

## Chapter V. DISCUSSION OF RESULTS

### V.1. Introduction

This section contains discussion of the results for the entire study. The three analyses performed will be commented; three of them will be compared with the previous research from chapter II. The use of graphics and figures is necessary to show the differences and similarities between the analyses. The objective of this chapter is to establish, if any, relation between the characteristics of the earthquakes executed and the behavior of the structure. It is also to predict, if possible, the behavior that the test structure will have during the test is going to be performed at the laboratory. The section is divided in three points, one for each of the analyses done.

### V.2. Limit analysis

After multiple attempts in chapter III to get the configuration of the structure, the final design was achieved. The limit analysis for that design, defined a structure with a large base shear coefficient 35%. This coefficient is for a base shear calculated with a triangular distribution of lateral loads. As it was explained earlier in the dissertation, Moehle [9] proposed an upper bound for the base shear. This upper bound is calculated with a uniform lateral load distribution, and the coefficient for this case is 40%. These two results imply almost double expectation of what will be considered normal for this type of building 20%. The main reason for this large value of base shear is to consider that all the width of the slab is yielding. But this hypothesis is realistic to assume, for various reasons: firstly previous research in Chapter II showed how the slab is more effective for low reinforcement ratios. Secondly the reinforcement is grade 60, for the calculations the nominal reinforcement strength has been assumed 483 MPa (70 Ksi) because in that way a little bit of strain hardening of the steel is considered. Even though, the re-hardening of the steel will be larger. The use of  $\phi$  13 (# 4) bars will increase that possibility because of its small diameter. Then it has been reasonable to consider the whole width

of the slab into the calculation for this structure. The worst case scenario is considered and the prediction for the behavior will be in the safety side.

The large base shear calculated, makes the system tough and a priori provides a good resistance against lateral load. This assumption will be further check with the dynamic analysis. The calculation also establishes a reference value to know if the structure is close to failure during the following analysis, static and dynamic.

### V.3.Static analysis

The results for this part were shown in Chapter IV in two figures with diagrams. The static analysis confirmed the results predicted by the limit analysis. It was performed with the same scheme than the limit one, one analysis for a linear lateral load distribution and another for a uniform load distribution. The results of both of them confirmed than the uniform distribution brings an upper bound coefficient. The major difference between the two different load configurations is the larger base shear for the uniform one. The rest is almost the same, having similar drift behavior. The main interest of this test is to see the relation between the drift ratio and the base shear. It is also interesting to be able to study the yielding history of the members along the loading.

During the discussion all the values will be mention between the upper and lower bound obtained from the two different lateral loads. The structure behaves elastically until the first cracks appear. That is around 10% of the Total tributary Weight (TTW). The mean drift ratio is around 0.1%. In the test case, the slabs will be previously cracked by added load in the slabs. Although the elastic part observed in figure 50 gives an idea that the building should behave without any trouble until load levels of 10% of its weight.

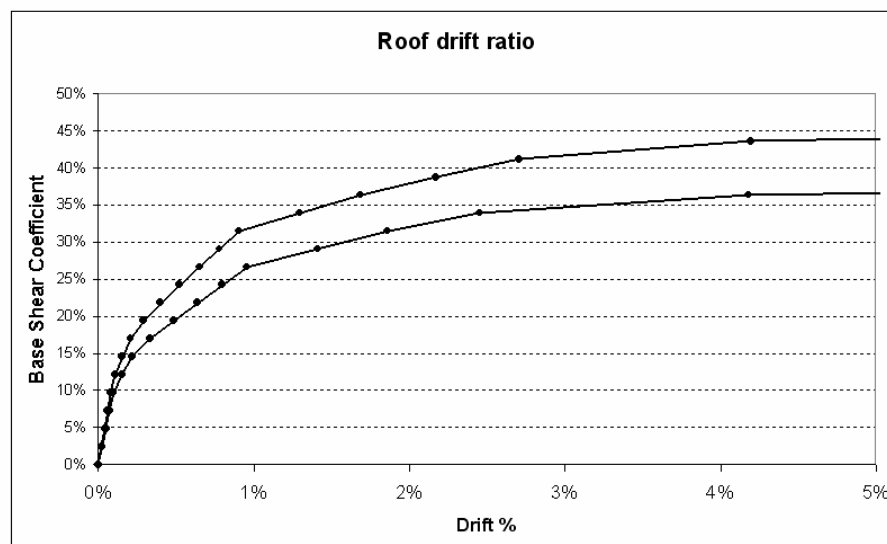


Figure 50. Upper and lower bound for the limit analysis

As it was expected, during the elastic range both load configurations give the same result. After the range of uncracked stiffness, the structure decreases in stiffness quickly, until it reaches a period of constant slope, as it can be seen in figure 50. This range goes on until the first yielding.

The first yielding of the structure appears around 1% mean drift and for a base shear between 578 to 712 KN (130 to 160 Kip) or expressed in coefficient between 26% and 32% of the weight. The first members to achieve yielding moments are the three base columns, in both load cases. These columns are the ones resisting the whole base shear as they are the connection to the ground. Then, it is expected to have in them the maximum level of stress of all the structure. After the first yielding occurs in figure 50 can be seen how the structure increases the drift linearly with the base shear, the stiffness is lower than before. The results show how the whole building is “rotating” around the plastic hinges created at the base columns. The stresses at the column slab joints increases with the drift increase. Thus in the following load steps, more members of the structure become to yield. In Appendix B there are two tables showing the moment values in each joint for every loading step. In those tables it can be seen how the first sides of the slabs to yield are the ones subjected to positive moment. They are followed in next steps by the negative sides generating the failure mechanism.

The overall yielding occurs when the mean drift ratio is about the 3% and the base shear is between 761 and 912 KN (171 and 205 Kip) 35-41% of the TTW. The criterion to say that the structure is failing is the review of the hinges generated; when these ones produce a mechanism the building is considered collapse. LARZ results show the ductility of the members, and then the plastic hinges can be found easily for each point with ductility 1. The result from LARZ shows the same mechanism as the one it was designed for. Once the mechanism is the same, the base shear is consistent and has values rather similar.

Comparing with Moehle [9], the structure is not as elastic as his model was. It seems not possible for the present structure to reach drift close to 5%. Because all the members of the system reach yielding far from that value. Looking at the history of the interstory drift ratios, the structure develops an “S” shape due to less stiffness at the second level. That level has before yielding the largest interstory drift ratios. Summarizing, with the static analysis using LARZ, the design developed in Chapter III is confirmed. A basic response of the structure has been established and should be compared with the dynamic analysis.

#### V.4. Dynamic analysis

The dynamic analysis in Chapter IV started with a study of the lower modes of vibration of the structure. Using the characteristics of the building and solving the eigen problem a period larger than expected was found. From the uniform building code a period around 0.3 sec was expected, but it turned out into a period almost double of the guessed one (0.56 sec). The reason for this structure to be more elastic than it was suppose too, is the use of the slab instead of beams. The slab, even if it is

used in its whole width, has a lot more elasticity than a beam. Thus the period found is larger. This big elasticity will be probably the main vulnerability for the structure.

In chapter IV 18 earthquakes records were applied to the structure with software so called LARZ. The results in drift for each record are plotted in figure 51.

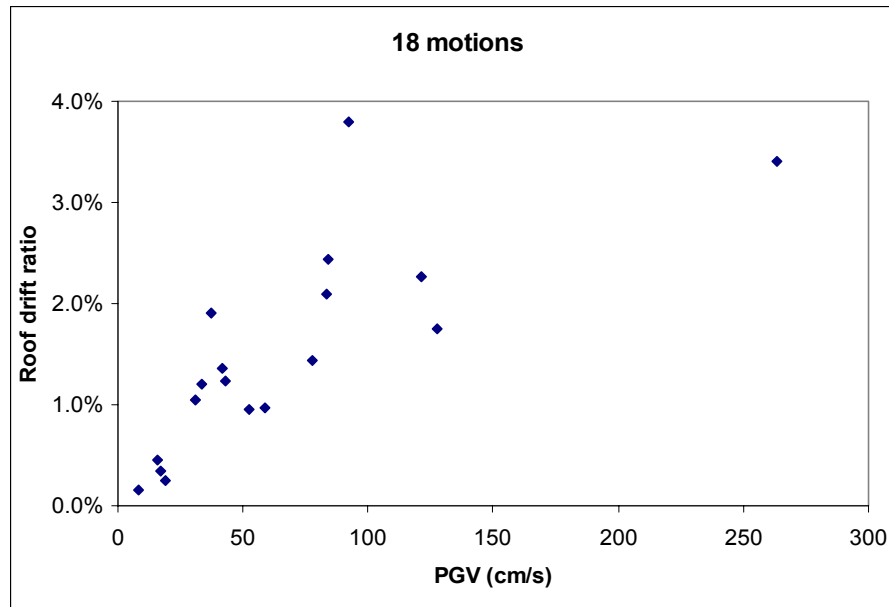


Figure 51. Drift-PGV diagram for the 18 records

In the diagram the drift trends to increase with the PGV of the earthquake. It is adventurous to establish a relation but it is clear that both increase. In Table 8 it can be seen the values for each point. There are two points in the diagram clearly out of trend. Those are Kobe with a drift of 3.80% and Chi-chi with mean drift ratio of 3.41%. Kobe is out of trend because its drift response is not around drift responses for other records with similar PGV, and Chi-chi is out because its PGV almost double the largest PGV of the rest of the records. In table 7 there is a column describing the damage to the structure, this column is based on the results of LARZ, the criterion to decide the level of damage in the structure, has not been based on the limit and static analysis earlier performed. Instead of this, it has been: if the response shows no plastic hinges in any joint, the damage is nothing. If there are a few plastic hinges in the members is called low damage. If most of the members are yielded, then severe damage has been written. Finally if all the members are yielded there is written collapse. When all of them are yielded, the structure can generate a mechanism and fail. Thus, when it says severe it means that the structure is really close to failure. Maybe it has already failed, but for the kind of output given by the software, it is impossible to know unless a mechanism is generated.

Table 8. Drift response for the 18 motions

Motion	PGV (cm/s)	Roof drift ratio	Base shear (KN)	Damage
Seattle	8.22	0.16%	286	Nothing
Kern County	15.60	0.46%	407	Nothing
San Fernando	17.16	0.34%	407	Nothing
Santa Barbara	19.25	0.25%	353	Nothing
Friuli	30.80	1.04%	660	Low
El centro	33.45	1.20%	689	Low
Miyagi Ken Oki	37.39	1.91%	675	Severe
Valparaiso	41.49	1.36%	784	Severe
Tokachi-Oki	43.00	1.23%	640	Low
Irpina	52.70	0.95%	638	Low
Kocaeli	58.80	0.96%	591	Nothing
Nothridge	77.88	1.44%	861	Severe
Duzce	83.50	2.09%	850	Collapse
Erzincan	83.90	2.44%	896	Collapse
Kobe	92.04	3.80%	1016	Collapse
Tabas	121.40	2.27%	1178	Collapse
Cape Mendocino	127.40	1.75%	1060	Severe
Chi-Chi	263.10	3.41%	864	Collapse

Looking at the results, in terms of PGV two approximate marks can be established: first one at 30 cm/sec, until this PGV the structure does not get damage and it behaves basically under the elastic and cracking range without reaching any yielding. Between this PGV and 80 cm/sec, the structure reaches different yielding levels, varying from low to severe. For motions with larger PGV than 80 cm/sec the structure tends to be collapsed. Then the response should be compared with the approximation from Lepage [6] for PGV. This approximation is based on the formula:

$$S_d = \frac{PGV}{\sqrt{2}} \cdot T \quad (16)$$

The period for this formula is the calculated using the proper method (0.56 sec). Using this value, the Lepage approximation can be plotted in the same diagram as the results, figure 52. In this figure the Lepage approximation is an upper bound of the response of the structure, but it starts to deviate for large PGV because of its linear relation. The path followed by the results is more like the curve described by a square root function. However the Lepage approximation seems to be in the safe side and working rather good until PGV around 100 cm/sec.

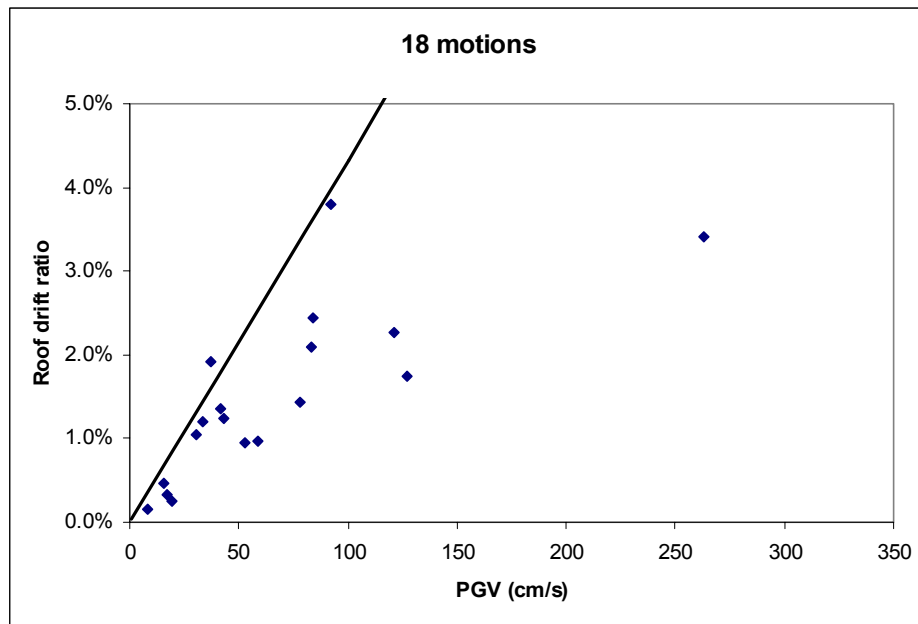


Figure 52. Lepage approximation compared with the results.

In terms of Base Shear, for the different motions the Base shear increases with the increase in drift. This relation is seen in Table 8 but also represented in figure 53. This figure is a diagram Base shear versus drift ratio; the interesting point would be to compare this relation for the different motions and the trend in the static analysis. Then, the diagram includes the results from the static analyses.

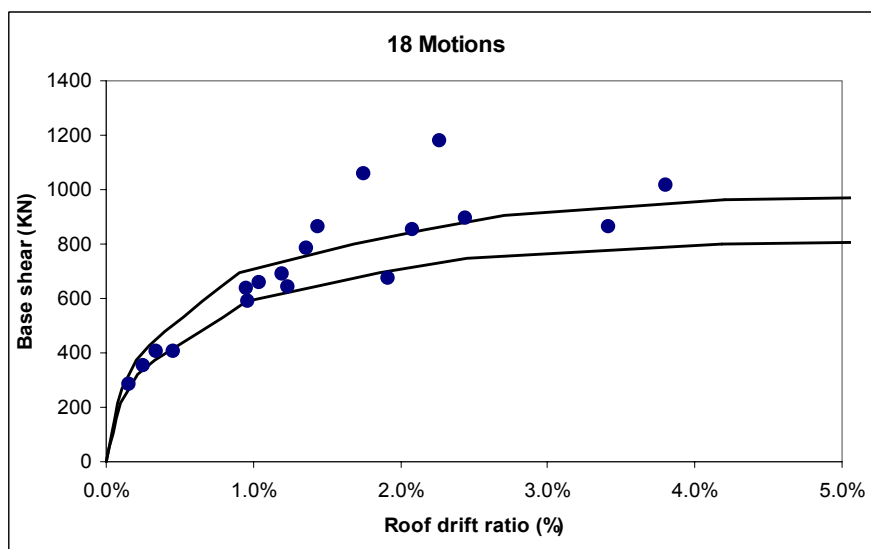


Figure 53. Base shear versus drift ratio compared with the static analyses results.

It is obvious the similarities between both analysis, dynamic and static. However the limit in drift is different. For the static analysis the structure reaches failure when the mean drift ratio is around 3%. In the dynamic approximation, the earthquakes reaching over 2% collapse the building or produce severe damage close to failure. Also for Base shear values over 800 kN (180 Kip) the structure is collapsed

or severe damaged. Then those values can be considered the limits for the structure to be competent.

Finally it is interesting to include the relation between drift response and the PGA of each motion. This is represented in figure 54, but as it is seen there is little relation between PGA and drift, this is also based on the brief paper written by Westergaard [16].

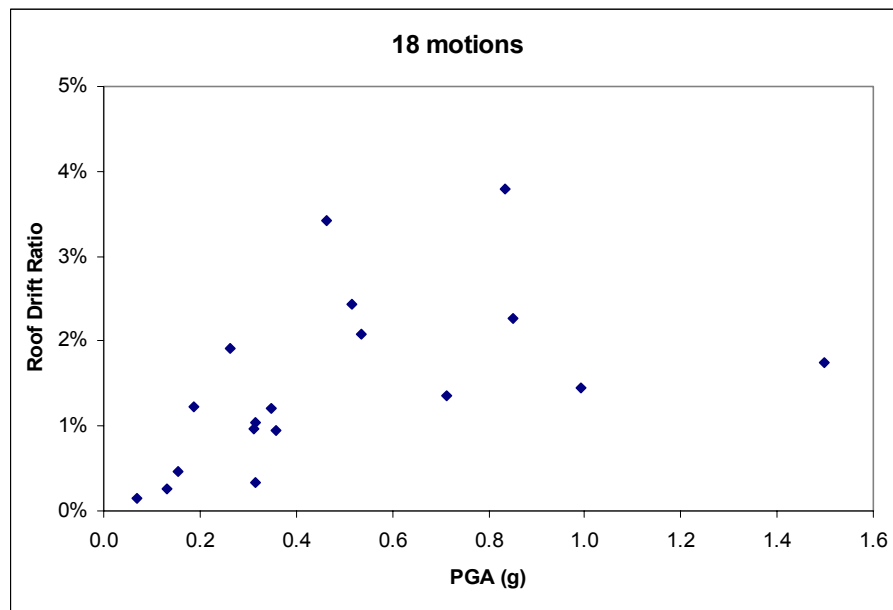


Figure 54. Drift ratio response versus PGA

Summarizing, the structure fails, in all the cases analyzed, in the mode it was designed for. Limits for its behavior can be established in terms of PGV, mean drift ratio and base shear. However the appropriate values to predict the laboratory test results should be closer to the dynamic analysis results than the static ones. This is because the test in the laboratory will be on a structure previously cracked with Dead and Live Load, and because the loading sequence will be cyclic. As it has been seen in the dynamic analysis, a cyclic load reduces the drift response of the whole system compared to the static analysis.