

Steel Rigid Frame Structural Behaviour at El Mayor Cucapah (Baja California) Earthquake April 4, 2010.



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SUMMARY:

The Earthquake of El Mayor Cucapah at Baja California, Mexico affected some buildings at the City of Mexicali. Three buildings were visited and studied to analyze their performance and structural safety after the Cucapah earthquake. This paper presents the results of a study of the structural behaviour from these three existing constructions and focus on the damages suffered by two of this steel structures with similar response period; all three structures were design according to the Mexican Seismic Code (Manual de Diseño de Obras Civiles, “Diseño por Sismo”, Comisión Federal de Electricidad–Instituto de Investigaciones Eléctricas, 2009, MSC). Maximum displacement response using design spectra and time history methods are compared; the damages suffered are consequence of the type of selection of the structural system and excessive seismic drifts.

Keywords: Rigid steel frame, structural damage, Story drift demands, damages, structural engineer.

1. INTRODUCTION

The earthquake that struck northern Baja California on April 4, 2010 affected the city of Mexicali, the biggest Mexican city near the surface faulting. Some short period structures suffered severe damaged because the nature of this near fault ground motion: large content of high frequencies, but also structures in between periods (medium) suffered structural damage because excessive story drifts.

Three buildings were structural inspected and studied. Two of them suffered severe structural damaged even though all this structures were design according the guidelines of Mexican Seismic Code (MSC); one has a structural system of gable steel semi-rigid frames (gable steel system) while the other construction had an exterior shear wall with steel rigid frames on the sides (shear wall - steel frame). The third building analyzed was constructed using modular steel frame with trusses as main structure (rigid truss-frame) presented only few non-structural damage.

These three structures were subjected to response spectrum analysis and linear time history analysis for comparing peak inter-storey drifts and drift limits established by the MSC. Acceleration and displacement response spectra are obtained to get a better perspective of their seismic behaviour.

The damages presented by two of these structures are consequences of: wrong choice of structural system in a near fault site and the acceleration imposed by this earthquake was greater than expected in seismic codes producing excessive drifts demands in this two buildings.

2. STRUCTURAL MODELS OF EXISTING BUILDINGS AND SEISMIC INPUT.

All cases studied are one storey building, located at the city of Mexicali (soft soil) and steel frame structures: i) Gable frame system with semi-rigid frames in all directions, except at two perimeter

seismic resistant frames with two lateral concentrically braced frames; ii) Rigid truss-column frames in one direction and semi-rigid frames with two lateral seismic eccentrically braced frames (exterior EBF's); iii) exterior masonry wall shear type and steel rigid frame at centre and perimeter. Each building was designed using the response spectrum method of analysis following the requirements of MSC. The response spectrum analysis used the computer program SAP2000 (Habibullah, 1994).

The buildings are regular as shown in Figure 1 where the analysis models of each building type are presented.

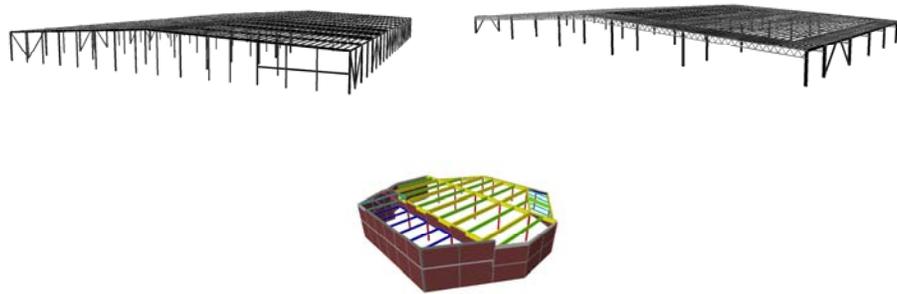


Figure 1. Mathematical models of existing buildings. From left to right: Gable Steel system, Rigid Truss-Frame system and shear wall - steel frame.

2.1. Seismic Input.

The input acceleration record used for time history analysis was the Cucapah Earthquake recorded by the CICESE (Scientific Investigation Centre and Superior Education of Ensenada) at the Baja California Government building at downtown.

Table 2.1. Earthquake Records

Location	Records	Component
Mexicali	Cucapah Earthquake CICESE	East-West
Mexicali	Cucapah Earthquake CICESE	North-South

Buildings were design considering a low level of ductility ($\mu = 2$). The steel frames are flexible buildings while the shear wall- steel frame is stiffer. The design response spectra for this site, according to the MSC is shown in figure 2.

Table 2.2. Building periods.

Building	X	Y
Gable steel system	0.65 sec	0.99 sec
Rigid truss-frame steel system	0.39 sec	0.69 sec
Shear wall - steel frame system	0.69 sec	0.79 sec

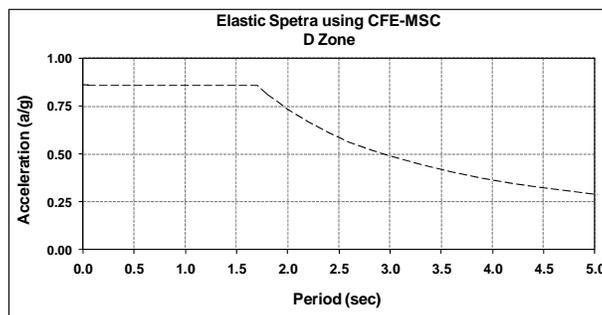


Figure 2. Design response spectra for Mexicali, according to MSC.

3. RESULTS.

The structural problems that were reported in this buildings during the structural inspection indicated that damages were produced by excessive deformations. All structures were designed for ductility under seismic loads and so deformations are more meaningful than forces. The parameter used to compare the response spectrum and the time history methods are: i) the peak inter-story drift; ii) Displacement spectra; iii) Acceleration spectra.

For the design response spectra method the drifts are based on elastic drifts modified by the "modification factor", $Q = 2$ (Normas Técnicas Complementarias de Diseño por Sismo, Mexico 2004). For the time history analysis, peak drifts were the maximum values occurring at any time step of the earthquakes. All time history analyses used the computer program SAP2000 (Habibullah, 1994). The peak drifts from each method of analysis are listed in Table 2.2, together with the ratio of time history drift to response spectrum drift.

Table 2.2. Peak Inter-storey drifts

	Time History (TH)		Design Response spectra (DRS)		Ratio TH / DRS	
	X	Y	X	Y	X	Y
Cucapah CISESE E-W						
Gable steel system *	1.90%	3.56%	2.31%	3.20%	0.82	1.11
Gable steel system w/ minimum live loads *	1.29%	2.40%	1.47%	2.00%	0.87	1.20
Rigid truss-frame steel system	0.39%	1.20%	0.43%	1.20%	0.90	1.00
Shear wall - steel frame system	0.26%	1.23%	0.39%	0.50%	0.66	2.46
Cucapah CISESE N-S						
Gable steel system *	2.10%	3.69%	2.31%	3.20%	0.90	1.15
Gable steel system w/ minimum live loads *	1.83%	2.61%	1.47%	2.00%	1.24	1.30
Rigid truss-frame steel system	0.37%	1.40%	0.43%	1.20%	0.86	1.17
Shear wall - steel frame system	0.26%	0.87%	0.39%	0.50%	0.66	1.74

* The gable steel system design with complete live loads as the Mexican seismic codes establishes presents in the seismic analysis enormous lateral displacements that would make the building collapse, and that didn't happened. Because of this phenomena it was considered in a second design state, just as it happened during the earthquake, that there was a minimum of live load at the roof and that helped this building for a better performance; this is the reason this gable systems appear twice at the comparative 2.2 table.

Even considering live loads under code values for the gable system, the peak story drifts suffered are beyond the maximum allowed by Mexican codes (maximum value = 1.2%). Why didn't this building collapsed? In section 3.2 it would be shown that the facade perimeter wall restricted the steel frame displacements suffered under the earthquake.

The rigid truss-frame steel system suffered minor non-structural damages because of this earthquake. This building behaved correctly even though with the N-S component of the seismic record the drifts obtained using the time history analysis exceeded than the ones reported using the design response spectra; this shows that even when a structure is design following the MSC response spectra, displacements suffered by buildings are higher than the expected by seismic codes. This behaviour is present in all buildings in the "Y" direction, (last column of table 2.2 has values greater than 1).

The shear wall - steel frame building, even though it should be a stiff structure because it has a perimeter shear wall in one side of the construction, has a relative high period with peak drifts obtained by the time history analysis under the E-W component record, exceeded 100% than the ones calculated using the design response spectra. This is the reason this building suffer great structural damages, without reaching collapse, but it was at the end demolished.

3.1. Response Spectrum.

Displacement response spectrum for both record components and for each type of building under the most adverse component earthquake record were calculated using SAP2000 (Habibullah, 1994) and are shown at figure 3.

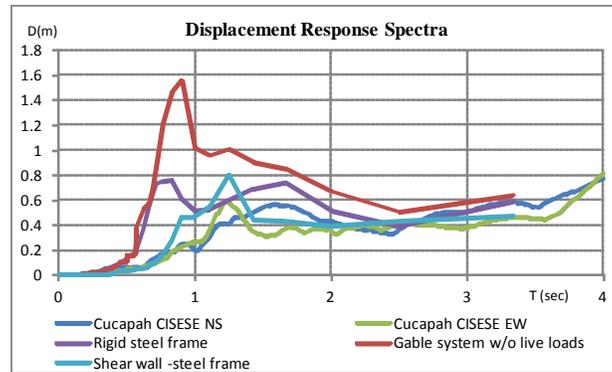


Figure 3. Elastic Displacement 5% damped Response Spectra.

The maximum displacements imparted to structures like the Gable semi-rigid steel frame will be 1.55m for a period of 0.8 sec and then diminishes for structures with longer periods. The rigid steel frame and shear wall - steel frame structure behaves in a similar way, with maximum displacements of 0.8 m for structures with periods between 0.8 and 1.2 sec. Finally the maximum displacement for single degree of freedom systems is 0.8m for structures with periods between 1.1 and 1.5 sec and after than diminishes to 0.5 m for 2.5 sec period structures, increasing to 0.8m for periods of 4 seconds or longer.

For all cases, there is a peak displacement value for periods within a range of 0.8 and 1.5 sec, then displacement diminishes and increases for periods larger than 3 sec. This means than not only short stiff structures suffer damages in a near fault zone, but also flexible structures of one storey height also are affected. The particular case or the Gable System and its response, is special because of a unfortunate structural decision of using semi-rigid connections and not placing a correct bracing system.

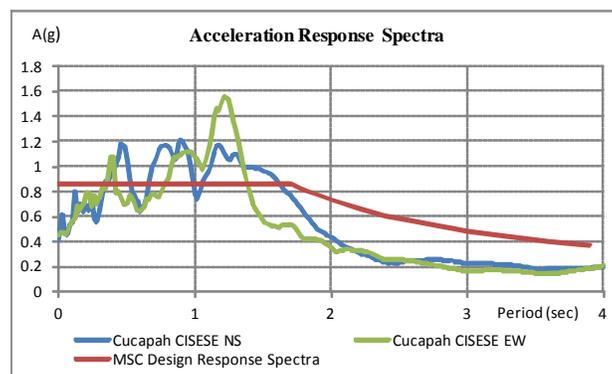


Figure 4. Elastic Acceleration 5% damped Response Spectra.

For the MSC there is a wide range of structures, from 0 to 1.8 sec, that must be design with an elastic acceleration $C = 0.86g$. All the structures studied should have been design with this value although displacements exceeded the code allowed value.

Comparing de response for the Cucapah Earthquake there are periods where de acceleration imposed was 186% greater than the design elastic value of acceleration. Because of this behaviour, the rigid-steel frame and the shear wall- steel frame reach their maximum drift code limit.

3.2. Structural inspection and synthesis.

During inspection there are many uncertainties about a building response; the limit is experience and the study of damages at place. With the analysis of all buildings it has been shown that two of the three structures suffer excessive drifts, greater or at the limit of the MSC limits. By the other hand, one structure with a fundamental period within the maximum displacement spectra range behave correctly proving that the solution of a structural system is the cause of all damage produce.



Figure 5. Gable steel system structure.

In photographs at figure 5 the building with a semi-rigid frame steel structure is shown. The columns are thin (254 mm x 254mm) and beams only simply support over them. The maximum height of this structure is 10 m. The facade wall was lateral restrained by connecting to the steel frames, this connection was only made to avoid displacement problems of the wall; in this case, the walls became part of the seismic resistant system, avoiding to reach displacements that could have produce a collapse of the structure. Also the roof at the moment of the earthquake had a minimum live load, helping to decrease the mass over the structure.



Figure 6. Gable steel system structure after the Cucapah Earthquake.

The external columns of the frames are Wide flange type. The steel frames pushed out de facade wall, moving it around 15 cm in the most affected zone that was at the centre of the building. This wall also

helped the structural system to reduce drifts in the other direction, producing diagonal cracks.



Figure 7. Semi-rigid connections at the gable structural system.

The displacement level was at the top roof the maximum suffered; figure 7 shows a bolt found at the floor level after the earthquake, it had a shear failure at the beam-column simple connection.



Figure 8. Shear wall- steel frame building after the Earthquake.

Torsional problems and excessive displacements were suffered by the structure presented at figure 8. Steel columns buckle and steel beams moved toward the lower level, making that the roof (composite slab) move down around 15 cm as it can be seen at left picture of figure 8. Right image is the same spot but from down under.



Figure 9. Shear wall- steel frame building with serious damages in shear walls

In figure 9 it can be seen that shear wall system produced torsional effect and cracked everywhere, damaging also the steel structure by increasing displacements and loads.

4. CONCLUSIONS

Three different existing buildings located at the City of Mexicali, Mexico, were inspected and studied after the Cucapah Earthquake. Two of them suffered important damages as an effect of excessive displacements even though all have similar fundamental response periods.

The acceleration imposed by this earthquake to the City of Mexicali in some cases was 186% greater than the design elastic value of acceleration; the three buildings studied were affected by this increment of seismic acceleration, as a consequence, greater level of drifts were obtained .

The peak displacement spectra presented two maximum points: i) a peak displacement value for periods within a range of 0.8 and 1.5 sec; ii) a displacement increase for periods larger than 3 sec. This means that not only short stiff structures suffer damages in a near fault zone, but also flexible structures of one storey height also are affected, like the gable system studied.

The structural system proposed for each building is as important as any other seismic parameter; a wrong selection of the resulting structural system can produce effects like the two buildings that have been studied and presented in this paper.

ACKNOWLEDGEMENT

Thanks are due to the members of the Seismic Department of the CISESE, and especially to Professor Luis Mendoza Garcilazo and Ing. Rogelio Reyes, for their help sharing the seismic records register at the City of Mexicali, BCN.

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