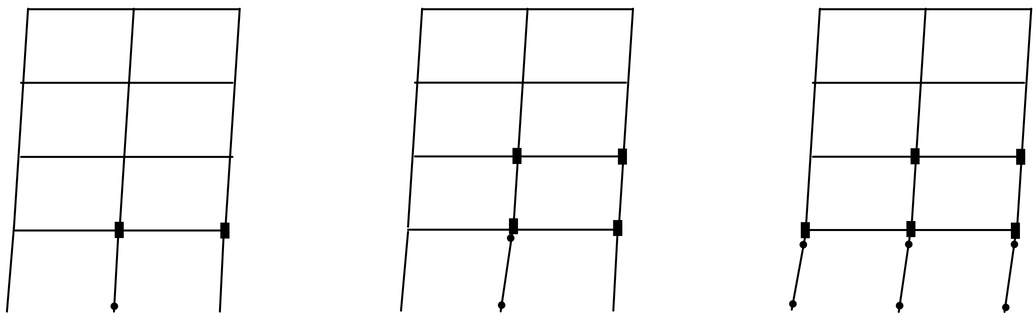
For 100% amplitude the structure formed a soft storey mechanism, forming plastic hinges at the top and bottom of the columns of the first level. All girders performed elastically due to its high bending strength and composite action with the concrete slabs, for all levels of shaking.

The overall dynamic response of the structure showed small ductility capacity prior to failure. Figure 2 illustrates the forming of the plastic hinges for each level of shaking until the formation of the yielded mechanism of the structure. This response showed the result of having similar moment strengths for the column and beam elements. These results also showed the little available moment redistribution and formation of additional plastic hinges prior to the formation of the soft storey mechanism for this structure.

Comparison of UBC Model Predictions and Experimental Results

The results from testing included envelopes of the peak response reached at each floor level during testing for the 60% amplitude of the Kobe earthquake for relative displacement. These envelopes were for values of relative displacement, absolute acceleration, shear force, overturning moment and interstorey drift.

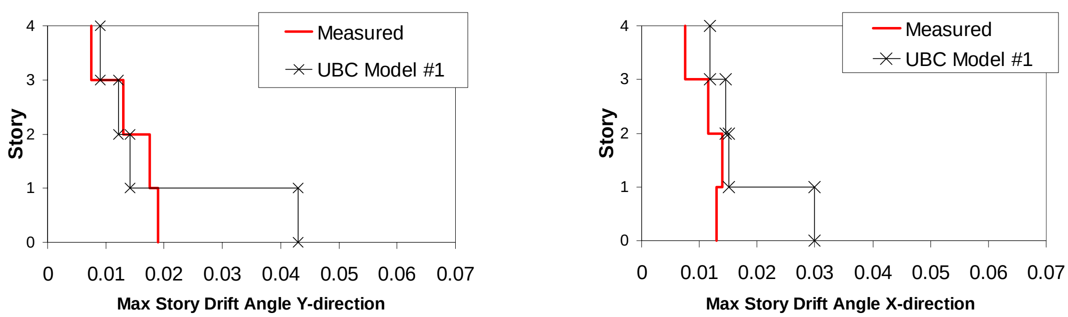
From the comparison of the predicted analytical results and the measured experimental results it was observed that the UBC model #1 accurately determined the proclivity to form a soft storey mechanism as well as all beams to remain elastic. However, during the test, this mechanism was not formed during the 60% amplitude test but for the higher level of shaking.



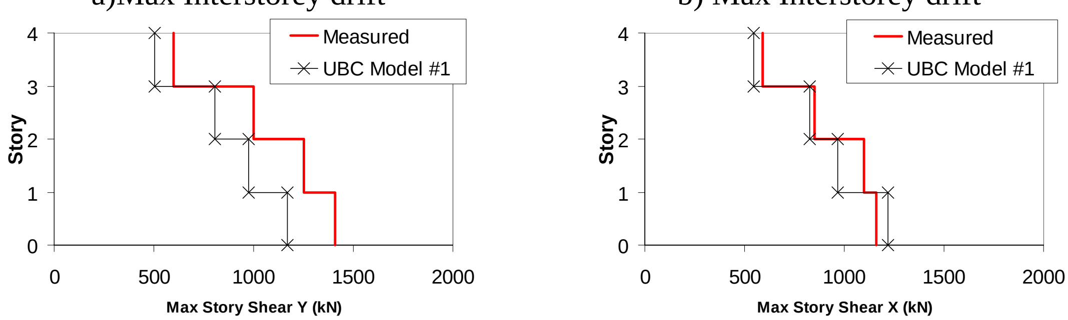
a) After 40% Kobe Acceleration Record | b) After 60% Kobe Acceleration Record | c) After 100% Kobe Acceleration Record

Figure 2 Distribution of formation of plastic hinges after different levels of excitation

Figure 3 presents a comparison of the measured results with the values determined from the UBC model #1. The demands predicted were shown to be larger interstorey drifts and lower lateral loads than what were measured during the experimental test. As a result the UBC model #1 was found to have predicted a higher level of damage for the earthquake ground motion than what was observed in the experimental test.



a)Max Interstorey drift | b) Max Interstorey drift



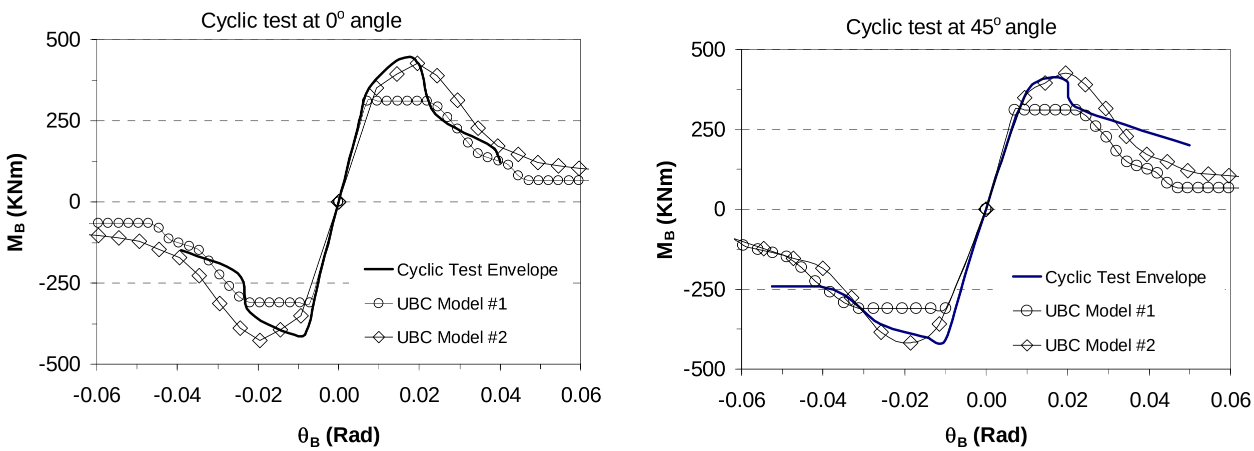
c)Max story shear (kN) Y direction | d) Max Story Shear (kN) X direction

Figure 3. Response Envelope over the height for 60% Takatori record

After the competition, information was presented showing that the measured period of vibration of the structure in the Y direction (T=0.76sec) differed from the UBC model #1 estimated period (T=0.96sec) and similarly for the X direction. This indicated that the structural model had not accounted for significant stiffness contribution from the non structural components.

Sensitivity Analysis

To evaluate the parameters that influence the differences between the UBC model #1 prediction results and the measured experimental results, a sensitivity analysis was carried out to find what model parameters needed correction. In this study, a second model, UBC Model #2 was developed varying the stress strain curve used in the fiber element modeling of the nonlinear portion of the steel columns. This variation was made to obtain more precise fit to the force deformation backbone obtained from the static experimental testing results. The variation in the stress strain model resulted in better estimation of force and deformation at yield and strength degradation when loaded with respect to the principal axis, and at 45 degree angle, as shown in Figure 4. The UBC Model #2 assumed a yield stress of 488 MPa at a strain of 0.0085, where UBC Model #1 had used 350 MPa at a strain of 0.002.



a) Backbone from loading at 0° angle   b) Backbone from loading at 45° angle

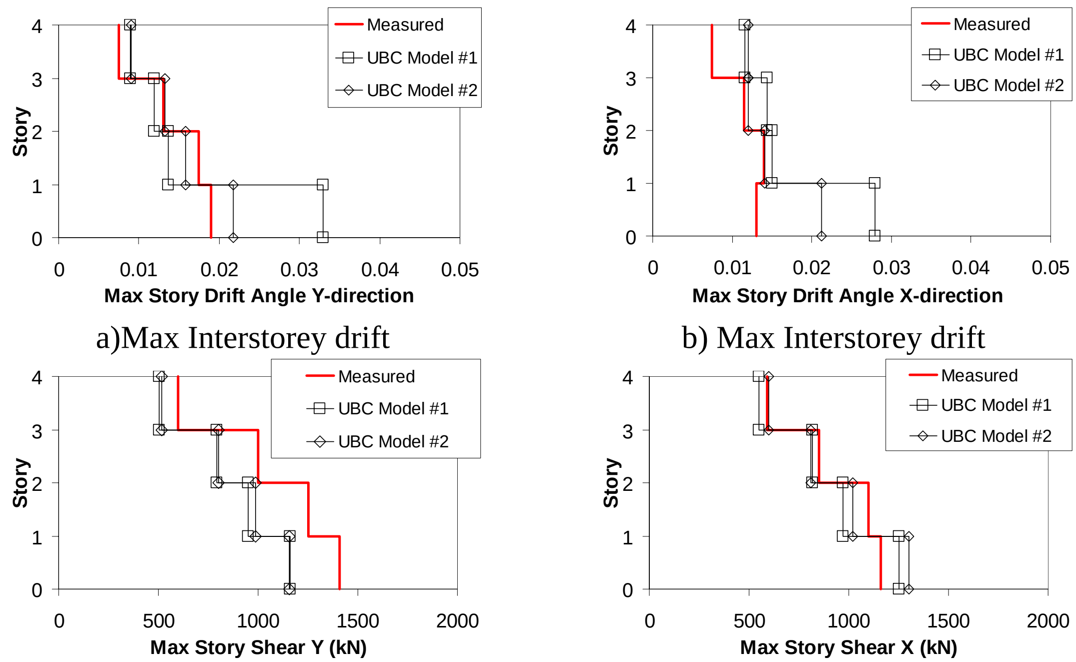
Figure 4. Load deformation backbone of test column with different stress strain properties.

The results of the sensitivity analysis for are presented in figure 5. The more precise force deformation backbone resulted in a significant improvement in the estimation of the interstory drifts of the first floor. However, the analysis also showed that the shear force demand was not significantly affected by the higher strength capacity of the structural elements.

It was found that accounting for a precise force deformation backbone capacity of the columns, the model did not result on a prediction close to the measured results. Similar sensitivity analysis of the damping and fiber discretization did not provide a significant improvement in the prediction results.

An addition analytical model, UBC model #3, was built to determine the effect of the contribution in lateral stiffness of the non structural components. The model starts from the

UBC model #1 and includes the ALC panels modeled as elastic shell elements accounting for all smaller panels following the assumption that they remain interconnected as the interstorey drifts are lower than the triggering drift for rocking behavior. To match the measured period of vibration, the effective shear stiffness for the shell elements were determined to be K_x = 6 kN/mm for walls along the X direction and K_y = 12 kN/mm in the Y direction.



a)Max Interstorey drift

b) Max Interstorey drift

c)Max story shear (kN)
Y direction

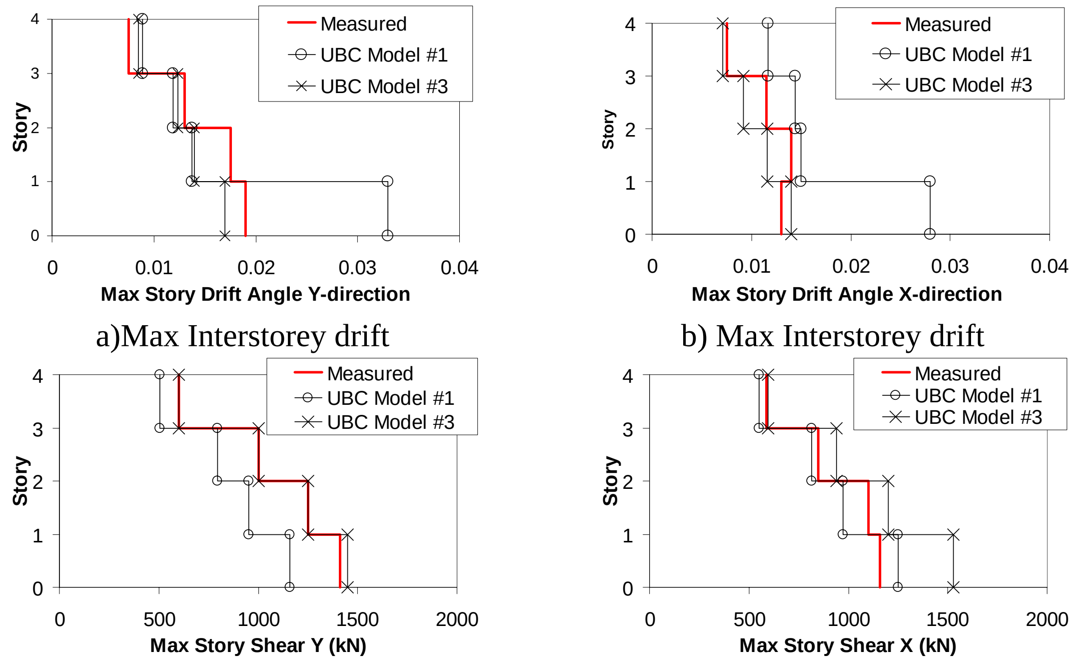
d) Max Story Shear (kN)
X direction

Figure 5. Sensitivity of results to variation in the stress strain values for the column for the shake table test at 60% amplitude

The results of this sensitivity analysis are presented in Figure 6, where it is shown that by including the contribution in lateral stiffness of the non structural walls there is an improvement in the estimation of the envelope of the response values of the structure, most noticeably for the Y direction. By observing the envelopes of the interstorey shear response, the estimation shows little error with respect to the measured response.

It is observed that for the second storey interstorey drifts in the Y direction there still remains an important error in the estimated results. This difference may be attributed to the inelastic shear deformation of the panel zones recorded for this level of shaking in that floor. In the X direction the analysis results show that there was error in drift for each floor indicating that the stiffness contribution in this direction was over estimated.

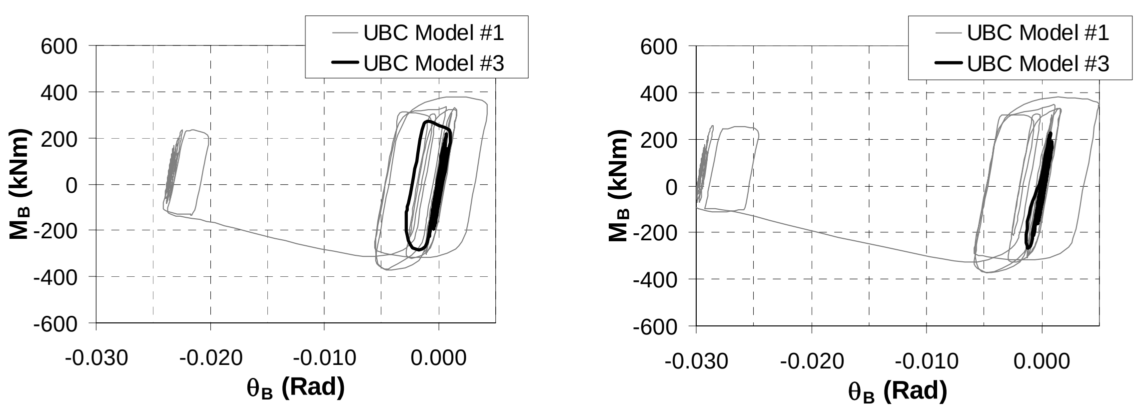
Figure 7 shows a comparison of the estimated moment rotation of the first floor at the bottom of the center west column and for the South West column of the first floor for the UBC model #1 and the UBC model #3. By including the non structural walls in UBC model #3, the analysis results are closer to the estimation in the plastic demand forced on the structural member during the response. The results allow determining that this level of shaking did not demand a high level of ductile response from the structure. The participation of the exterior walls allowed to avoid the complete formation of the soft storey mechanism, as the perimeter column on the south showed close to linear response, as seen in figure 7b for the southwest column. These observations are shown to be consistent with those from the experiment.



a)Max Interstorey drift b) Max Interstorey drift

c)Max story shear (kN) Y direction d) Max Story Shear (kN) X direction

Figure 6. Sensitivity of results to contribution of stiffness from non structural ALC wall for the shake table test at 60% amplitude.



Center Column South West Column

Figure 7 Effect of Non Structural Components on Nonlinear demands on Columns

Conclusions

From the results of this study it was found that the ALC panels increased the lateral stiffness of the building and provided additional base shear capacity. Furthermore, the analytical results showed that the contribution of the non structural walls reduced the nonlinear demands in the columns and prevented the formation of the soft storey mechanism at the 60% amplitude of the ground motion.

A performance based design of this building would have required to model the participation of the non structural components to better assess the seismic demands and the formation of plastic mechanism of the building.

Future Studies

Further studying is required to determine the shear capacity of the anchoring system and the triggering drift for rocking of the ALC panels

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