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ANALYSIS OF DIAPHRAGM FORCES IN A FIVE-STORY MINIATURE STEEL BUILDING DURING SHAKING TABLE TESTS

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SUMMARY

Large floor horizontal accelerations have been recorded in buildings in earthquakes. Such accelerations are responsible for inertia forces causing damage to services and are a major reason for structural damage even building collapse. This paper describes the dynamic response on a shaking table of a five-story-steel frame building with removable fuses. Floor accelerations measured in the specimen are compared with predicted values resulting from the use of current code approaches and a proposed method.

INTRODUCTION

Floor accelerations in buildings during strong earthquakes can reach values higher than those considered in the design, which may lead to the collapse of the floor system, even though structural elements, as columns or walls, are responding in the elastic range (Rodriguez et al, 2002). Floor accelerations produce in-plane diaphragm forces which when exceed the diaphragm resistance, the floor system could not transfer these forces to the vertical structural elements. Evidences of this behavior have been observed in some precast buildings during the Northridge earthquake, California in 1994 (Fleischman et al, 1998).

The current Mexico City Building Code (MCBC, 1993) approach for the evaluation of in-plane diaphragm forces is simplistic and on the unsafe side (Rodriguez et al, 2002). According to this code, these forces can be assumed equal to the set of forces leading to the maximum base shear. The new Mexico City Building Code (MCBC, 2004) has improved these provisions since specifies a set of lateral forces higher than that specified for the structural system.

A five-story miniature steel building designed for seismic resistance was subjected to unidirectional input ground motion on a shaking table. The specimen was designed following details used for a similar specimen in a previous research conducted at the University of Canterbury, New Zealand (Kao, 1998). A relevant feature of the specimen was the use of replaceable fuses for joints located in potential plastic hinges in beams and base columns. Prior to the shaking table tests, representative joints in the specimen were tested with reverse cyclic loads in order to measured their rotational stiffness and strength. Results

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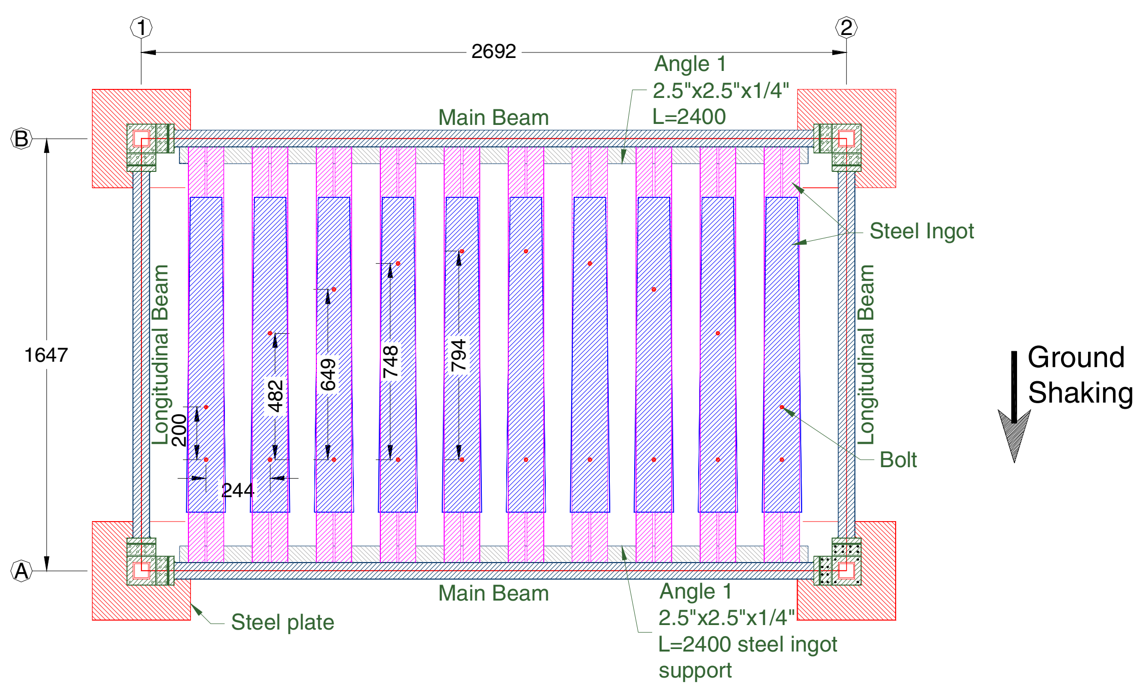
from these tests were incorporated in the analytical studies of the specimen. Results from the shaking table tests are described in the paper and compared with prediction using code approaches and a proposed method.

DESCRIPTION OF THE SPECIMEN

The specimen represents a three-dimensional five-story frame structure and was 2.8 m high. The specimen was considered a miniature building, therefore the seismic records used as input in the shaking table tests were not scaled. The specimen was designed following the Guerrero code (NTCDS-EG-89). According to these provisions, the elastic specified seismic coefficient, c, was equal to 0.4, the performance reduction factor Q' was equal to 4 and the allowable interstory drift limit d_r was equal to 0.012. The design gravity load per level was equal to 5.0 kPa.

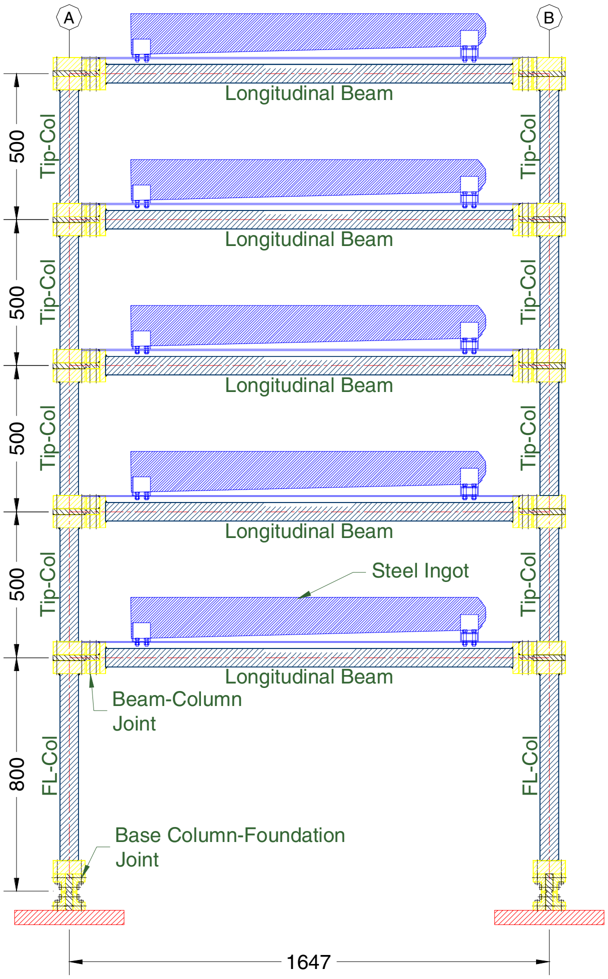
The geometry and dimensions of the specimen are shown in Figure 1. Seismic mass at each floor were introduced using steel ingots supported by steel plates. These plates were supported on the transverse beam (main beam in Figure 1a). A 3/16" topping steel plate at each level were bolted to the ingots forming a parabolic arch arrangement, and therefore defining a load path designed for avoiding bending of transverse beams.

A lateral elevation of the building and fuse location are shown in Figure 1b. The specimen was designed to respond in the inelastic range during a selected ground motion, which was applied in the specimen's longitudinal direction (see Figure 1a) in the short side of the plant. Figure 1c shows typical dimensions of a beam-column joint. Figure 2 shows a view of the specimen during construction on the shaking table.

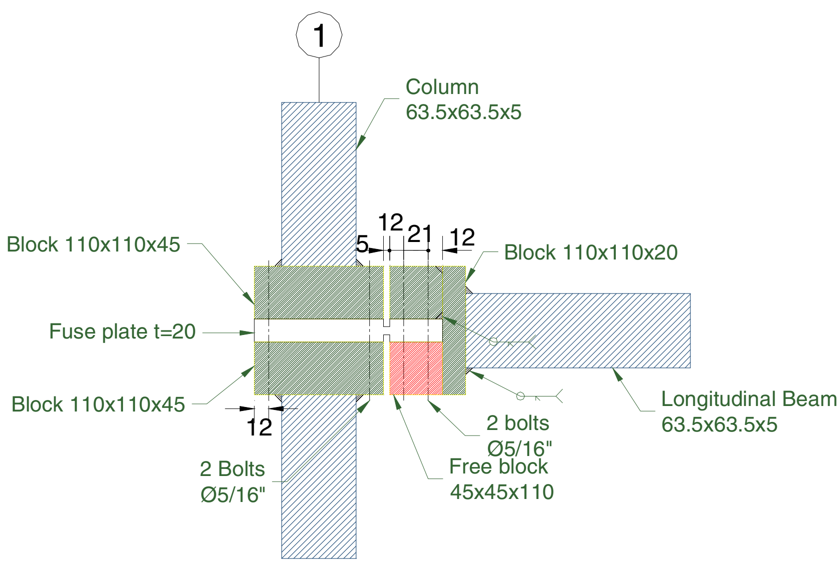


a) Plan

Figure 1. General details of the specimen tested in the shaking table (dimensions in mm)



b) Elevation



c) Beam-Column Joint

Figure 1. General details of the specimen tested in the shaking table (dimensions in mm, continuation)

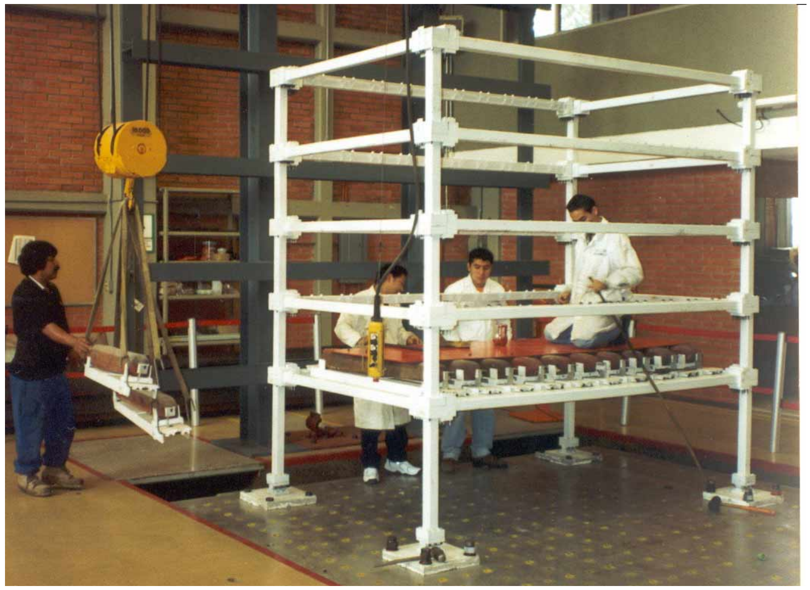


Figure 2. View of the specimen during construction

DESCRIPTION OF TESTS

Test prior to the shaking table tests

Test on beam-column joint specimens

Quasi-static reverse cyclic loading test on four beam-column joint specimens were performed to investigate the behavior of the joints. Results from the testing were incorporated in the analytical dynamic studies. A typical beam-column joint specimen is shown in Figure 3.

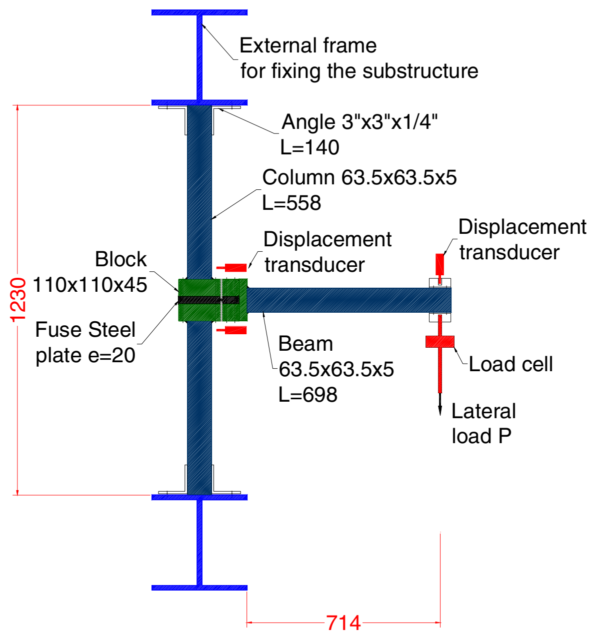


Figure 3 Beam-column joint set-up (dimensions in mm)

Lateral loads were applied at the beam ends of the beam-column joint specimen, see Figure 3. The lateral loading history used in the tests was based on force control during elastic response of the specimen, followed by displacement control during inelastic excursions. Results from these tests were used for defining strength and stiffness properties of typical fuses in the specimen.

As seen in Figure 1, for defining a plastic hinge location the fuse has a thickness reduction with dimensions of 15, 12 and 10 mm. These different fuses were distributed along the several members of the specimen to comply with the specimen's design requirements.

Free-vibration test

Prior to the shaking table test, the specimen was subjected to free-vibration tests in order to evaluate dynamic properties such as natural frequencies and damping ratios. These tests were conducted by first pulling the building by using a weight of 0.5 kN and a cable attached to the fifth floor and then releasing the specimen by cutting the cable. The response was measured with accelerometers placed at each floor level.

Shaking table tests

The specimen was subjected to two levels of ground shaking. In a first level of shaking the specimen was expected to respond in the elastic range, and this level of testing is labeled as "Low-Level Shaking Table Test". A second level of testing was designed for expected severe damage in the specimen, and is labeled as "High-Level Shaking Table Test". The ground motion record for the Low –Level Shaking Table Test was obtained using the LLolleo record obtained in the 1985 Chile Earthquake with accelerations affected by the factor 0.1.

Figure 4 depicts a typical plant of the building showing the location of accelerometers and displacement transducers. Three accelerometers were used at levels 1 to 4, two of them oriented in the North-South direction (N-S) and the third oriented in the East- West direction (E-W). Four accelerometers were placed on level 5 of the specimen, two of them oriented in the N-S direction and the other two oriented in the E-W direction.

Figure 5 shows the specimen placed on the shaking table. This figure also shows the transducers for measuring hinge rotations and lateral displacements. A steel structure was constructed on the shaking table besides the specimen for supporting transducers installed for measuring the specimen's lateral displacements (see Figures 4 and 5).

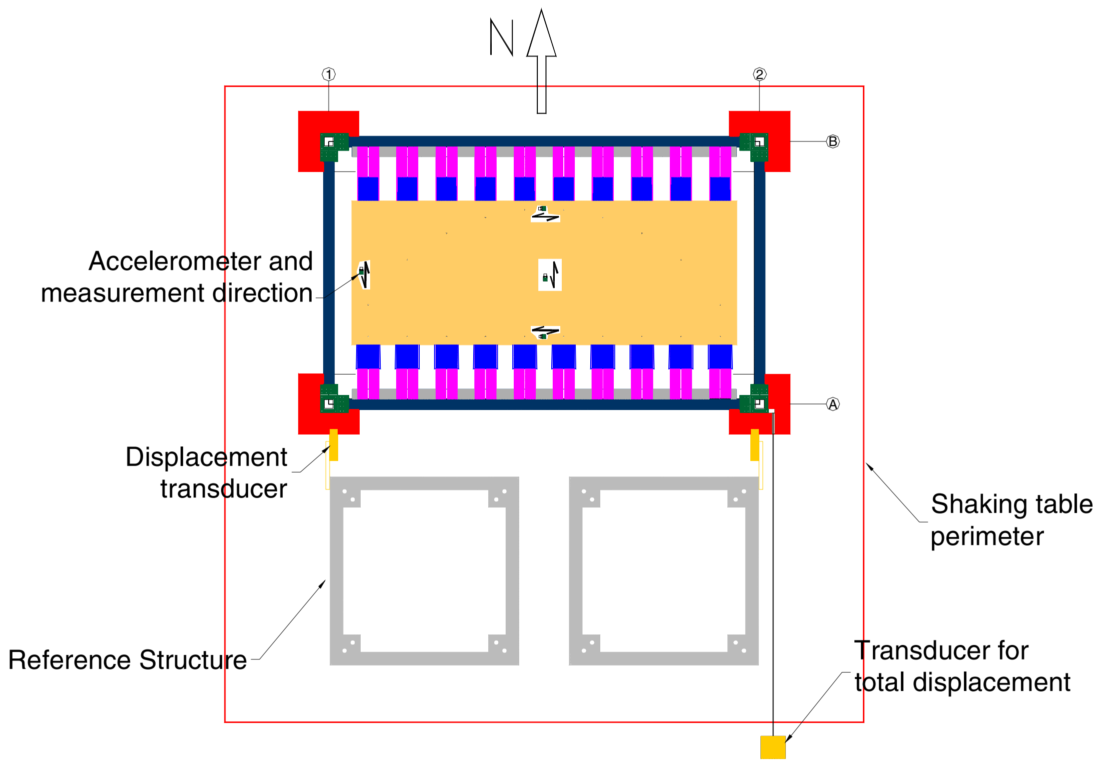
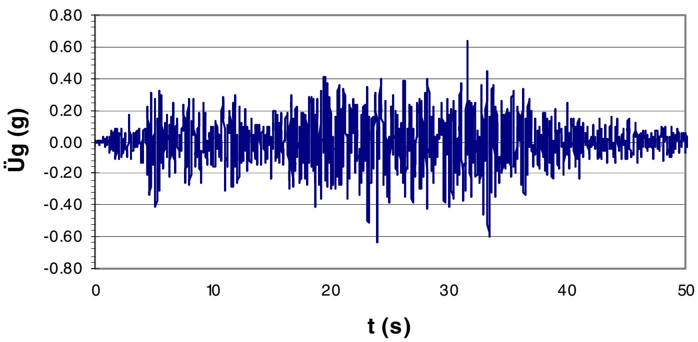


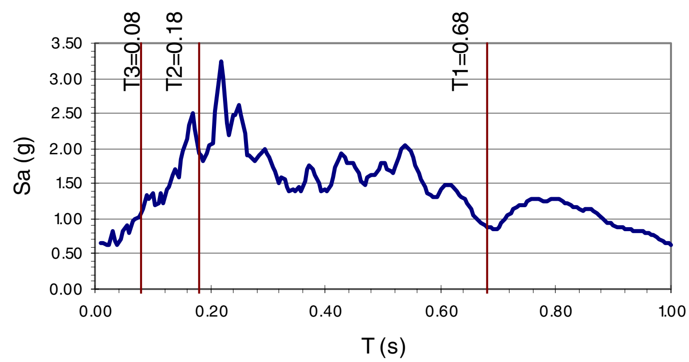
Figure 4. Plan building at level 5 in the shaking table

Figure 5. Building on the shaking table

Figure 6a shows the input accelerations for the high-level shaking tests, and Figure 6b shows the acceleration response spectra for this record considering a critical damping ratio, ξ, equal to 0.03. The later figures also identifies the modal periods (T) calculated for the specimen.



a) Accelerogram



b) Pseudo-acceleration elastic response spectra (ξ=0.03)

Figure 6. Accelerogram used as input ground motion in the high-level shaking table tests and corresponding pseudo-acceleration response spectra.

TEST RESULTS

Some results from the experimental program conducted in this study are evaluated in the followings. Figure 7 shows moment-curvature curves measured in a beam-column joint specimen. In this case, the thickness of the fuse was 19 mm. A comparison of measured effective stiffness and predicted stiffness (using gross section properties) in this beam-column joint showed that the former was about 0.2 times the later. These results were incorporated in the prediction of the specimen's response later described. Figure 7 also shows an envelope of the hysteretic curves and values of the plastic and ultimate moments, M_p and M_u , respectively.

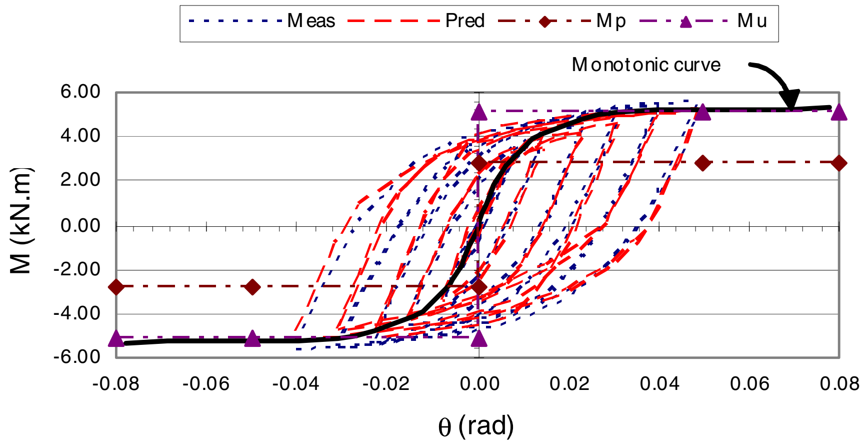


Figure 7. Moment-rotation relationships of beam-column joint specimen with 19 mm fuse thickness

Damping ratio for the first mode of vibration was evaluated from the logarithmic decay curve (Clough and Penzien, 1993) of floor accelerations recorded during the free-vibrations tests. This parameter was equal to 0.02.

Figure 8 shows the transfer function of the floor accelerations recorded at level 5 for the low and high-shaking table tests and corresponding measured modal frequencies. The later values are also listed in Table 1. The thick and thin lines in Figure 8 correspond to results of the low-level and high-level shaking table tests, respectively. As seen in Figure 8, the ratio of the amplitude of the transfer function in the inelastic case to that of the elastic one was 0.25 times. For the second mode this ratio was close to 0.5. This finding does not support current seismic code approaches in which the inelastic response is obtained reducing the elastic response using a single factor for all modal contributions.

Table 1. Frequencies and vibration periods

Mode	f (Hz)	T (s)
1	1.48	0.68
2	5.42	0.18
3	11.91	0.08

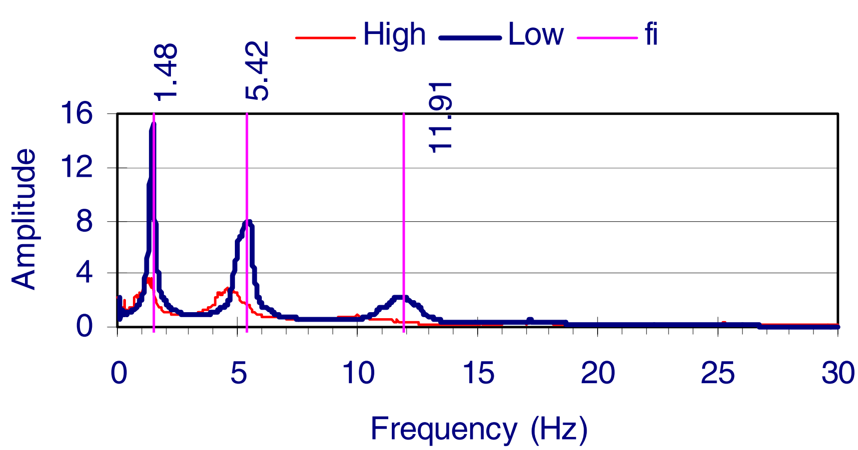


Figure 8. Transfer function for floor accelerations measured in level 5 of the specimen for the low and high-level shaking table tests

The peak ground acceleration measured at the base of the structure, $\ddot{U}_{gmax}$, was equal to 0.89·g. The ratio $\ddot{U}i$ / $\ddot{U}_{gmax}$, where $\ddot{U}_i$ is the acceleration at level i measured in the high-level shake test, measured between 0.44 and 1.14.

ANALYTICAL MODEL

Analytical studies using the RUAMOKO computer program (Carr, 1998) were carried out to study the inelastic response of the specimen. The specimen was modeled as a bidimensional frame taking advantage of its symmetry. A total of 27 elements were used for analyzing this frame, of which ten elements represent columns, five elements represent beams, and twelve elements represent the specimen's fuses, see Figure 9.

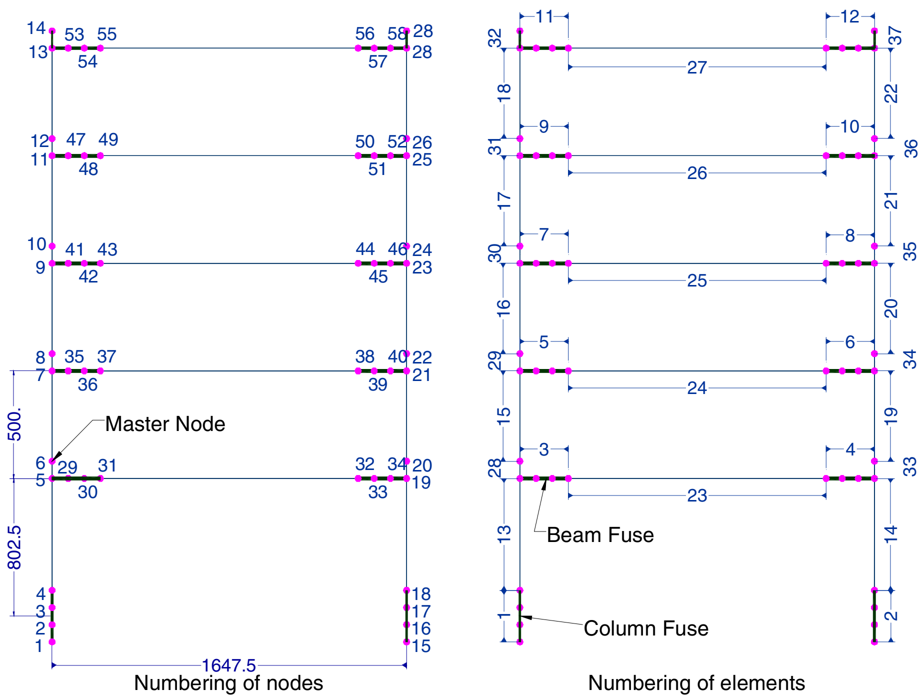


Figure 9 Analytical model of the frame building (dimensions in mm)

Predicted and measured stiffness for the specimen's fuses are shown in Table 2. Measured stiffness values were obtained from experimental testing of substructures as that showed in Figure 3. In Table 2 the parameters b and h represent the width and thickness of the fuses, respectively. The elastic module is represented by S, and the parameters I and I_{eff} represent the gross and effective moment of inertia, respectively. The elastic module and the fuse length were equal to 196 GPa and 5 mm, respectively. The predicted gross rotational stiffness was defined as EI/L, and the parameter K_θ is the effective rotational stiffness, which was defined as the ratio I_{eff}/I times EI/L.

Predicted strengths for the specimen's fuses are shown in Table 3 and were obtained using measured mechanical properties of the steel obtained in coupon tests. Equations (1) to (7) were used for the evaluation of typical parameters needed for the computer program for defining the moment-rotation curves of the specimen's fuses. These parameters were the plastic moment, M_p, the ultimate moment, M_u, the yield rotation, θ_y, the rotation at first hardening, θ_{sh} and the ultimate rotation, θ_u.

$$S = \frac{b \cdot t^2}{6} \tag{1}$$

$$Z = 1.5 \cdot S \tag{2}$$

$$Mp = Z \cdot fy \tag{3}$$

$$Mu = R_f \cdot Mp \tag{4}$$

$$\theta_y = \frac{Mp}{K_\theta} \tag{5}$$

$$\theta_{sh} = 1.16 \cdot \theta y \tag{6}$$

$$\theta_u = \frac{2 \cdot \varepsilon_u}{h} \cdot L \tag{7}$$

In previous equations parameter f_y is the measured yield strength of the steel fuses and was equal to 285.2 MPa. The ratio between yield and maximum strength R_f was equal to 1.83. Parameter ε_u in Equation (7) is the ultimate deformation and was equal to 0.17.

Table 2. Stiffness evaluation of fuse tested using results of substructure test

Fuse	b (cm)	h (cm)	S (cm^3)	I (cm^4)	EI/L (kN-m)	I_{eff}/I	K_θ (kN-m)
Column	11.0	1.9	6.62	6.29	2459.8	0.20	492.9
Beam L 1,2	11.0	1.2	2.64	1.58	621.3	0.30	74.5
Beam L 3,4	11.0	1.0	1.83	0.92	359.7	0.26	93.5
Beam L 5	11.0	0.6	0.66	0.20	77.7	0.20	15.5

Table 3. Strength and strain evaluation of the fuses tested using real properties of steel

Fuse	Mp (kN-m)	Mu (kN-m)	θ_y (rad)	θ_{sh} (rad)	θ_u (rad)
Column	2.83	5.18	0.0057	0.0067	0.0895
Beam L 1, 2	1.13	2.07	0.0152	0.0176	0.1417
Beam L 3, 4	0.78	1.43	0.0073	0.0084	0.1700
Beam L 5	0.28	0.52	0.0182	0.0211	0.2833

Figure 10 shows the envelope of measured floor accelerations (identified there as "Exp") for the high-level shaking test and are compared with predicted envelopes for this parameter. The line with square marks identifies results using the modal superposition technique and the square-root-of-sum-of-the-squares approach (SRSS) without any reduction in the modal contributions. The line AMR identifies results using current code approaches based on all modal contributions affected by the same reduction factor. The line LFd identifies the values of floor accelerations obtained from lateral loads associated to the maximum base shear calculated with the current Mexico City Building Code. As seen in Figure 10, in most cases when comparing measured values and predicted values based on current code approaches, the later values lead to unsafe results.

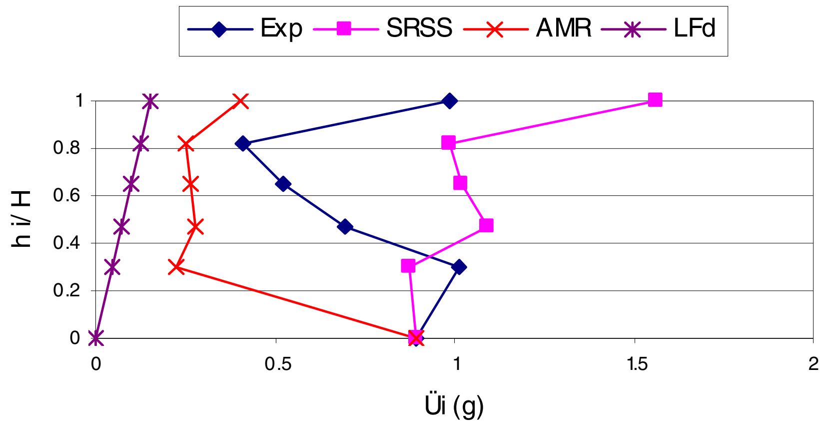


Figure 10 Measured and predicted absolute floor accelerations using code approaches

Figure 11 shows measured absolute floor accelerations and predicted values (FRM) using the first mode reduced method (Rodriguez et al, 2002). This method is based on the SRSS technique in which only the contributions of the first mode is reduced. A comparison of measured and predicted values shown in Figure 11 indicates that the proposed method leads to a better agreement between these values than that discussed for predictions whose results are shown in Figure 10.

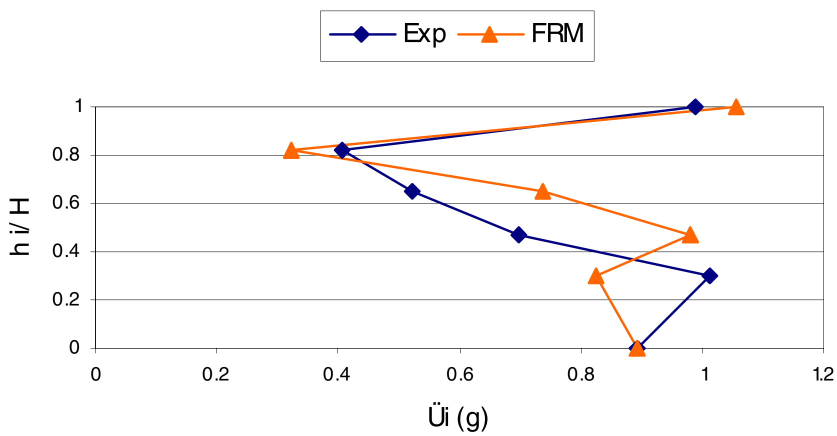


Figure 11 Measured and predicted absolute floor accelerations

CONCLUSIONS

A five-story miniature steel building was designed using replaceable fuses located in potential plastic hinges. This building was subjected to unidirectional input ground motions on a shaking table. Some conclusions obtained from this study are the following.

An evaluation of measured floor accelerations in the specimen for the low and high-intensity shaking table tests showed that typical code approach for predicting these floor accelerations, in which the modal contributions is reduced by the same factor, might be on the unsafe side.

This study explores the use of a previously proposed method for the evaluation of floor accelerations in buildings during strong earthquakes. This method is based on the assumption that that structural ductility affects only the contribution of the first mode of response. Results showed in this paper indicate that the use of this method leads to a better agreement between measured and predicted floor accelerations than that using current code approaches.

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