Chapter III. DESIGN OF THE TEST STRUCTURE

III.1. Introduction

The process followed to design the structure has linked the desired strength of the system in order to test it at the lab, and the ACI building code requirements for structural concrete. The initial draft of the structure was done looking at the characteristics of flat-plate buildings constructed in the mid 20^{th} century. This initial draft was reviewed to accomplish roughly the ACI. After this, a limit analysis was performed to calculate two parameters, the base shear coefficient of the structure and the failure mode under lateral loads. As is shown in forward sections the structure is subjected to multiple changes, principally in column geometry and in slab reinforcement. After reaching the desired mode of failure with the limit analysis, the structure is checked to accomplish with the ACI code requirements. Once the design is checked with the ACI code, in another point the final structure configuration is explained.

III.2. Limit analysis

There are two different objectives in this limit analysis. The first one has been to design the appropriate structure by doing a simple analysis. The structure is going to be subjected to a test; it is not desired then, that the structure fails in a manner that does not contribute with interesting data. Thus by using limit analysis reasonable expected response of the lateral strength of the building has been guaranteed. Since it was not good from the starting draft, a sequence of try and error has been helpful to design the outlines for the reinforcement and the geometric configuration of the structure. Once the design is achieved, the second objective is to get a lower and upper bound for the base shear of the structure. With this result it is possible to have an approximation of the loading capacity that is going to be needed at the lab. Also this boundary base shear will be used to compare the analysis performed in Chapter IV.

III.2.1. Method

The limit analysis method is based on the same principles as the yielding lines analysis for reinforced concrete plates. In this case, the structure is assumed to be two-dimensional, and the elements are considered only in their length to reduce the specimen to a wire frame. Plastic hinges are applied to the joints of the system to generate a mechanism; these plastic hinges are the same element as the yielding lines used in limit analysis for plates. On the structure lateral forces are arbitrary distributed over the stories. Then applying a virtual movement is possible to calculate the external work generated by those lateral forces. This virtual movement also generates a virtual rotation; with this rotation at each hinge and the plastic capacity of each member, the internal work of the system is calculated. The principle of energy conservation establishes that the external work has to be equal to the internal work. With this equilibrium the lateral forces applied to the structure can be computed. Once these magnitudes are achieved, the base shear coefficient is calculated as the sum of all the lateral forces. Then one parameter has to be introduced; the base shear coefficient. It is a parameter that expresses the ratio between the base shear and the total tributary weight of the structure.

The lateral forces applied to the structure to find the base shear must be representative of the force distribution during an earthquake. The most common distribution is to divide the forces linearly with the height and weight. In this manner for the present structure will be an inverted triangular pattern. Also previous research by Moehle [9] suggested that the more representative will be a uniform distribution over the height of the building. Moehle also concluded that the uniform distribution is the upper bound of the base shear. Thus in this dissertation both approximations will be used, in order to get two boundaries (upper and lower) for the lateral strength of the structure.

To make the calculation more accurate some corrections are introduced to the wire frame. The length of the slab members is reduced to take care of the differential rotation induced by the plastic hinges. This differential rotation is due to their generation at the face of the columns instead of at the center of the joints. Also the clear length of the column is corrected because the slab thickness. The plastic hinges can be located in slab and column members, it is necessary to study all kind of possible mechanism to find out which one occurs at lower loading level. The one which happens at lower level will establish the minimum base shear of the building. The four possible structural mechanisms for this structure are shown in Fig 2.

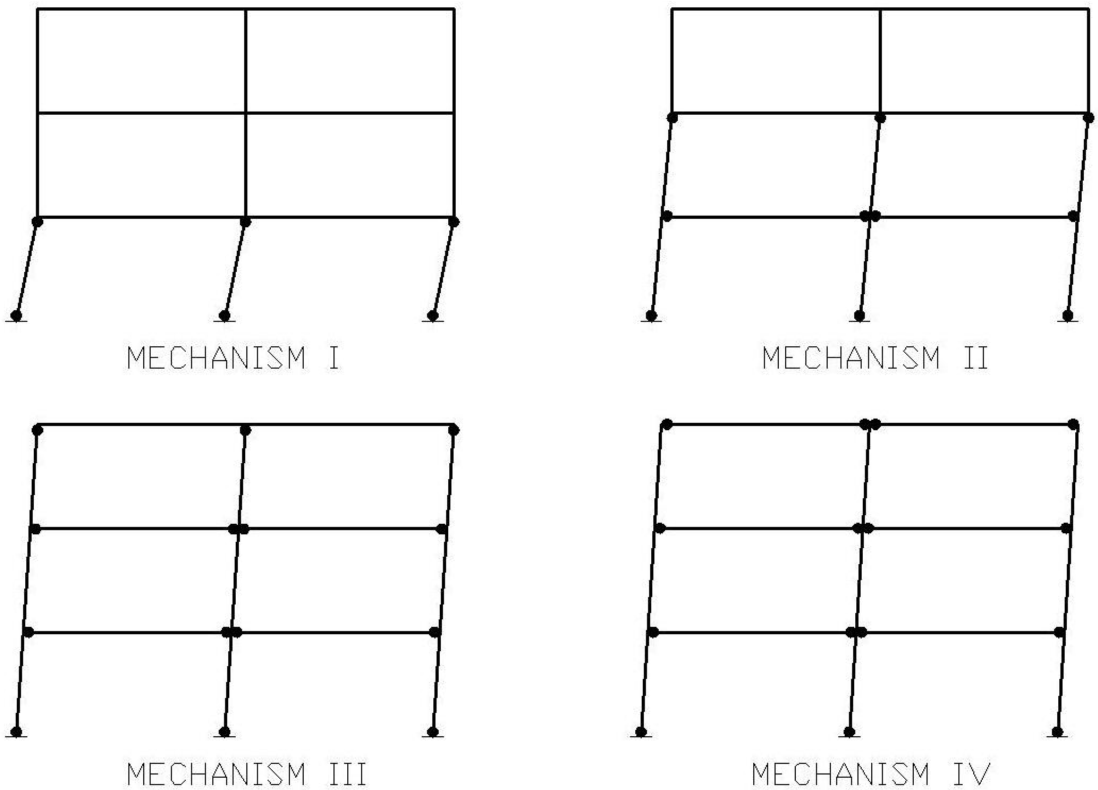


Fig. 2 Four mechanisms considered in limit analysis

Looking at the different possible mechanism for this system it is necessary to reach mechanism number IV or at least III in order to achieve a structural failure. It is vital to avoid the first two mechanisms. If a column failure is achieved (mechanism I and II) the test will be useless. The potential of the structure will not be developed and only the column capacity of the first level will be tested. Thus in the next point when the limit analysis is performed the objective is to find mechanism IV as mode of failure.

III.2.2. Structure model

To analyze the structure it has to be modeled like a planar unidimensional frame. For that the columns and slabs have to be converted into horizontal and vertical members with a determined flexural capacity. In order to be safe in the calculations, it has been considered yielding along the whole width of the slab. There are two reasons to consider this option. First of all, in this way the worst case scenario is considered for resistance of the slab members. If the slab is considered as strong as possible, in case is not Mechanism IV will be even easier to get. As a second reason, previous research by Morrison [10] has shown how the effective width of the slab changes with the amount of reinforcement at the joints. The prototype is trying to model a structure built in the 50's so small amounts of reinforcement at the slab are expected. Thereof is reasonable to consider that the slab will be yielding in its full width.

The structure has been modeled as two frames of columns and beams. The columns are the same as the real columns. For the beams, the slab is divided into two beams in each span. The structure will be then two identical frames. Thus it will be a three story structure with nine columns per frame, two spans, and four beams per floor.

The reinforcement detail of the slab has to follow the ACI regulations. In terms of ACI the slab is divided in different column and middle strips. Fig 3 shows the slab division into strips and the slab division in beams for the later model.

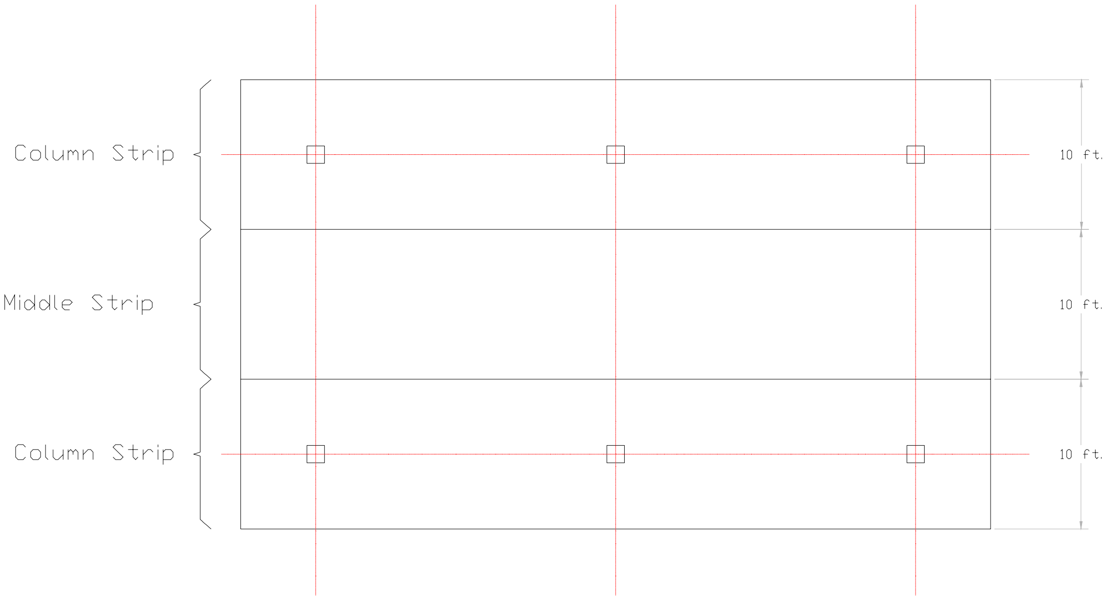


Figure 3. Column and middle strips

For the slabs, ACI establishes that the positive reinforcement of the slab must be continuous at the joints to prevent progressive collapse. However for calculations only the column strip bars have to be considered as the ACI specifies. For the negative reinforcement, all the bars are continuous in the joint, then both middle and column strips are considered for capacity. Consequently the horizontal members of the frames are the slabs treated as beams, the positive reinforcement for these beams are the column strip bars. The negative reinforcement for the beams is the negative column strip of the slab and half negative middle strip. For the columns capacity all the reinforcement is considered.

Yielding capacities of the members had been calculated with an excel sheet developed by Santiago Pujol at Purdue University 2000. This excel sheet is shown at the Appendix A section A.1 and contains a moment curvature diagram. The inputs are the member characteristics, material properties and the concrete deformation at the compressive face. The properties of the structure and materials were explained in the Structure description at section III.3. However it is good to remember the change in the yielding tension of the reinforcement. Instead of 60 Ksi the use of 70 Ksi is recommended to take care of steel strain hardening. This change is only for calculation purposes and it will be kept for all the analysis during this dissertation.

Now the process to model the structure has been established, in the following point this process will be applied to any different geometric or reinforcement configuration.

III.2.3. Design process

This section will do a summary of the most important steps in the limit analysis to find a configuration that satisfy the demanded failure mode. This is an explanation of the process followed to obtain the final configuration; in next section is where the results from the analysis are recapitulated. For each configuration tried is a table showing in one side the column reinforcement, and in the other the slab reinforcement. The slab reinforcement is presented in number of bars and percentage of steel; is also divided in two different strips with negative and positive bars. In Appendix A section A.2 is shown a detailed calculation for the definitely configuration of the structure. There are explained all the steps followed in the limit analysis in order to get the base shear strength of the building. All the calculations for the different configurations tried are not included due to the extension of the dissertation.

During the design process some parameters will not change because they are result of space requirements at the lab, and implied in the experiment itself. The structure has three stories, the height between stories is 3.05 m (10 ft) and each story measures 9.14 m (30 ft) wide and 15.24 m (50 ft) long

The starting characteristics of the draft for the structure are in Table 2. This will be called Configuration 1 from now on. Using these properties the plastic capacity of each member is calculated and the limit analysis performed. The analysis results in a Mechanism I were only the first floor columns failed. At the end of this section table 5 summarizes the results for the Base shear obtained for the different configuration. Then the columns are too weak in relation to the slab. For a better mechanism to occur is needed then that the columns are strengthen or the slab weakened.

Table 2. Configuration 1

Slab thickness	178 mm (7 in)	
	Column strip	Middle strip
Negative	24 Ø 16 (24 # 5) 1,08%	8 Ø 16 (8 # 5) 0,36%
Positive	10 Ø 16 (10 # 5) 0,45%	8 Ø 16 (8 # 5) 0,36%

Column size	356x356 mm (14x14 in)
Column reinforcement	4 Ø 22 (4 # 7) 1,22%

The second try (Configuration 2) only differs from the first one in the column amount of reinforcement. For this attempt the column reinforcement is increased slightly over 2% to 4 Ø 29 (4 # 9). Nonetheless the use of 2% reinforcement ratio starts to be uncomfortable. That is a common amount used nowadays, but around a 1% ratio represents better the prototype building. This change does not represent any change in the failure mode; the system still fails under mechanism I and far away from the mechanism IV in terms of base shear. The problem now is that the reinforcement ratio at the columns can not be increased because the structure will not be representative of the prototype. Consequently the columns have to be increased in size to increase the plastic capacity without increasing the reinforcement ratio.

Third attempt changes only in the column configuration as well. In this case the slab is kept as it was but the columns are increased until 457 mm (18 in) square and the reinforcement ratio is decreased down to 1.5%. To get this ratio 8 Ø 22 (8 # 7) were chosen because fit better spacing requirements for longitudinal bars. Using these new characteristics the limit analysis shows a Mechanism II, although the minimum base shear obtained is closer to the III mechanism. Thus for the next step will be recommend to decrease slab capacity; that could change the order of the mechanisms and make the fourth one the one with the minimum base shear.

Configuration 4, in table 3, has the lesser amount of bars possible for Ø 16 (#5). It is not possible to set less bars than this because is already out of the spacing requirements. The column has been kept the same as the Configuration 3. With these characteristics the limit analysis results into a mechanism IV, the desired mechanism. However as it is shown in Table 5 the differences between Mechanism II, III and IV are too small. Consequently it can not be assured which mechanism will be reached first.

Table 3. Configuration 4

Slab thickness	178 mm (7 in)	
	Column strip	Middle strip
Negative	12 Ø 16 (12 # 5) 0,54%	6 Ø 16 (6 # 5) 0,27%
Positive	6 Ø 16 (6 # 5) 0,27%	6 Ø 16 (6 # 5) 0,27%

Column size	457x457 mm (18x18 in)
Column reinforcement	8 Ø 22 (8 # 7) 1,48%

This is a point of inflexion in the research because now the columns can not be increased either in reinforcement or size. An increase in the size of the bars will cause the columns to be unrealistic compare to the slab. The other point is to reduce slab capacity either reducing the size of the bars to keep the spacing inside ACI limits, or reduce the slab thickness to lose inertia. Bars Ø 16 (#5) are the most common reinforcement for this kind of structure, consequently reducing the diameter to Ø 13 (# 4) will not be too representative of the prototype. Also reducing the thickness of the slab to 152 mm (6 in) instead of 178 mm (7 in) will make the building more alike to a laboratory model than to a real structure. Finally the decision was to continue reducing bar size; reducing the slab thickness with bars number 5 as reinforcement the ACI spacing requirements will compromise the adequacy of the configuration.

Then, Configuration 5 is made decreasing slab bars to Ø 13 (# 4), and the column stays in the same configuration. As it is shown in table 4 only the reinforcement at the column negative strip is different, the rest of the slab is in the same arrangement. The limit analysis shows a failure mode in mechanism IV, and it is also checked that the rest of the possible Mechanism are far enough to be safe in the experiment.

Table 4. Configuration 5

Slab thickness	178 mm (7 in)	
	Column strip	Middle strip
Negative	18 Ø 13 (18 # 4) 0,50%	9 Ø 13 (9 # 4) 0,25%
Positive	9 Ø 13 (9 # 4) 0,25%	9 Ø 13 (9 # 4) 0,25%

Column size	457x457 mm (18x18 in)
Column reinforcement	8 Ø 22 (8 # 7) 1,48%

III.2.4. Results

The results of the five steps explained previously are in Table 5. This table shows the base shear for each Mechanism. It is calculated only for the linear lateral load pattern and the five different configurations.

Table 5. Base shear for linear loading of the different configurations and mechanisms

KN	Reinforcement Configuration				
Mechanism	1	2	3	4	5
I	556	765	1303	1303	1303
II	663	778	1050	899	895
III	801	890	1103	863	847
IV	996	1041	1148	792	762

Kip	Reinforcement Configuration				
Mechanism	1	2	3	4	5
I	125	172	293	293	293
II	149	175	236	202	201
III	180	200	248	194	191
IV	224	234	258	178	171

Table 6. Base shear for uniform loading for the definitely configuration

Reinforcement Configuration 5		
Mechanism	**KN**	**Kip**
I	1303	293
II	985	222
III	990	222
IV	890	200

Looking at the table, configuration 5 fails in the mode desired. Takes up the maximum capacity of the structure developing plastic hinges in all the slabs, and the minimum base shear is lower than the second feasible mechanism. Also the difference between them is 20 Kip, large enough to guarantee that the first failure will be under hypothesis IV.

To calculate the base shear coefficient is needed the Total Tributary Weight (TTW) of the structure; its calculation is detailed as well in Appendix A section A.2. The self weight of the structure and the projected load for the structure are 537 kg/m^2 (110 psf) then the TTW is 2202 KN (495 Kip).For Configuration 5, using the triangular distribution the solution is a base shear 36% of the TTW of the structure. If the uniform pattern is taken, the calculated capacity increases until 40% of the system's weight. Although this is the maximum calculated capacity, it is expected to measure higher base shear during the test. The steel has been understood to be 70 Ksi, but due to strain hardening major strengths will be achieved. More reason to believe the strain hardening is the use of small diameter bars such Ø 13 (# 4).

This base shear is very high compared with what was expected. For this kind of building a base shear close to 20% of the tributary weight will be expected. The reason for this high base shear is divided into two causes. The main one is because a complete structural failure was looked for, the structure is working at its maximum capacity when failure, to counteract it a high base shear is needed. Also, the full width of the slab is taken into account, thus the slab has more strength and the columns have been increased in size to resist better than the slab, hence more resistance has been added to the structure.

III.3. Direct design

After the process of the limit analysis, the overall strength requirements for the structure have been bounded. This section will explain the design method from ACI and check that the amount of reinforcement suggested at configuration 5 accomplishes its requirements. It will also check the bar spacing requirements. The chapter for seismic load requirements of ACI will not be used. The system has just been analyzed as a flat-plate structure without any special characteristic.

For slab design, the code use direct design method which is basically a gravity load method. This method is intended for design the moment distribution along and across the slab. Although is good for vertical loads, it has limitations for lateral loads and one-way slabs. This method has three different steps, {1}. Determination of the total factored static moment, {2} distribution of the factored moment to negative and positive sections, {3} distribution of the negative and positive factored moments to the column and middle strips. A detailed calculation of the direct design method is shown in Appendix B. In any case here is a summary of the main calculations. The method starts calculating the static moment:

$$M_o = \frac{w_u l_2 l_n^2}{8} \tag{4}$$

In this formula w_u is the factored load per unit of area, l_n is the length of clear span in the direction of the moments, and l_2 the length of the transverse span from center to center of column. The factored load used for design is the typical used in this kind of buildings, the self weight of the slab, a Dead Load of 5 psf and a Live Load of 50 psf. This is only a design load, then in other calculations 15 psf is going to be used as a representative Live Load. The slab is divided into column and middle strips. Each strip is 10 ft. wide; the same distribution is done for the transversal axis. For the moment distribution, the slabs are considered without beams and all of them interiors; as it was explained earlier only interiors spans are being modeled. Then, the factors for distributing the moment found at the tables in ACI code are 65% to the negative side of the slab, and 35% to the positive side. Once the moment is divided, it is also shared into negative column strip 75% and negative middle strip 25%. The positive column strip gets 60% and the positive middle strip the 40%. Once the static moment has been distributed to every zone of the slab, it is compared with the factored moment capacity of that zone of the slab. The moments generated by the static moment are smaller than the factored capacity of the slab, thus the design is valid. All these calculation are in detail in Appendix B. The difference between design load and capacity is not significant, that means the structure is not over designed for gravity loads. The design loads generated are between a 50% and 85% of the capacity.

For the spacing requirements, the code obliges to do not be larger than 18 in. in any case. This limitation is at Chapter 7 of the code, looking in the conditions at Chapter 13, is found that the maximum spacing can not be larger than two times the thickness of the slab. Consequently the latter provision is the restrictive one. The lesser amount of bars placed in a strip are 9, the strips are 3.05 m (10 ft) each, that gives a spacing of 338 mm (13.3 in). The bars are not going to be placed exactly at that distance during construction; some of them will be at 356 mm (14 in) and some at 305 mm (12 in) because of the work conditions. In any case the calculation satisfies the code.

The review of the ACI requirements has shown then that the Configuration 5 is the appropriate for the experiment necessities and is also a realistic structure.

III.4. Structure description

In this section the definitely test structure properties are described, the purpose is also to make the reader familiar with the building. These properties are result from the design developed in chapter III of this study. The structure is a full-scale three story building. Figure 5 shows the main views and dimensions of it. It is 9.14 m (30 ft) wide and 15.24 m (50 ft) long, the slab thickness is 178 m (7 in) and the columns are distributed in two frames of three columns in the longitudinal axis. Columns are 457 mm (18 in) square. The columns have 1.5% reinforcement distributed in 8 Ø 22 (# 7). In the longitudinal axis, the slab is divided in three strips 3.05 m (10 ft) wide each one. There are two column strips and one middle strip. The negative reinforcement is 0.50% in the column strip and 0.25% in the middle. The positive reinforcement is 0.25% in both column and middle strip. The slab bars are Ø 13 (# 4). In the transversal axis the reinforcement is the same as the longitudinal. All the reinforcement details are shown in Figures 4 and 6. These parameters already mentioned are the main and the important for the present study, although a lot of other details have been and will be design in order to essay the specimen at the laboratory. Is not an objective of this dissertation to design the rest of the characteristics; however, the objective is determining the load necessary to perform the test. Thus, following paragraphs refer to the footings, loading system and materials used in the experiment. All the figures corresponding to these following paragraphs are in Appendix A.

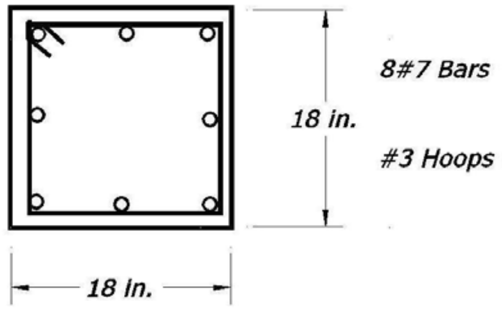


Figure 4. Detail for the column reinforcement.

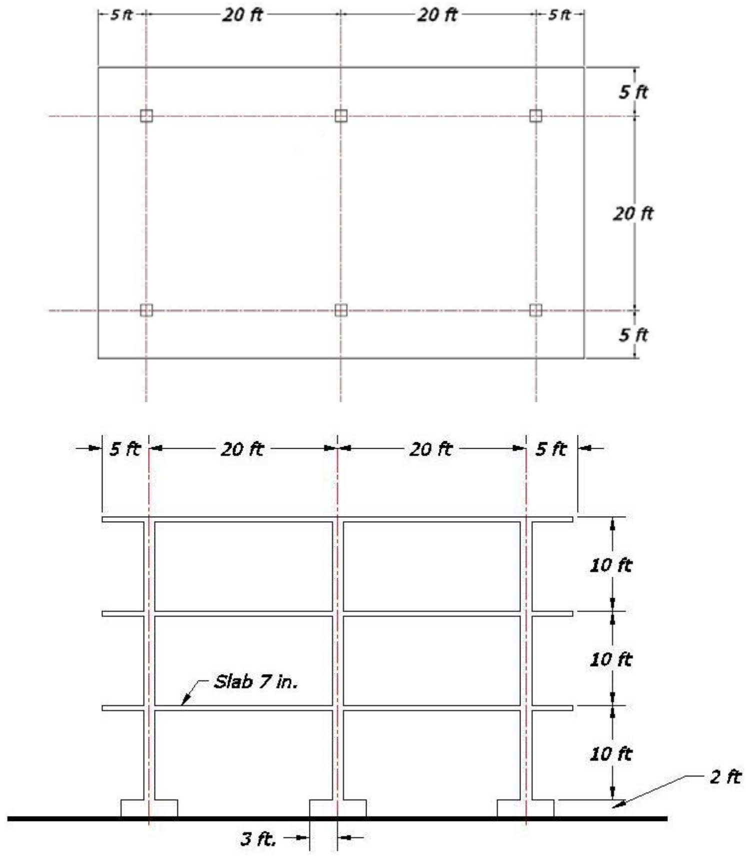
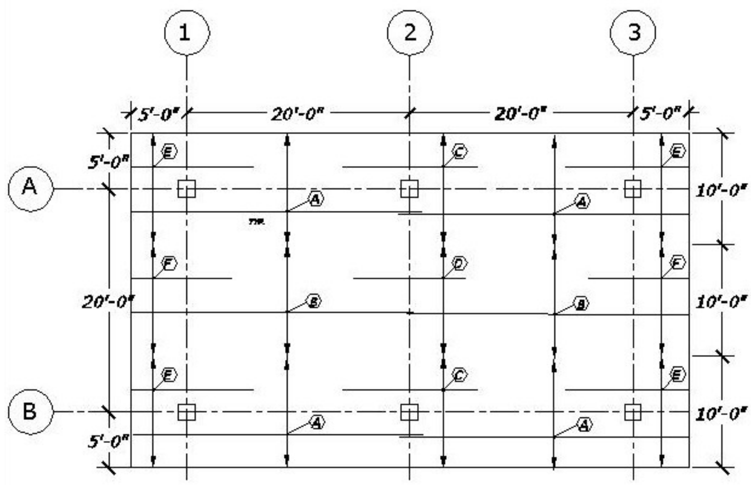


Figure 5. Structure layout in plan and side view.



REINFORCEMENT SCHEDULE		
MARK	REINFORCEMENT	NOTES
A	9-#4 x 26'-0" BOTTOM	
B	9-#4 x 26'-0" BOTTOM	
C	18-#4 x 13'-0" TOP	
D	9-#4 x 10'-0" TOP	
E	18-#4 x 12'-0" TOP	
F	9-#4 x 10'-0" TOP	

NOTES:
1. MINIMUM 2 SLAB BARS TOP & BOT SHALL PASS THROUGH COLUMN.
2. SEE FLOOR SLAB LOADING SHEET FOR CONNECTION LOCATIONS AND DETAIL K FOR ADDITIONAL REINFORCEMENT.

Figure 6. Slab reinforcement layout.

The loading configuration for the tests will be four RAM's, two of them at the top floor and one RAM in each of the other floors. This configuration has been adopted to keep control over the rotation of the system. This scheme was adopted in a Full-scale test performed in Japan for a seven story building [15]. When constructing the building it will be small eccentricities in the connections to the loading and small inaccuracies in the structure. Due to them, during the loading the structure can rotate and generate torsion on the system. To avoid this situation the two top RAM's will correct the loading so rotation is not generated. The structure will be fixed to the ground by placing footings in each support. The strong floor of the lab has lines of holes to fix the specimens to test; these holes are located in rows along the lab. Each one of these rows is formed by another three rows of holes separate each other 0.61 m (2 ft) from center to center. Then the footings have been design to act as a fixed point for the structure as well as to adapt to the configuration of the lab floor holes. A drawing of the footings is shown in Appendix A. How to transfer the loads from the RAM's to the structure has been another problem of major concern. Finally the solution adopted was to attach one channel to the side of the slab, connecting it to the slab in the middle of the spans. The connections are shown in more detail in Appendix A. However these connections for the loading experiment do not concern to the present study.

Also the dimensions of the RAM's were analyzed. It is not desirable to have a structure stronger than the elements to load it. The limit analysis previously explained solves this question. After the calculation was done, the base shear of the structure is 40% in the upper bound. Although the load pattern for the test will be inverted triangle, this base shear is the adequate to calculate the RAM's capacity. The 40% base shear requires a force at the top floor of 440 KN (99 Kip), at the second floor 294 KN (66 Kip) and at the first floor 147 KN (33 Kip). Looking at the required load and pursuing ensure the test, the capacity of the top floor RAM's is 556 KN (125 Kip) each. The use of 334 KN (75 Kip) and 222 KN (50 Kip) is for the second and first story respectively. Finally the RAM's will be able to develop a load equal to the 75% of the TTW. Maybe they can be considered over-designed but they will be useful for future experiments at the lab.

The materials used in the construction have been selected according with standards. ASTM C-150 Type I cement, ASTM C-33 & INDOT Spec #23 sand, and #8 gravel ¾ inch. These characteristics will be used for all the structure. The concrete is batch in a commercial ready mixed concrete plant. The concrete stress is 24.1 MPa (3500psi) having a 7 day average strength of 21.1 MPa (3059 psi). The 28 day average strength is 31 MPa (4500 psi). Although the f'_c is 24.1 MPa (3500psi), the calculations for capacity have been done assuming 27.6 MPa (4000 psi) because it is closer to the real strength. This is because 24.1 MPa (3500psi) is the minimum required, thus the expected resistance should be larger. For the reinforcement Grade 60 deformed reinforcement bars are used. The same assumption as concrete is done, 414 MPa (60 ksi) is the minimum required yielding strength, so a 483 MPa (70 ksi) is more appropriate to use for the calculations.