Chapter IV. ANALYSIS OF THE TEST STRUCTURE

IV.1. Introduction

After the design process, an analysis is needed to understand the behavior of the structure. Two analysis will be done, first one static that will show an approximation of what will really happen in the test; Second a dynamic analysis where the structure will be subjected to different earthquakes. The static analysis will be compared to the limit analysis previously performed. The dynamic analysis will evaluate the resistance of the structure to ground motions. All the analysis will be performed with software called LARZ. The objective of this chapter is to explain how the software works, and how the structure is modeled to fit the program. It is also to explain the characteristics of the static analysis and behavior of the structure; finally the dynamic analysis will be explained. This last point implies to explain the approximation to get the period of vibration of the building, one is the code approximation and the other is the exact approximation for the dynamic equilibrium of the structure. After that, an introduction to the ground motions selected for the analysis and the reason to select them. Finally will be the explanation for the dynamic analysis performed with LARZ and the detailed results for every motion executed.

IV.2. Software used

The software chosen for the purpose of this dissertation is LARZ; it is a program for nonlinear analysis of reinforced concrete frame structures. The software was originally developed by S. Otani at the University of Illinois at Urbana-Champaign and its initial name was SAKE. Along its history has been subjected to various modifications, mainly by M. Saiidi at University of Illinois, and lately by R. López at University of Puerto Rico. In the beginning the program had two different versions: one static, intended to calculate the response of the system under lateral load or displacement increments. Another dynamic version prepared to subject the system to earthquake motions. At the Last modification by R. López, both versions were put together in the same application.

The program is written in FORTRAN language; the input is basically a description of the characteristics of the structure followed by a description of the earthquake motion or the static loading depending on the demanded analysis. The output is a bunch of .txt files with the moment and shear history of the members of the system; it also contains the base shear history and the drift response along the duration of the earthquake.

The program is intended for frame structures, but the actual structure is a flat-plate; then the structure has to be modeled as a frame. In order to transform the slab in beams and create a frame, the most important parameter to decide is the equivalent beam width. From the previous study by Morrison [10] observed that for low reinforcement levels in the slab, the slab is fully effective. Morrison tests developed full slab capacity for reinforcement ratios of 0.65% at the slab. The structure now under study has a maximum ratio of 0.56%. Thus, for this case is proper to consider the slab as full width effective. Each slab is divided into two longitudinal beams 15 ft. wide and the columns kept as they were. ACI requires negative reinforcement to be continuous in all the strips, and then the negative moment capacity will be calculated with the 15 ft. wide beam. The positive reinforcement is only required to be continuous in the column strips to avoid progressive collapse of the slabs, and then only the column strip has been taken to calculate the beam positive moment capacity. The structure is modeled as it was in the limit analysis of chapter III. The input has to name every structural element on the structure and that will be the key to understand the output and for the correct execution of the program. The system is introduced as degrees of freedom with a specific mass each one. Then some supports are defined for each degree of freedom (columns), and the beams are introduced as flexural members at their corresponding height. Finally the joints and the members of the structure have to be numbered to relate them in the space. All this is explained in detail in appendix B, in figures B.1 and B.2 the members and joints numbers are shown.

The input also needs important parameters for the nonlinear calculation. Following the relevant parameters to understand LARZ analysis are explained. The program uses a hysteretic model from Takeda-Sozen, and the exponent chosen for the decreasing slope is 0.5. The most critical input parameter is the capacity of every member in the frame. It is introduced writing three points of the moment curvature diagram of each member. The three points are: cracking point, yielding point, and another point in the yielding slope so the slope is calculated by LARZ. The software does not understand about failure, it simply yields the structure until the infinite. LARZ accumulates all the rotation in the yielded point, if the element has a large yielding slope, then the program will assign more load to that element than in reality will happen. Consequently the slope given for the yielding is really important. The solution adopted to reduce the problem is to reduce to the minimum the yielding slope of each element. In the input the cracking and yielding points are the real ones, but the point in the yielding slope has the maximum curvature value accepted by LARZ with just 0.1 KN-m (1 Kip-in) more than the yielding point. Also the column needs another modification. Since the columns have three reinforcement layers, the moment curvature diagram has two yielding points: one for the exterior layer and another one for the middle layer. To model the structure has been done: calculate the capacity of a hypothetical column with the same reinforcement but distributed in two layers. Summarizing, Figure 8 is a no dimensional diagram showing the solution adopted for the input and the real diagram for the column capacity. Figure 9 is a no dimensional diagram showing the input for the slab capacity and the real diagram.

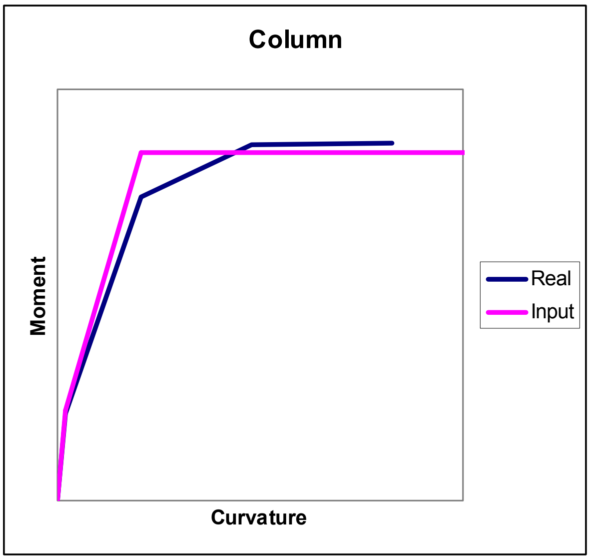

Figure 8

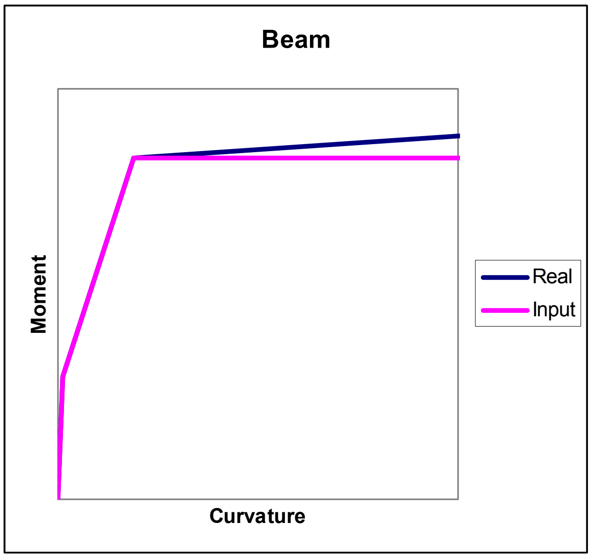

Figure 9

IV.3. Static analysis

In this section the software is used for first time. All the modifications to the test structure described in previous point are necessary. There are two types of load configuration, first an inverted triangle distribution and then a uniform distribution. As it was previously explained is intended to get an upper and lower bound of the base shear. The input for LARZ contains the total load increment for every loading step, this increment is defined in 53.4 KN (12 Kip). This step has been chosen to get approximately 15 increments of load for failure. As explain before, LARZ does not reach failure, just yielding. Thus the objective of the analysis is to get a mechanism that can represent failure in reality. There are two ways to interpret the failure: the slope gets flat at the output diagram Base shear – drift, or looking if the generated plastic hinges create a possible mechanism. Then the main output variables are the mean drift ratio and the stresses in each member. Also is known that the limit for a flat plate structure will be around a 3% of mean drift ratio. Following are the description of the load history and results of the analysis.

IV.3.1. Linear load distribution

The load step is distributed in the following manner, 26.7 KN (6 Kip) per step at the top floor, 17.8 KN (4 Kip) per step in the second floor, and 8.9 KN (2 Kip) per step to the first floor. The moment sign criterion to input in the software is not trivial. Then looking at the moment response for the first load step is checked that the output has the correct moment sign for the loading configuration. The moment signs at the first step are distributed as shown in figure 10. These signs will not change during the elastic load range.

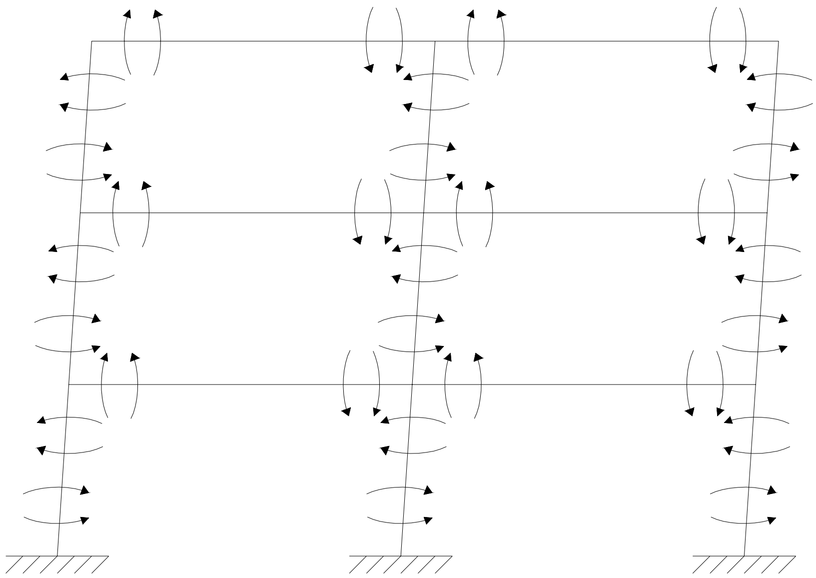

Figure 10. Moment signs.

The first loading steps get an elastic response from the building. Until the fourth load step 213.5 KN (48 Kip) the structure remains uncracked. In this point, the three columns at the base crack, and the positive slabs from first and second floor, are cracked also. This translates into a reduction of the stiffness so the slope of the curve base shear–displacement decreases Figure 11. The slope keeps decreasing while the different members of the system start cracking. By the lateral load 427 KN (96 Kip) all the columns and slabs appear to be cracked and the overall stiffness turns up to be a fifth of the initial uncracked stiffness. At this point the roof drift ratio is 0.5%. During this first range of the behavior, the first and the third story have a similar interstory drift ratio. The second one has this ratio higher because its position with columns above and below. In terms of stiffness, the first story has the larger one. Between the two others, the third story is slightly stiffer than the second due to have no columns above.

There is a transition period between the cracking of the members and the first yielding. During it, the overall stiffness keeps roughly constant so approximately a linear deformation is developed. The interstory drift keeps constant during period of loading; however the first story shows a much higher stiffness than the upper two. The stiffness at the first level has been affected only by the cracking at the column, when the stiffness at the other two has been affected both by column and slab cracking. This effect is reflected at the path of the interstory drift, while the top stories are over 1% ratio, the first one is still around 0.75%. The first yielding occurs at the load 587.2 KN (132 Kip) and mean drift ratio of 1%. The first elements to yield are the three base columns, those three elements are the ones subjected to more stress. Since they are fixed in the bottom they are transferring the entire lateral load to the foundations. Until this position the structure has behave strongly. As prove of it the base shear coefficient is already a large 27% of the total tributary weight. The system is still far from developing its last capacity because only three members have yielded.

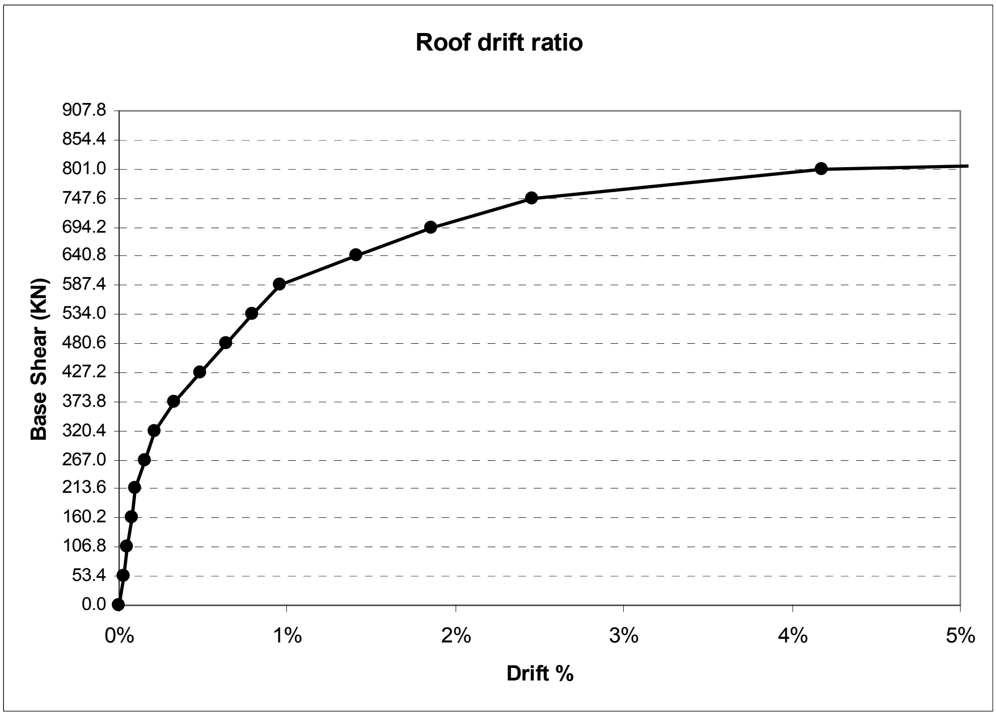

Figure 11. Base shear against Roof drift ratio.

When first yielding occur the overall stiffness of the building experiments the biggest change. The three bases become a point of rotation so the drift increases rapidly in the three degrees of freedom. At this loading status, it is when the software starts to apply the modified yielding slope introduced at the input. When the structure was modeled for the input of the program, it was really important to keep the yielding slope as flat as possible. The software still adding rotation to the hinge, depending on the slope the level of load that the member can carry will be too high. Keeping the slope almost flat we assure that the software does not assign to any member a non reasonable load.

After the 587.2 KN (132 Kip) load step, the yielding starts to appear in the different members of the system. The output can not be taken as absolute truth because the program does not understand about failure of members. However it gives an almost exact approximation of what is happening in the building. Around the 693.9 KN (156 Kip) load, all the positive loaded slabs appear to be yielded, but the overall stiffness of the structure does not seem to be affected. Having hinges at the positive sides of the joints does not increase the drift of the structure; the negative side of the slab at the other side of the column is preventing the joint to increase significantly the rotation. Then as the load keeps increasing more hinges are develop until reach a mechanism somewhere slightly larger than 747.3 KN (168 Kip). It is higher than that value because the load increments are 53.4 KN (12 Kip), and in next load step the structure has already failed. Then all the slabs are yielded in both negative and positive sides. The columns at the base have plastic hinges; therefore the structure can rotate free around them because it the joints have not resistance to rotation any more. The roof drift ratio at this point is 2.5% where the interstory drift for the first and third floor are approximately the same, and at the second floor is vaguely higher. Therefore the deflection shape of the building looks linear. For the further discussion has to be observed the fact that the program is assigning more load than physically possible to some members. The columns at the base, according to LARZ, have a load higher to their ultimate moments.

IV.3.2. Uniform load distribution

The increase of load will be 17.8 KN (4 Kip) in each floor. The purpose of this configuration as mention before is to obtain an upper bound of the base shear of the structure. The general behavior of the structure is the same as it was on the linear load shape; nonetheless some different points are remarkable enough to be explained next.

The moment signs distribution is the expected one, as it was for the linear shape, and serves to proof that the program has a correct input. The elastic part of the loading comprehends form zero until 213.5 KN (48 Kip) of load. During this range the entire system remains uncracked. The overall stiffness of the system is constant and almost equal to the one developed under different load distribution. The first story is the stiffest one due to its fixed columns at the bottom. This elastic zone has rather small drifts, with a mean drift ratio of 0.08% at the maximum loading. The cracking starts at the base of the column in the three columns connecting to the ground, and continues propagating through the members as the load increases. As it can be seen at the figure 12 the stiffness decreases as long as more members crack. The cracking seems to affect the structure until the load step 480.4 KN (108 Kip) approximately. By this time, the stiffness is just around a fifth of the uncracked stiffness.

In the following load increments the variation in slope seems to be stabilized and the structure keeps at barely constant stiffness until its first yield. By that moment the system has reached a mean drift ratio of 0.4% which is significantly smaller than the yielding drift with linear distribution. During the range before yielding, the first story develops stiffness twice high as the other two. In general the behavior of the building is being stiffer whit this new load shape as it was expected. The first yield also appears first at the base columns, but at a higher load 693.9 KN (156 Kip). This means a base shear 32% of the total tributary weight of the building. Although the load is larger, the mean drift ratio is still close to the 1%. The larger interstory drift ratio occurs at the second level, while the first remain the smaller and the third in between. This makes the building to have and S-shape deflection. After the three bases yield, the entire structure tries to rotate around them, and generates new hinges within the different joints.

The small yielding slope introduced at the software keeps the moments down in these three already yielded members. During this loading period, the stiffness is significantly reduced and it is approximately twenty times less than the uncracked. Within the next few steps the building absorbs the increasing load by developing new plastic hinges in different joints. The elements yielded are the positive loaded sides of the slabs, starting to generate those hinges around 800.7 KN (180 Kip). The structure reaches a mechanism after the load step 907.4 KN (204 Kip). It is after this because in the next one the structure is totally failed. The Roof drift ratio as it is observed in Figure 12 is vaguely over 2.71%. At this load step the interstory drift ratio generates an S-shape to the building. The second story has a larger drift than the other two.

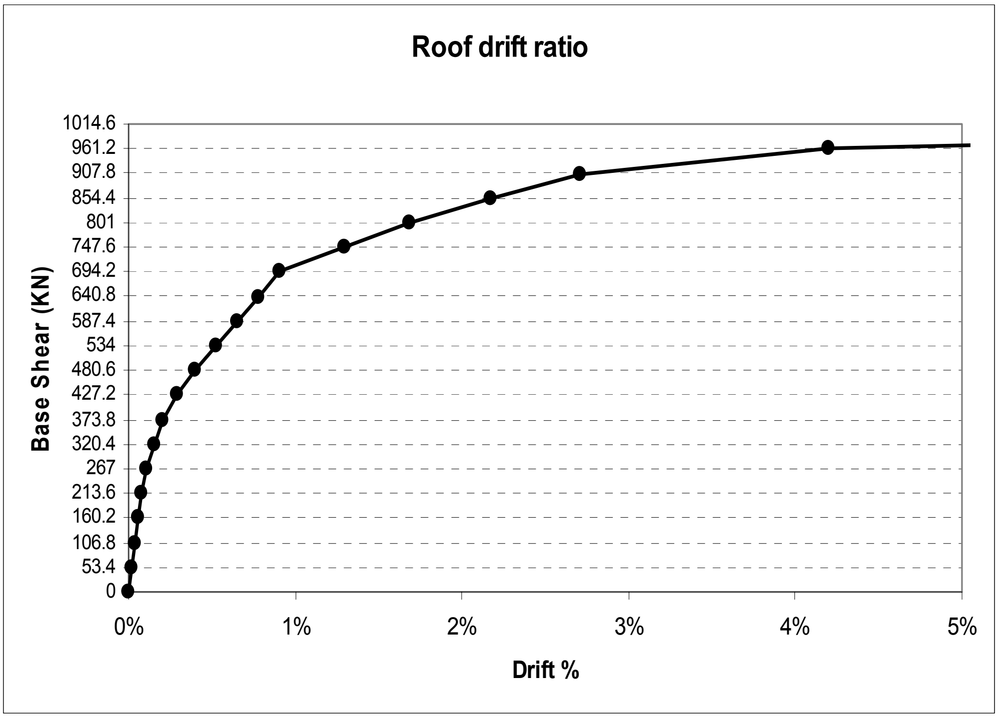

Figure 12. Base shear against Roof drift ratio.

As summary for this section, the static analysis developed with LARZ provides two boundaries for the base shear of the structure 747.3 KN (168 Kip) and 907.4 KN (204 Kip). Also is an approximation of the behavior of the building until collapse, showing the cracking and yielding points and relating them to drift ratios. In Appendix B section B.2 are the tables showing the detailed stresses and drift history tables for each member, these tables will be used for the discussion of the results.

IV.4. Dynamic analysis

In this section earthquakes effects are going to be simulated using LARZ. First of all a selection of the ground motions will be done. Then will be calculated the period of vibration of the structure. Two methods are going to be used, the one used by the uniform building code as a simplification, and an analytical method to find the exact solution for the three modes of vibration of the structure. The period is not important itself for the dynamic analysis, but gives a good idea of the structure elasticity and also allows a basic spectrum analysis. After this the structure is going to be shaken with 18 different earthquakes. These earthquakes are enumerated in Table 7. The earthquakes have been carefully selected. The initial idea was to run the LePage set, in this way a option to compare the response offered by LARZ with the LePage approximation can be done. Afterwards this set of ten earthquakes was extended until eighteen selecting a variety of strong motion records. These ones were selected in order to get different values for Peak Ground Velocity PGV and Peak Ground Acceleration PGA. Also a variety of regions around the world was selected. Motions from West pacific, East Pacific, Anatolian, Italian, some more motions. The intention is to see the behavior of the building subjected to earthquakes from light to severe grades and also to compare the response to Lepage approximation. The structure response will be analyzed in terms of Base shear, drift, and yielding levels.

Table 7. List of applied earthquake motions.

Motion	Country	LePage set
Cape Mendocino	U.S.A.	No
Imperial Valley	U.S.A	Yes
Kern County*	U.S.A	Yes
Northridge	U.S.A	Yes
San Fernando	U.S.A	Yes
Western Washington	U.S.A	Yes
Valparaiso	Chile	Yes
Friuli	Italy	No
Irpina	Italy	No
Erzincan	Turkey	No
Duzce	Turkey	No
Kocaeli	Turkey	No
Tabas	Iran	No
Chi-chi	Taiwan	No
Miyagi ken Oki	Japan	Yes
Kobe	Japan	Yes
Tokachi Oki	Japan	Yes

*Kern county earthquake is applied twice from a different station record

IV.4.1. Uniform Building Code:

The code actually uses one formula to approximate the lower period of a reinforced concrete frame structure. However before 1997 an older formula was used, it is interesting to comment it because its simplicity. The analytical method is going to be used further in this chapter will provide a close solution to the exactness, but these two formulas provide an easy starting point. The first formula until the code of 1997 was:

$$T = \frac{N}{10} \tag{4}$$

Where N is the number of stories. This gives a period of 0.3 sec. for the present building. In 1997 the formula was replace by this one

$$T = 0.0731 \cdot h^{\frac{3}{4}} \tag{5}$$

Where h is the height of the building in feet. This last formula brings a period of 0.38 sec.

As summary, these two approximate periods are a starting point to think for the dynamic analysis of the building. However these two equations do not represent the effect of mass and stiffness of any member. These changes in mass and stiffness distribution affect the period. Although their simplicity these two periods represent what the UBC expects from this structure.

IV.4.2. Proper method:

The derivation of this method is directly from the equilibrium equations of the system. The system is modeled as shown in figure 13 without any damping device in any degree of freedom. The system will be subjected to an undamped free vibration of a multi degree of freedom system. The building is assumed to be a wire frame; with this assumption the increase of stiffness at the joints due to the rigid parts of the members is avoid. The result model is as it has been in previous chapters: two frames, three stories, 2 bays in the longitudinal direction and one bay in the transversal one. The system is a multi degree of freedom, modeled as three concentrated masses linked by shear springs. Each of these springs represents story stiffness. The beams are understood to be flexible; therefore they affect the stiffness of the story. In case the beams were assumed rigid, it would be an underestimation of the period.

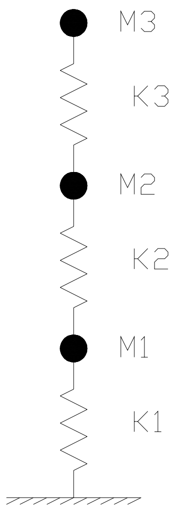

Figure 13. Structure model.

The members have stiffness equal to their moment of inertia divided by their length. The modulus of elasticity or Young's modulus is assumed 24821 Mpa (3600 Ksi) as it is for all the calculation in this dissertation. Once the story stiffness is calculated the building is modeled as concentrated masses connected by springs. To find the equilibrium equations, D'Alembert's principle of dynamic equilibrium (8) is used. A system of three equations is achieved and expressed in matrix form (9). The system is assumed to have a harmonic solution and then the standard Eigen problem is found (11). Once the eigenvalues and eigenvectors are solved, the mode shapes, and mode periods are reached. The main steps of the process are described below in analytical form:

Stiffness of the members:

$$K = \frac{Inertia}{Length} \tag{6}$$

Story stiffness:

$$K = \frac{24 \cdot E}{H^2} \cdot \frac{1}{\frac{2}{\sum K_c} + \frac{1}{\sum K_{ga}} + \frac{1}{\sum K_{gb}}} \tag{7}$$

The sub index c means Column, ga Girder Above and gb Girder Below

Equilibrium equation:

$$[m] \cdot \ddot{x} + [K] \cdot x = 0 \tag{8}$$

Operating it:

$$[I] \cdot \ddot{x} + [m]^{-1} \cdot [K] \cdot x = 0 \tag{9}$$

$$[D] = [m]^{-1} \cdot [K] \tag{10}$$

If a harmonic solution is assumed, the displacement vector can be expressed as:

$$[x] = \{A\} \cdot \sin w \cdot t \tag{11}$$

Then the acceleration vector is:

$$[\ddot{x}] = -\lambda \cdot \{A\} \cdot \sin w \cdot t \tag{12}$$

$$\lambda = w^2 \tag{13}$$

Substituting (10) and (11) into (9):

$$[[D] - \lambda \cdot [I]] \cdot \{A\} = \{0\} \tag{14}$$

And this is the standard eigenproblem. The solution is composed by three eigenvalues and three eigenvectors from which the modes of vibration and the periods can be found:

$$\lambda = \begin{bmatrix} 127.891 \\ 955.539 \\ 1796.833 \end{bmatrix} \qquad w = \begin{bmatrix} 11.309 \\ 30.912 \\ 42.389 \end{bmatrix}$$

$$T = \begin{bmatrix} 0.56 \\ 0.20 \\ 0.15 \end{bmatrix} \qquad \Phi = \begin{bmatrix} 0.303 & -1.092 & 3.024 \\ 0.746 & -0.897 & -2.567 \\ 1 & 1 & 1 \end{bmatrix} \tag{15}$$

IV.4.3. Ground motions

To execute LARZ in this section the structure was modeled exactly the same as it was for the static analysis. It is a wire frame structure, two frames, three stories and two bays in the longitudinal direction. There are some variables different in the program input for the dynamic case. All these differences are explained next.

To compute the stiffness of the structure and to see how it changes with the time, the program needs an assumption for the damping coefficients of the two first modes of vibration. The damping coefficients are assumed to be 2% for both modes. This damping only affects the elastic behavior of the reinforced concrete, after cracking and yielding, hysteretic damping takes part. A 2% coefficient has been selected because is the minimum damping that reinforced concrete can have. With these two assumptions LARZ calculates the alpha and beta coefficients. Basically they are the mass damping coefficient and the stiffness damping coefficient. In the input there is a variable related to damping; it defines the number of computation steps between the stiffness recalculation. This variable was originally design to save computational time; nowadays a normal PC is powerful enough to run the program this variable is setup as 1.

At the end of the input, the seismic motion is introduced. In this part there is one critical parameter: the interval of time for computation. This interval has to be kept equal or below a tenth of the motion time interval. When the program is integrating the motion is important that reads between the peaks of the motion, in other case would cause numerical problems and non correct answers. At the same time, as the time interval gets smaller the output data becomes larger. Then a simple variable reports the output every defined number of load increments, helping for disk storage economy.

The structure was subjected to 18 earthquakes simulations, below the representative data of each earthquake is explained. For each earthquake the relevant data is included; the data of the motion itself with geographical and magnitudes specifications. It is also included the combined response spectrum (spectral displacement, pseudo spectral acceleration and pseudo spectral velocity) and the base shear-roof drift ratio response. In each earthquakes the structure behavior will be explained in order to elaborate the afterwards conclusions. For more detailed data, all the response given by the software is included in the Appendix B along with the different motion records. In this Appendix the response is not on the original support format, it is processed into excel format to make its comprehension better.

An earthquake is a natural phenomenon; hence every motion is different and has singular characteristics. During the recent history different manners have appear to classify earthquakes. Into these multiple manners the two principals are: Group the motions by the energy they release at the source or classify them by the physical measured properties as PGA or PGV. The first method is a quantitative manner, measures the energy based on the maximum amplitude recorded. The most known scales in this group are Richter (Magnitude-scientific) and Modified Mercali Intensity (Intensity-journalistic). The second method has only been developed for a few decades because the distribution of seismic accelerograms around the world has been slow and limited to obvious economic resources. For the purpose of this study, mainly focused in strength and drift analysis, the proper approach will be either PGA or PGV. The motions in this paper are arranged by PGV; because the relation between drift and velocity has been always more clear than drift and acceleration. In the analysis of each ground motion, two figures are included. The first one refers to the motion and is its combined response spectrum. From the spectrum, if scaled in the paper, the three components of the earthquake can be easily read: pseudo spectral velocity, pseudo spectral acceleration and spectral displacement. The second one is a plot Base shear-Roof drift ratio, this plots represents the behavior of the structure, should be linear while the structure is elastic. It has been carefully dimensioned: the limits are ±890KN (±200 Kip) and ±3% because these are the expected limits for the collapse of the structure. Then any plot can be quick evaluated looking at its linearity and the closeness to the limits.

Western Washington:

Year	1949
Magnitude	6.9
PGA	0.068 g
PGV	8.22 cm/s
Station	Seattle Army Base S02W

This motion is part of the LePage set; the characteristic of the recorded motion were small acceleration and small velocities. The record duration was 66 sec. In figure 15 is the response spectrum. The spectrum shows how the earthquakes excites a wide band of periods, however is remarkable the excitation of periods around 1 sec. The motion acceleration record in the Appendix B figure B.3 has no remarkable characteristics due to its low acceleration. In figure 14 is the response given by the software. It does not behave outside the linear range. The base shear keeps a linear relation with the drift, thus the maximum base shear corresponds to the maximum drift ratio. The maximum base shear is 284.7 KN (64 Kip) or the 13% of the weight of the structure. It is a low number compared to the base shear found in the limit and static analysis. Thus the response shows no yielding or big deformations. The maximum mean drift is 0.16%. This number also indicates the building is subjected to small loads. The deflected shape looks like an “s”. The first and the third floor are stiffer than the second. It confirms then the same behavior observed at the static analysis in the elastic range, where the middle floor was subjected to larger interstory drift ratios.

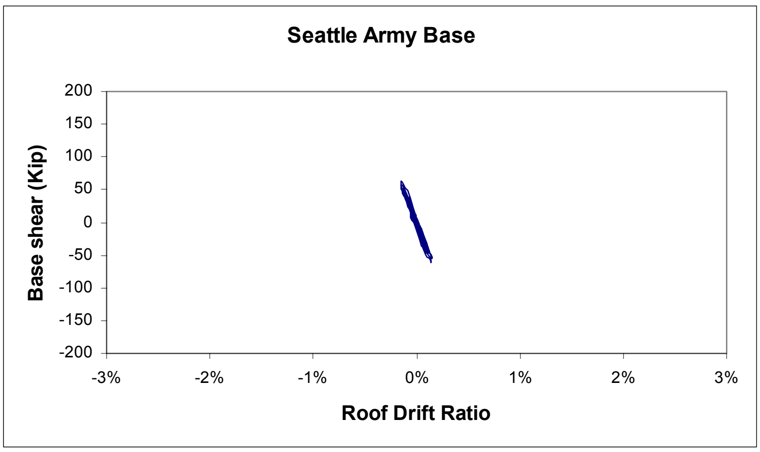

Fig14. Base Shear-Roof Drift Ratio response

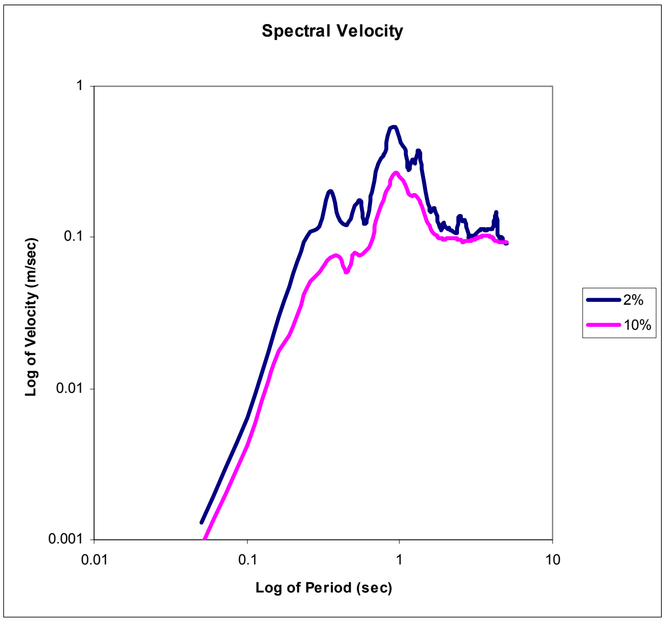

Fig 15. Response spectrum for Seattle earthquake

Kern County:

Year	1952
Magnitude	7.4
PGA	0.156 g
PGV	15.60 cm/s
Station	Taft Lincoln N21E

This motion is part of the LePage set also; its characteristics are medium-small acceleration and small velocities. The duration of the motion was 45 sec. The response spectrum in figure 16 is flatter than before. Thus more periods are excited by the motion.

In the response shown at figure 17 even if it is still a small earthquake, it can be appreciated how the structure behaves out of the linearity. The path moves around a line but this line is getting thicker as the earthquakes increases in strength. The maximum base shear calculated for the structure is 409.2 KN (92 Kip) or 19% of the weight. Consequently important stresses appear at the structure members, but none of them is close to yielding. The maximum ductility reached by a member is 0.58 which is still far from yielding. The maximum drift remains at 0.46%, as comparison with the static analysis, the structure remains close to the elastic behavior therefore no risk of failure exist. The deflected shape at maximum roof drift looks like an "s" also. The interstory drift ration for the maximum roof drift indicates the third floor stiffer than the first one. For the previous motion both were the same, indicating no that the third floor stiffness gets similar to the second one as long as the load increases.

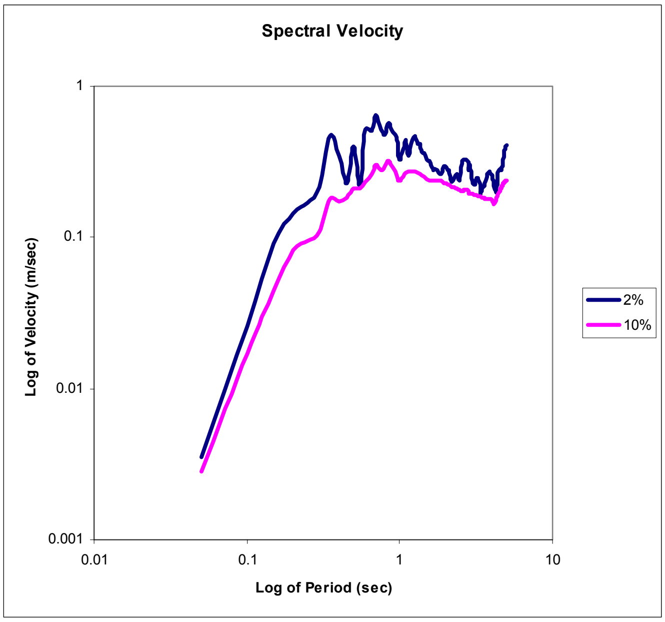

Fig 16. Response spectrum for Kern County earthquake

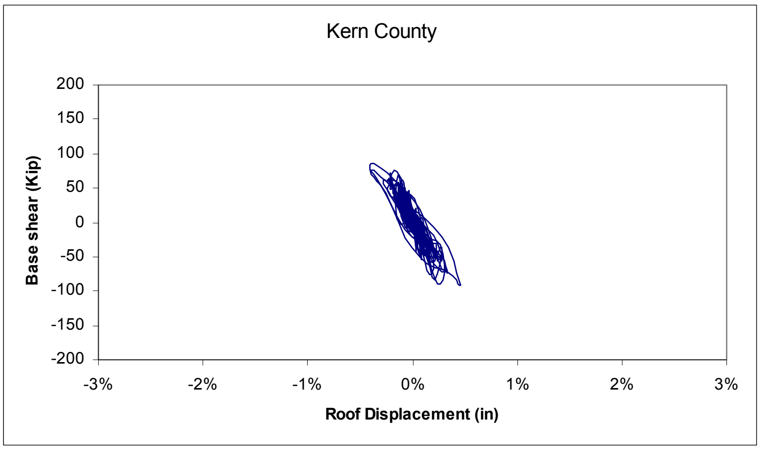

Fig 17. Base Shear-Roof Drift Ratio response

San Fernando:

Year	1971
Magnitude	6.6
PGA	0.316 g
PGV	17.16 cm/s
Station	Castaic N21E

This motion is also part of the LePage set; its characteristics are medium to strong acceleration and medium-small velocities. The duration of the motion was 30 sec. The response spectrum in figure 18 is more constant in the constant velocity range. It does have a peak around one second period, but is not as obvious as it was in the previous earthquake.

As seen in Fig 19 the earthquake does not produce large excitations to the building. The structure is mainly kept inside the linear range except some points. The maximum base shear corresponds to maximum drift in general. The whole structure stays without any yielding, observing that the yielding is smaller than in the earthquake before. Anyway the increase in PGV was too small to expect a large increase in the response. The maximum base shear remains in 19% or 409.2 KN (92 Kip). The maximum roof drift ratio decreases till 0.34% which means no risk for the structure integrity. The interstory drift ratio is low as well, being the second floor the higher one. The deflected shape for the maximum roof drift shows how the first floor is stiffer than the others. This behavior was expected due to no yielding on any member.

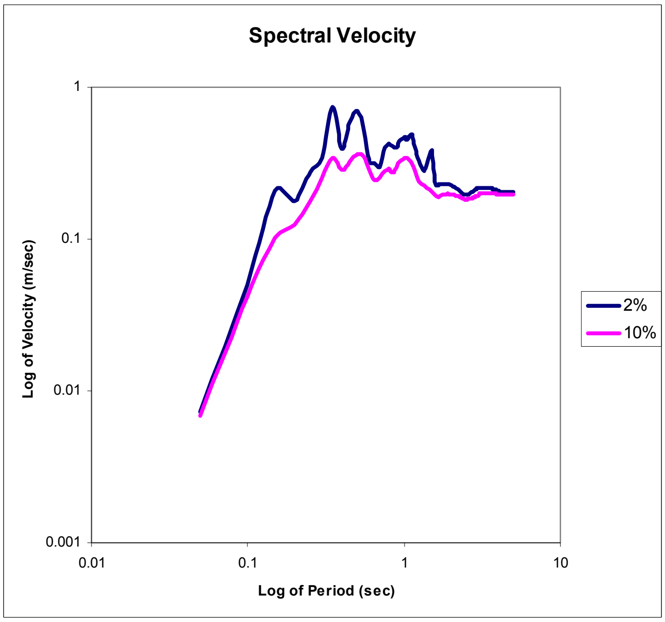

Fig 18. Response spectrum for San Fernando earthquake

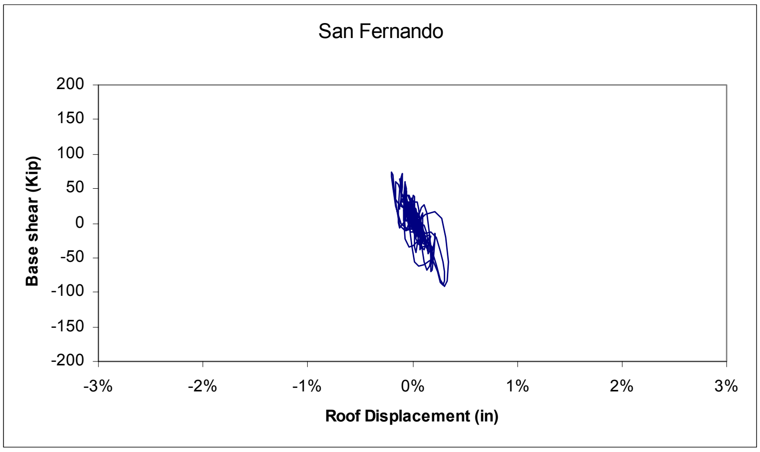

Fig 19. Base Shear-Roof Drift Ratio response

Kern County:

Year	1952
Magnitude	7.4
PGA	0.131 g
PGV	19.25 cm/s
Station	Santa Barbara Court House S48E

LePage included this motion in its investigation. It is the same earthquake than before (Kern County) but the record is from different station. It can be observed in the spectrum that this record has narrower spectrum than the one measured in Taft Lincoln. Our calculated period of the structure 0.56 sec. is less affected; hence the response obtained is smaller than before.

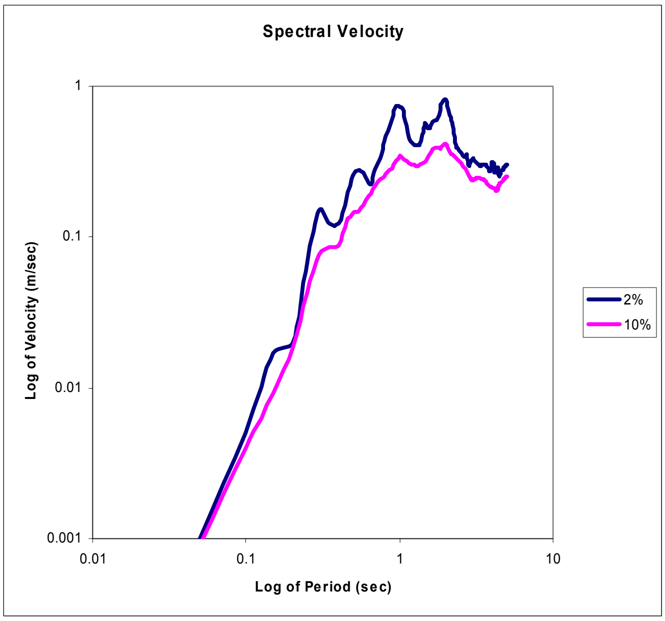

Fig 20. Response spectrum for Kern County earthquake

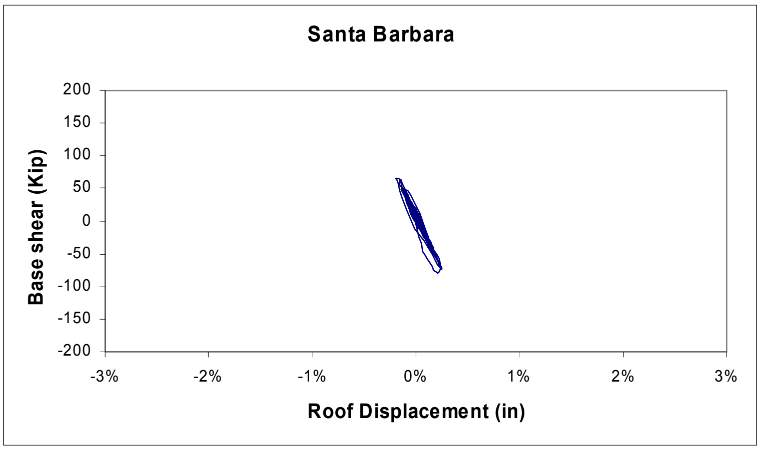

Fig 21. Base Shear-Roof Drift Ratio response

As it can be seen in Figure 21 the motion excites the building in its linear range. The excitation is clearly less than for the other station recorded in the same earthquake. Even if it was expected higher response due to slightly higher velocity, the probably reason is the more narrow spectrum. It barely appears some cracking in a few members of the structure. The base shear remains low 16% compare to the base shear calculated for the limit analysis, and the drift is not significant (0.25%). At the deflected shape for the maximum displacement at top, an S shape is observed. This means that the stills inside the elastic range. Being the first floor the stiffer one.

Friuli:

Year	1956
Magnitude	6.5
PGA	0.315 g
PGV	30.80 cm/s
Station	Tolmezzo 270

This motion does not belong to the LePage set, but is one of the Italian earthquakes selected for the analysis. The earthquake has significant values of PGA and PGV, and it is close to the well known Imperial Valley earthquake in terms of those values. Looking at the spectrum response, it is narrow and exciting intensely the periods slightly lower than one sec. In the Fig 22 it is important to notice the change in scale of the ordinates axis. From now on the spectrum PGV scale will be between 0.01 and 10 m/sec. The four preceding motions where between 0.001 and 1 m/sec. Observing the motion in the Appendix B figure B.7 it is seen that is basically comprised into five seconds. At this time is the maximum excitation but during most of earthquake the motion is so small that elastic behavior can be expected.

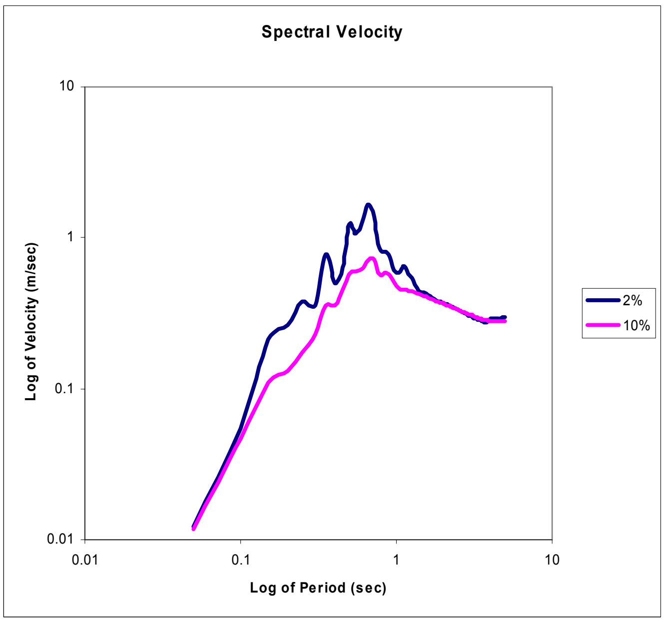

Fig 22. Response spectrum for Friuli earthquake

The response of the structure is higher than any earthquake before. The plot on Fig 23 shows how the relation between base shear and drift is linear for some time but in the moments of high excitation it gets a little erratic. The maximum Base shear does not correspond to the maximum drift, and the structure does not behave in the linear range. The maximum drift is 1.04%, and as it was expected from the static analysis, some yielding appears at the base columns. Also some slabs are close to yielding with ductility values of 0.9. Due to this small yielding some deformation remains at the end of the motion. It is only a 0.04% but is representative of the yielding occurred during the earthquake. The base shear is getting near to the calculated for the limit case, and it is a 30% of the weight of the structure 658.3 KN (148 Kip). The deflected shape for the maximum displacement shows the first story stiffer than the other two. The second and third stories have the same interstory drift ratio.

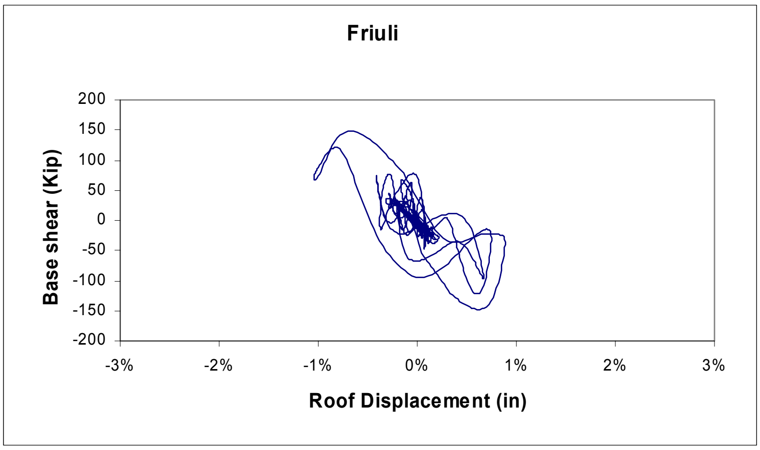

Fig 23. Base Shear-Roof Drift Ratio response

Imperial Valley:

Year	1940
Magnitude	7.0
PGA	0.349 g
PGV	33.45 cm/s
Station	El centro array 180

This motion is one of the most known in the earthquake engineering environment. That is because it was one of the first motions to be recorded; it also displays a pretty harmonic shape with peaks rather well distributed along time. The spectrum includes a good band of periods figure 24. The first region of the spectrum follows quite well the standard of constant acceleration; this is because the slope is close to 45 degrees. In terms of PGA and PGV the values are not high. It is though a strong earthquake due to its excitation of most of the common periods. The motion was 40 sec long. It is considered the standard motion, and usually taken a good model to design drift theories.

In the calculated response, the building gets into in non elastic behavior. Figure 25 shows how the structure is out of the linear range for a long time during the earthquake. The characteristics are similar to the previous earthquake (Friuli) the response is similar too. Base shear reach a maximum at 689.5 KN (155 Kip) or 31%

of the weight. The drift increases a little bit but that is due to the increase in velocity. Maximum mean drift ratio is 1.20%; it is achieved right after the moment of PGA. The yielding appears only at the base columns again, but it develops higher moments. Some slab members are close to yielding too. The moments reached at the middle base column is close to failure. LARZ gives higher moments than reality after yielding due to its infinite yielding slope. However some compression cracking and damage at least in the middle base columns may be considered. At the end of the motion a small permanent deformation is also observed. This value is about 0.08% roof drift ratio probably due to the damage in the base columns. The deflected shape for the maximum drift looks like the one from Friuli. The first story is stiffer than the others. Third and second appear to have the same interstory drift.

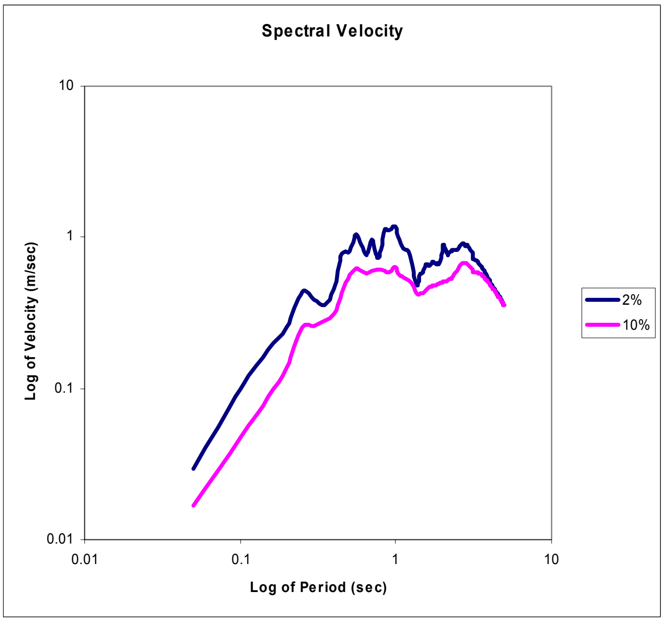

Fig 24. Response spectrum for Imperial Valley earthquake

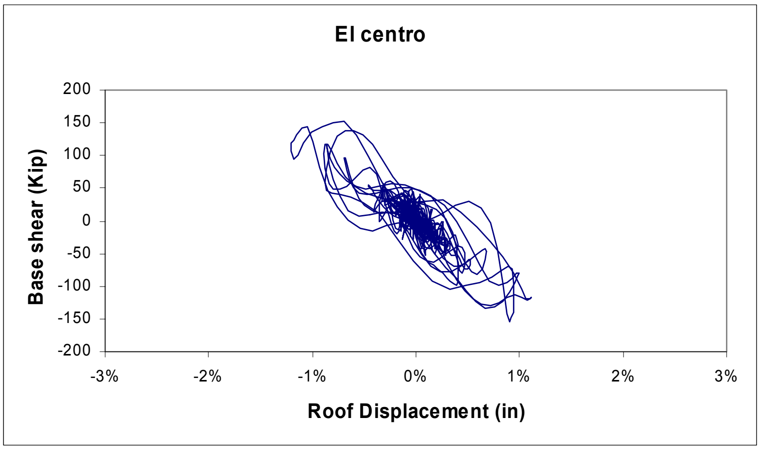

Fig 25. Base Shear-Roof Drift Ratio response

Miyagi-ken Oki:

Year	1978
Magnitude	7.5
PGA	0.263 g
PGV	37.39 cm/s
Station	Sendai NS

This motion is also inside LePage set of earthquakes. It has significantly smaller acceleration than the previous but an increase in the velocity. Looking at the motion record in the Appendix B figure B.9 an important issue can be observed: the acceleration has almost constant amplitude during half of its 40 sec duration. The peaks are also rather equally distributed along the time axis. This can be the explanation for the kind of narrow response spectrum in figure 25, exciting structures strongly with periods vaguely over one second. The velocity is higher in relation to the PGA due to the big spacing between acceleration peaks.

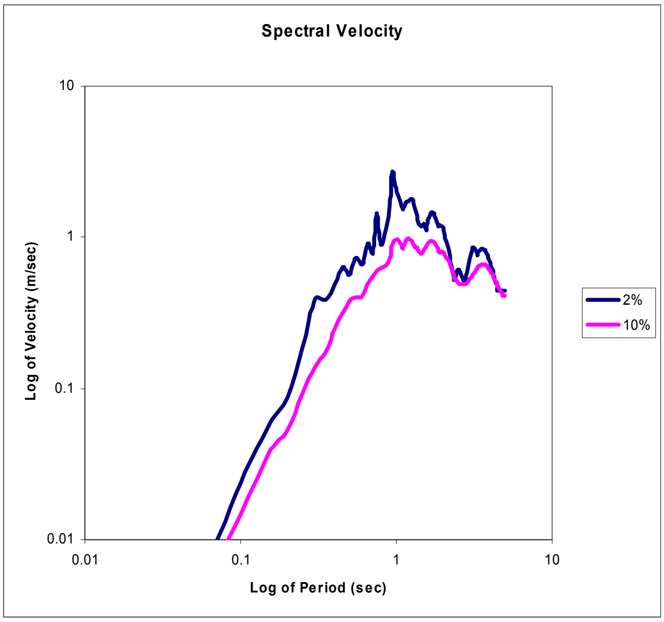

Fig 26. Response spectrum for Miyagi-ken Oki earthquake

The structure responds quite badly to this earthquake. Both base shear and displacement plotted against time look like the motion. Peaks are equally distributed and have similar amplitude for half of the record. This regularity is shown as well in the figure 26 because the plot follows a path of linearity. The problem is that the roof drift ratio is brought over 1% for most of that time. This translates into a high moments at the base columns and the slabs. The slabs go over yielding in every joint, and the columns at the base are yielded in excess. LARZ does not tell if they reached failure, but judging by the ductility it is obvious that permanent damage is achieved. However at the end of the record no permanent deformation is observed, probably due to the regularity of the shaking. The structure moves back and forth equally so no permanent drift remains on it. Even if the building gets a high mean drift ratio of 1.91%, the base shear remains as it was for previous earthquakes, which is around

30% of the weight. The deflected shape at the maximum roof displacement looks almost linear for the three stories, meaning that the interstory drift ratio is similar for all the levels. This is basically due to the hinges distribution. They form a mechanism similar to the one developed in the static analysis.

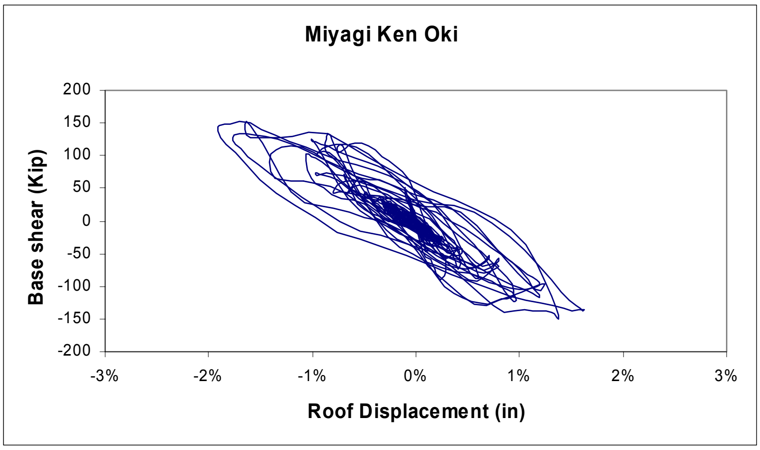

Fig 27. Base Shear-Roof Drift Ratio response

Valparaiso:

Year	1985
Magnitude	7.8
PGA	0.713 g
PGV	41.49 cm/s
Station	Llolleo N10E

This earthquake also pertains to the LePage set. It was a long record (70 sec) and its main characteristic was the rapid variation in acceleration. It has a large PGA and not that large PGV. This is consequence of the small time interval between acceleration peaks. The motion has large an amplitude during approximate 30 sec. For this time, the acceleration is always around 0.3 g, having larger peaks in individual points. The response spectrum, figure 27, looks like a broad band spectrum, comprising periods form 0.3 to 6 sec approximately. It has a peak exactly over 0.55 sec, thus high excitations can be expected on the structure under study.

The drift response for the building is not as high as it was with the precedent earthquake. Nonetheless looking at figure 28 the response looks “thick”. This is due to the regularity showed by the base shear response. The base shear is rather constant during the mid part of the motion as it can be seen in Appendix B figure B.43. This makes that the drift changes but the base shear not. Then the plot in figure 28 gets a little bit of square shape, for the same base shear bunch of drift values are obtained. The maximum base shear for the response is 782.8 KN (176 Kip).

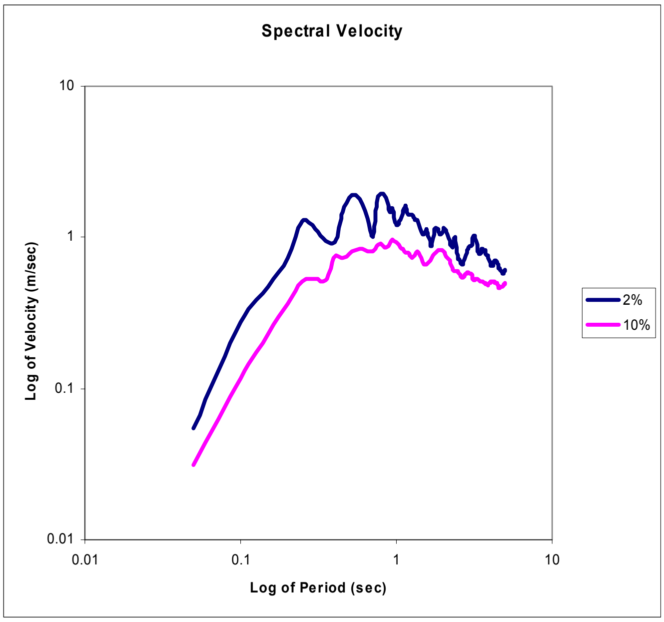

Fig 28. Response spectrum for Valparaiso earthquake

In a different plot, the drift response against time seems to have three different parts. In the first one the excitation is around 0.5% roof drift. The second part around the middle of the motion has the largest drifts, reaching there the maximum drift for the whole record 1.36%. In this part is where the building yields and gets a permanent deformation. Then in the latter part the structure is excited around 1% mean drift ratio amplitude. In this final part of the plot the structure vibrates around the new “zero” created by the permanent deformation. This remaining deformation at the end of the shake is approximately a 0.2% mean drift ratio. Yielding just occurs at the base columns members; nevertheless all the slabs are so close to yielding that is difficult to say they do not reach yielding point. The deflected shape for the maximum deflection looks like the most of the previous responses, stiffer at the first floor and equal for second and third.

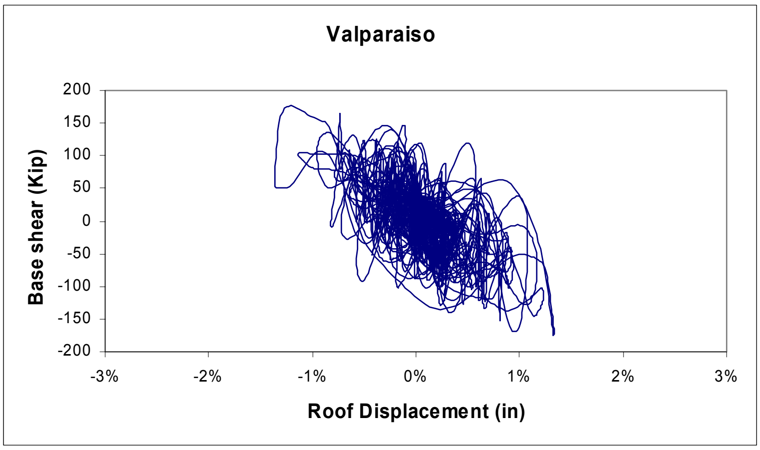

Fig 29. Base Shear-Roof Drift Ratio response

Tokachi Oki:

Year	1968
Magnitude	7.8
PGA	0.187 g
PGV	43.00 cm/s
Station	Hachinoche EW

This earthquake is quite opposite to the last on. In this case the PGA is not large but the PGV is little bit larger than Valparaiso. This is possible because the acceleration peaks at the record are more spaced in the time, so the velocity is bigger even if the acceleration is small. Also the fact that these peaks are quite evenly distributed in time makes the response spectrum narrower. The spectrum is plotted in figure 29 and shows the larger excitations for periods higher than one sec. This record was also used by Lepage to elaborate his theory.

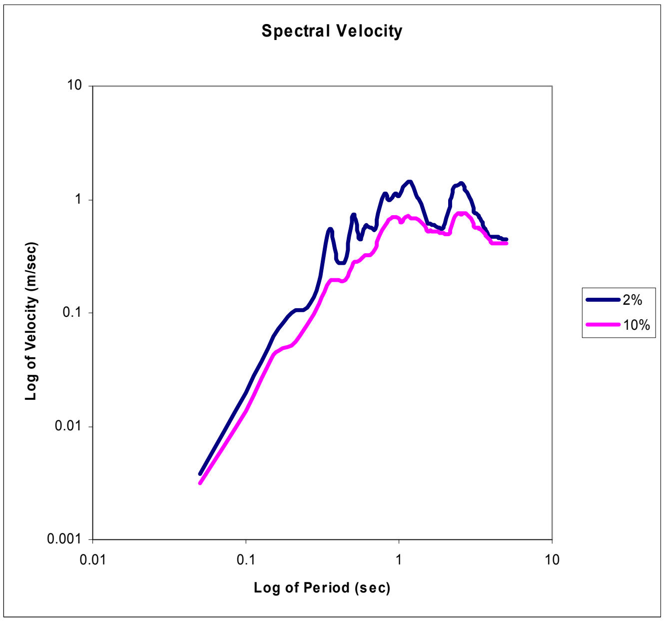

Fig 30. Response spectrum for Tokachi Oki earthquake

The response for this earthquake looks quite linear in figure 31. The base shear increases with the drift increase. However for the larger base shear values, the path does not follow well the linearity. The maximum base shear for this motion is 640.5 KN (144 Kip) or 29% of the weight. This occurs for the maximum roof drift ratio 1.23%. The structure have its higher response at the beginning of the motion and the rest stay vibrating with an approximately 0.3% amplitude. The building does not have overall yielding, but it does at the base columns, the rest of the member specially the slab members only develop about a 70% of its moment capacity. The deflected shape appears to be stiffer at the first level. The building reaches its maximum interstory drift ratio 1.39% at the second story.

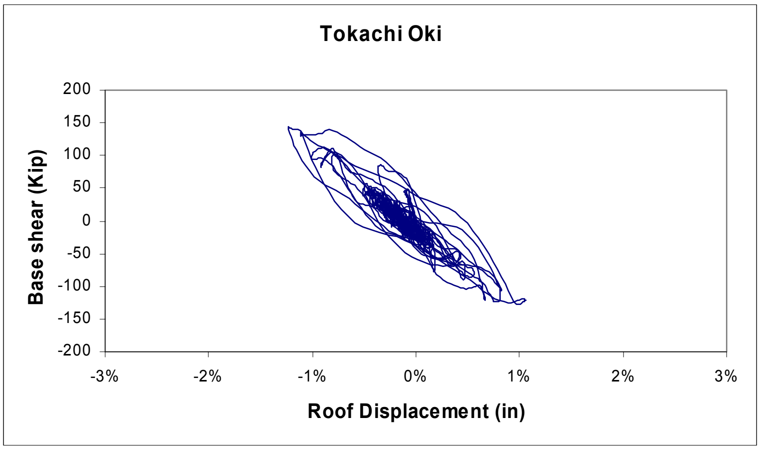

Fig 31. Base Shear-Roof Drift Ratio response

Irpinia:

Year	1980
Magnitude	7.2
PGA	0.358 g
PGV	52.70 cm/s
Station	Sturno 270

This earthquake pertains to the Italian region. The motion does not have any special characteristic, except at the beginning, where the acceleration peaks are rather separate. This generates three velocity peaks at the first third of the record. After that the earthquake is decreasing rapidly. The response to this earthquake is low compared to its PGV. For the past two motions the response showed a decreasing trend with PGV increase. This trend goes against Lepage approximation. The probable reason is in the population of earthquakes used for this study; these motions are probably lower bounds. The maximum mean drift ratio is barely 1%. Yielding only appears at the building in the base columns. Consequently, the damage in the structure is moderate and no risk of collapse appears. Nevertheless around 1% of roof drift ratio is expected to have severe damage in the masonry and the inside of the building. At the end of the motion a permanent deformation is observed. Nonetheless this deformation due to the yielding at the base is really small, 0.05% mean drift ratio. The base shear remains fairly linear with the drift. The maximum correspond to the maximum drift, and only for the highest values shows a behavior out of the linearity. The maximum base shear is 29% of the total weight of the structure. The interstory drift ratio shows no strange values, and the second floor has less stiffness as it has been observed in previous earthquakes.

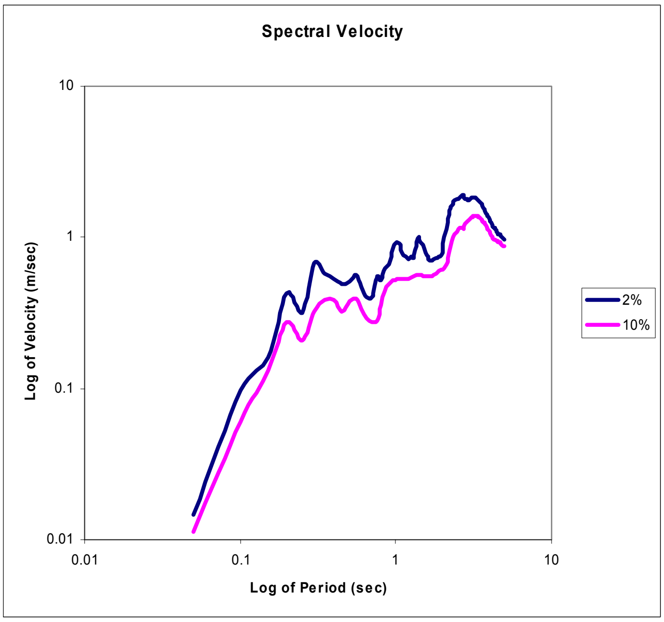

Fig 32. Response spectrum for Irpinia earthquake

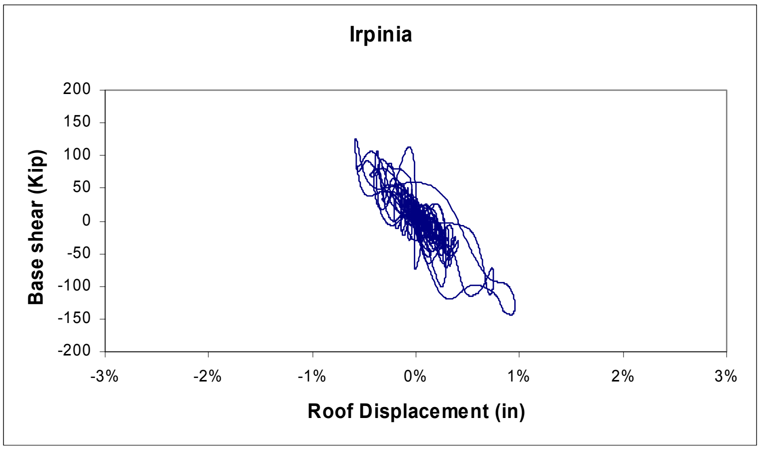

Fig 33. Base Shear-Roof Drift Ratio response

Kocaeli:

Year	1999
Magnitude	7.4
PGA	0.312 g
PGV	58.80 cm/s
Station	Duzce 180

Kocaeli pertains to the Anatolian earthquakes, is the first of the two big motions registered at Marmara region in 1999. The response spectrum shows a wide band of periods affected by the motion figure 34. The drift analysis for Kocaeli gives a maximum mean drift ratio around 1%. This is consistent because the characteristics do not change too much from the Irpina earthquake just analyzed. However, now the earthquake is strong most of the 28 sec of duration, while in Irpina it was only strong at the beginning. The structure only achieves yielding at the columns fixed to the ground. Looking at the overall structure, except those columns the rest of the members are slightly over their half capacity. The base shear plotted against roof drift ratio, figure 35, follows vaguely a linear trend. The base columns yielding cause a small permanent deformation about 0.03% at the end of the earthquake. The maximum values calculated are in Appendix B table B.15. The base shear gets a maximum of 591.6 KN (133 Kip), about a 27% of the weight, and the roof drift ratio gets a maximum of 0.96%. Looking the results of the calculations, no big structural damage is observed. This response contrasts with the reality because it was a major earthquake with a large number of casualties. The interstory drift ratio looks not alike with the mean drift ratio, because the maximum is 1.23% for the third story, relatively large compared to the 0.96% of the building. The deflected shape for the maximum roof drift shows this behavior where the first story is stiffer than any one else, and the second stiffer than the third.

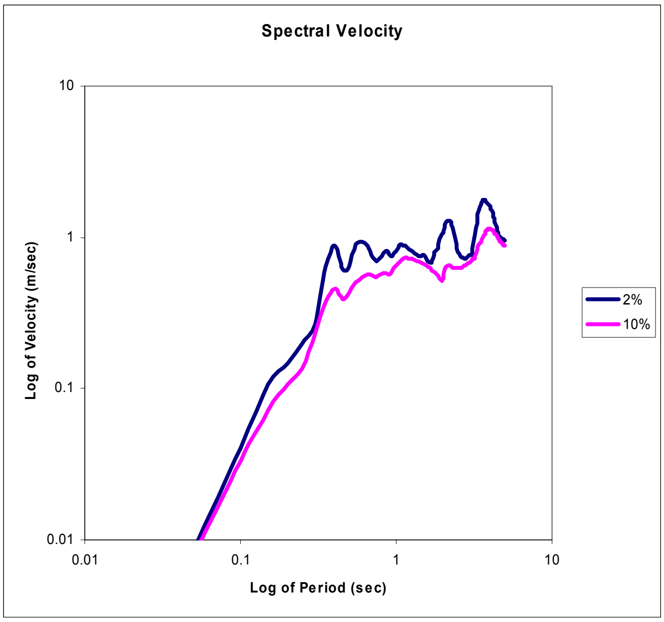

Fig 34. Response spectrum for Kocaeli earthquake

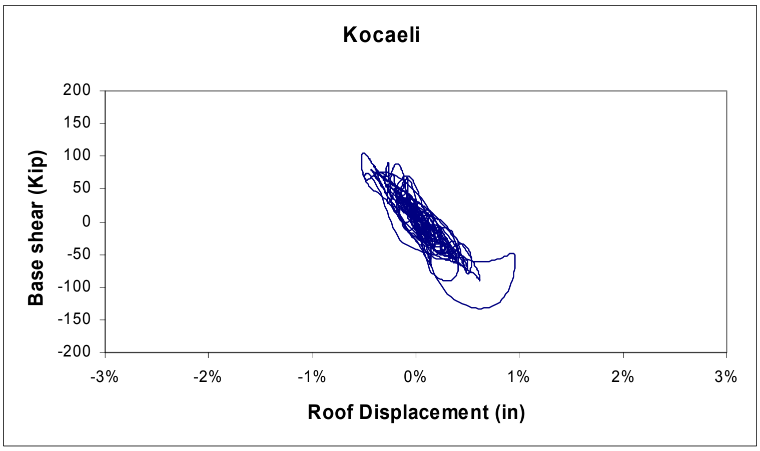

Fig 35. Base Shear-Roof Drift Ratio response

Northridge:

Year	1994
Magnitude	6.7
PGA	0.991 g
PGV	77.88 cm/s
Station	Tarzana NS

This major earthquake is also one of the LePage selections. Its PGA value is the second higher out of the 18 earthquake selection in this study. The response spectrum figure 36, affects specially those periods near the present structure period. It is a high demanding motion for a forth of its time, displaying large acceleration and velocities during this time. The building was shaken for drift amplitudes over 1% most part of the motion, Appendix B figure B.54. Thus the structure is severely damaged. The maximum roof drift throughout the record was 1.44%. In figure 37 the evolution of base shear against drift appears not following linearity. The static analysis presented a base shear limit for the structure in the range 800.7-889.6 KN (180-200 Kip), and during the motion this level of base shear is achieved various times. The yielding level is close to generate a structural failure at the structure. Plastic hinges appear in the second and third slab; the columns at the base are yielded and get close to the ultimate capacity. It can be appreciated also how the second levels of columns is starting to yield at the bottom. Thus the building can not be considered collapsed because all the members are still under ultimate limits. But stern conditions are reached at the structure. The interstory drift ratio is higher than the mean for the second and third story. The latter one is almost reaching 2% ratio. This shape can be seen in Appendix B figure B.56 where the third floor seems to be the lesser stiff one, generating a cantilever shape.

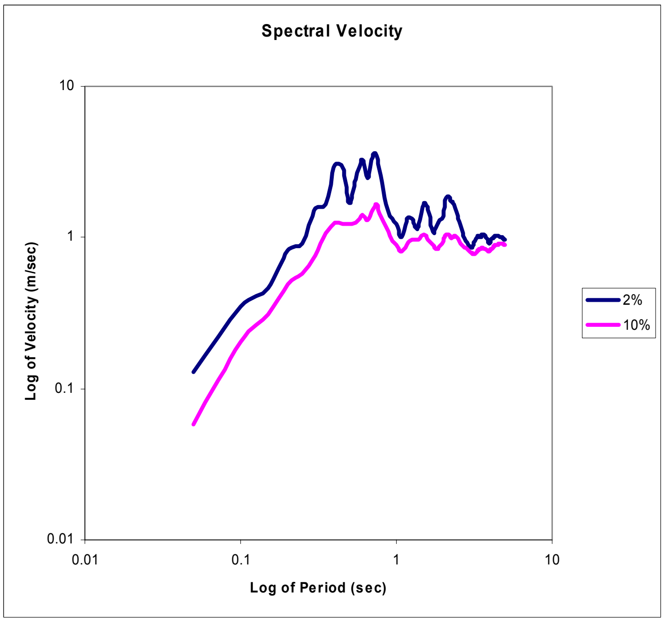

Fig 36. Response spectrum for Northridge earthquake

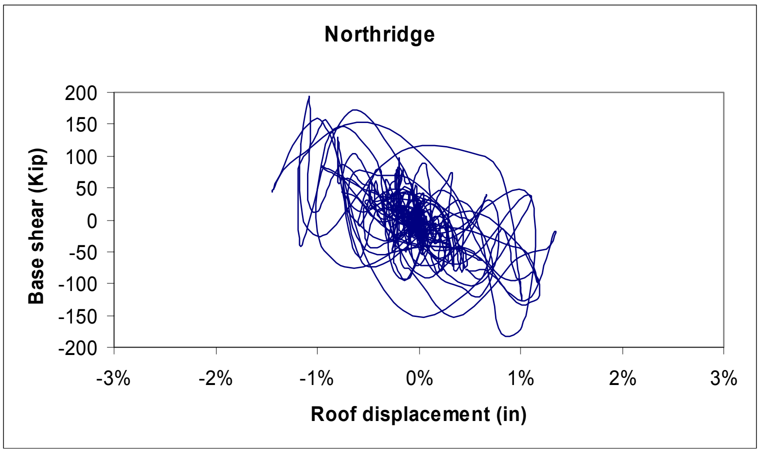

Fig 37. Base Shear-Roof Drift Ratio response

Duzce:

Year	1999
Magnitude	7.1
PGA	0.535 g
PGV	83.50 cm/s
Station	Duzce 270

This Anatolian earthquake hit the same region than Kocaeli just 3 months after that event. The response spectrum in figure 39 shows an earthquake increasing response with period increase. Thus the zone of constant acceleration is quite long. The characteristics of the motion are for a devastating one. The PGA is over 0.5g and the velocity is close to 100 cm/sec. The drift analysis provided a result within the limits of collapse. In figure 38 the base shear has peaks at the same time than drift, but the linearity is uncertain. Both of them are up to the limit levels derived form the static analysis. The base shear reaches its maximum in 849.6 KN (191 Kip) or 39% of the weight; the maximum roof drift ratio is 2.09%. Although the static analysis draws the limit around a 3% drift, looking at the yielding levels induce by this earthquake collapse is considered. Appendix B table B.17 shows all the horizontal members yielded, and the base column with ductility levels of 4.29.

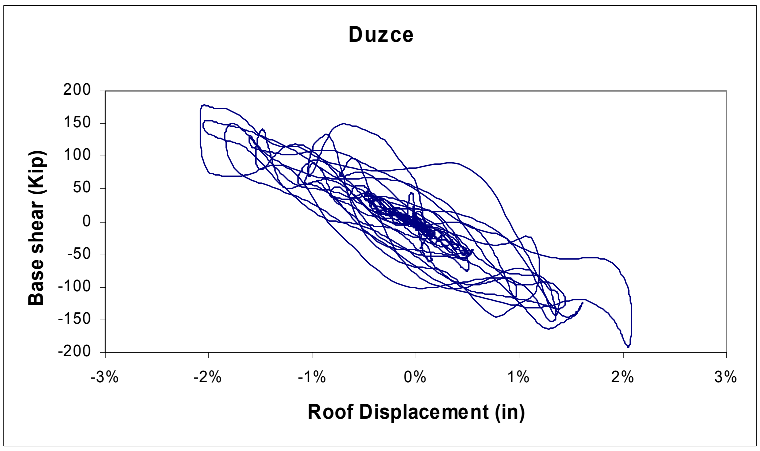

Fig 38. Base Shear-Roof Drift Ratio response

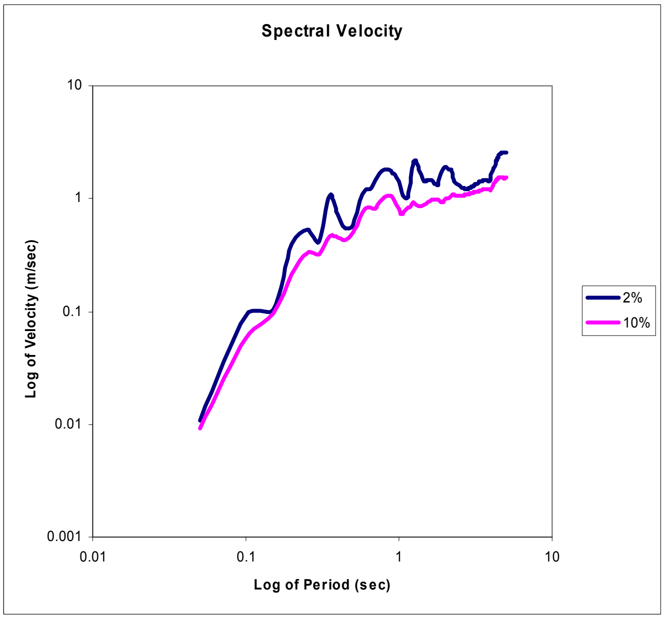

Fig 39. Response spectrum for Duzce earthquake

Those results, as previously explained, cannot be considered real because the software never calculates failure. However with that ductility level given by the program the columns must be at the ultimate capacity. This factor added with all the slabs under yielding can be considered as a collapse. The maximum interstory drift ratios during the earthquake are larger than the maximum mean ratio; all levels go over a 2% drift. The deflected shape for the maximum top displacement Appendix B figure B.59 shows all the stories practically aligned. This is a sign that the structure has reached a structural mechanism and collapsed.

Erzincan:

Year	1992
Magnitude	6.9
PGA	0.515 g
PGV	83.90 cm/s
Station	Erzincan NS

Erzincan completes the series of Anatolian earthquakes selected for this study. In terms of physical parameters is almost the same than Duzce, but the response spectrum (figure 40) is totally different as well as the acceleration record shape. This motion displays a narrow spectrum, highly exciting periods around 2 sec. The acceleration record shows a large peak at the beginning and then decrease rapidly to stay below 0.1g for the rest of the motion. This characteristic is observed at the drift response figure 41 with two large peaks almost three times larger than the rest of the peaks. This behavior is showed at figure 41 where only one line corresponding to those two peaks is plotted outside the linear behavior.

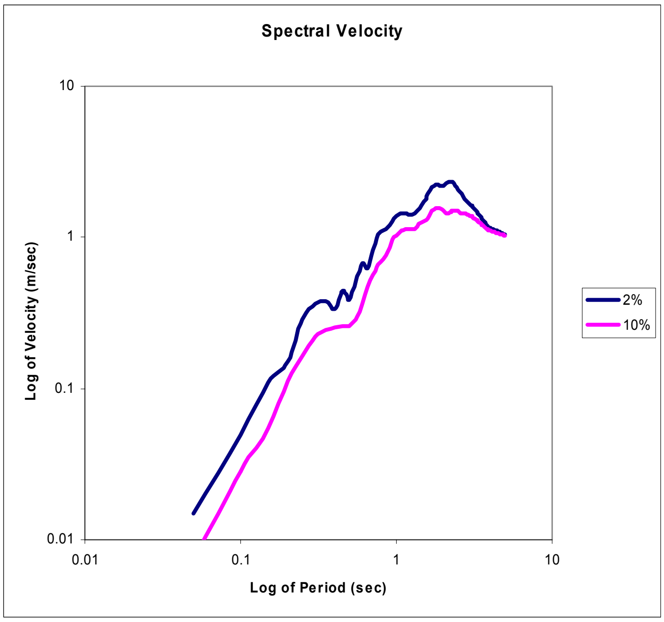

Fig 40. Response spectrum for Erzincan earthquake

Looking at Appendix B table B.18 the stress levels and the ductility indicate sign of collapse. The base columns of the building are completely yield and with ductility limits over the ultimate capacity. All the slabs are yielded in the joints and the rest of the columns have developed around half of their capacity. The deflected shape confirms the collapse option because shows all the levels with similar drift, which means than a probable mechanism has been achieved. Then the structure must be collapse somewhere in the two large peaks at the very beginning of the motion. The reason to have plotted all the response along the whole motion is because LARZ does not understand about failure. The maximum roof drift ratio appears to be 2.44%, and the base shear 894.1 KN (201 Kip) or 41% of the weight.

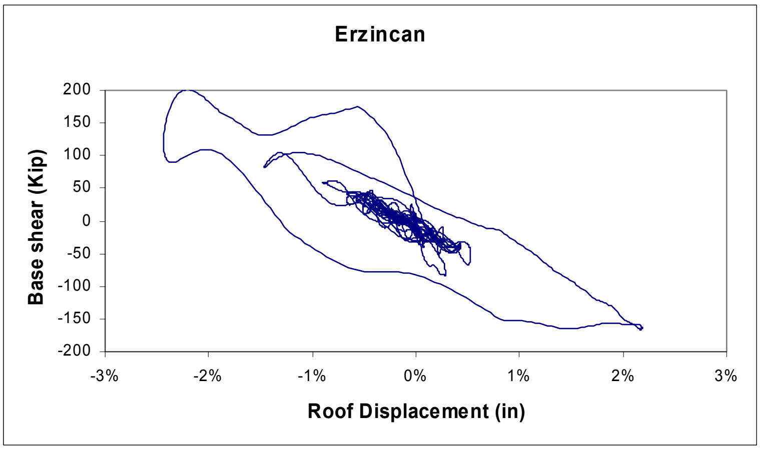

Fig 41. Base Shear-Roof Drift Ratio response

Kobe:

Year	1995
Magnitude	6.9
PGA	0.835 g
PGV	92.04 cm/s
Station	Kobe NS

Well known modern earthquake used by LePage to establish his theory. In figure 42 is the narrow spectrum developed by the motion. The record in Appendix B figure B.17 shows a high demanding earthquake; quite large accelerations sustained for almost a third of the duration. The calculated response to the earthquake reaches 3.8% roof drift ratio. This can be translated directly into a collapse of the building. That large drift peak occurs around 6.2 sec. However the building arrives so far because LARZ have not failed it yet. The first large peak of the response has a 2% roof drift ratio. the next peak at 4.8 sec has a drift ratio of 3.58%. Then the building can be considered collapse by that time so it will never arrive to a bigger displacement. The Base shear reached before the collapse is 889.6 KN (200Kip) or the 40% of the weight. The yielded members of the structure are the base columns and all the slabs, generating a mechanism that can generate the deflected shape shown in Appendix B figure B.65. This study is focused in a drift analysis to establish an upper and lower bound response for the structure. Thus for discussion effects the maximum values of the response will be kept even if failure should have been reached before.

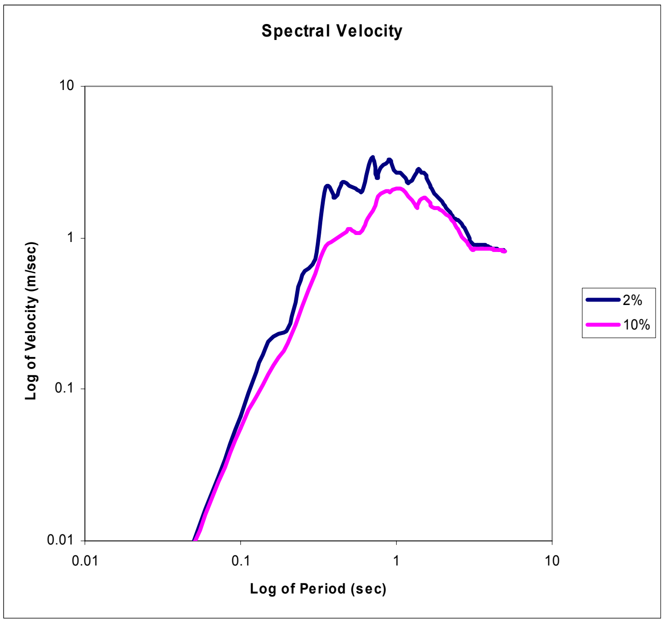

Fig 42. Response spectrum for Kobe earthquake

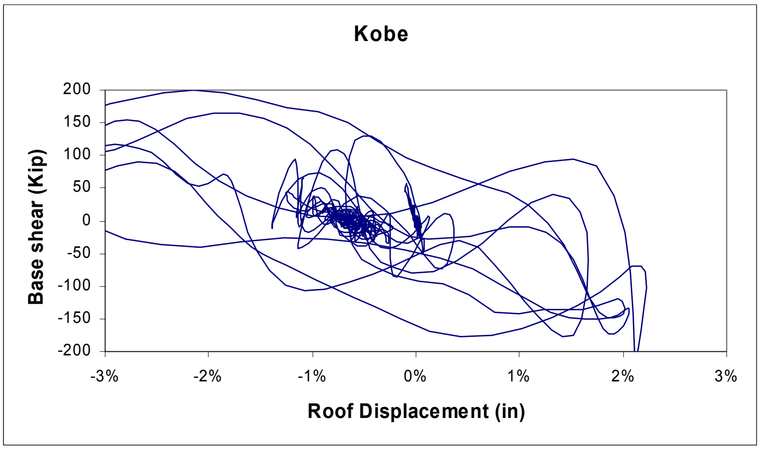

Fig 43. Base Shear-Roof Drift Ratio response

Tabas:

Year	1978
Magnitude	6.9
PGA	0.852 g
PGV	121.40 cm/s
Station	Tabas TR

Tabas in not on the LePage list of Earthquakes, but is a major earthquake with a high velocity and high acceleration that fits well the range of PGA's and PGV's needed for the study. The earthquake is high demanding showing accelerations over 0.2g for the most of the motion, Appendix B figure B.18. The response spectrum in figure 44 is a broad band spectrum severely affecting almost all the common structural periods, form 0.2 sec until 5 sec. The earthquake produces a large response, reaching a maximum drift ratio of 2.27%. This ratio is not excessively large comparing to the static analysis but other parameters indicate collapse of the structure. The yielding level of the building is total; all the slabs are yielded at the joints, also the base columns and in the first floor. In addition the bottoms of the columns of the third level are almost in yielding. Thus the possibility of a structural mechanism is a reality. The interstory drift ratios indicate higher levels than the mean, with a 2.5% at the second level. In figure 45 the base shear lost its initial linear relation with drift, and reaches a maximum of 1178.8 KN (265 Kip) or 54% of the weight. This latter parameter compared with the limit and static analysis can be considered as the definitely indication of failure.

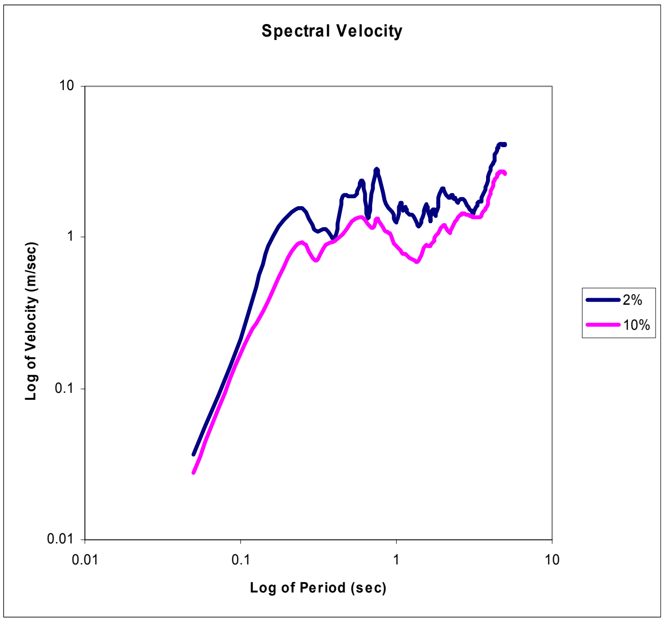

Fig 44. Response spectrum for Tabas earthquake

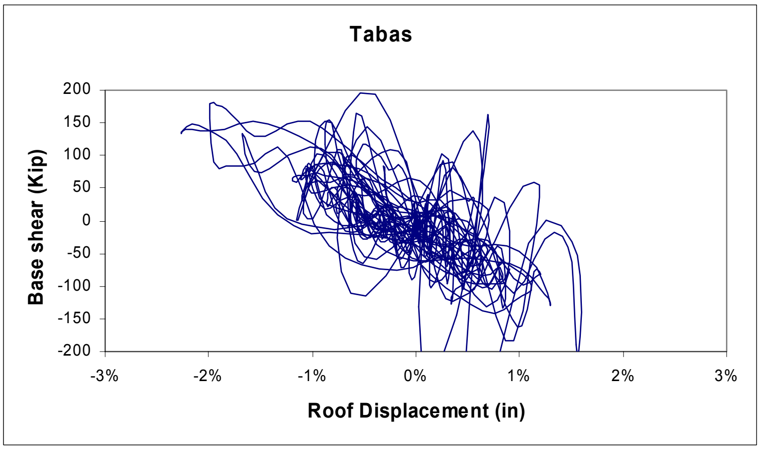

Fig 45. Base Shear-Roof Drift Ratio response

Cape Mendocino:

Year	1992
Magnitude	7.1
PGA	1.497 g
PGV	127.40 cm/s
Station	Cape Mendocino 000

Cape Mendocino does not pertain to Lepage list of Earthquakes. It is the earthquake with larger PGA of all the analyzed in this study. The record main characteristic is an enormous acceleration peak, and then the rest of the motion is in lower levels. Also the motion has a high velocity, the second largest in the list, thus severe damage is expected in the structure. The spectrum in figure 46 is broad band, affecting most of the periods.

Looking at the response of the structure, the building reaches its maximum drift and base shear at the time of the maximum PGA. In figure 47 it can be observed how the system stays most of the motion between ±445 KN ± (100 Kip) and between ±0.5% roof drift ratio. The maximum roof drift is 1.75%, is not excessively large compared to previous earthquakes, although the yielding levels are too high to not consider the collapse of the structure. For that peak, the base columns are absolutely yielded, almost all the slabs are yielded as well, and over 1% drift there is major damage to the building contents. Thus a mechanism can be easily reached. The base shear has a 1059 KN (238 Kip) maximum also during the acceleration peak. As in some of the previous earthquakes the structure presents signs of collapse but in lower levels of drift ratios than in the static analysis. This point will be analyzed in the further discussion of results.

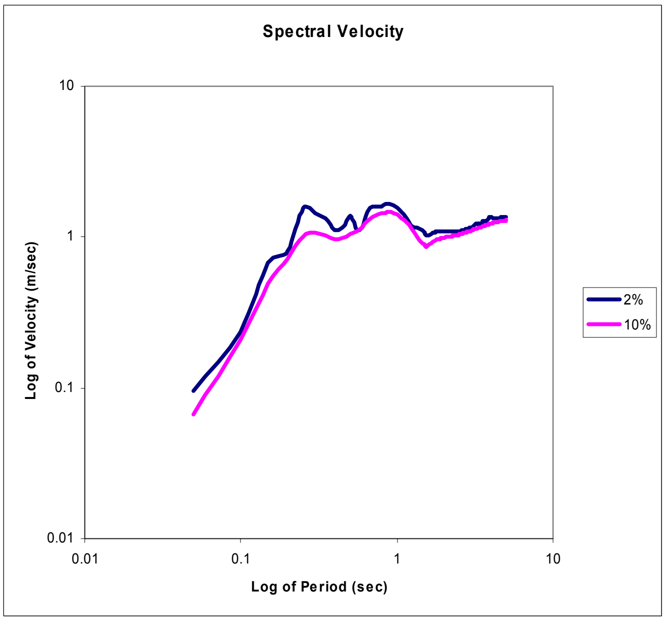

Fig 46. Response spectrum for Cape Mendocino earthquake

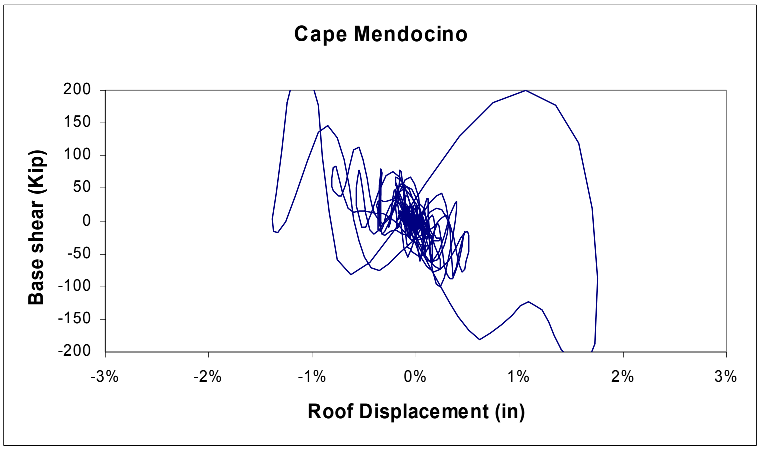

Fig 47. Base Shear-Roof Drift Ratio response

Chi chi:

Year	1999
Magnitude	7.6
PGA	0.462 g
PGV	263.10 cm/s
Station	TCU 068

This earthquake is not on Lepage list again, but was chosen because of its enormous velocity. It is an extraordinary record getting the amount of velocity due to the time lap between peaks in the acceleration record. This is shown in Appendix B figure B.20. The spectrum shows a long range of constant acceleration periods, affecting more to periods over 1 sec.

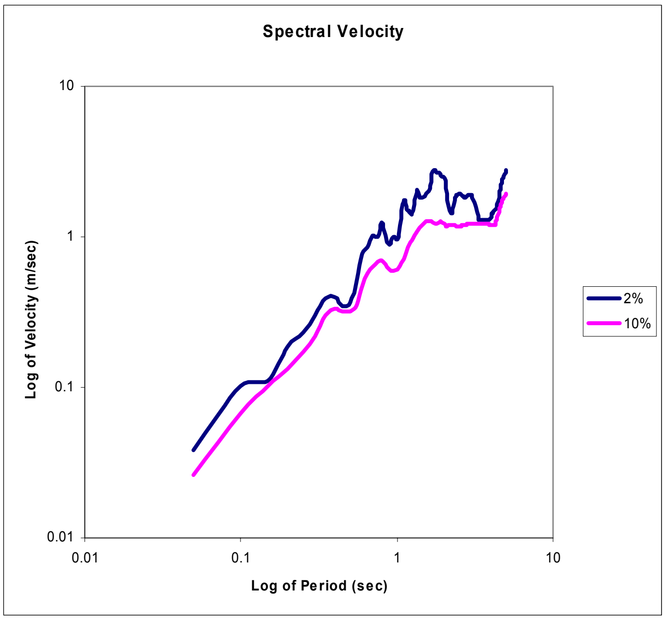

Fig 48. Response spectrum for Chi chi earthquake

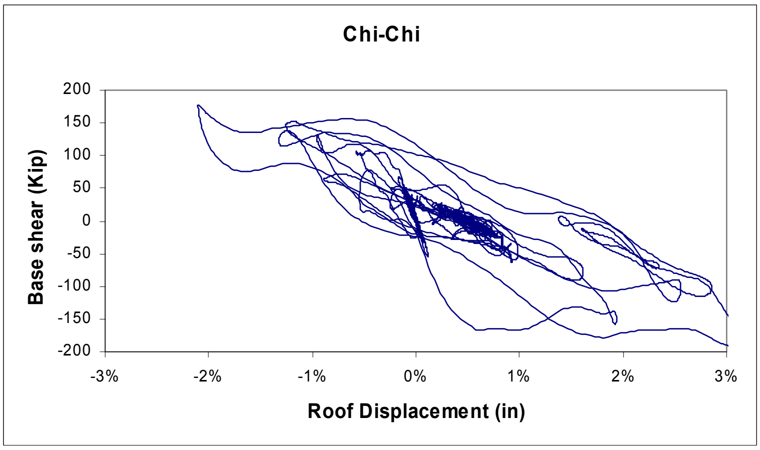

Fig 49. Base Shear-Roof Drift Ratio response

The earthquake causes clearly collapse of the structure under study. This can be appreciated in most of the parameters. The base shear gets a maximum of 863 KN (194 Kip) which is around the maximum for the static analyses. The drift gets over 3% as a peak, and reaches over 2% various times in the response. The structure is completely yielded, the base columns are over their ultimate capacity and the slabs have ductility values over 2. Thus the structure response accomplishes the expectation for the PGV of the motion.