Chapter II. REVIEW OF PREVIOUS RESEARCH

II.1. Introduction:

The purpose of this chapter is to review first the basic concepts for earthquake engineering. It is not intended to review the basic concepts about earthquakes properties but to review basic concepts of the response of structures under seismic loads. The chapter is focused in the different parameters that govern the structure and in the different ways to model and predict the structure developed by earlier authors.

Once the basics have been reviewed, this chapter will look for similar researches developed by different investigators around the United States. There is actually a wide spectrum of different researches concerning to lateral load and earthquake design on reinforced concrete buildings. This review is focused only in those experiments or analysis fulfilled comparable to the structure object of this dissertation. Therefore can be use to extrapolate hypothesis to the structure here analyzed. Also the aspects from the code that refer to lateral load flat plates, or just simply flat plates will be evaluated. Finally it will be an evaluation point; for this object the present structure will be presented in its original configuration trying to compare it with previous research and get some useful hypothesis.

II.2. Basic concepts:

This point intends to acquaint the reader with some of the basic concepts of the earthquake engineer:

- Dynamic analysis: the structure response to a seismic excitation or ground motion is a vibration. In this manner the structure dissipates the energy transmitted by the earthquake. The structure is a system with foundations, structural and non structural elements. This system has a complicate response under ground motion; therefore how to model it to perform the analysis is basic. For a dynamic analysis the way to model the structure is to transform it into a system of masses connected one to each other through springs and dampers. All the structures can be modeled as a single degree of freedom (SDOF): one mass connected to the ground by one spring and one damper. This model gives a simple system easy to understand. However is difficult to believe that is a representative model of the real system. Usually the buildings are not simple structures then they are tried to be modeled as multi degrees of freedom (MDOF): a series of masses connected between each other by springs and dampers. Once the system is defined, if the structure is subjected to a motion in its base, three forces appear: inertial forces, spring forces and damping forces. Then a dynamic equilibrium equation is written in terms of a differential equation. From it is obtained the nominal period of the structure. Although calculation will be seen in detail in chapter V it is remarkable than the period found is the larger period of vibration of the structure. This value is important to determine the flexibility of the system as well as to perform a spectrum analysis.

- Drift analysis: The solution for the dynamic equilibrium equation shows the movement of the different degrees of freedom (DOF) of the system. These movements allow computing the drift of each DOF or story. Consequently a drift response analysis of the structure can be done. Drift is the main control parameter to know how much an earthquake has affected the structure. The drift parameter is presented in different ways in this dissertation: mean drift ratio or roof drift ratio, it is the ratio in percentage between the lateral displacement at the top of the building and its height. Another parameter is the interstory drift ratio which is the ratio in percentage of the relative displacement between floors and the floor height. The boundaries for drift control are considered around 3%, which will be the mean ratio of collapse for a normal reinforced concrete building. Around a 1% of mean drift ratio or interstory drift ratio the contents of the building have suffered a severe damage, most probably due to the collapse of masonry walls.

- Demand analysis: For engineers interested in measuring the intensity or the demand of an earthquake, it has been difficult to identify a parameter as the real measure of its demand. From an earthquake record the most important parameters are the velocity and the acceleration. These two parameters come in an irregular wave form recorded at the seismic stations. From these records the most important parameters are the peak ground acceleration (PGA) and the peak ground velocity (PGV). Nowadays the PGV is considered the parameter to measure the demand of an earthquake; the reasons to do it come from a paper written by Westergaard in 1933 [16] and clearly explained by Sozen [12] in 2003. Therefore in this dissertation the earthquakes will be classified basically under PGV considerations. Also in 1997 Andres LePage [6] wrote a paper relating PGA and PGV to the spectral displacement of a structure. LePage established an upper boundary for spectral displacement linearly related to the period of the structure. His work has been an important reference to select the seismic motions in this dissertation and it is explained further in this chapter.

II.3. Experimental research

Jack P. Moehle nowadays at the University of California, Berkeley has been one of the most interested researchers on the behavior of flat plate structures subjected to lateral loading. Also at University of Washington a project sponsored by the National Science Foundation was developed along the 1970's mainly by Neil M. Hawkins. This project was an investigation on the seismic resistance of concrete slab to column and wall connections. Then this chapter is mainly focused on those works.

II.3.1. Scale models

In 1984 Jack P. Moehle et al. [9] tested a flat-plate frame under lateral load. The structure was subjected to simulated earthquakes motions in a shake table. The prototype structure used was really similar to the one is going to be tested in Purdue University as it is explained in section II.6.

It was a two story building with 6.10 m (20 ft) spans in both directions, 3.05 m (10 ft) height between levels. It was supported in 457 mm (18 in) square columns and the slab thickness was 203 mm (8 in). A spandrel bean 457 mm (18 in) wide and 356 mm (14 in) depth was along the perimeter of the slabs. Characteristics of concrete and steel were: 27.6 MPa (4000 psi) for the concrete and 414 MPa (60 ksi) for the steel. The structure was designed following the ACI 318-83 code to make it realistic. Then all the proportions and materials were reduced to a 0.3 scale model.

The system was subjected from low till high-intense earthquake simulations over a shake table. In these tests was observed that the overall yielding of the structure did not start until the drift ratio was 1.5%. The system was found very flexible, and it exceeded 5% of drift ratio without collapse. The calculated period for the structure before the test was 0.21 sec. This was calculated with a low-amplitude free vibration test, and it matches almost perfectly with the old approximation from the Uniform Building Code [14]. This period increased during the earthquake simulation test, and afterwards the free-vibration period was increased up to 0.24 sec. Lateral stiffness calculated with the equivalent frame and the equivalent width method were larger than the stiffness measured in the experiment. Therefore, it was recommended to use a cracked inertia of the elements equal to one third of the uncracked inertia. Also the initial measured lateral stiffness was less than the stiffness calculated by elastic methods, indicating that the cracking due to non-working loads was significant. Concerning to the base shear, the distribution of inertial forces during the base shear peaks was uniform with the height, so was recommended to assume this pattern in order to calculate an upper bound for the building strength. Even the limitation inherent to the method, and some inaccuracies a limit analysis was able to predict the 80% of the system strength.

In 1999 Jack P. Moehle and Shyh-Jiann Hwang [4] tested a nine-panel flat-plate frame, performing vertical and lateral loading. The characteristics of the model were also similar to those of this dissertation; the prototype structure was an intermediate level of a multistory building. The spans were 4.70 m (15.5 ft) and 6.90 m (22.5 ft). The system had 3 spans in each direction. The story height was 3.05 m (10 ft) and the slab thickness was 203 mm (8 in), and they used a 0.4 scale prototype to test it on the lab. All the design of the testing element followed the ACI 318-83 code. Gravity moments were design following the direct design method, and the lateral moments were designed following an effective beam approximation.

They conclude that the stiffness of the system was overestimated by methods, the equivalent frame and the equivalent beam width. The resistance at reasonable drift, like 1%, was less than the ACI design strength. Also the ACI methods for moment distribution did not reproduce the moment fields. Finally the structure reached punching failure at large drifts, but did not collapse because the continuity of the positive bars at the columns.

II.3.2. Slab column connections:

During the 1970's an investigation was developed at the University of Washington into the Seismic resistance of concrete slab to column connections and wall connections. The investigation was sponsored by the National Science Foundation-Research Applied to National Needs. This research was divided into several reports each of them covering different periods of the study. In the following pages only the relevant reports for the current dissertation are analyzed.

In 1974 the first part of this project was summarized by N. M. Hawkins et al. [2]. It was a test of 6 slab-column specimens transferring moment and shear. The main objective was to establish the effect of severe cycles of moment transfer reversals on ductility, strength and energy absorption and dissipation. Also a secondary objective was to establish the effect of the geometric form for the slab and the loading history on strength and ductility. The prototype structure was a flat-plate with 6.10 m (20 ft) spans and 2.74 m (9 ft) height between stories and 152 mm (6 in) as slab thickness. The elements tested were approximately full scale of it. The specimens were intended to simulate the connection extending the slab and columns until the point of contra flexure. In the specimen to test, the slab was 3.96 m (13 ft) long and 2.13 m (7 ft) wide. The columns were 305 mm (12 in) square and extended 1.07 m (3.5 ft) above and below the slab. The reinforcement for the column was four bars 25 mm (1 in) diameter. The reinforcement for the slab had three different ratios, 1.29% 0.90% and 0.57% for the top mat. The bottom reinforcement was approximately half of the top layers for all the specimens. The elements were subjected under a preselected loading history. It was alternative loading cycles with different magnitude for the peak moment transferred to the column.

The principal conclusions for this study were: the magnitude of the lateral load yielding the bars that go through the column is the critical load. If more cycles are done at this load level, the connection fails in a deformation about 50% higher than at yielding. Intense high reverse loads can result in the connection having 20% less capacity than those loaded monotonically. For flat-plate structures designed following ACI 318-71 the column stiffness has little effect on the lateral load response, the main factors are the characteristics of the slab column connections. Rotations at the slab-column connection give form 60% to 80% of the energy absorption of the specimen.

In 1976 N. M. Hawkins et al. [3], tested five reinforced concrete slab-column specimens continuing the investigation initiated at University of Washington. The elements were subjected to high intensity shears and transferring reversed cycling moments. The objective of this research was to see the effects of this loading on the strength, ductility, energy absorption and dissipation of the connections. The main variables were the flexural reinforcement in the slab and the provision of integral beam shear reinforcement.

The prototype structure was again the same than in the previous, a 6.10 m (20 ft) spans flat-plate, 2.74 m (9 ft) height between stories and 152 mm (6 in) slab. The specimens were approximately full scale of it only representing the connection and the necessary length of slab and column to represent until the contra flexural point. So the slab was 3.96 m (13 ft) long 2.13 m (7 ft) wide and columns 305 mm (12 in) square and extended 1.07 m (3.5 ft) above and below the slab. The column reinforcement was four bars 25 mm (1 in) diameter. The reinforcement for the slab was two-ways and had three different configurations, 0.57% 0.9% and 1.15% of reinforcement at the top mat. For the bottom mat, the reinforcement ratios were 0.58% 0.50% and 0.28%. The specimens had various shear reinforcement configurations; some of them did not have any shear reinforcement, and others had different size and extend of the closed stirrup integral beam shear reinforcement extending out of the column. The gravity load was tried to keep constant, based on a live load of 391 kg/m^2 (80 psf) and the self weight of the structure. Although at the last specimen this amount was smaller to account the lesser reinforcement in the slab.

The conclusions of the research were various, here only main ones are exposed; Considerable ductility can be achieved with adequate detailed stirrup reinforcement. The connections with those details can develop significantly higher rotations. Damping ratios increase as flexural reinforcement in the slab decreases. The deflections at the edge were due in 10% by the column, 25% by the slab, and 65% due to concentrated rotation at the slab-column connection. Although is not on the conclusions on the paper, having a look at the results a trend between flexural capacity of the slab and reinforcement ratio can be appreciated. The relation between measured resistance and the calculated ultimate resistance is shown in this table

Table 1. Hawkins test results

Reinforcement ratio at top of slab	1.15%	0.90%	0.57%
$M_{measured} / M_{calculated}$	0.83	0.88	1.11

Looking at these results it can be observed that the slab with less reinforcement is more efficient.

In 1981 Denby G. Morrison [10] tested 8 slab-column connection specimens. The elements were loaded with lateral loads. Five of them were tested statically and three dynamically. The object of the experiment was to analyze the strength and stiffness behavior of the connection, also the energy-dissipation behavior under lateral forces.

The prototype structure was a normal high flat-plate building. The models were approximately a third of a scale of the connections at the lower level. The dimensions were 1.83 m (6 ft) plate, 76 mm (3 in) slab thickness and 305 mm (12 in) square column. That makes the prototype building significantly different than the one is under research here. In that case, it was approximately a flat-plate structure with 5.49 m (18 ft) spans in both directions, 229 mm (9 in) slab thickness, roughly height of 3.35 m (11 ft) between floors, but the main difference was that the columns that were a 914 mm (36 in) square. This made the column stiffness big enough so the column contribution to the rotation of the joint was negligible. In this way the slab was the only member contributing flexibility to the connection, so all the results will be due because of the slab behavior at the connection.

Different amounts of reinforcement in the slab were tested. Also for the static test three different vertical loads were tested intended to observe the behavior of the connection. It was three kinds of reinforcement, 0.65 %, 0.98% and 1.31%. The reinforcement was uniformly distributed along the width of the slab and both bottom and top were equal.

The dynamical test was performed with three different motions, a random motion simulating the north component of El Centro earthquake, and also steady state tests were developed having sinusoidal motions in different frequencies. The static tests were done by cyclic increases of rotation of the connection. After the last cycle was done, the elements were loaded till failure. The dynamic tests developed more capacity than the static ones, but this increase in strength was attributed to higher levels of strain. Although, this behavior can not be transferred to full scale buildings were the fundamental frequency of the connections would be lower.

The main and more useful conclusion is that: with the increase of reinforcement was observed that the efficiency of the structure was lower. In other manner, comparing the ratio between moment developed and moment capacity of the slab, the higher was the reinforcement the lower was the ratio. The elements with less reinforcement developed an equivalent width of 100% of the slab, and the higher reinforced elements only developed a 60%. In terms of absolute resistance higher reinforcement gives higher strength, but if the objective is too take maximum profit of the structure capacities is better to reduce the reinforcement ratio.

In 1992 Jack P. Moehle and Austin D. Pan wrote “An experimental study of Slab-Column connections” [11]. Four slab-column sub-assemblages at 60% percent scale of the prototype building were tested. The prototype was a flat plate structure with 6.10 m (20 ft) spans, 3.05 m (10 ft) story height, 457 mm (18 in) square columns and a 203 mm (8 in) slab. The geometric proportions of the prototype building were the same than the model tested in 1984 and almost like the one is on the present research. The reinforcement details for the slab were: maximum of 0.76% at the negative, concentrated at the column strip. Uniform 0.25% was used for the entire positive. The reinforcement was equal in both ways of the slab and the bars were all 10 mm (0.375 in).

The study had the objective to find the consequences of biaxial loading, behavior for different vertical loads, as well as the post failure behavior of the connection. Two specimens were tested under biaxial load and other two under uniaxial load, in both cases different gravity loads were applied. The load was applied once at a time in the biaxial test, with different increments of load, changing direction in each addition. Models for calculating the strength of the structure were compared with the experimental data. These models were the eccentric shear stress, the equivalent frame method and the equivalent beam width.

It was mainly concluded that, the biaxial loading of the structure reduces the overall response of the system, less strength, less drift capacity and less stiffness. The magnitude of the gravity load is a main variable in the behavior, increasing of the vertical load affects the lateral behavior. The continuous bottom reinforcement evicted the progressive collapse of the structure. The eccentric shear stress model was conservative to predict the moment transfer, but both equivalent frame and equivalent width overestimated the capacity, so was recommended to reduce one third the stiffness of the slab to account with cracking. This reduction results in a conservative approximation of the structure stiffness compared to the experimental data.

II.4. Analytical studies

In 1974 Hawkins N. [3] at the same time than the experimental investigation initiated at University of Washington, an analytical study of flat-plates was developed. The overall objective of this analytical phase was to develop a rapid and realistic method to analyze flat-plate structures laterally loaded in the inelastic range. A preliminary study shown that an analytical solution based on the differential equation for plates was too complicate, then it was seen that the equivalent frame method from ACI 318-71 was reasonably accurate and simply enough to extend to nonlinear range of the structure behavior. Equivalent frame method was developed by Corley only for gravity load use, however ACI permits its use for gravity and lateral loads. In this report the method was checked for various flat-plates structure and was found that overestimates the transferred moment from the slab to the column due to the overestimated elasticity of the column. Then the equivalent frame method was redeveloped with the complementary energy method. This improvement treats the slab at its central part bending in one-way, and twisting at the rest. The outline was to derive more consistent method for both Corley and ACI approximation, and then derive an equivalent frame treated as a discretisation of a continuous problem based on an energy method. Following that, compare the derived method with the solution of a finite element analysis, which is considered a more exact solution. After the research it was concluded that the energy method gives a solid theoretical base to develop an equivalent frame method for analysis of flat-plate structures. Although the energy method was accurate for the tested structures, it was recommended that a further study be done to confirm the accuracy of its parameters.

In 2000 Jack P. Moehle and Shyh-Jiann Hwang [5] compared the two most important models for laterally loaded slab-column frames. One is the effective beam which represents the slab action as a slab-beam framing directly on the columns. The second is the equivalent frame method which models the slab action as a combination of flexural and torsional actions. Both models were compared with experimental results from a lateral test on a multi-panel slab scaled at 40%.

The effective beam width method tries to represent the part of the slab that is working as a flexural member. When the connection is subjected to rotation, the slab nears the column rotates with it, but the portion of slab away of the column has less rotation. The width coefficient is trying to not consider this portion that does not rotate equally with the column. A wide variety of previous analytical studies were explained at the report. The variety of results came with the two treatments of the joint flexibility. One way was to consider the slab rigid within the plan of the column, and the other one considered the joint equally flexible as the surrounding slab. The equivalent frame method is the described by the ACI code; the model interconnects the column with the slab-beam with torsion members. The torsion transverse member flexibility is directly proportional to the transverse length of the slab. Then the effect on the lateral stiffness by varying the transverse length of the span is notorious. The research consist in elaborate a finite element method to compare the results with both members. The effects on these models of slab aspect ratio was studied, and the effect of cracking on connection stiffness. For the finite element method was observed that the lateral load stiffness was almost independent of the transverse span length. Hence the equivalent beam width represents better the behavior because of its independence on the slab aspect ratio. When the effects of cracking are included, both methods match better with the response measured from the finite element solution.

It was concluded that both methods were good to predict the elastic behavior of the structure under lateral loads. Even though the equivalent width method trends to calculated higher stiffness than real for irregular frames, and the equivalent frame method trends to calculate them lower. No method matched the real elastic stiffness, so it was propose to reduce one third the stiffness of the beam (in the equivalent width) and the stiffness in the torsional member (in the equivalent frame) to find a reasonable lower bound of the true elastic stiffness.

In 1997 Lepage Andres [6] presented his thesis on drift estimates for reinforced concrete structures under seismic loads. His work is also summarized in a paper for the Eleventh World Conference on Earthquake Engineering in Acapulco Mexico. He proposed a method to calculate seismic drift, using an idealized linear response spectrum modified by a factor that accounts for nonlinear action. The evaluation of the method was done with results from nonlinear dynamic analysis for various hysteresis models. Also was evaluated with earthquake simulation test in the University of Illinois. To develop his theory Lepage starts with the approximation that the spectral values for acceleration velocity and displacement are related like:

$$S_a = \omega^2 \cdot S_d \quad (1)$$

$$S_v = \omega \cdot S_d \quad (2)$$

Assumed a system with a 2% damping ratio, and derived the spectral acceleration using the peak ground acceleration and an amplification factor. For that distinguished between structures with period above or below the characteristic period for ground motion. Using the relation in (1) and (2) derived idealized displacement response spectra. This spectra is defined in equation (3)

$$S_d = K \cdot T \cdot \sqrt{2} \quad (3)$$

This formula is his main conclusion, where K is a constant with length/time dimension, determined from displacement spectra for damping factor 2%. It is must fit the place, for stiff soil and ground motions with effective peak ground accelerations of 0.5 g, was proposed K=250 mm/sec (10 in/sec). T is the period for the structure, and it is factorized with square root of two for considering the effects of cracking in the structure.

Lepage compared the approximation to actual results for linear systems subjected to El Centro 1940 NS Imperial Valley earthquake. His approximation was an adequate representation of the response spectra generally on the higher side.

II.5. Code

This point is focused on the treatment of flat-plates structures by the main structural concrete code at the United States. The ACI318-02 Building Code Requirements for Structural Concrete [1] is a code that covers the proper design for concrete structures. It states the necessary requirements to provide public health and safety. Is written in a manner that it can be taken as a reference in building construction but has not legally status unless the government body declares it. For the case concerning right now, the ACI code considers the structure formed by columns and two-way slabs. For structures laterally loaded ACI code recommends the use of approaches like the effective beam width, the equivalent frame method, or plate-bending finite element models. Also the specifications for design for seismic resistance are covered in the chapter 21. But this research is trying to evaluate the response of a structure design only for gravity loads. Then the structural and design necessities are mainly comprised in other two chapters. Chapter 13 applies for the design of slab system with flexural reinforcement in more than one direction, independently of having beams or not. There are two design methods, the direct design which is based on the distribution of a factored static moment over the slab, and the equivalent frame method, based on the stiffness of the slab, column and a defined torsional member. In both of them there is no special treatment for flat-plates, is just that had to be applied for slab without beams between interior supports and no edge beams. Chapter 10 apply to members subjected to flexure and/or axial loads; in a flat-plate structure the columns are subjected to axial load, and flexure due to transferred moments from the slab.

Summarizing, ACI code does not have any special section for flat-plates, treats them as two way slabs. In Chapter III of this dissertation the structure is designed according to the mentioned ACI chapters and also to accomplish with spacing and cover requirements from ACI chapter 7.

II.6. Evaluation

In this point the reviewed studies are going to be evaluated in a manner that they are useful for the current research. For each study some expectations are extrapolated to the present structure, some of them serve to orientate the analytical research, and some of them will be useful to compare results with.

From the studies with scale models: the limit analysis was able to predict the 80% of the system strength that means that it can be applied now as a useful tool to approximate the behavior of the structure. Also it was said that the uniform load pattern fits better the base shear peak inertial forces of a ground motion. Then both linear and uniform distributions are going to be calculated in order to get two boundaries for the limit analysis. The test of 1984 by Moehle is the most similar to the one is being performed now in dimensions. But two main differences can make the results to diverge; the one by Moehle was a 30% scale model. It was also excited with earthquakes simulations when this one is going to be tested statically at the lab. In any case it will be interesting to compare results like the yielding ratios between the experimental dynamic analysis and the theoretical dynamic analysis developed in chapter IV.2.

From the studies with column-slab connection specimens:

Morrison dissertation brings out a point really interesting, the slab works more efficiently when is less reinforced. Then it can be applied for the present study because the level of reinforcement is even less than in Morrison test. As it is designed in Chapter III the slab has a 0.57% as a maximum reinforcement ratio, and for Morrison test the slab that developed a 100% of its capacity had a 0.65% of reinforcement. From Moehle in 1992 some hypothesis can be extrapolated. Biaxial load considerably reduces the response of the structure; for this study only the uniaxial load is considered, then a weaker response has to be expected for a real earthquake with both components. The gravity load modifies the response of the system under lateral loads; then the gravity load for the test has to be carefully design in order to represent the reality of the structure. With a good approximation of the real gravity load, the test will be more representative. From the studies at University of Washington, the whole study program was focused more on shear reinforcement at the connections and that is not the present case of study. Although they record a lot of data and experiments on connections without shear reinforcement. Looking at their results the same observation than Morrison can be conclude, the slab develops its full capacity with lesser amount of reinforcement. The deflections come in its major part form the concentrated rotation accumulated at the connections. The column stiffness has little effect on the lateral response. And finally the slab-column connection is the main contributor to energy absorption on the system.

From the analytical review; those studies have been focused primarily on the discussion of how to model the stiffness of the structure. This has been about the two principal models for stiffness, the equivalent beam width and the equivalent frame method. Moehle compared these models with test results and with finite element calculations, that they both overestimate the stiffness, and that the inertia of the elements have to be reviewed in order to account with crack effects. Other kind of research have been oriented in the convenience of one method over the other, but always observing that both are not taking into account the effects of cracked sections. Finally in the investigation developed at University of Washington was focused to improve the equivalent frame method with an energy based method. They improved for the specific structures that were tested there, but a suggestion for further investigation was done in order to ascertain the parameters that control their new method. The study is going to be used more in this dissertation is Lepage approximation of displacement response spectra. In his study he used 11 ground motions with a broad band of PGA and PGV's, consequently at least these earthquakes are going to be used in this study, then his approximation can be compared to the results obtained with LARZ.