PIGEON ROCK CASINO

THESIS

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ACKNOWLEDGEMENT

THE AUTHOR WISHES TO EXPRESS HIS DEEP THANKS FOR GENEROUS CONSULTATION ON DIFFERENT MATTERS CONCERNING THE WRITING OF THIS THESIS TO PROF. FERRUH KOCATAŞKIN OF ROBERT COLLEGE, ISTANBUL.

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Chapter I

INTRODUCTION

" Beirut, the capital of Lebanon, has been called the heart of the Orient. It boasts a fine International Airport and a harbour rated among the first in the Mediterranean. It is a smiling city of over five hundred thousand inhabitants and a busy, modern mercentile and shipping center, which however, knows how to relax in its leisure hours. It counts a large number of ultra-modern hotels, cabarets, cinema, theatres, etc.

Long stretches of smooth sandy beaches equipped with modern swimming facilities are at everybody's disposal.

Numerous restaurants renowned for their exquisite cuisines offer excellent European-Continental and Oriental foods that appeal to every taste. Cosy bars and tea-rooms are found in great number

The gay night life of Beirut has earned it the name of "Paris of the Orient." The city has several types of night spots.

Cinemas in Beirut are all of modern design, are almost all air-conditioned and offer American, French, English, Italian, German, Russian, and Arabic productions.

For the holidays, cabarets, night clubs, bars, restaurants have special shows imported from European cities, at great expense. The casino du Liban is of course the leader among all places of entertainment.

Hotels in Beirut deserve a special mention. Those in the luxury and first class categories constitute an attraction in themselves whether from the architectural standpoint or the lavish decorations of the interior salons and suites." Ref. 13 .

All such facilities are available to encourage tourists and attract vacationeers from different parts of the world to visit Lebanon. It is well recognized that a big part of the income in Beirut comes from Tourism.

Although the preceeding discussion gives an impression of Beirut as being saturated with all leisure facilities one recognizes the further need of new ones.

The idea of greater number of tourists coming to Lebanon is always favored. More tourists means more income and more income means a higher standard of living, a thing that all the nations in the world are looking forward to attain.

The proposed idea is to add one more constructive item to this smiling city and its ever flourishing business of tourism.

The idea is to build a casino on top of the Pigeon Rock (details follow in later chapters).

This idea is not new. Several people thought of making something out of this rock but nobody thought of it seriously. The proposal is as follows: A casino is to be built on top of the Rock. A stairway and elevator system is to run from the bottom of the rock to the top of it from the south-western side.

Furthermore a bridge is proposed to run from the bottom of the stairs to the landscape on the southern side of the Rock.

A small building on the shore at the beginning of the bridge is to be constructed, and this will be used for reception, administration, starage, and staff.

The casino is proposed to consist of two floors, a basement and a roof. The basement will include the main kitchens.

The first floor is proposed to consist of a nightclub. The top floor is to be a restaurant. The roof will consist of a snack bar and a terrace.

It is further suggested that the casino's restaurant and night club be all surrounded with glass running from the ceiling to the floor to overlook the beautiful surroundings of the Meditteranean on one side and the Raouche area on the other.

The proposed plan is show in Fig. I-1 .

The aim here isn't to make the complete planning of the casino but to make a reasonable architectural design to give an idea of what could be done in the interior and exterior. An economic survey is attempted to help in the validity of the proposed project.

Description of the Rock including geology of it preceed the stru

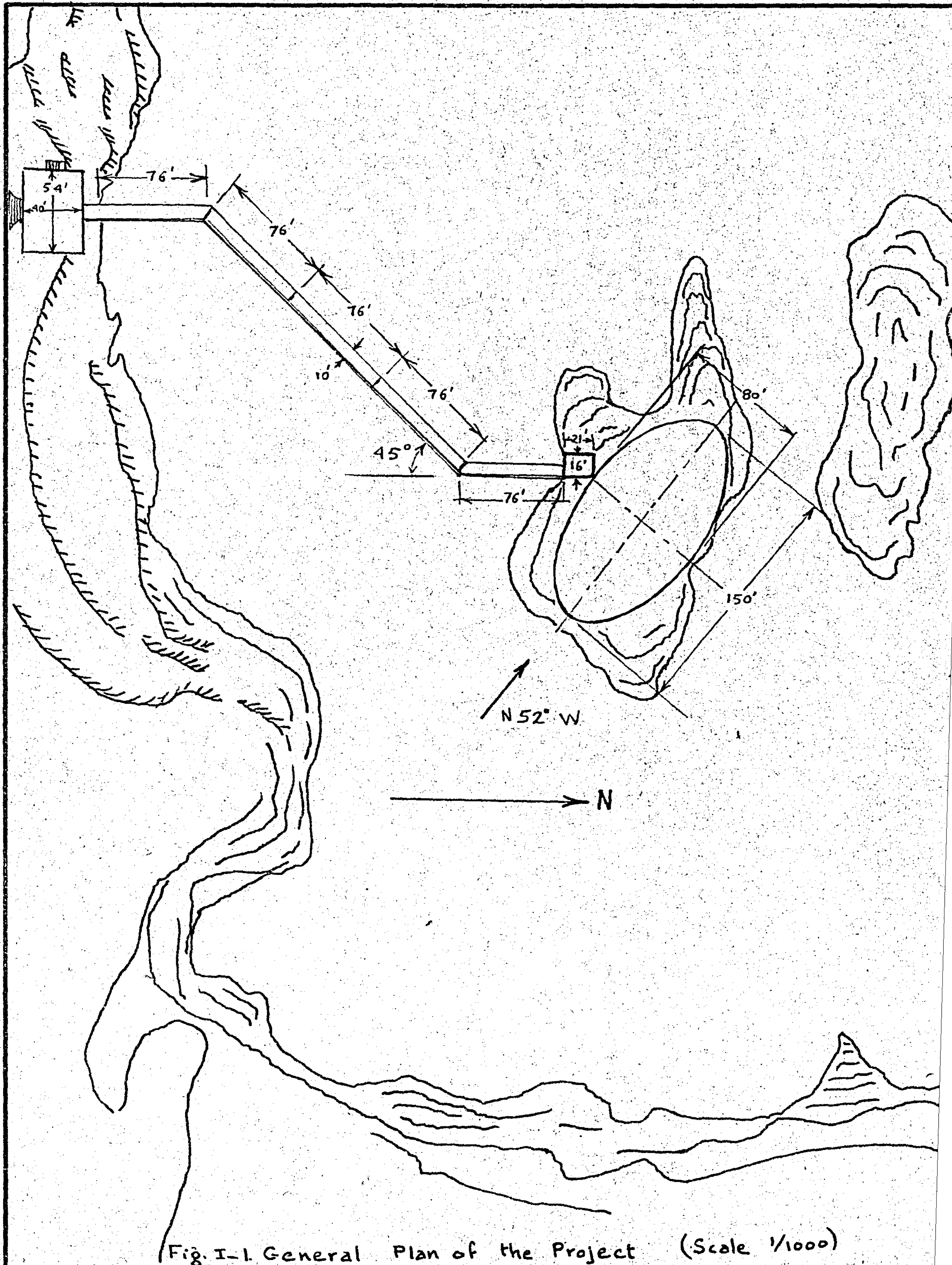

Fig. I-1. General Plan of the Project (Scale 1/1000) tural designs.

The structural designs af the four parts namely, the casino, bridge, stairway and shore building will be worked out in more detail and a bill of materials will be formulated.

Procedure and methods of construction are suggested in Chapter IX.

The thesis is going to be ended with a general conclusion.

Chapter II

ECONOMIC SURVEY

Tourism in Lebanon

The following quotations are taken from articles published by the M.E.A. and the office of tourism in Lebanon, Ref. 13 & 14. Although the purpose of such articles is propaganda yet they give a true picture of the situation.

" A land of guaranteed sunshine, sandy beaches and mountains, the Mediterranean republic of Lebanon is a utopian playground for the away-from-it-all twentieth century tourist.

Both historically and geopraphically, a meeting place of east and west, this legendry country contains every possible holiday and sightseeing attraction. It is a place in the sun where the ancient and the ultra-modern mingle in vividly contrasting splendour.

The climate is ideal for year-round holiday making. Off-shore summer breezes prevent it from getting too hot along the beaches and during the winter months thick carpets of snow in the mountains provide alpine skiing conditions. Sailing, water-skiing and skin diving are popular diversions at the resorts dotted along the 140-mile long coastline. "

" Not so long ago, vacationers and tourists of neighbouring countries flocked to Lebanon, Businessmen and salaried workers who had limited holidays preferred to spend them in this country instead of passing a good portion of them aboard a ship which took several days to take them and bring them back to and from Europe."

But the airplane has changed this state of affairs in a few hours, it brings the Middle East within the reach of the farthest of tourists. A new type of customers was thus acquired, entering, on the stroke of a wing, the circuit of Great International Tourism, which groups more than 150 countries.

Statistics are from this viewpoint quite eloquent- a country which counts 1,600,000 inhabitants, received during these past years, a number of visitors which exceeded that of its population....which is a kind of record."

Development of the Area

" The first locality that meets the eye of the traveler as he arrives in Lebanon is the city of Beirut, the capital of the country.

Beirut from the sea and the air is unforgettable sight. Risin behind it are the beautiful high mountains bathed in sunlight, or should the traveller arrive in winter, capped by snow or hidden by mist.

Beirut enjoys for the greater part of the year the ideal type of climate mentioned before, comparable to that of the Cote d'Azur. It has, however, the added advantage of being within easy reach of the summer and winter resorts of the Lebanon, rising to an altitude of about 6550 feet above sea level.

Fig.II-1 General View of the Rocks as seen from the ai

Fig.II-2 Development of the Area.....

The sumptuous and dainty villas, the beatiful gardens, the suburban forests add to the charm of the natural site making Beirut one of the prettiest cities of the Orient and an ideal centre for residence and tourism." Ref. 13 .

The latest and most developed area in Beirut is the Raouche area where the Pigeon Rocks are located.

High buildings, first-class restaurants, dance halls, coffee houses and bars spread all over the area. The Raouche area is becoming the new center of night spots and night life in Beirut.

Interviews

I tried to carry out a sort of an interview concerning the subject of my project with more than forty different people.

I've chosen these people as representatives of different branches of the Lebanese society. They included people of different educational, cultural, economic, and social standing. Thus I thought that their opinions might help me in evaluating my project from the economic point of view.

I was careful about one thing though, that the people I've picked for the interview were mature enough to judge. That's to say they were mature from the point of view of age, education and thinking on one hand and that they could tell something about the subject. All of them knew the Raouche area and the Pigeon Rocks well enough to be able to judge.

Those people I've consulated included the businessman, the employee, the artist, the travel agent, the airlines manager, the architect, the engineer, the restaurant owner, the casino owner, and different representatives of different kinds of business.

In the interview I asked a person to tell me his idea about my project. I told him briefly about it and clearly of what I am planning to do. I said what would the casino inclde and how it would look like with some more details.

As the reader will notice fnom the results of the interviews that the opinions differed. Some thought that the ideawas excellent but others didn't. Some were in between the two. From such interview I tried to derive my conclusion which shows that my project, if brought to reality, would be very valuable and successful.

* This is a very good idea. I wonder if anybody thought of it seriously of bringing it to reality.

* I think if the rocks stay as they are without spoiling the view it would be better. I'm worried that if any attempts are made to make something out of these rocks it would just ruin their natural beauty.

* If this idea is brought to reality, it would certainly be great. The casino would be of much greater fame than any other one in the country after Casino du Liban, not because of its size but because of its uniqueness.

* I tell you right now, after listening to what you said that I am ready to invest in such a project. Get through with your idea and design and I hope that we can work out something together if you like.

* I wonder whether such an idea can be brought to reality. But if it ever be brought to reality, undoubtedly it will be great.

* This will just add one more thing to the beauty of our city.

*This will be a second step after the construction of the Casino du Liban. I hope that Lebanon will have several of such unique things.

* Architect: " After thinking of your idea I agree with it provided that the following two things are carefully takǡn into consideration:

a) The architectural design should be beautiful enough so that it would add a further beauty to the rock itself.

b) There would be no bridge that would ruin the view , but an underwater tunnel, so that the casino wold look like an eagle's nest.

* Well! I think that Beirut is very crowded. I wish at least the rocks would stay as they are so that people on the shore would see something of this nature left untouched by human hands.

* Businessman: " Oh! this will give the people of the city inclding businessmen a chance to relax and be away from the city's noise and trouble at least for a short while.

* Airlines Salesman: "There will be another thin in our hand to advertise with full confidence. The tourists would surely enjoy going to such a place."

* Casino Partner: " I hope if such a thing is brought to reality I would be a partner, Because the reputation of such a place would outshine the reputation of the best night clubs and restaurants in the city.

* Restaurant owner at Raouche: " People coming to the Raouche area and to our restaurants would prefer to be further from the city and nearter to the water, especially in summer time. For sure they will prefer to be an the rock rather that to watch it from

far. This would just be bad for our business, because the project would be so great."

* Civil Engineer: " I think that such a construction has many structural difficulties, but it is worth the trouble."

* Man ! if such a project is brought to reality the profits in three or four years would cover all of its cost."

* The rock is too high and further more it is in the middle of the water. Many people prefer places on the shore with little elevation. But still, you would have enough of customers all the time.

* Airport passengers manager: " I think it would be worth while then shifting all our airline busses to pass through Raouche on their way to the city because we would have one more thing to show and to invite our tourists to.

* Municipality official: " I would support and encourage any idea that would add to the tourism business in our country. Your idea is just excellent.

* Being fond of nature I hope that the rocks would stay as they are. I'm worried that such an increase of population wold change our little country to a big city, for almost every area in Liban is being ppulated.

* I hope you can build your casino on the shore and leave the rocks undistmmbed, they look much nice as they are.

* Banker: " I liked you idea very much. I hope you will be able to work out your idea for your own sake and for the flourishing of our country."

* The whole country in its success and failure depends on your educated young men. Your idea is very constructive from many points of view. I wish many others would think lile you to create new things for this country.

* This is what we want for this country : enthusiastic, creative, young men like you. Your idea is just fine. I hope you'll be able to bring it to reality."

* Tourist Agency Manager: " Lebanese are business minded. This is a very good idea to extract more money from the tourists. Believe me that many of the transit tourists staying in Beirut for even

few hours would visit that casino, to dine, to watch the show at night, or to sit at the terrace and enjoy the sun during the day-time, the sunset in the evening or the sea breeze in hot summer nights.

In such a place you can have prices double those at other places of the sort in the city."

* Businessman: " This would be a quiet place and a very attractive one. I wouldn't mind dining there several times a week. This would release the stress I've at the office during the day hours. I think that hundreds of other business men in Beirut would agree with me."

* The area of the rock is small; but if the design is the right one a beauty can be made out of this rock. Furthermore, I think that the project from the economic point of viewwould be very profitable."

* The project is worth the adventure. This can be a very profitable project especially if it is run by the proper staff.

* I wonder if anybody ever thought of this project seriously. I congratulate you for thinking of bringing the idea to reality.

* It would be a big step to bring such an idea to reality. It should be thought of very seriously so as to make the best out of this rock. The idea is excellent.

* There is no doubt that such an idea is just excellent. I would invest in such a project.

* Municipality, Eng'g section:" I think you're the first one to consider the idea seriously for others have mentioned such a thing but as fas as I know no one has ever paid a serious thought to that project. I advise you to go and register your idea if you can before someone else does that, for the idea is a very good one.

* If the government gives you permission to bring such a project to reality and you make the design of such a structure successfully, you'll gain a great reputation in Lebanon.

* He who designs such a casino and brings it to reality would gain a great reputation in Lebanon as a creative engineer, provided that the beauty of the structure would fit that of the rock.

* If the proper design is doen for such a casino this would greatly add to the beauty of the rock.

* Is the rock big enough for a casino ? If it is so then there is no doubt that such a project would be one of most inventive ones in this country.

* If the idea was realizable then why hasn't such a structure been built before. I think that the idea is sophisticated and I wonder if such a thing can be brought to reality.

* If the government gives you permission for that project there would be a big number of businessmen who would like to invest in such a project. I think that the project is convincing and I'm sure that the government would pay the necessary attention to it.

* The idea is very good. If you can bring it to reality you will gain two benefits from the design:

a. the cost of your design.

b. a big reputation as an inventive engineer which will be very valuable to you in the future.

* I think that the idea is very good, but for the project to be complete I would suggest that you make use of the two rocks together. Furthermore it wold be a good idea to build a swimming

place at the bottom of the rock from the western side, with row-boats, flat boats, water baats, etc. The project them would be more complete, more profitable and one would find more satisfaction in going there.

* I think that the rock isn't strong enough to carry such a structur But if this project is realisable from the structural point of view then from all the other points it would be perfect.

* I can dream of such a place. It would add to the beauty of the area and to the beauty of our city.

Conclusion

These interviews show quite clearly that, provided the technical difficulties can be overcome the project would be warmly welcomed in both business and official circles. The fact that at this early stage several business men have expressed their willingness to invest in such a project demonstrates its practicability.

It is also generally agreed that the project would be a valuable asset to the amenities of Beirut, and hence Lebanon.

Chapter III

DESCRIPTION OF THE ROCK

Location

"Not far from the American University of Beirut, visitors will see the picturesque "Pigeon Rock Grotto", rising out of the deep blue waters." Ref. 13 . These rocks are located in a sort of bay near to the shore. They look like small islands. They are about 40 feet away from each other at the nearest point. Fig. The big rock is the one of our interest. It isabout 110 feet above water level and about 120 feet at its nearest point from the shore. It's about 300 feet away from Chourane Str. which is the main street in the vicinity.

The rock rises almost vertically from the water level. The contours are so close that it is too difficult to climb up to the top of the rock without the aid of the necessary climbing equipment.

At the bottom quarter of the rock one can see a natural tunnel which adds to the beauty of the rock, that goes down below the water level. It is easy to pass by small boat under this tunnel from one side of the rock to the other.

Fig.III-1 The Rocks as seen from the Eastern Side.

Fig.III-2 The Rocks as seen from the Western Side.

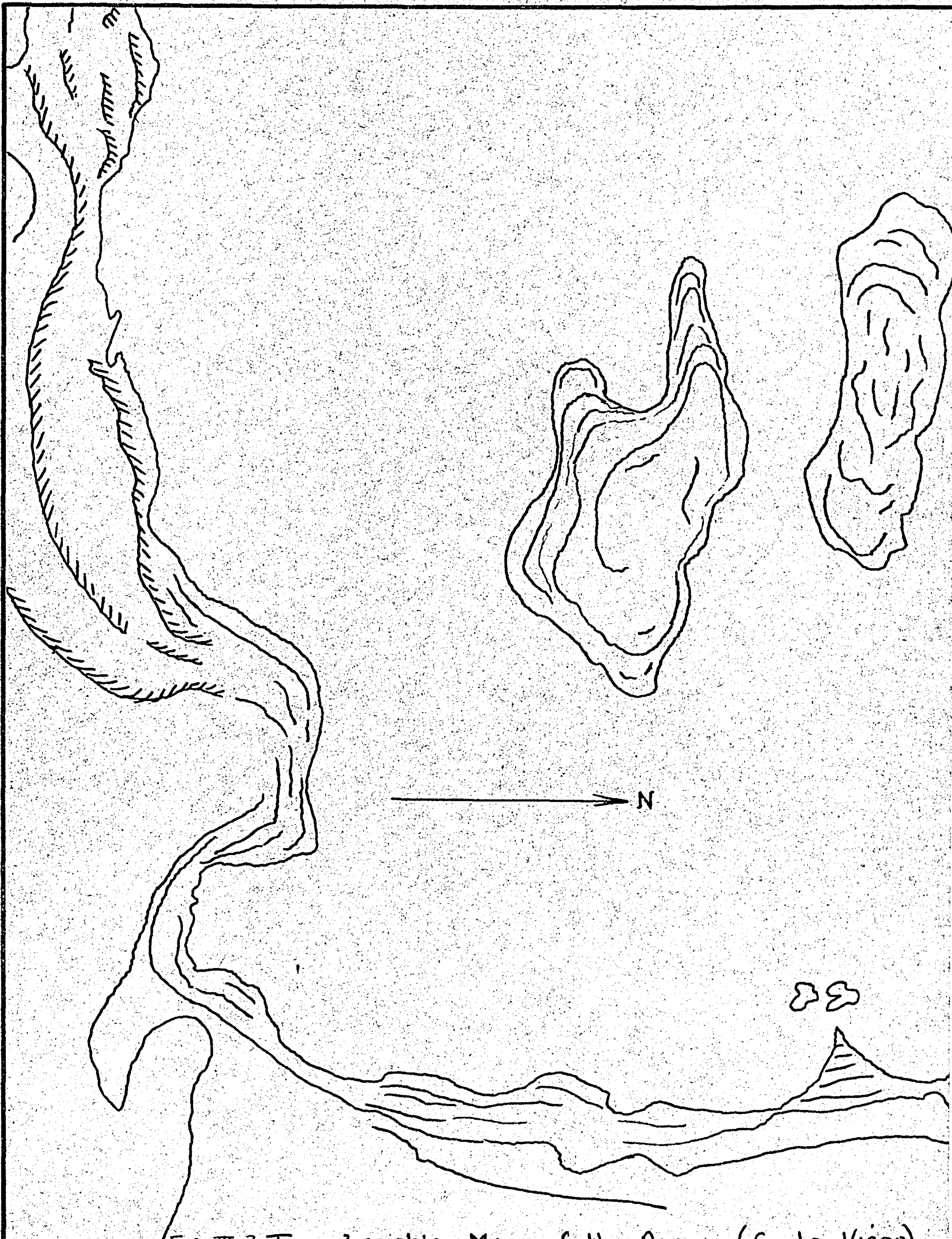

Fig. III-3 Topographic Map of the Area (Scale 1/1000)

GEOLOGY OF THE ROCK

The following article is taken from Ref. 7 p. 528 under the heading of Limestones. It is included here to help in making a comparison between our rock and limestones.

Structural features

Limestones are always stratified, but the beds vary in thickness in different quarries or even in the same quarry. Those deposits which show massive bedding will naturally be of greater value for extracting dimensian stone. In most districts where limestones are quarried for structural work the beds lie flat or nearly so, but at times owing to folding of the rocks the beds may be tilted at varying angles. Jointing is rearely absent, and since limestones are more soluble in surface waters than sandstones the rock along these joints is sometimes more or less weathered by solutions.

Vertical and horizontal variations may occur. Thus thick beds may alternate with thin ones, or shaly seams with lime stones. Certain beds may be of even character, while others interbedded with them may be of cherty nature. As a result a good series of beds occurs at one level, while at a higher or lower level the beds may be worthless. Again, the limestones if fallowed up along the strike sometimes became shaly, or change in composition, Bearing these facts in mind, it will be realized that in searching for a quarry site, the engineer should not base his condusions on one or two outcrops.

Properties of Limestones:

Texture: - Limestones show a variable texture, but the majority are fine-grained. Those which are coarse grained are either strongly fossiliferous or else coarsely crystalline. The finer-grained ones split more evenly and have better weathering qualities. The texture does not necessarily bear any direct relation to the absorption.

Hardness:- Dense limestones are usually quite hard, while the more porous ones are likely to be soft.

Color:- A pure limestone whether calcitic or dolomitic is white, but clayey or carbonaceous impurities tend to give it a grayish color and the former may also make it grayish or brownish black. Many of the later fade slightly on exposure to the atmosphere

Durability and mineral impurities:- Both limestones and dolomites of dense and massive character, as well as those free from mineral impurities, are of good durability, although not as long-lived as dense sandstones and granites.

Limestones weather primarily by solution, that is to say, rain or surface water may slowly attack the rock, but the solution of the surface is likely to go on very unevenly. If certain portions are silicified, such as fossils replaced by silica, or if quartz veins are present in the rock, these resist the solvent action of the surface waters more than the surrounding calcareous parts of the rock and are left standing out in relief, giving the stone a rough appearance

Dolomites do not weather so readily by solution. Some coarse-grained ones disintegrate, breaking off a grain at a time.

Certain mineral impurities interfere with the value of the stone.

Pyrite is an undesirable one, not only because it weathers to rusty limonite, but for the reason that in this change sulphuric acid is set free, which attacks the rock.

Another common impurity in some lime rocks, the nodules usually being strung out in bands along the stratification planes. It not only causes the rock to weather unevenly, but interferes with the dressing of it, in drilling through it, and lastly imparts to the stone a tendency to split along the lines of the chest concretions when exposed to frost action.

Fire resistance:- The resistance of limestone to fire, at temperatures below that required to convert the stones into quicklime, is usually fair, although lime rock, like other stones, is apt to spall badly under the combined attack of fire and water.

Crushing strength:- Most hard limestones show a good crushing strength, ranging from 9000 to 12,000 pounds per square inch, or sometimes very much higher.

Turning back to our rock we see that it hassall the properties of limestone including the stratification of the beds which lie almost flat at all elevations.

Jointing is present at different locations in the rock and the rock along these joints is more or less weathered with time. Generally speaking we can say that the rock doen't have a change in composition, and it is more or less composed of the same material of limestone. This rock that has been standing for ages against weathering factors proved to be of the same chemical composition. Salter parts at the joints or faults were weathered leaving behind them cracks like canitics extending from top of the rock to the bottom of it and inclined at different angles.

The natural tunnel might be an indication ofthe former existence of salter material that was weathered leaving this cavity.

The physical properties of lime stones coincide with the properties of our rock. The rock is hard and dense. It has almost white color but because of clayey or carbonaceous impurities the color is grayish.

The top part of the rock is weathered primarily by solution, that is to say, rain water. As a result, that part is uneven and shows a rough appearance.

The resistance of limestone to fire is usually fair and this is a fairly desirable property.

The minimum crushing strength is 9000 psi, but in design a much lower value is going to be taken as a safety factor.

Blasting

All what we've here concerning blasting is made of hard rock. The blasting is going to be quite costlys but there isn't much choice because the area is limited and because the project is worth the cost.

A. Blasting the Rock

This is going to include three parts.

1) Top leveling: The top of the rock is going to be cut off to a maximum depth of 12 feet from the peak of the rock for the following reasons.

a) To level the rock, and make it suitable for the construction.

b) To get a bigger area by cutting the top contours off.

c) To get rid of the eroded and unreliable material for the construction.

2) Basement: Below the leveled surface, the area framd by the hexagon inside the six columns, is going to be blasted to a depth of twelve feet. The reason for this is to gain space and to use all this place as a kitchen and stove for the casino.

Other minor side excavations include a small tunnel sanitary pipes.

3) Side Blasting: This is mainly meant to include some parts on the lower half of the rock to provide the necessary space for the stairway structure.

B. <u>On the shore</u>

This includes rock blasting in place of the minor structure for lowering the rock level and for leveling the area for the structure.

The approximate amounts of rock to be blasted will be estimated in later chapters.

Chapter IV

DESIGN OF THE CASINO

The design of the casino is more or less the principal part of this thesis. All the other parts may be considered as minor parts of the project compared to that of the casino.

The shore building, the bridge, and the stairway structures are nothing but suppliments to the main structure.

The casino in its architectural and structural designs, and peculiarity of its location forms the major part of the problem. The problems of the structural design with its safety and economy form the main work here.

The problems that were faced in the design of the casino, their solutions, and some comments on these solutions are illustrated in this chapter.

Architectural Design

The first problem is that of the architectural design. Here it might not be possible to chose the architectural shape that is desired. The top area of the rock is limited and the shape of it is limited too.

An elliptical form of the casion floors seems to be the most suitable if most of the available top area of the rock is going to be used.

The shore building is designed to help in reducing the amount of space of the casino needed for auxiliary functions as administration, storage, and staff rooms.

The first floor and the top floor have similar designs with regard to space. The space in each of these floors is cut by six columns only.

The basement which is the hexagonal area framed by the six columns is designed to be used for kitchens and storage. Toilets and washrooms may be included in the basement.

The first floor will consist of a night-club and a bar ,Fig. IV-2

The top floor will consist of a restaurant. Fig. IV-3

The roof will include a snack bar and a terrace. Fig. IV-4

The two floors and the snack bar were suggested in Chapter I to be surrounded with glass from floor to ceiling so as not to cut the view from any side.

One should have in mind though that the design of the interior as shown in Fig IV.2-3 is not a final decision but a proposal. As our purpose here is only to show how the interior may be arranged and to show that the structural design is based on some achitectural considerations.

Any additions of partitions, floor finish, floor tiles, and false ceiling are considered in the structural design with respect to their additional loads on the structure. This is accounted for to add to the flexibility of the design of the interior.

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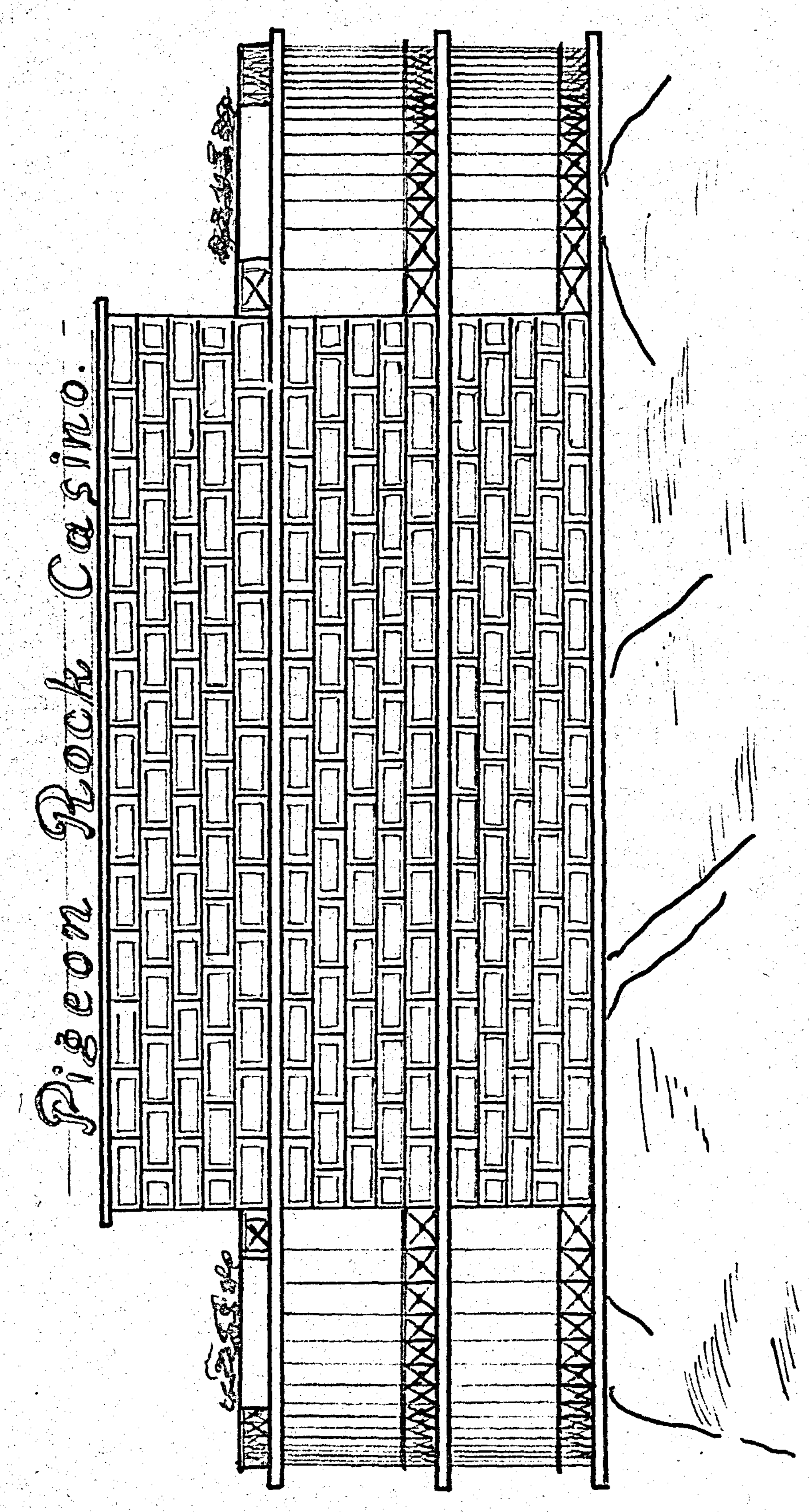


Fig. IV-1. General View of the Casino Looking From North-East (Scale 1/200)

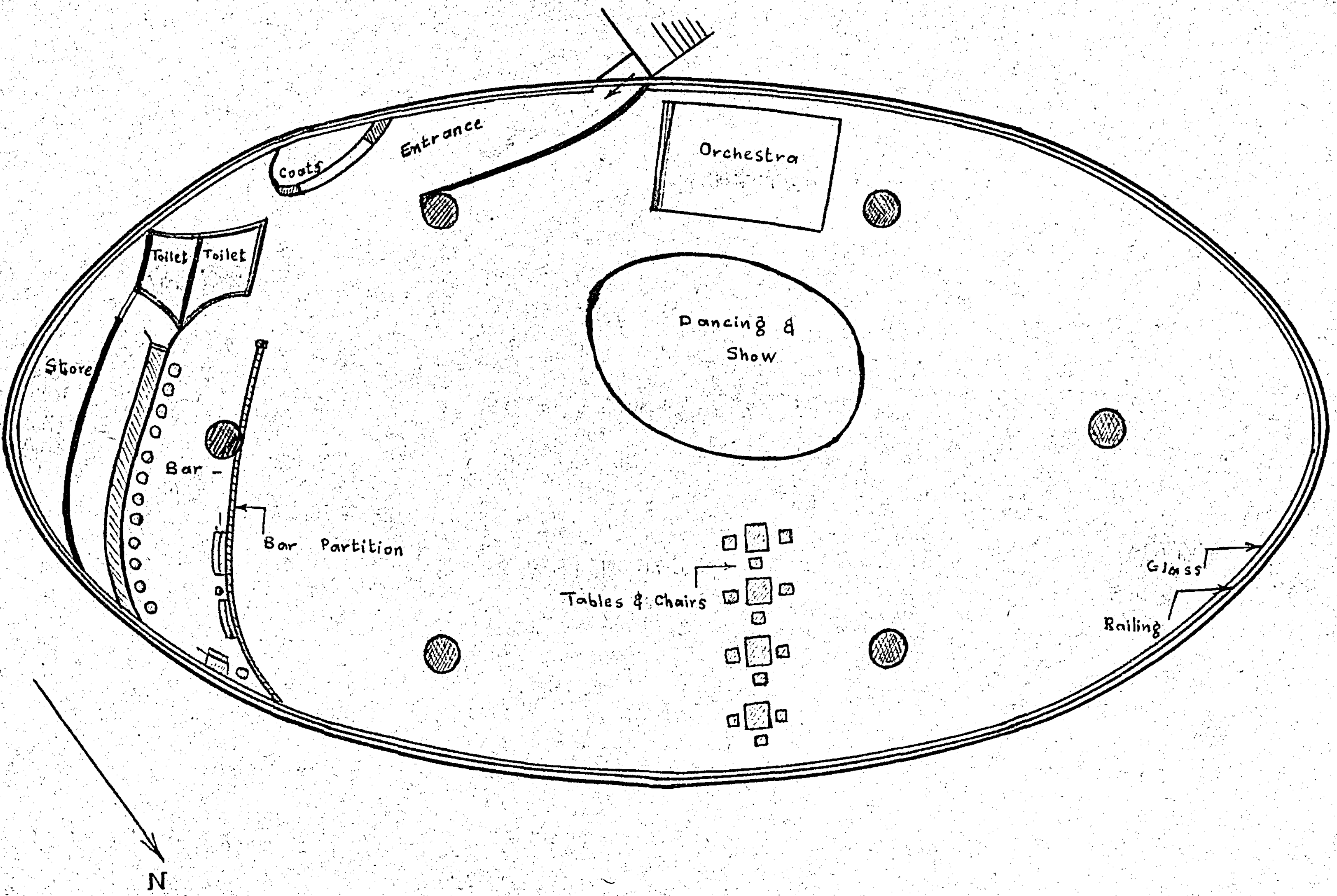


Fig. IV-2 Interior Plan of the Casino's Night Club & Bar (Scale 1/200)

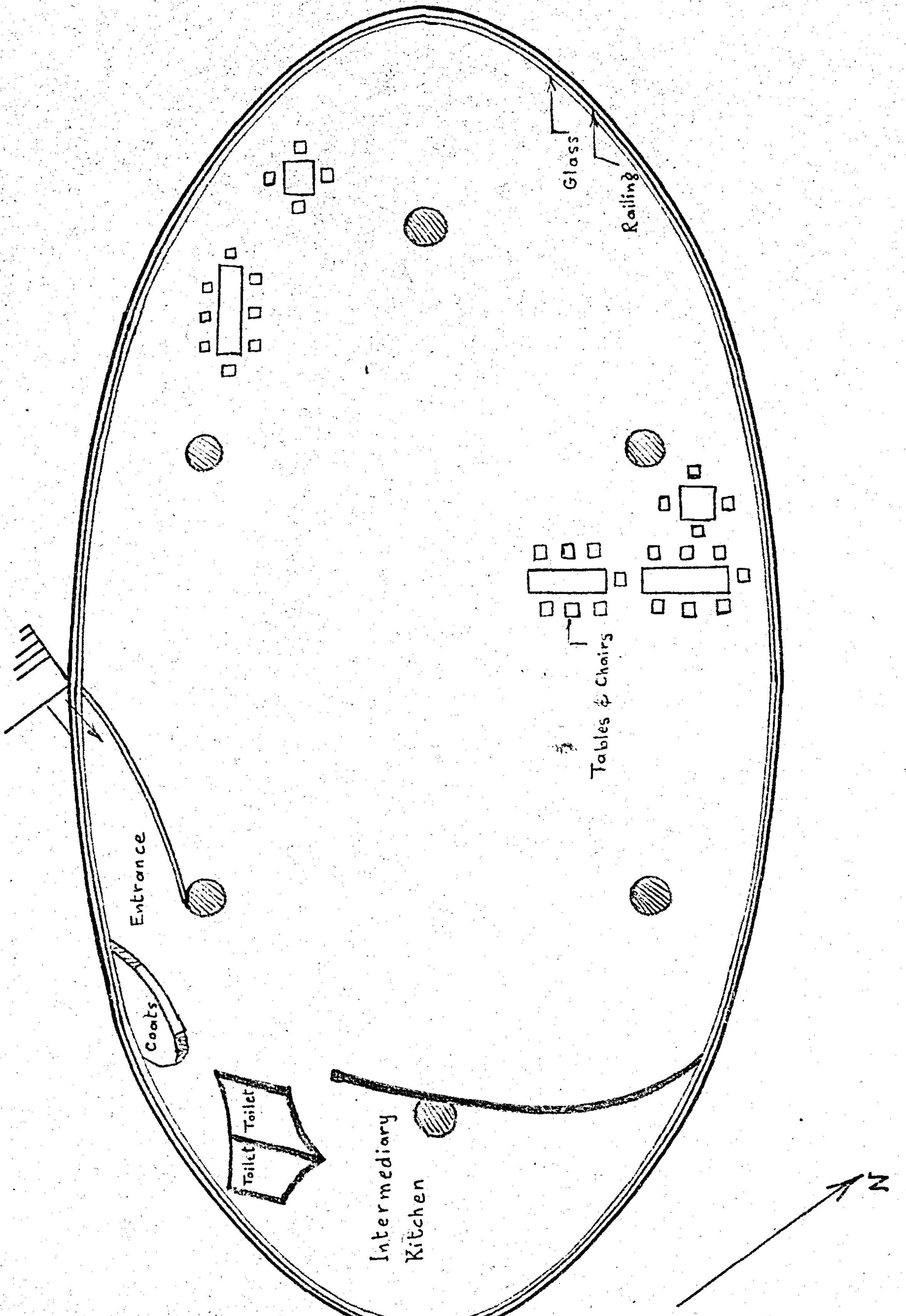


Fig. IV 3 Interior Plan of the Casino's Restaurant (Scale 1/200)

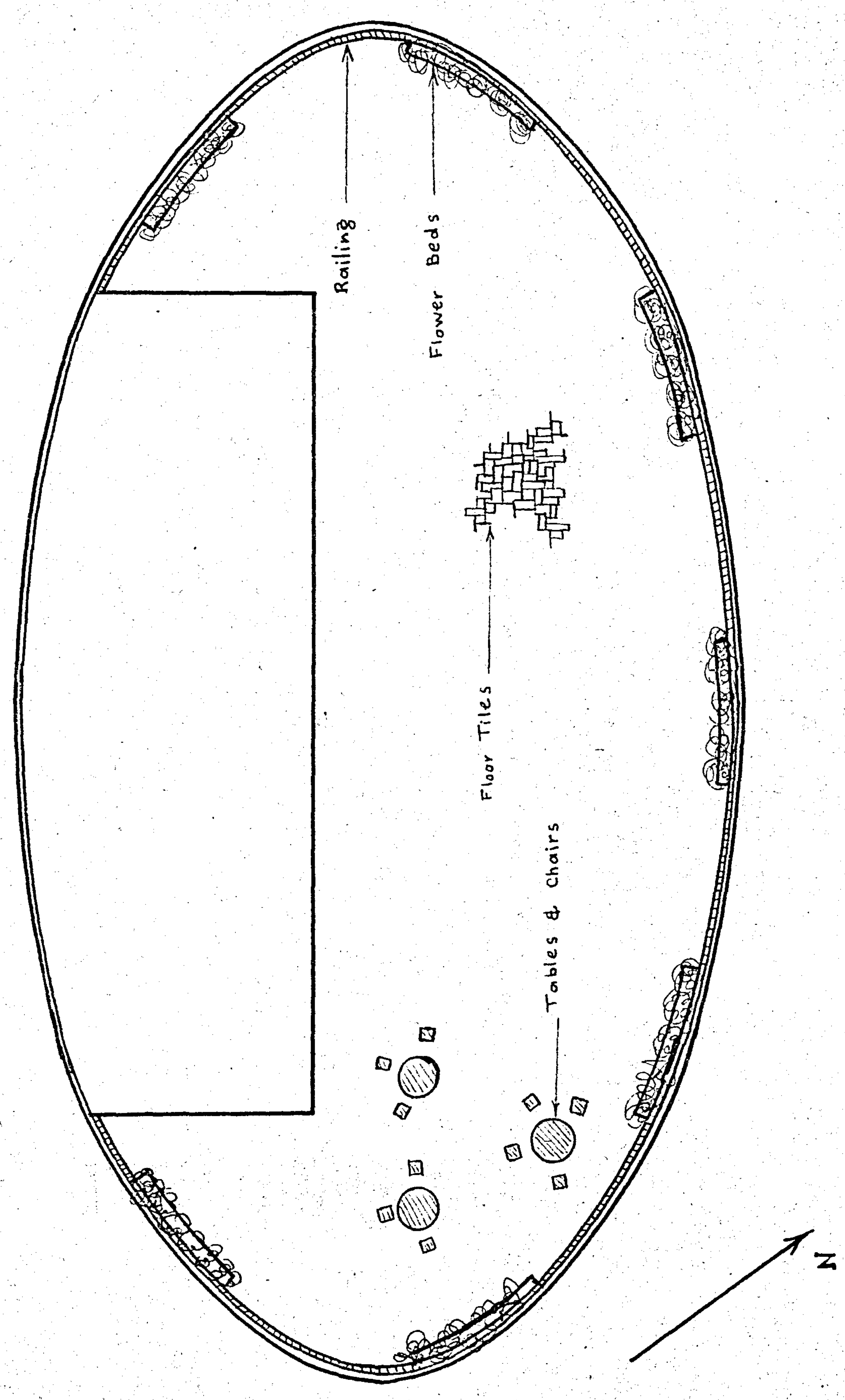


Fig. IV-4. Plan of the Casino's Terrace & Snack Bar (Scale 1/200)

Structural Design

Illustrations

* As this structure is important from several pointsof view, relatively high safety factors are going to be taken in the design.

* Due to the fact that full fixity might not be the case in fixed-fixed reinforced concrete beams, safety measures are going to be taken into consideration.

1- For fixed-fixed beams with uniformly distributed loads positive moment is taken equal to w /16 instead of wl /24.

2- For fixed-fixed beams with concentrated loads, the calculated positive moment will be multiplied by a factor of 1.5 for steel area calculation.

* Shears and moments in beams are taken with = total length of the beam, i.e. from center to center of supports.
* Steel used for the casino is high structural steel of 30,000 psi tensile strength.

* 28 day compressive strength of concrete used for the casino should have a minimum f'c = 3000 psi.

* Additional length of steel is indicated when necessary for embedment. There is no additional length when the steel is carried over from one member to the other.

* Steel of the same diameter and characteristics is preferred in most cases if it satisfies economic and structural conditions. The reason for this is to simplify the work and avoid mistakes of mixing of bars of different diameters during construction.

* All bars used are deformed bars except those used for stirrups, tie bars, or spiral reinforcement.

* All bending of bars including bent-up bars, stirrups, tie bars, and hooks should be made according to ACI Code.

* Wherever stirrup spacing is greater than 3 inches for No.3 bars it is O.K. Otherwise No.5 bars or higher are used.

* Steel chairs are used whenever necessary to keep steel in position.

* Steel bars are welded together whenever the design length exceeds the available standard length.

DESIGN OF THE SNACK BAR

For stability of structure and simplification of the design, the roof of the snack bar is going to be designed in such a manner so as to have the two columns carrying the roof under pure compression and to have the supporting beam free from torsion. This is because the area of the snack bar is not of major importance, and the smaller the area is the bigger will be the area of the terrace. So, the following procedure is followed, Fig. .

Section A-A:

$$\frac{wl^2}{2} = \frac{w(50)^2}{12}$$

$$= 20.4 \text{ ft.}$$

Section B-B:

Loads: L.L. $= w = 25$ lbs./sq.ft.

D.L. = D.L. of slab + D.L. of T-beams

D.L. slab = $0.33 \times 150 = 50$ lbs./sq.ft.

D.L. of T-beam varies with distance from the support.

Calculate for P, Fig.IV-5.

(P is the sign load per T-beam, i.e. per three feet of slab).

$$P(15) + \frac{3 \times 50 \times (15)^2}{2} + \frac{3 \times 25\ (15)^2}{2} + 0.5\frac{(0.17+1.67)}{2}\frac{(15)^2}{3} \times 150$$

$$= \frac{3 \times 50\ (17)^2}{2} + \frac{3 \times 25(17)^2}{2} + 0.5\frac{(0.17+1.67)}{2}\frac{(17)^2}{3} \times 150$$

P = 196 lbs /lin.ft.

P = 196 X 3 = 588 lbs / T-beam.

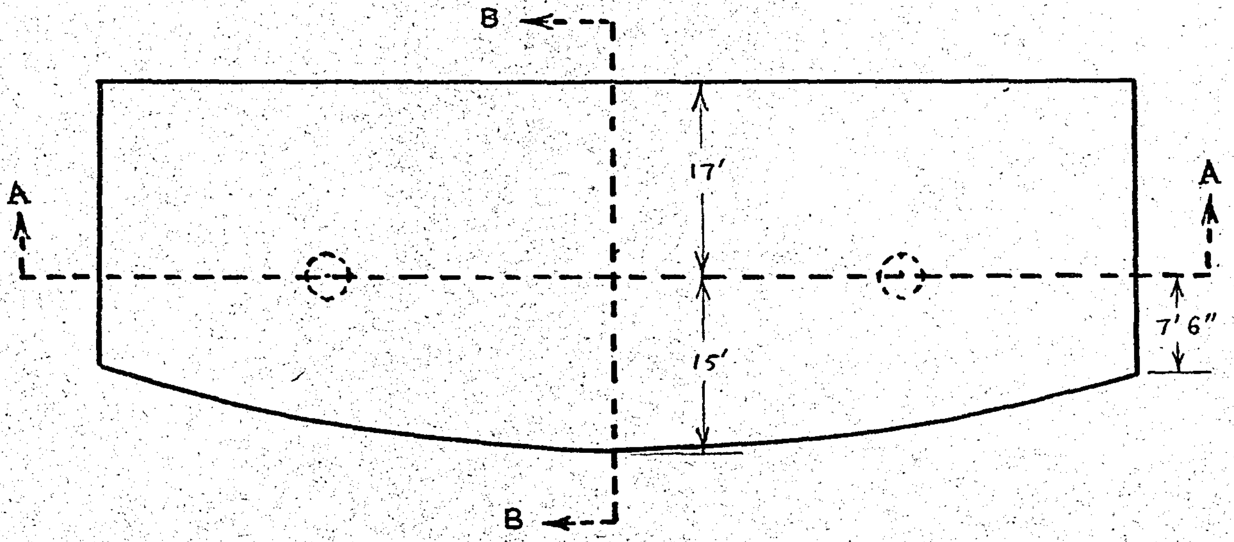


Fig. IV-5-a Plan of the Snack Bar Slab

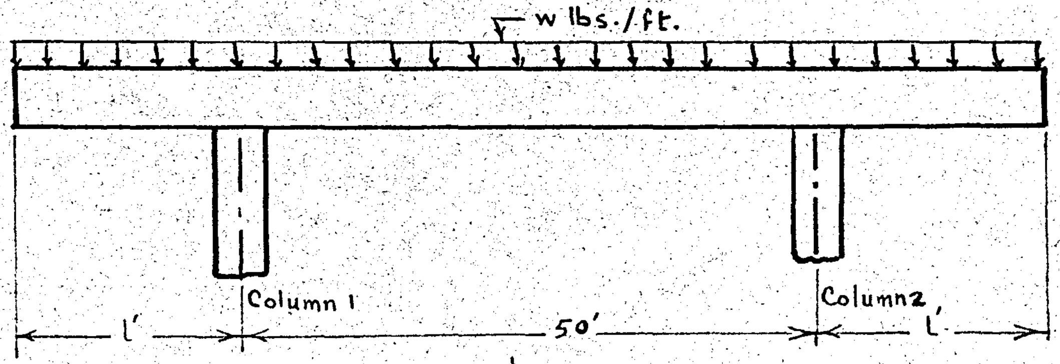


Fig. IV-5-b Section A-A

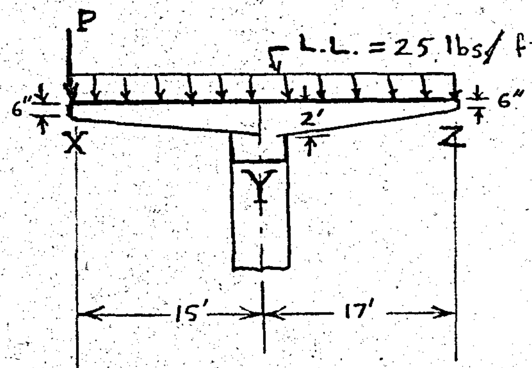


Fig. IV-5-c Section B-B

This load can be increased as we go away from the mid span of the center beam. The reason is because the T-beams under the sign are shorter as shown in Fig. IV-5.

* Design of T-beams:

One X-Y beam and one Y-Z beam are designed and the rest are the same Fig. IV-6 .

Beam X-Y

$$(-)\ M = 588(15) + \frac{3 \times 50(15)^2}{2} + \frac{3 \times 25(15)^2}{2} + 0.5\frac{(1.84)}{6}(15)^2(150)$$

$$= 39.4 \text{ kip ft.}$$

$$(-)As = \frac{M}{Jdf} = \frac{39.4 \times 12}{7/8 \times 20 \times 30} = 0.9 \text{ sq.in.}$$

use: 2 No.5 .... full length = 15'

1 No.5 .... half length = 8'

Spacing: Checks O.K.

Shear: $V = 588 + 3 \times 50 \times 15 + 3 \times 25\ (15) + 0.5\frac{(1.84)}{6}(15)(150)$

$$= 4307 \text{ lbs.}$$

$$v = \frac{V}{bJd} = \frac{4307}{6 \times 7/8 \times 20} = 41 \text{ psi} \angle 98 \text{ psi} \quad \text{O.K.}$$

Bond: $u = \frac{V}{oJd} = \frac{4307}{5.9 \times 7/8 \times 20} = 34.8 \text{ psi} \angle 210 \text{ psi} \quad \text{O.K.}$

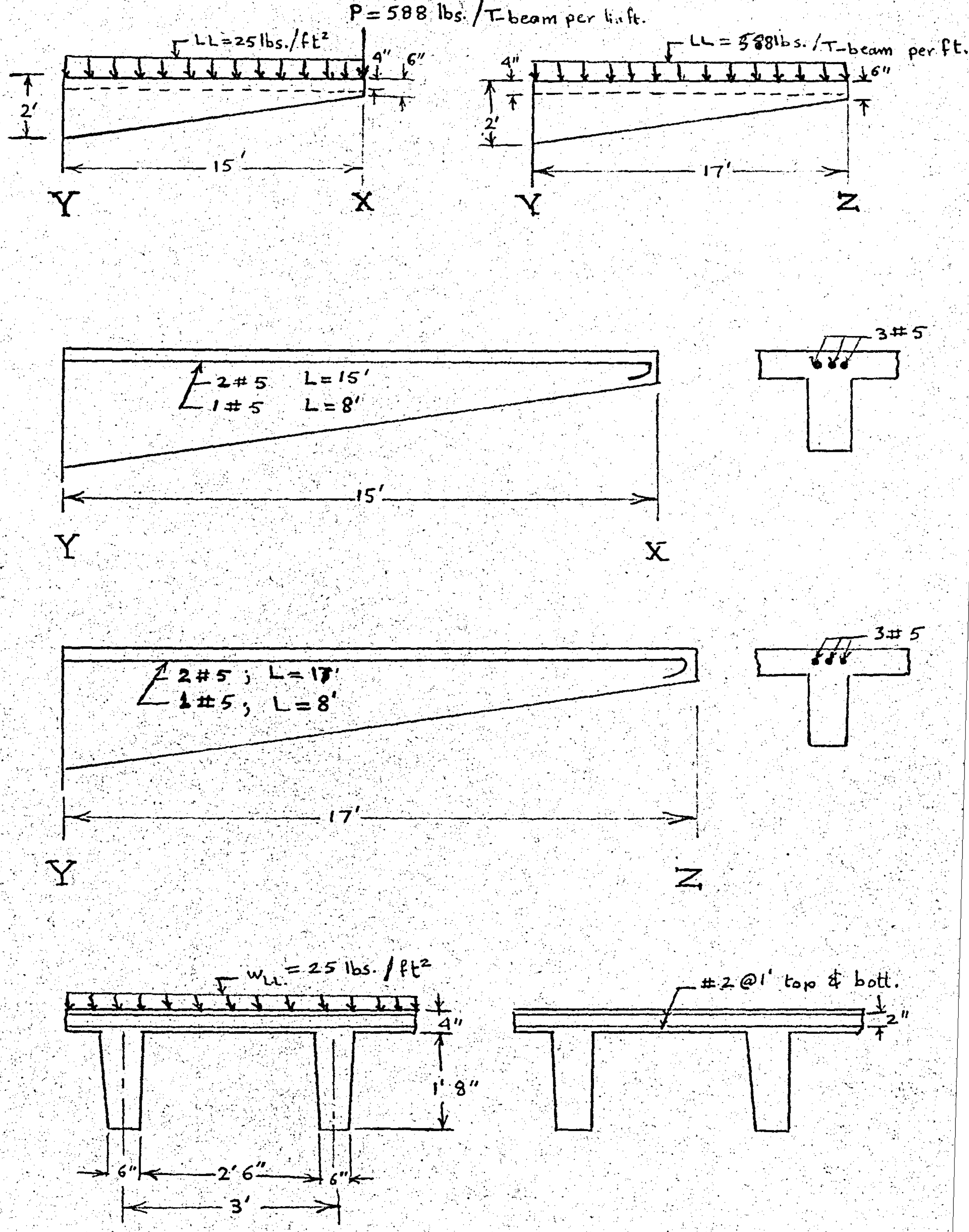


Fig. IV - 6 T-Beams of Snack Bar

Beam Y-Z

$$(-)\ \mu = \frac{3 \times (50)\ (17)^2}{2} + \frac{3 \times 25\ (17)^2}{2} + 0.5\left(\frac{1.84}{6}\right)(17)^2(150)$$

$$= 39.33 \text{ kip-ft.}$$

$$(-)As = \frac{M}{Jdf} = \frac{39.33 \times 12}{7/8 \times 30 \times 20} = 0.9 \text{ sq.in.}$$

use: 2 No.5 .... full length = 17'

1 No.5 .... half length = 9'

Spacing: Checks O.K.

Shear: $V = 3\times 50\ (17) + 3 \times 25(17) + 0.5\left(\frac{1.84}{2}\right)(17)(150)$

$= 4850$ lbs.

$v = \frac{V}{bJd} = \frac{4850}{6\times 7/8 \times 20} = 46.3$ psi $<$ 90 psi O.K.

Bond: $u = \frac{V}{OJd} = \frac{4850}{5.9\times 7/8\times 20} = 47.0$ psi $<$ 210 psi O.K.

Lateral Reinforcement Fig. IV-6.

Use 20 ksi steel for lateral reinforcement.

L.L. ..... w= 25 lbs/sq.ft.

D.L. ..... w= 0.3 X 150 = 50 lbs. /ft^2.

Total w= 75 lbs/ft $\longrightarrow$ 100 lbs./ft^2.

$(+)\ M = wl^2/16 = 100\ (3)^2/16 = 56$ ft-lbs.

$(-)\ M = wl^2/12 = 100\ (3)^2/12 = 75$ ft-lbs.

$(+)\ As = \dfrac{56 \times 12}{7/8 \times 3 \times 20{,}000} = 0.013$ sq.in/ lin.ft.

$(-) As = \dfrac{75 \times 12}{7/8 \times 3 \times 20{,}000} = 0.017$ sq.in/ lin.ft.

use No,2 at 1ft. top.
No.2 at 1ft. bottom.

* Design of Supporting (Major) Beams Fig.IV-5-b.

Loads: w_1 = max. shear at T-beams
= 4.307 + 4.850 = 9.157 kips/lin.ft.
w_2 = D.L. of beam
= 2 X 3 X 150 = o.9 kips/linft.
$w = w_1 + w_2$ = 10.057 kips/lin.ft.

Center beam:

$(+)\ M = \dfrac{wl^2}{16} = \dfrac{10.06\ (50)^2}{16} = 1560$ kip-ft.

$(-)\ M = \dfrac{wl^2}{12} = \dfrac{10.06\ (50)^2}{12} = 2080$ kip-ft.

$(+)\ As = \dfrac{1560 \times 12}{7/8 \times 30 \times 30} = 24$ sq. in.

$(-)\ As = \dfrac{2080 \times 12}{7/8 \times 30 \times 30} = 32$ sq. in.

(+)As use .... 9 No,10 Straight bott.
9 No.10 bent.
2 No.10 Straight bott.

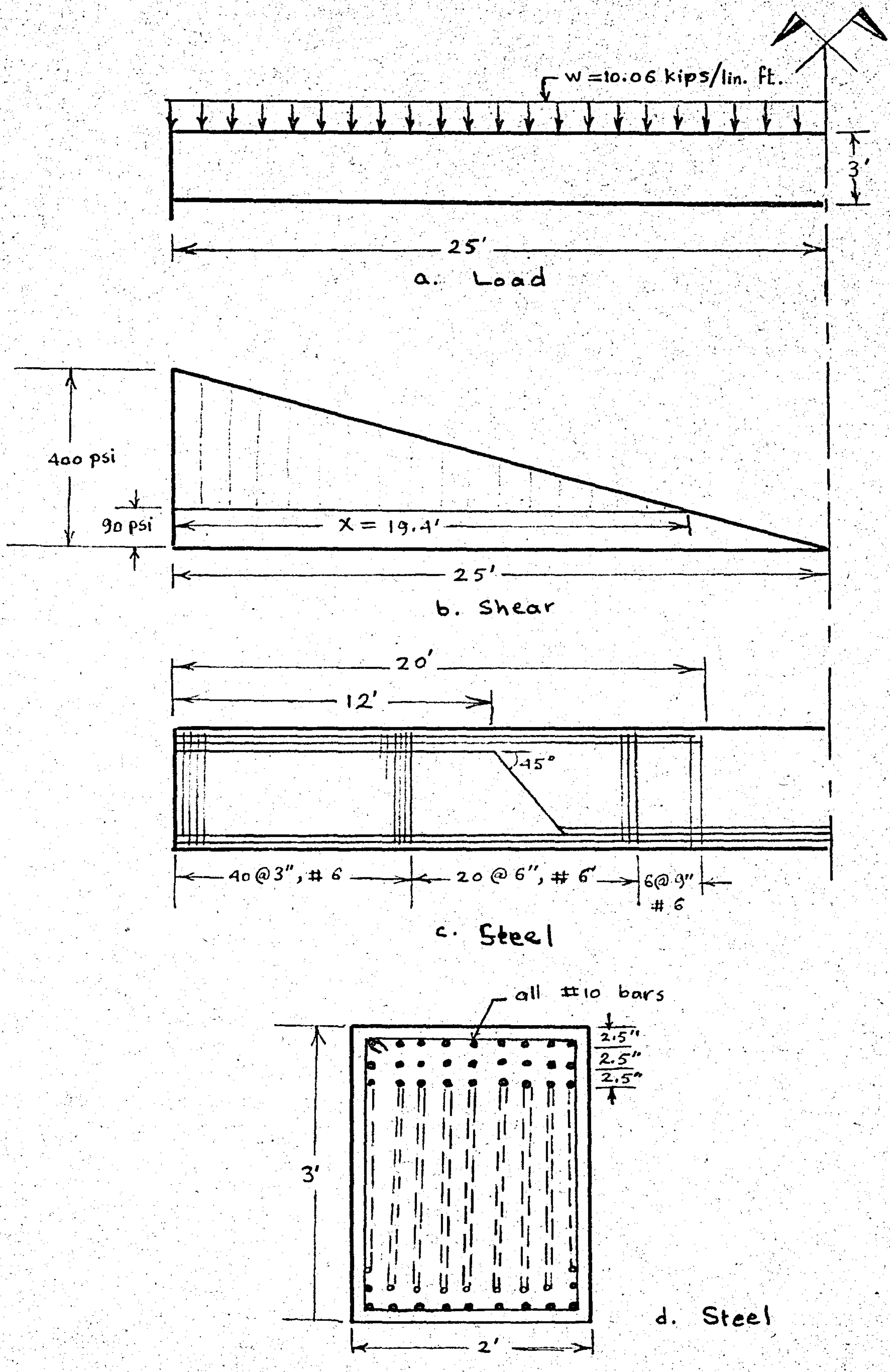


Fig. IV - 7 Center Beam of Snack Bar.

(–)As use .... 9 No. 10 top.

9 No. 10 top.

9 No. 10 bent.

Shear: $V = \frac{wl}{2} = \frac{10.06 \times 50}{2} = 251$ kips.

$v = \frac{V}{bJd} = \frac{251000}{24 \times 7/8 \times 30} = 400$ psi $>$ 90 psi.

use stirrups. Fig. IV-7.

$x = 25 \times \frac{310}{400} = 19.4$ ft.

Stirrups No.6 bars

$S = \frac{A_v F_v}{v' \ b} = \frac{0.88 \times 30000}{310 \times 24} = 3.6$ in. .... 3 in.

Bond: $u = \frac{V}{0Jd} = \frac{251000}{84.9 \times 7/8 \times 30} = 113$ psi $<$ 210 psi O.K.

Cantilever Beams:

(–) $M = \frac{wl^2}{2} = \frac{10.06 \ (20)^2}{2} = 2012.0$ kip-ft.

(–) $As = \frac{2012 \times 12}{7/8 \times 30 \times 30} = 30.6$ sq.in.

(–) As use .... 9 No. 10

9 No. 10

7 No. 10

carry over from center beam 25 bars.

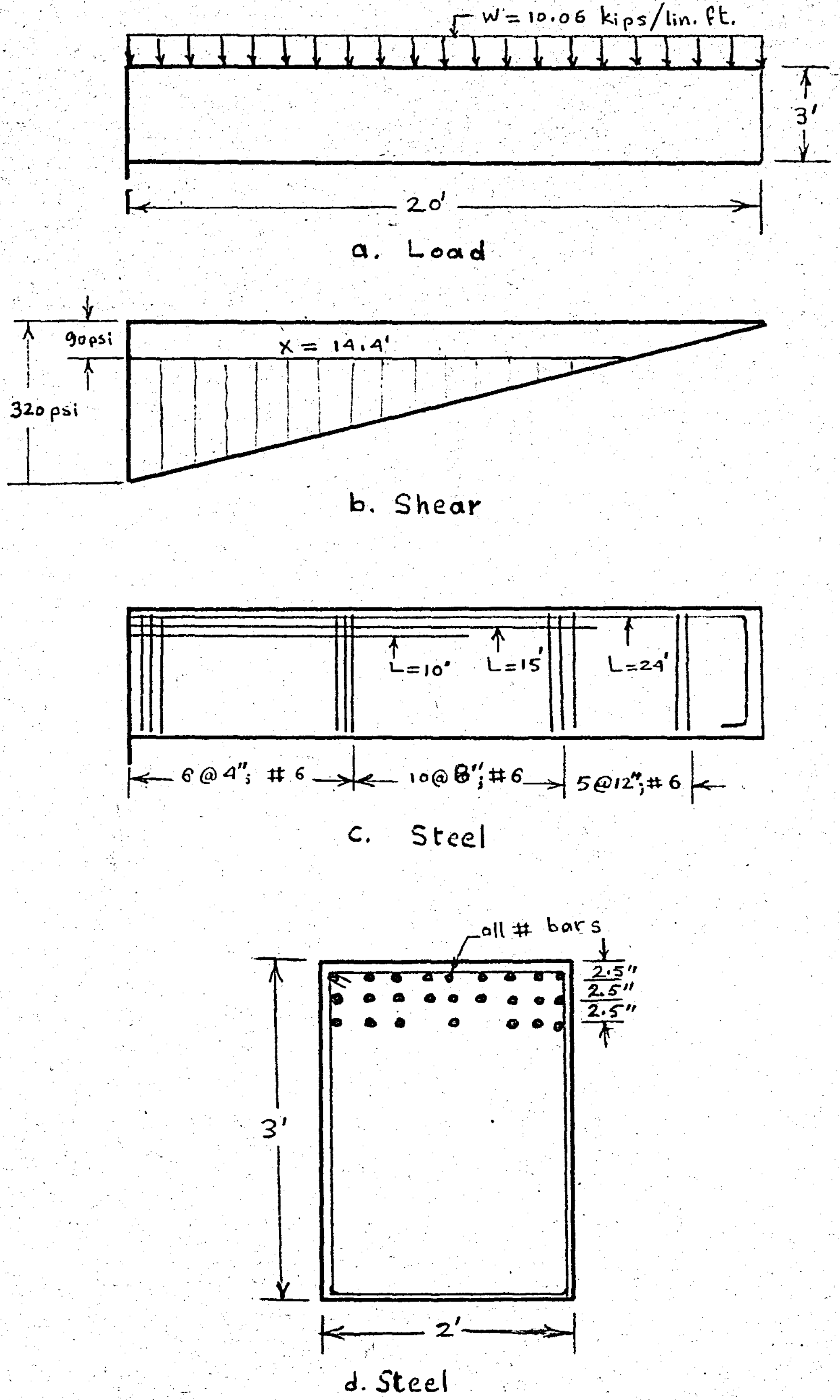


Fig. IV-8 Cantilever Beam of Snack Bar

Spacing: Checks O.K.

Shear: $V = wl = 10.06 \times 20 = 201{,}2$ kips.

$$v = \frac{V}{BJd} = \frac{201200}{24 \times 7/8 \times 30} = 320 \text{ psi} > 90 \text{ psi}$$

use Stirrups, Fig. IV-8.

$$x = 20 \times \frac{230}{320} = 14.4 \text{ ft.}$$

Stirrups No. 6 bars

$$S = \frac{A_v f_v}{v' b} = \frac{0.88 \times 30{,}000}{230 \times 24} = 4.8 \text{ in} \ldots 4 \text{ in.}$$

Bond: (similar to center beam) Checks O.K.

Design of Snack Bar Columns:

Load: (concentric)

$P = V_1 + V_2 = 251 + 201 = 452$ kips.

$A_g = \pi r^2 (12)^2 = 452$ in.

$f'c = 3000$ psi.

$fs = 30{,}000$ psi.

Vertical Reinforcement:

$$P = A_g (0.225 f'c + P_g f_s)$$

$$452000 = 452 (0.225 \times 3000 + 30{,}000 P_g)$$

$P_g = 1.12$ per cent

$As = 452 \times 0.0112 = 5.1$ sq.in.

Use 6 No.9 bars

Spiral Reinforcement:

$$P' = 0.45 \frac{A_g}{A_c} - 1 \quad \frac{f'c}{f's}$$

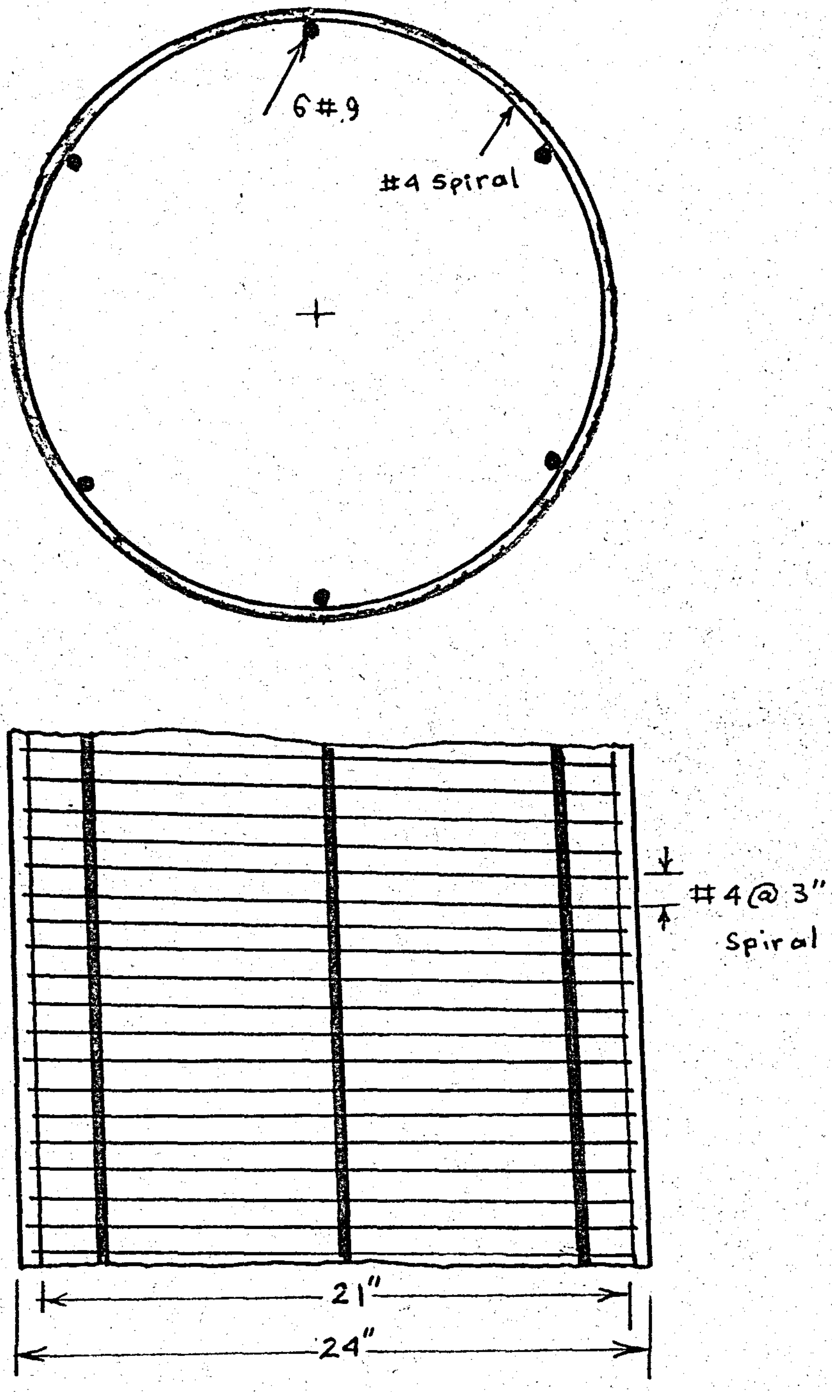


Fig. IV-9 Columns 1 & 2 of Snack Bar

$$A_c = \frac{\pi}{4}(24 - 3)^2 = 346 \text{ sq.in.}$$

$$P' = 0.45\left(\frac{452}{346} - 1\right)\frac{3000}{30000}$$

Volume of Spiral in one foot.

Length of Column $= 12 A_c P'$

$= 12(346)(0.0135) = 56$ cub.in.

Spacing: $\frac{d}{6} = \frac{21}{6} = 3.5$ in.

Max. spacing = 3 in. governs.

$\pi (21) \times \frac{12}{3} = 265''$ per foot.

Cross section $= \frac{56}{265} = 0.21$ sq.in.

Use No. 4 bars.

Bill of Materials of Snack Bar

Steel and Concrete

Member	No. of Parts	Bars	Shape	Length ft.	Wt/ft. lbs.	Total Wt. lbs.	Total Concrete ft^3
Floor XY	1 X30	2 No.5	Straight	15	1.043	940	3150
	1 X 30	1 No.5	"	8	1.043	250	
	1	32No.2	"	90	0.167	480	
Floor YZ	1 X 30	2 No.5	"	17	1.043	1060	3570
	1 X 30	1 No.5	"	9	1.043	282	
	1	32No.2	"	90	0.167	480	
Center Beam	1	11No.10	"	50	4.303	2380	300
	1	9 No.10	Bent	54	4.303	2110	
	1	18No.10	Straight	2X15	4.303	2340	
	1	66No.6	Stirrup	2X10	1.502	1980	
Cantilever Beams	2	9 No.10	Straight	20	4.303	1560	140
	2	9 No.10	"	15	4.303	1160	
	2	7 No.10	"	10	4.303	602	
	2	36No.6	Stirrup	10	1.502	1080	
Columns	2	6 No.9	Straight	15	3.400	6200	76
	2	23No.4	Spiral	14	0.668	430	
					Total	17654	7236

Design of Floors

The three floors of the major structure (the casino) seem from the structural point of view to be the most important part of the project.

Several proposals are going to be studied before the find decision is made.

The best two proposals are going to be worked out. For each one, the bill of materials, cost, and workmanship are going to be estimated. On the basis of these considerations the final design is going to be decided.

One should have in mind though, that the designs to be tried are not going to be carried to the end with full specific details. As our purpose here is to go on with design to such a point where we shall be able to judge which of the two designs is better.

The final design is going to be worked out in full details.

Proposal No. 1

The 1st proposal is a flat slab, where no beams are used.

"For ordinary spans with heavy loads, under average conditions, the flat-slab floor is more economical than the beam-and girder floor." (Ref. 1 P.299)

The floors in question have long spans with comparatively light loads, so the design of a flat slab is out of question.

Proposal No.2

This consists of a steel-girder system with ribbed floors. Fig. IV-10. "Ribbed floors are economical where the live loads are fairly small and the spans comparatively long." (Ref. 1 p.259)

This applies to the case of the casino floors and it will be tried.

Proposal No.3

Here we have the same thing as proposal No.2 but with reinforced concrete beams instead of steel-girders. As we have a small number of beams and as the spans are long this design doesn't seem to be economical and it will not be tried.

Proposal No.4

This is reinforced concrete beam system with ribbed floors. Fig. IV-

In this design there is an important point to be considered. As different sub-floors have triangular shapes the preparation of form work and the steel-pan fillers for the ribbs is going to be difficult and costly. This design doesn't seem to be preferable to others and will be neglected.

Proposal No.5

This consists of a criss-cross beam system. Fig. IV-12. Here there are major beams and minor beams. The minor beams are

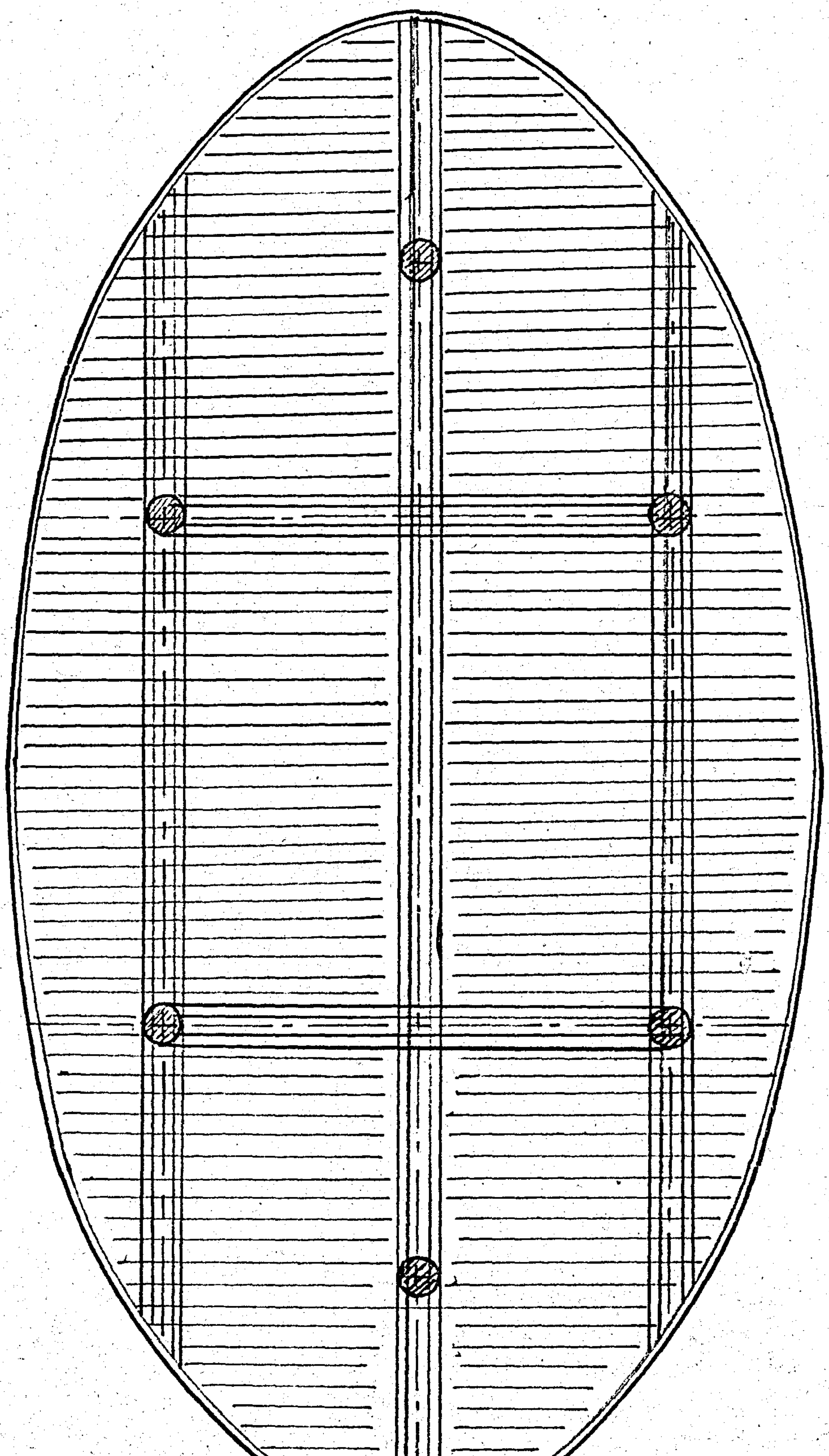


Fig IV-10 Proposal No: 2 (Scale 1/200)

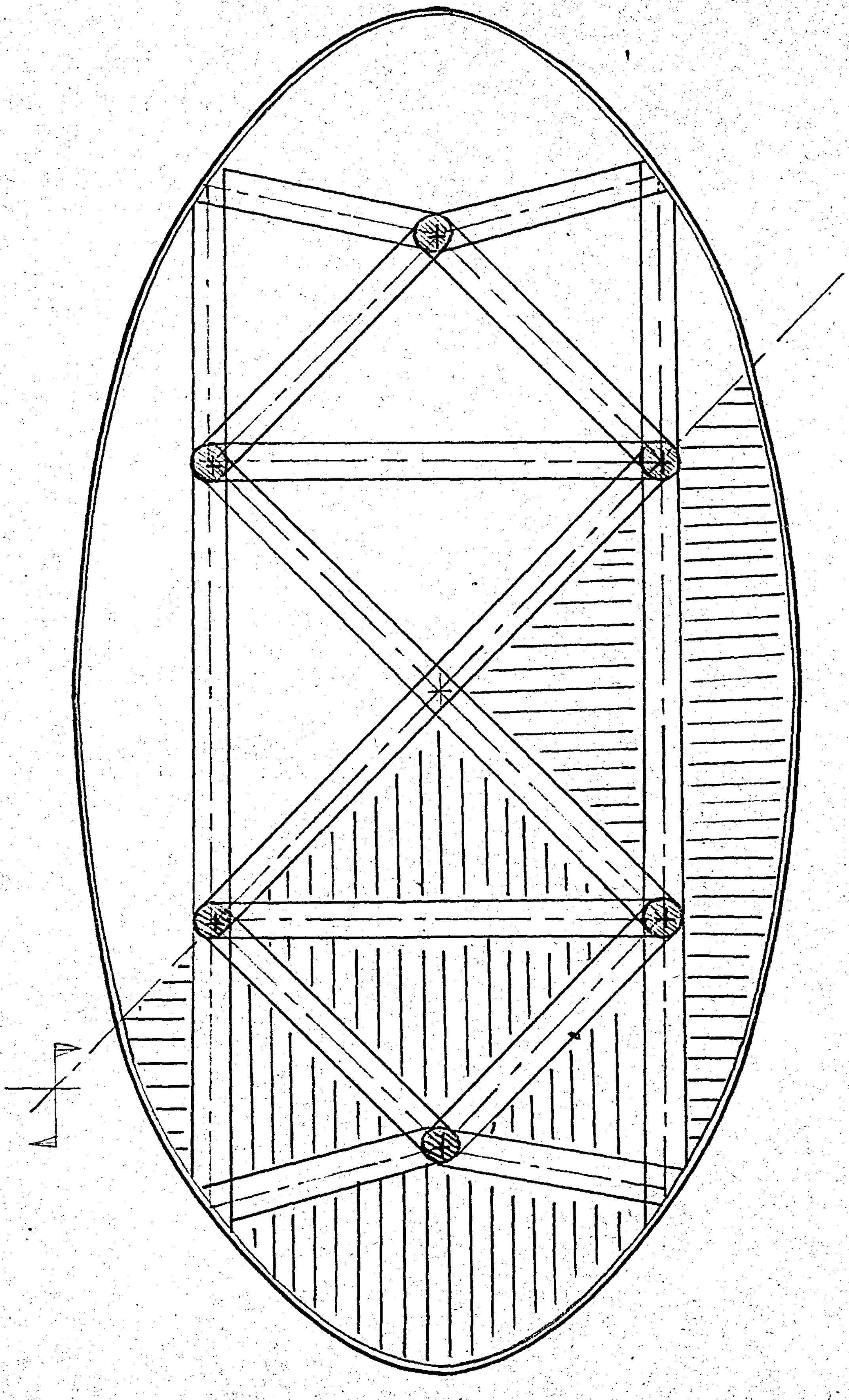


Fig. IV_11 Proposal No. 4 (Scale 1/200)

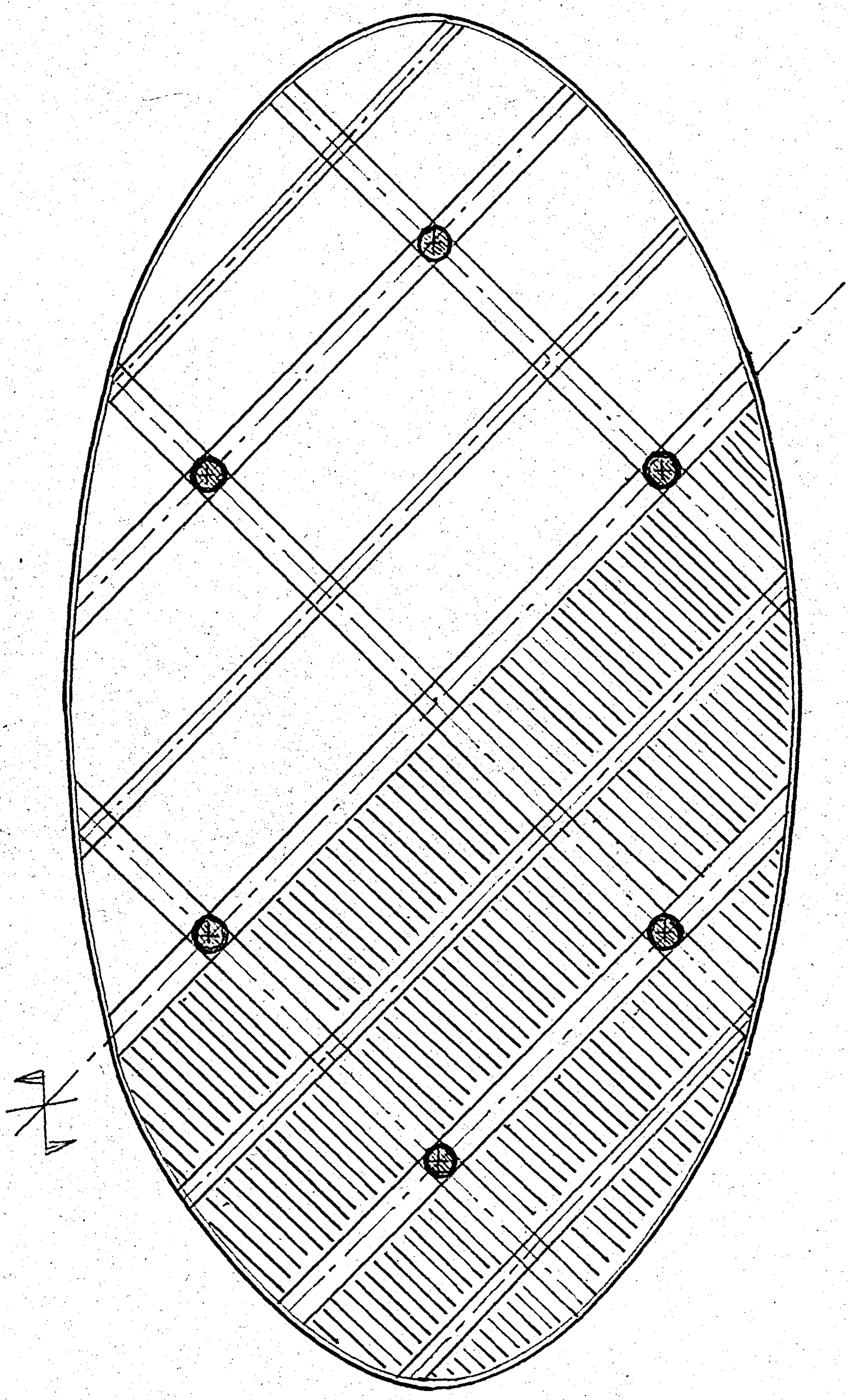


Fig. IV-12 Proposal No. 5 (Scale 1/200)

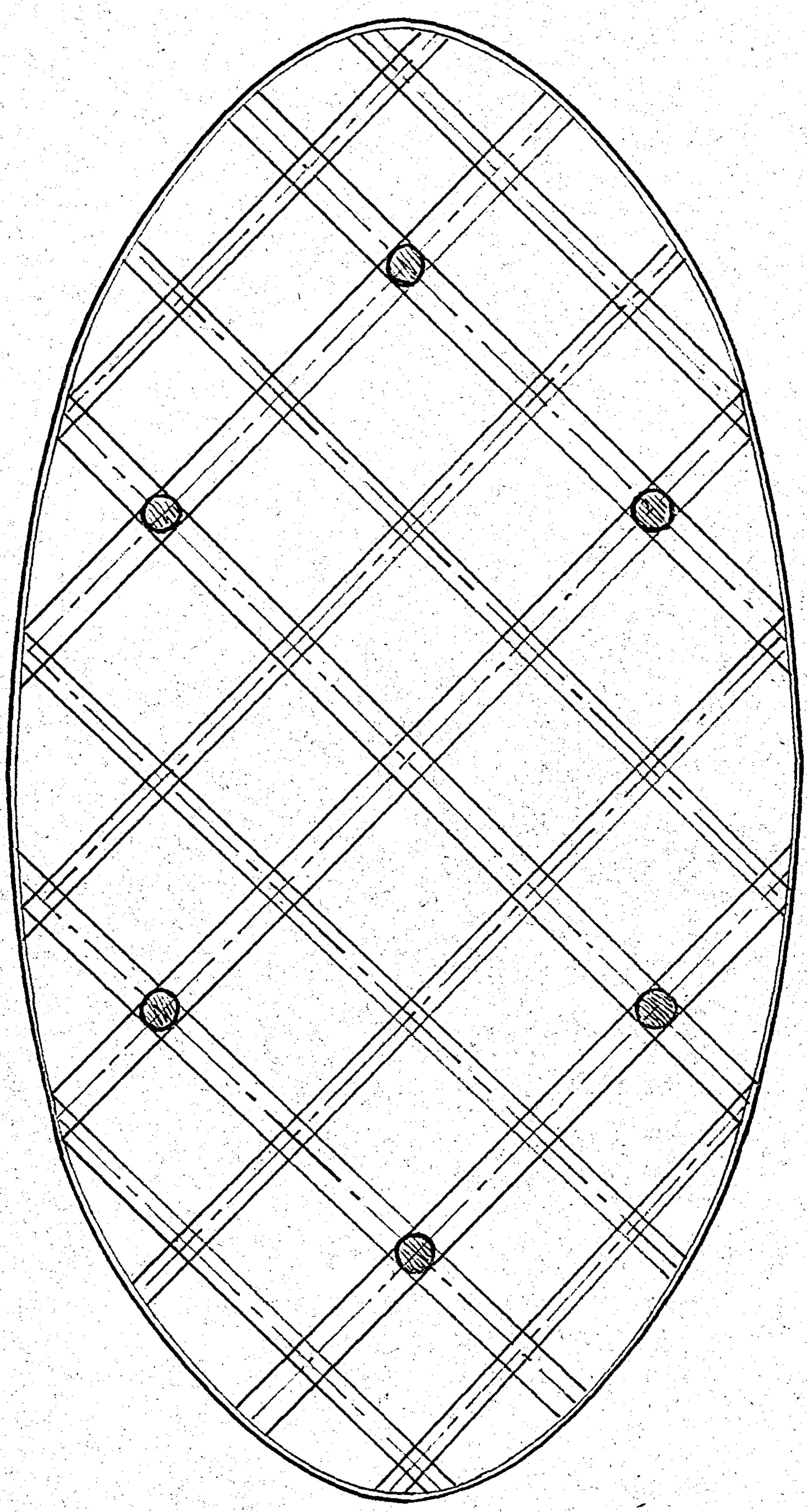


Fig. IV-13 Proposal No. 6 (Scale 1/200)

supported by major beams and the major beams are supported by columns. All crossings of beams are at right angles.

The floors consist of ribbs of medium lengths.

This design seems to be reasonable and economical. The form work including the steel pan fillers is straight forward.

The design of this floor is going to be worked out.

Proposal No.6

It is similar to proposal No.5 except that the number of minor beams is doubled. Fig.IV-13.

The floors consist of two-way slabs. Generally speaking ribbed floors are more economical than two-way slabs. Furthermore the No. of minor beams being double that of proposal No.5. It's reasonable to neglect this trial.

Proposal No.2

Illustrations

* For girder beams, the same kind of WF section will be used in most cases to secure perfect alignment and equal depths in continuous beams.

* Where the beams' negative or positive moment requires a bigger steel cross section, cover plates will be used.

* Connection of steel members may be either done by riveting or welding, whichever more practical and cheaper will be chosen.

* WF beams will be punched along the web to allow the bars of T-beam reinforcement to pass through from one panel to another.

* The same steel will be carried from one T-beam to another along the same axis as long as economy still prevails.

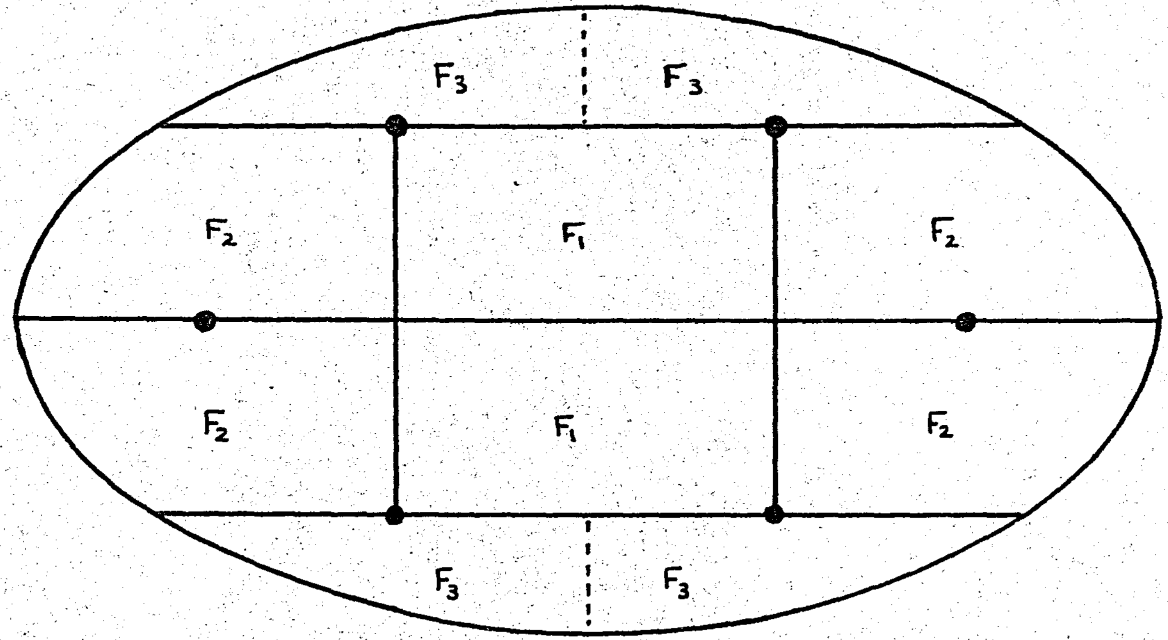


Fig. IV-14, a Floors of Proposal No. 2.

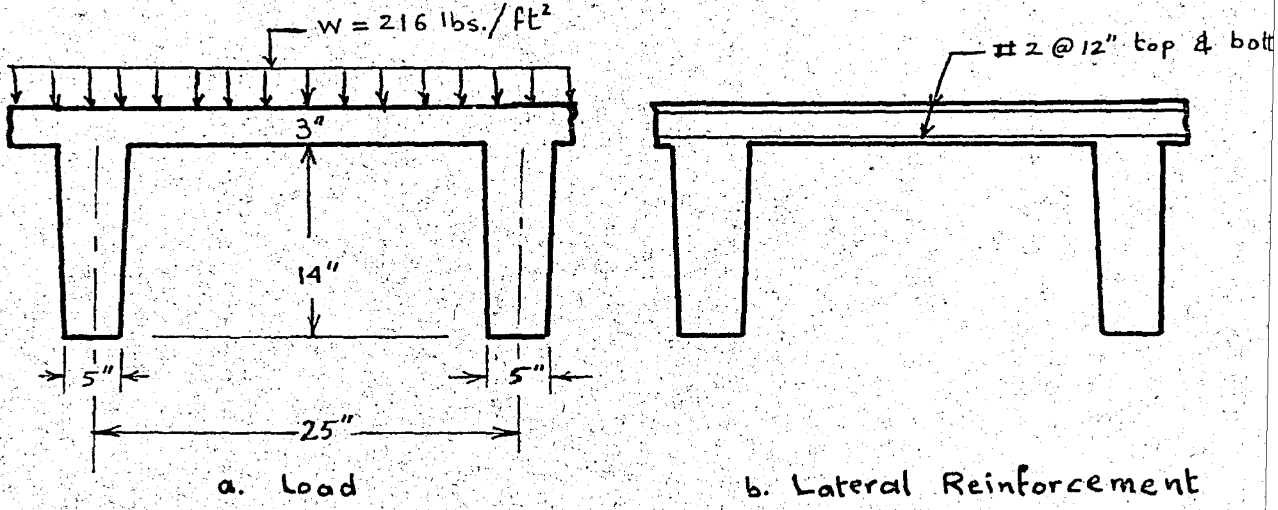


Fig. IV-14 b T-Beams of Floors of Proposal 2.

Floors

All floor ribbs will have a cross sectional dimensim as shown in Fig. IV-14-b.

Loads: w (D.L.) = 81 lbs./ sq.ft. .... Ref. 1

w (L.L.) = 100 lbs./sq.ft.

Partitions = 10 lbs./sq.ft.

Plastered Ceiling = 10 lbs./sq.ft.

Floor finish = 15 lbs./sq.ft.

Total Load = 216 lbs./sq.ft.

Design of Floor Ribbs

* Design of Floor 1   Fig. IV-15.

No. of T-beams = $\frac{50 \times 12}{25}$ = 24 beams.

L = 25 ft.

Load = 216 lbs./sq.ft.

w = $\frac{216 \times 25}{12}$ = 450 lbs/lin.ft.

Moments: $(+)M = \frac{wl^2}{16} = \frac{1}{16}(0.45)(25)^2 = 17.6$ kip-ft.

$(-)M = \frac{wl^2}{11} = \frac{1}{11}(0.35)(25)^2 = 25.6$ kip-ft.

Steel: $(-)As = \frac{(-)M}{Jdfs} = \frac{25.6 \times 12}{15.5 \times 7/8 \times 20} = 1.14$ sq.in.

$(+)As = \frac{(+)M}{Jdfs} = \frac{17.6 \times 12}{15.5 \times 7/8 \times 20} = 0.78$ sq.in.

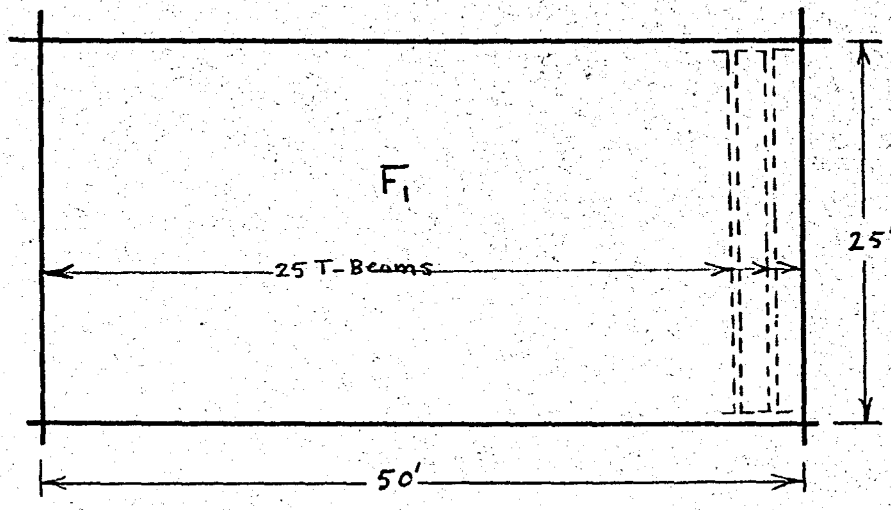


Fig. IV-15 Floor 1 of Proposal No. 2

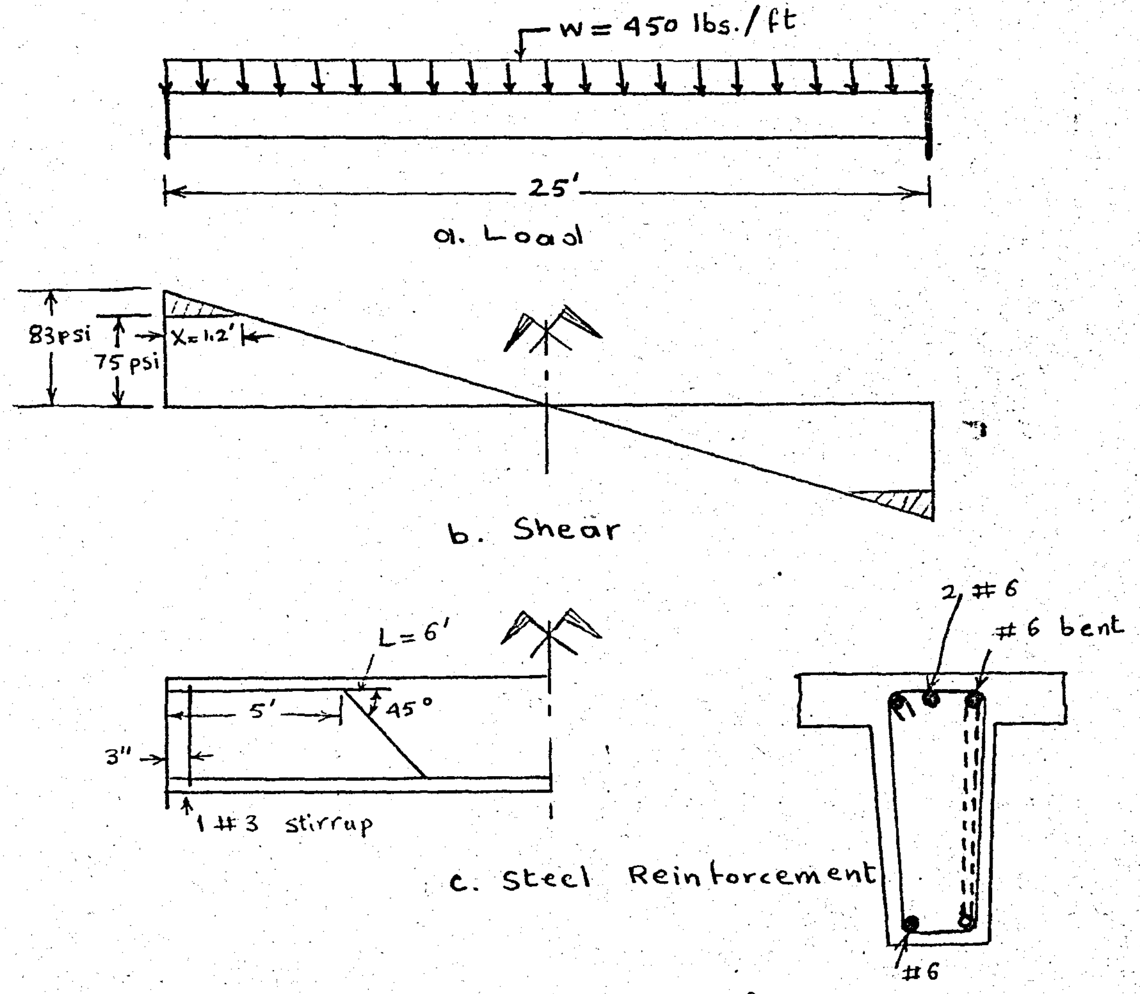


Fig. IV-16 T-Beams of Floor 1

(+)As .... 1 No.6 str. bott.

1 No.6 bent.

(-)As .... 1 No.6 bent.

2 No.6 str. top.

Shear: $V = \frac{wl}{2} = \frac{450 \times 25}{2} = 5625$ lbs.

$v = \frac{V}{bJd} = \frac{5625}{5 \times 7/8 \times 15.5} = 83$ psi $>$ 75 psi.

Use stirrups, Fig.IV-16.

$x = \frac{8 \times 12.5}{83} = 1.2$ ft.

Stirrups No.3 bars.

$S = \frac{A_v f_v}{v'b} = \frac{0.22 \times 20{,}000}{8 \times 5} = 110$ in.

Use .... 1 at 12 in. from supports.

Bond: $u \doteq \frac{V}{OJd} = \frac{5625}{7.1 \times 7/8 \times 15.5} = 59$ psi $<$ 175 psi O.K.

* Design of Floor 2 Fig. IV-17.

No. of T-beams:

$$o - 35 \text{ ft.} = \frac{35 \times 12}{25} = 17 \text{ beams.}$$

$$35 - 50 \text{ ft.} = \frac{15 \times 12}{25} = 6 \text{ beams.}$$

Design of the 17 fixed-fixed beams is similar to that in Floor 1.

Design of Cantilever Beams: Fig. IV-17.

Design of T-beams 1,2,& 3 = Fig. IV-17.

L (beam 1) = 23 ft.

w = 450 lbs/lin.ft. (similar to F_1)

Moments: $(-)M = \frac{wl^2}{2} = \frac{0.45\ (23)^2}{2} = 118$ kip-ft.

Steel: $(-)As = \frac{(-)M}{Jdfs} = \frac{118 \times 12}{7/8 \times 15 \times 20} = 5.4$ sq.in.

(–)As .... 3 # 9

3 # 8

Shear: $V = wl = 23 \times 450 = 10300$ lbs.

$$v = \frac{V}{bJd} = \frac{10300}{5 \times 7/8 \times 15} = 157 \text{ psi} > 75 \text{ psi.}$$

Use stirrups, Fig. IV-18.

$$x = \frac{23 \times 83}{157} = 12.2 \text{ ft.}$$

Stirrups No.3 bars

$S = \frac{Avfv}{v'b} = \frac{0.22 \times 20{,}000}{82 \times 5} = 10.9$ in. .... 10 in.

Use .... 28 No.3 at 10 in.

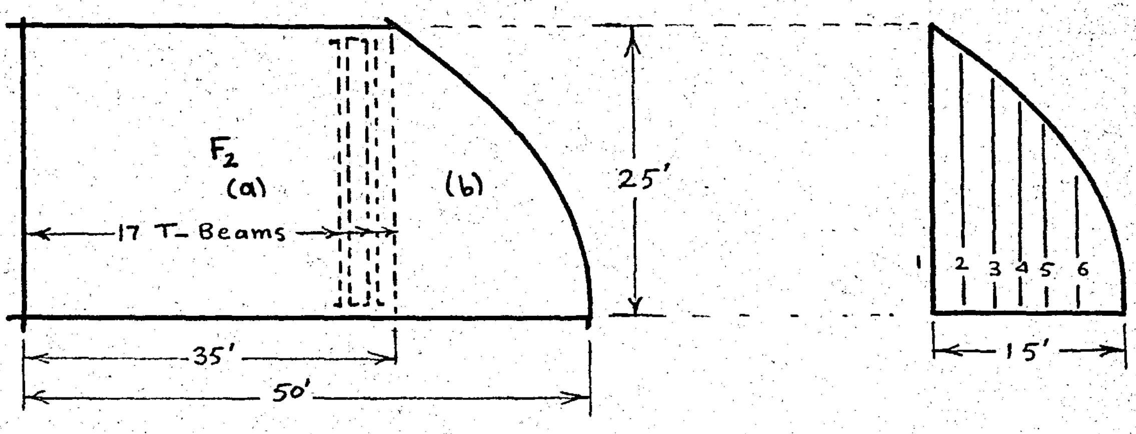


Fig. IV-17 Floor 2 of Proposal No. 2

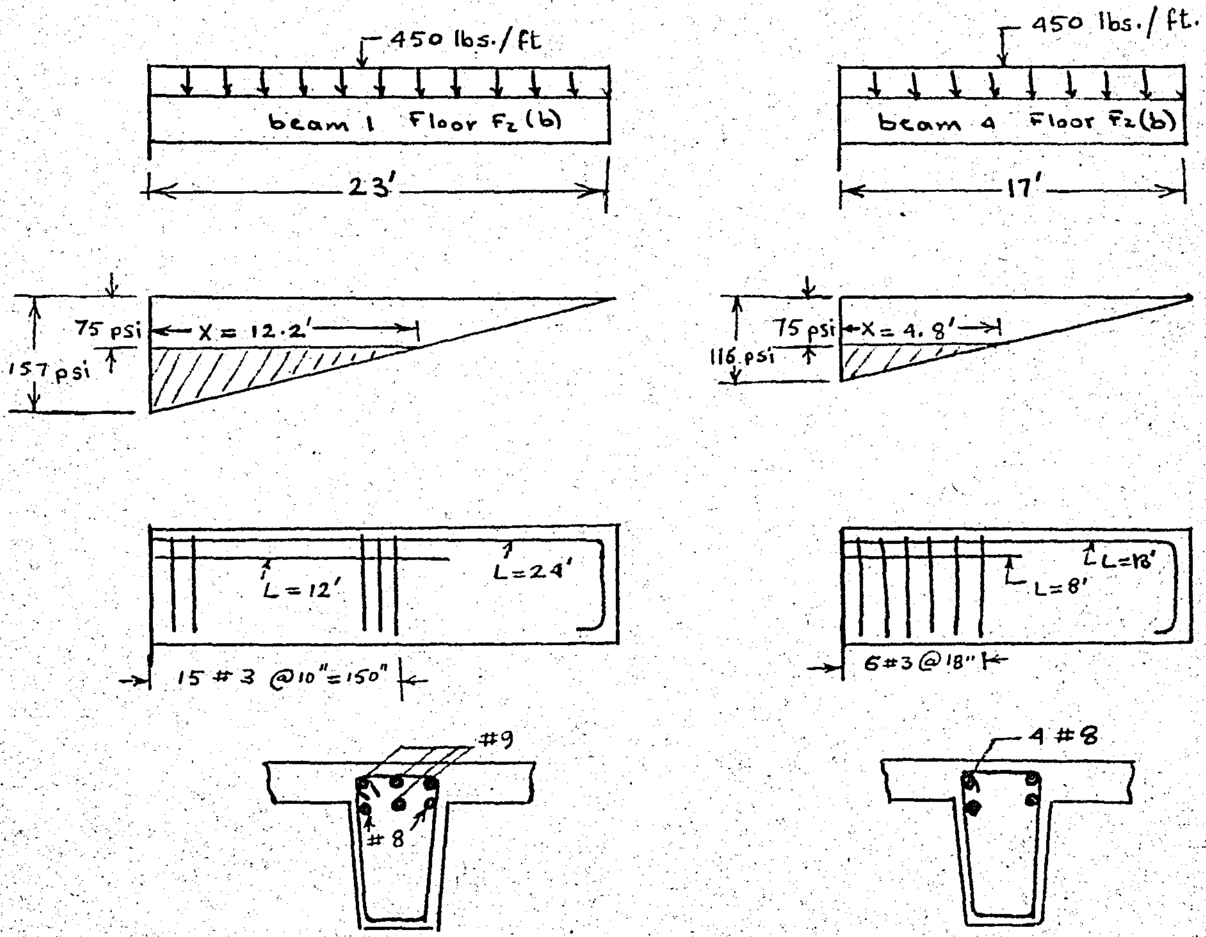


Fig. IV-18 T-Beams of Floor 2 (b)

Bond: $u = \frac{V}{OJd} = \frac{10300}{20.5 \times 7/8 \times 15} = 38.5$ psi 175 psi. O.K.

<u>Design of T-beams 4,5, & 6</u>

L = (beam 4) = 17 ft.

w = 450 lbs/lin.ft. (similar to F_1).

Moment: $(-)M = \frac{wl^2}{2} = \frac{0.45\,(17)^2}{2} = 64$ kip-ft.

Steel: $(-)As = \frac{(-)M}{Jdfs} = \frac{64 \times 12}{7/8 \times 15 \times 20} = 2.96$ sq.in.

(-)As 2 No.8

2 No.8

Shear: $V = wl = 17 \times 450 = 7660$ lbs.

$v = \frac{V}{bJd} = \frac{7660}{5 \times 7/8 \times 15} = 116$ psi $>$ 75 psi.

Use stirrups, Fig. IV-18.

$x = \frac{116 \times 17}{41} = 4.8$ ft.

Stirrups No.3 bars:

$S = \frac{Avfv}{v'b} = \frac{0.22 \times 20{,}000}{41 \times 5} = 21.5$ in.

Use .... 4 No.3 at 22 in.

Bond: $u = \frac{V}{oJd} = \frac{7660}{12.6 \times 7/8 \times 15} = 46$ psi $\angle$ 175 psi O.K.

* Design of Floor 3 Fig. IV-19.

No. of T-beams $= \frac{(60-1)\ 12}{25}$ 28 beams.

$w = 450$ lbs/sq.ft. (similar to F_1).

Design of T-beams 1 - 14

L (beam 1) $= 15$ ft.

Moment: $(-)M = \frac{wl^2}{2} = \frac{0.45\ (15)^2}{2} = 51$ kip-ft.

Steel: $(-)As = \frac{(-)M}{Jdfs} = \frac{51 \times 12}{7/8 \times 15 \times 20} = 2.34$ sq.in.

$(-)As$ 3 No.8

Shear: $V = wl = 15 \times 450 = 6750$ lbs.

$v = \frac{V}{Jdfs} = \frac{6750}{5 \times 7/8 \times 15} = 103$ psi $>$ 75 psi.

Use stirrups, Fig. IV-20.

$x = \frac{15 \times 28}{103} = 4.1$ ft.

Stirrups. No.3 bars.

$S = \frac{Avfv}{v'b} = \frac{0.22 \times 20{,}000}{28 \times 5} = 31.5$ in.

Use ...... 3 No.3 at 18 in.

Bond: $U = \frac{V}{OJd} = \frac{6750}{9.4 \times 7/8 \times 15} = 56$ psi $\angle$ 175 psi O.K.

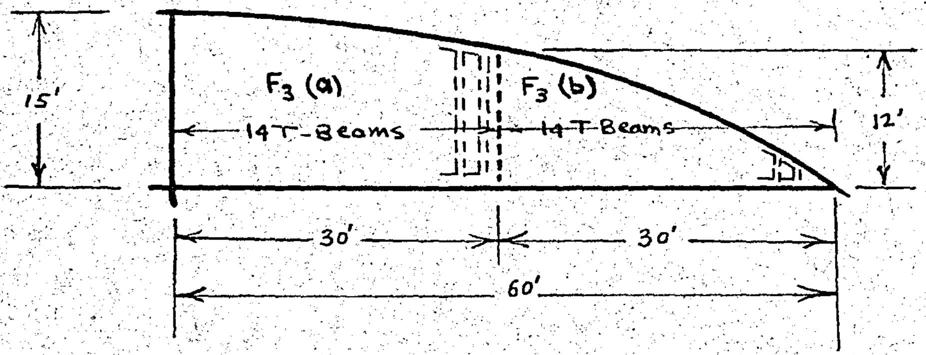


Fig. IV-19 Floor 3 of Proposal No. 2

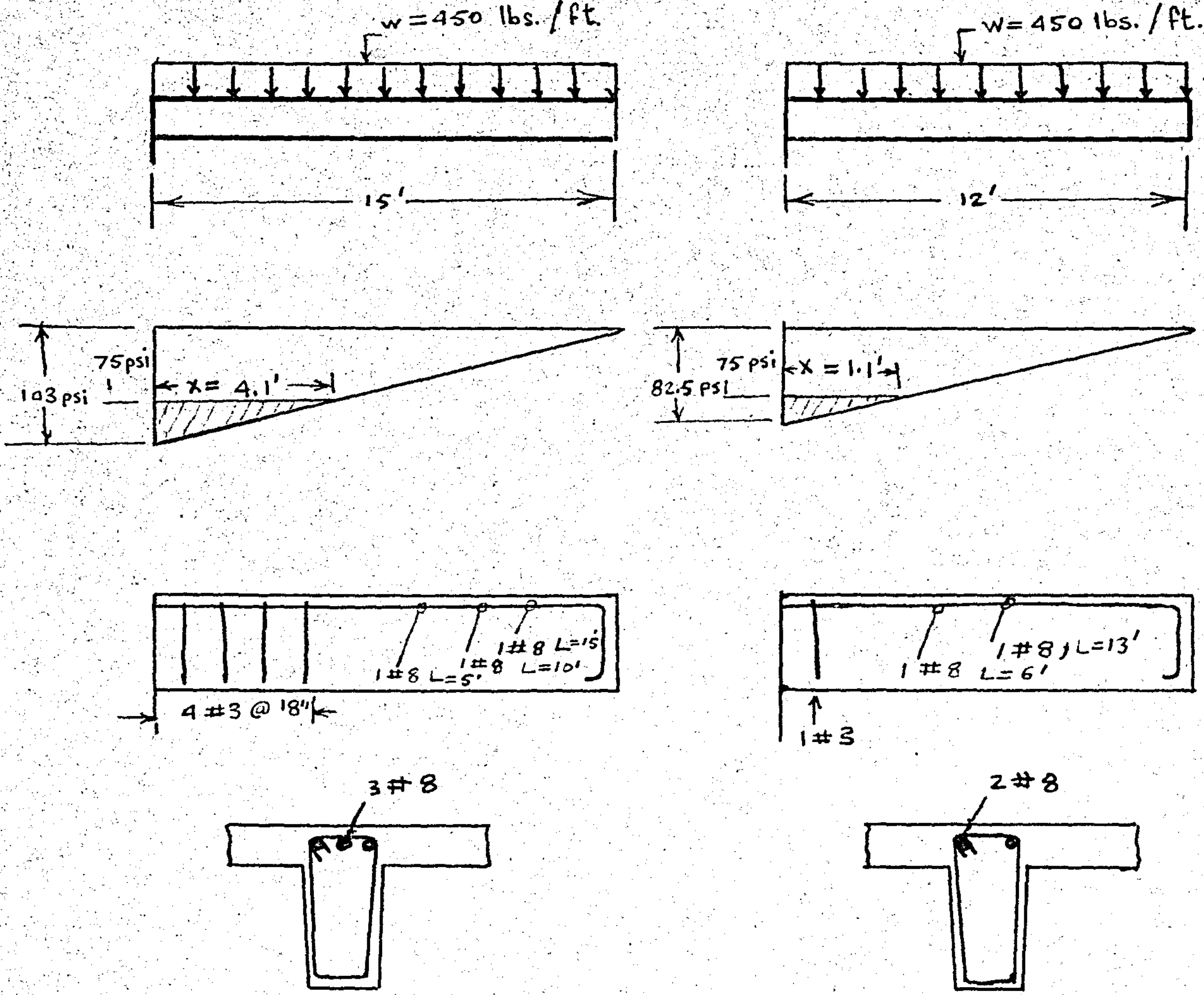


Fig. IV-20 T-Beams of Floor 3

Design of T-beams 15 – 28

L (beam 15) = 12 ft.

Moment: $(-)M = \frac{wl^2}{2} = \frac{0.45\ (12)^2}{2} = 32.4$ kip-ft.

Steel: $(-)As = \frac{(-)M}{Jdfs} = \frac{32.4 \times 12}{7/8 \times 15 \times 20} = 1.48$ sq.in.

(-)As 2 No.8

Shear: $V = wl = 450 \times 12 = 5400$ lbs.

$v = \frac{V}{bJd} = \frac{5400}{5 \times 7/8 \times 15} = 82.5 \text{ psi} > 75 \text{ psi.}$

Use stirrups. Fig. IV 20.

$x = \frac{12 \times 7.5}{82.5} = 1.1$ ft.

Stirrups No. 3 bars

$S = \frac{Avfv}{v'b} = \frac{0.22 \times 20{,}000}{7.5 \times 5}$

Use 1 No.3

Bond: $u = \frac{V}{OJd} = \frac{5400}{6.3 \times 7/8 \times 15} = 65 \text{ psi} < 175 \text{ psi}$ O.K.

Lateral Reinforcement:

Total w = 216 lbs/sq.ft.

L (Flange) = 2 ft.

$(-)M = \frac{wl^2}{12} = \frac{216 X(2)}{12} = 72$ ft-lbs.

$(-)As = (+)As = \frac{M}{Jdfs} = \frac{72 X 12}{7/8 X 20,000} = 0.025$ sq.in.

(-)As .... No.2 at 12 in top.
No.2 at 12 in bottom.

Sum of Lateral Steel:

Area of Floor (approx.) $= \frac{2}{3}$ X length X width

$A = \frac{2}{3} X 150 X 80 = 8000 ft^2$.

L (Lateral steel) = 8000 X 2 = 16000

Sum of Concrete for Ribbs:

$V = \frac{8000 X 81}{150} = 4300 ft^3$.

Steel and Concrete for Floor Belt:

Width = 4 in.

Depth = 3 ft.

Steel = 4 No.3

L (belt's steel) = 400 X 4 = 1600 ft .

V (belt's concrete) = $400 ft^3$.

Girder Beams

Beam Analysis

Reaction Between Intersecting Beams: Fig. IV-22.

The deflections at the point of intersection in both beams are equated to each other and the force P is calculated.

*Deflection of the Long Beam

Deflection at P from "a", Fig. IV-23.

The beam is loaded with the M/EI diagram, (conjugate beam method).

Loading check:

$$\frac{w_2 l_2^2}{8EI} \times l_2 \times \frac{2}{3} \stackrel{?}{=} \frac{w_2 l_2^2}{12\,EI} l_2 \qquad \text{Checks O.K.}$$

Deflection: $$y_a = \left(\frac{3\,w_2 l_2^2}{32\,EI} \times \frac{2}{3}\right) \frac{3X}{8} - \frac{w_2 l_2^2 X}{12\,EI} \cdot \frac{X}{2}$$

$$= \frac{-7 w_2 l_2^4}{24(16)^2 EI} \quad \text{down.}$$

Deflection at P from "b", Fig. IV-23.

Total M_A = Total M_B

$$= \frac{P\,(3/4)\,l_2 (l_2/4)^2}{l_2^2} + \frac{P\,(3/4)^2\,l_2^2\,(l_2/4)}{l_2^2}$$

$$= \frac{3\,Pl_2}{16}$$

The beam is loaded with the M/EI diagram (conjugate beam method)

Loading Check:

$$\frac{3\,Pl_2^2}{16} \stackrel{?}{=} \frac{Pl_2}{4}(l_2/2) + \frac{Pl_2^2}{4} \cdot \frac{l_2}{4} \qquad \text{Checks O.K.}$$

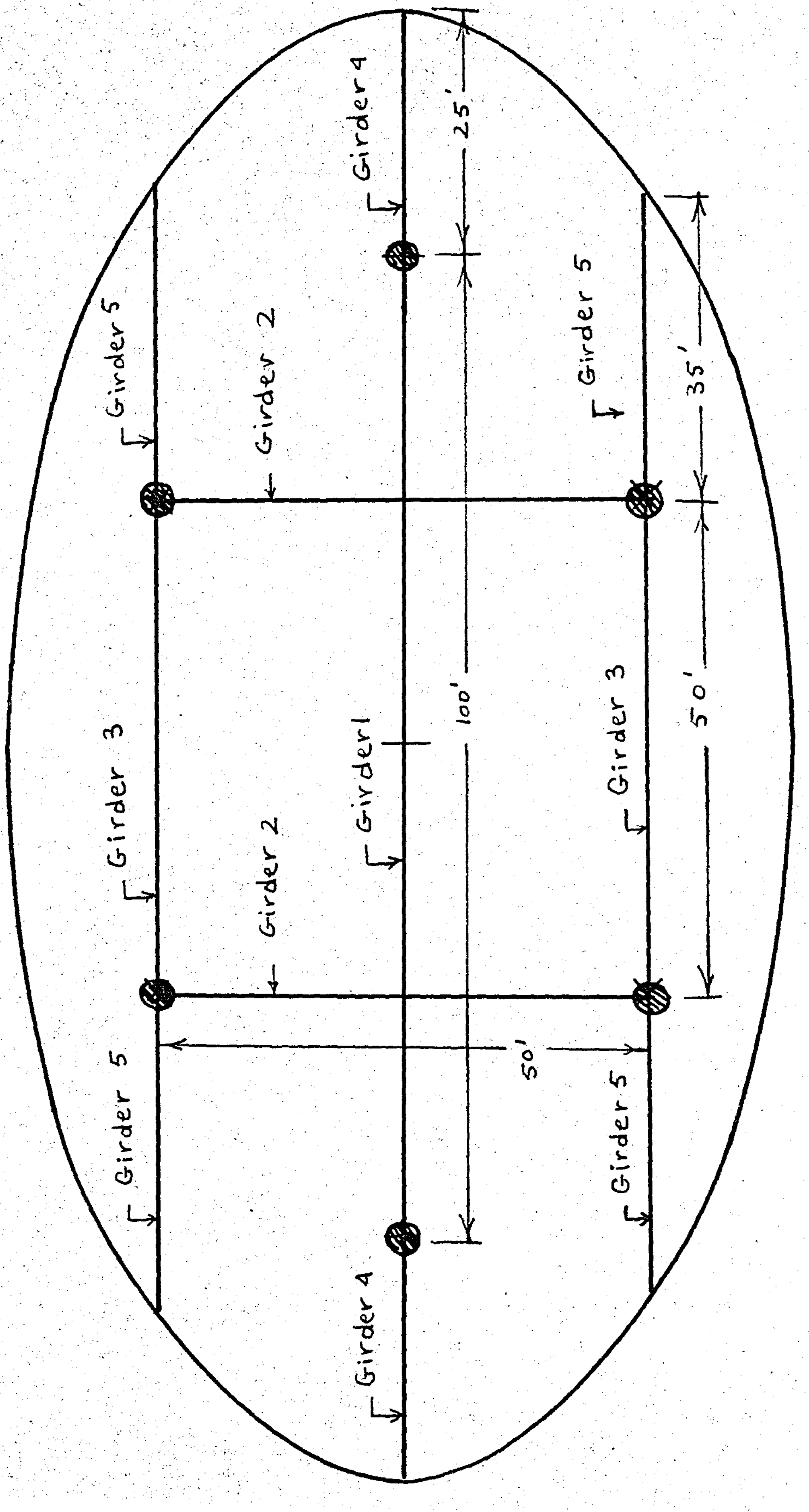


Fig. IV-21 Plan of Floor Indicating Girder Beams of Proposal No. 2.

Deflection $y_b = \frac{3\ Pl_2}{EI\ 16} \cdot \frac{l_2}{4} \cdot \frac{l_2}{8} - \frac{1}{EI} \cdot \frac{Pl_2}{4} \cdot \frac{l_2}{4} \cdot \frac{1}{2} \cdot \frac{l_2}{4} \times \frac{1}{3}$

$$= \frac{Pl^3}{12\ 8EI} \times \frac{5}{12}$$

Total deflection $y_2 = \frac{7\ w_2 l_2^4}{24\ (16)^2 EI} - \frac{Pl_2^3}{128EI} \cdot \frac{5}{12}$

Deflection of the Short Beam: Fig. IV 24.

$$y_1 = \frac{w_1 l_1^4}{384EI} + \frac{Pl_1^3}{192\ EI}$$ Ref. 6

Force of Reaction:

$y_1 = y_2$:

$$\frac{7\ w_2 l_2^4}{24(16)^2 E_2 I_2} - \frac{Pl_2^3}{128\ E_2 I_2} \cdot \frac{5}{12} = \frac{w_1 l_1^4}{384\ E_1 I_1} + \frac{P\ l_1^3}{192\ E_1 I_1}$$

Simplify:

$$P = \frac{\frac{7\ w_2 l_2^4}{32\ E_2 I_2} - \frac{w_1 l_1^4}{2\ E_1 I_1}}{\frac{l_1^3}{E_1 I_1} + \frac{5\ l_2^3}{8\ E_2 I_2}}$$

$E_1 = E_2$

$I_1 = I_2$

$l_1 = 2\ l_2$

$$P = \frac{350\ (w_2 - w_1)}{12}$$

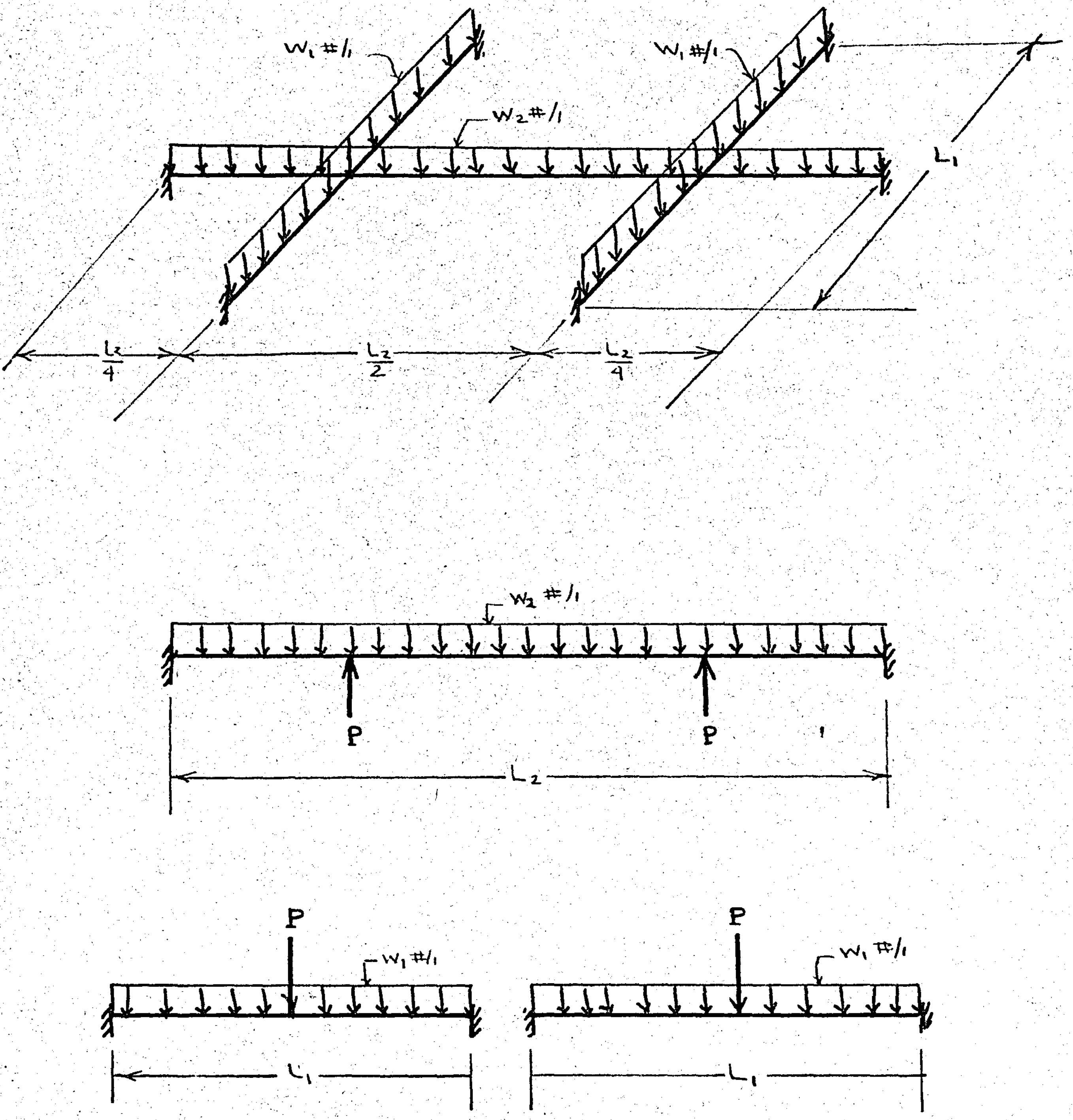


Fig. IV_22 Loads on Intersecting Beams

Loads on Beams:

Total Load w = 216 lbs./sq.ft. of floor.

Load on beam = load of floor + D.L. of beam

D.L. of beam (assumed) = 300 lbs/lin.ft.

Load on beam = 216 X 25 + 300 = 5700 lbs/lin.ft.

$$P = \frac{0.350 \; (w_2 - w_1)}{12}$$

$$P = \frac{0.350 \; (5700 - 300)}{12} = 157.5 \text{ kips.}$$

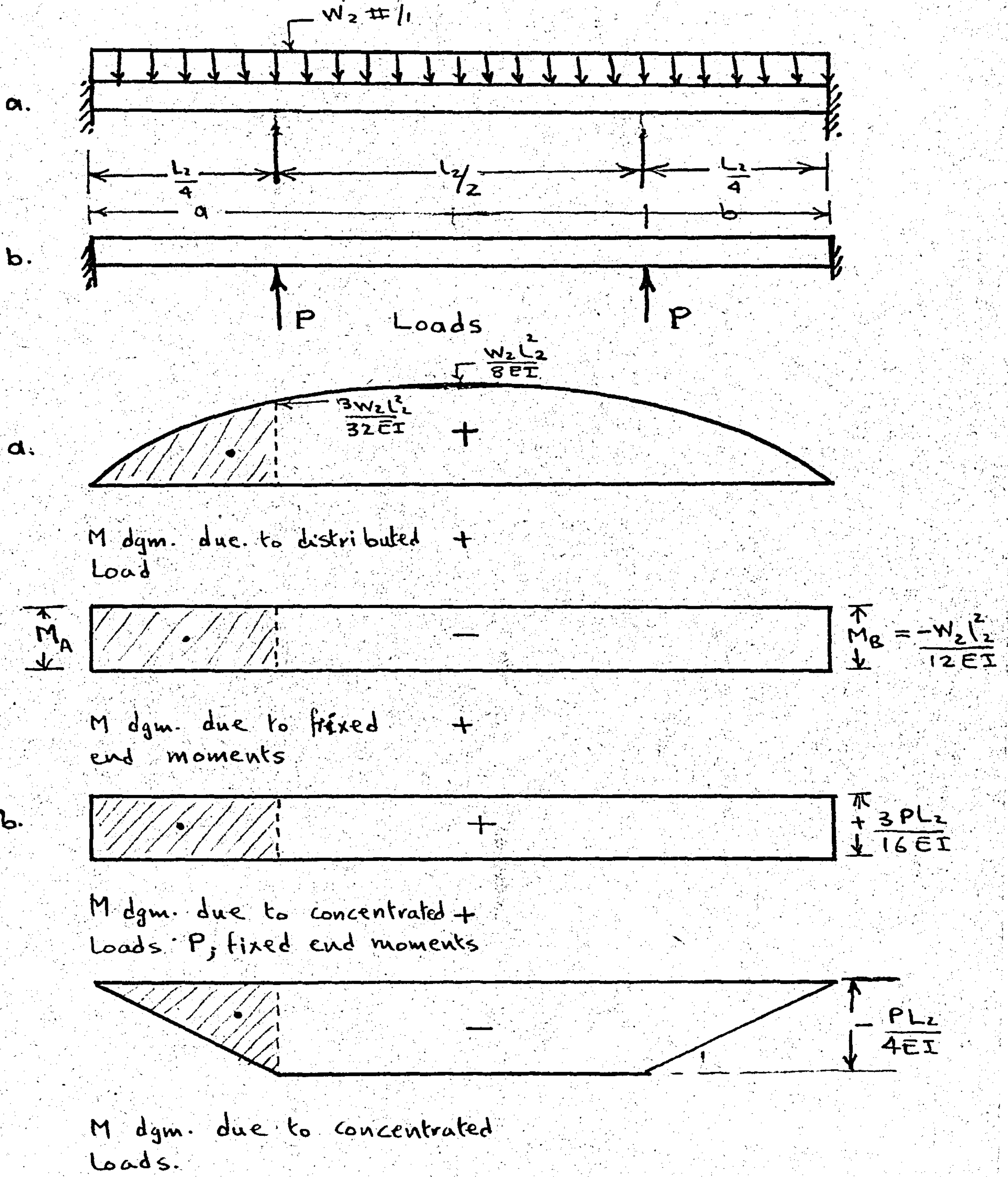


Fig. IV-23 Loads & M Diagrams of Beam L_2, Girder 1.

Design of Beam Girders

Design of Girder 1: Fig. .

Loads: Fig. IV-23

Moments: Fig. IV-23

$$(+)M = \frac{w_2 l_2^2}{24} - \frac{Pl_2}{4} - \frac{3Pl_2}{16}$$

$$= \frac{5.7\,(100)^2}{24} - \frac{157.5 \times 100}{16} = 1390 \text{ kip-ft.}$$

$$(-)M = -\frac{w_2 l_2^2}{12} + \frac{3\,Pl_2}{16}$$

$$= -\frac{5.7\,(100)^2}{12} + \frac{3\,(157.5)\,100}{16} = -1800 \text{ kip-ft.}$$

Steel: $(+)As = \frac{(+)M}{fsd} = \frac{1390 \times 12}{36 \times 20} = 23.7$ sq. in.

Use .... 36 WF 16 1/2

t = 1/2 in. corner plates 10 ft. long.

$A_{c.p.} = 30 - 23.8 = 6.2$ sq.in.

$t_{c.p.} = \frac{6.2}{16.2} = 0.38$ in 1/2 in.

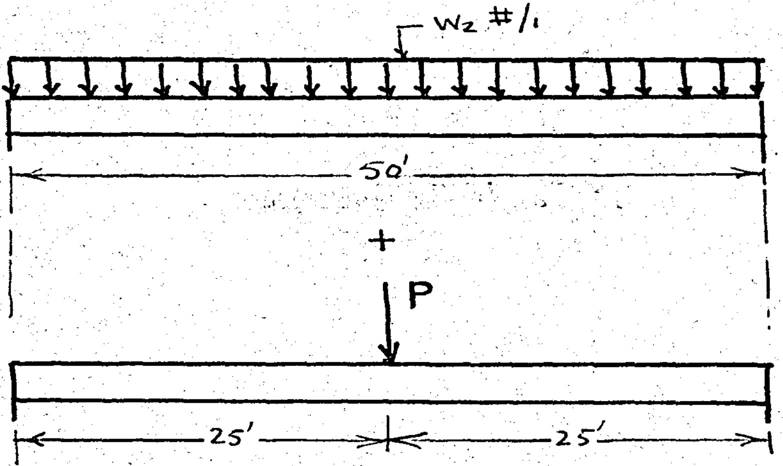
Design of Girder 2: Fig. IV-24.

Loads: Fig. IV-24.

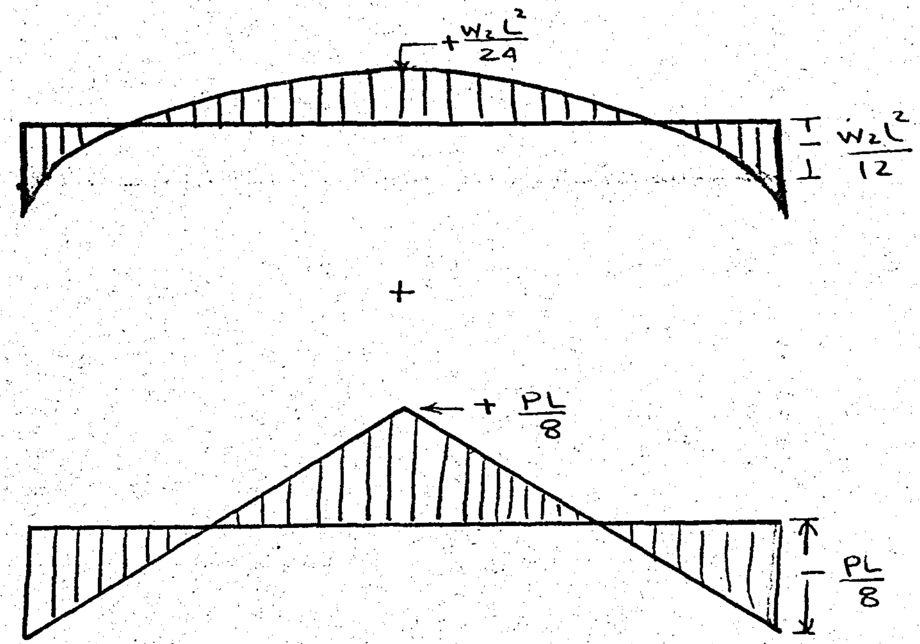
Moments: Fig. IV-24.

$$(+)M = \frac{w_2 l_1^2}{24} + \frac{Pl_1}{8}$$

$$= \frac{0.3\,(50)}{24} + \frac{157.5\,(50)}{8} = 1003 \text{ kip-ft.}$$



a. Loads



b. Moment Diagrams

Fig. IV 24 Loads & Moments on Beam L_1, Girder 2.

$$(-)M = \frac{Pl_1}{8} - \frac{w_2 l_1^2}{12}$$

$$= -\frac{157.5\ (50)}{8} - \frac{0.3\ (50)^2}{12} = -1035 \text{ kip-ft.}$$

Steel: $(+)As = \frac{(+)M}{fsd} = \frac{1003 \times 12}{36 \times 20} = 16.7$ sq.in.

Use .... 36 WF 16 1/2

$(-)As = \frac{(-)M}{fsd} = \frac{1035 \times 12}{36 \times 20} = 17.3$ sq.in.

Use .... 36 WF 16 1/2.

Design of Girder 3

Loads: Fig. IV - 25.

$$w = 0.216\ (\frac{25}{2} + 15) + 0.3 = 6.3 \text{ kips.}$$

Moments: Fig. IV-25.

$$(+)M = \frac{wl_1}{24} = \frac{6.3\ (50)}{24} = 656 \text{ kip-ft.}$$

$$(-)M = \frac{wl_1}{12} = -\frac{6.3\ (50)}{12} = 1312 \text{ kip-ft.}$$

Steel: $(+)\ As = \frac{+M}{fsd} = \frac{656 \times 12}{36 \times 20} = 10.93$ sq.in.

use .... 36 WF 16 1/2.

$(-)As = \frac{(-)M}{fs\ d} = \frac{1312 \times 12}{36 \times 20} = 21.86$ sq. in.

use .... 36 WF 16 1/2.

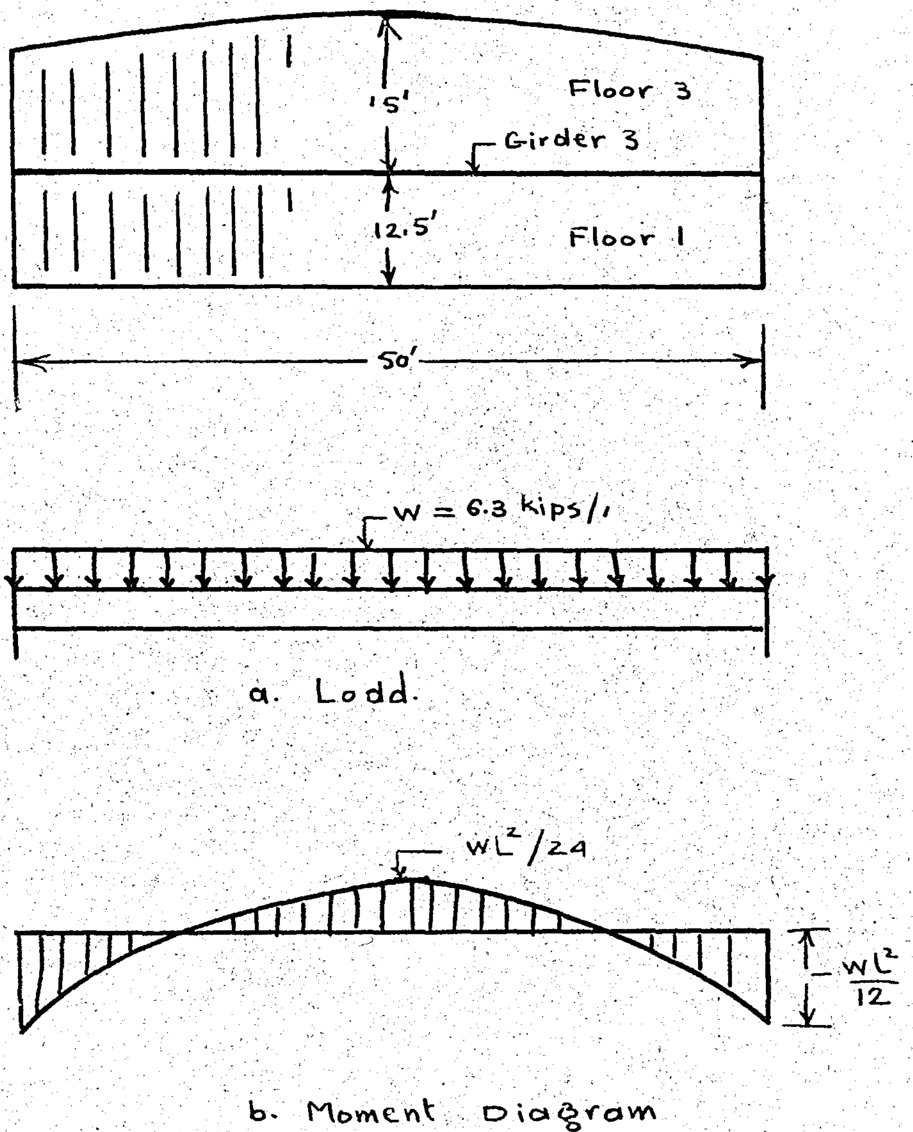


Fig. 25 Loads & Moment Diagrams of Girder 3

Design of Girder 4

Loads: Fig. IV - 26.

Moments: Fig. IV - 26.

$$(-)M = -\frac{25\ (216)(8)^2}{2} - \frac{50}{2} \times 216 \times 17 \times 13 - \frac{30\ (25)^2}{2}$$

$$= -\ 145 \text{ kip-ft.}$$

Steel: $(-)As = \frac{(-)M}{fs\ d} = \frac{1451 \times 12}{36 \times 20} = 24.2$ sq.in.

use .... 36 WF 16 1/2

t = 1/8 in. cover plate 5 ft. long.

$A_{c.p.} = 24.2 - 238 = 0.4$ sq. in.

Design of Girder 5

Loads: Fig. IV - 27.

Moment: Fig. IV - 27.

$$(-)M = 24.5\ (5)\ 2.5 + 22.5\ w \times 5\ (7.5) + 21w \times 5\ (12.5)$$

$$+\ 19.5\ w\times5\ (17.5) + 19w \times 5\ (22.5)$$

$$+\ 17\ w \times 5\ (27.5) + 14w \times 3\ (31.5) + \frac{0.3\ (33)}{2}$$

$$= 2150 + 165 = 2315 \text{ kip-ft.}$$

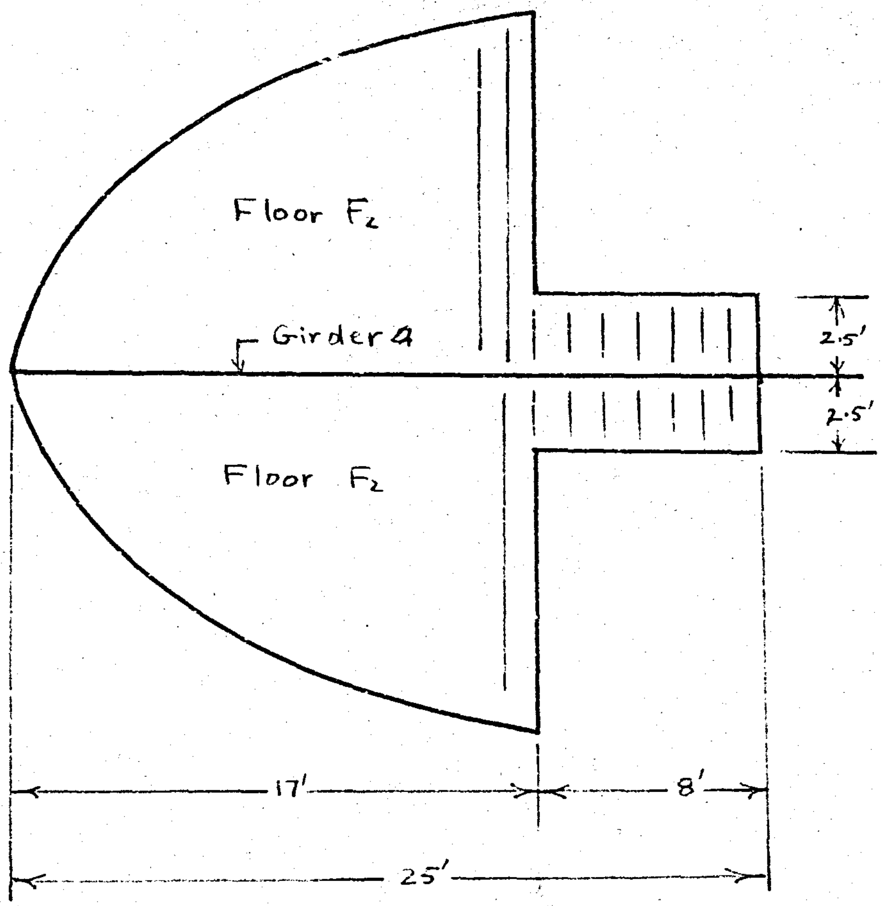
Steel: $(-)As = \frac{(-)M}{fsd} = \frac{2315 \times 12}{36 \times 20} = 38.6$ sq.in.

use .... 36 WF 16 1/2.

t = 1 in. cover plate 15 ft. long.

$A_{c.p.} = 38.6 - 23.8 = 14.8$ sq.in.

$t_{c.p.} = \frac{14.8}{16.5}$ ..... 1 in.



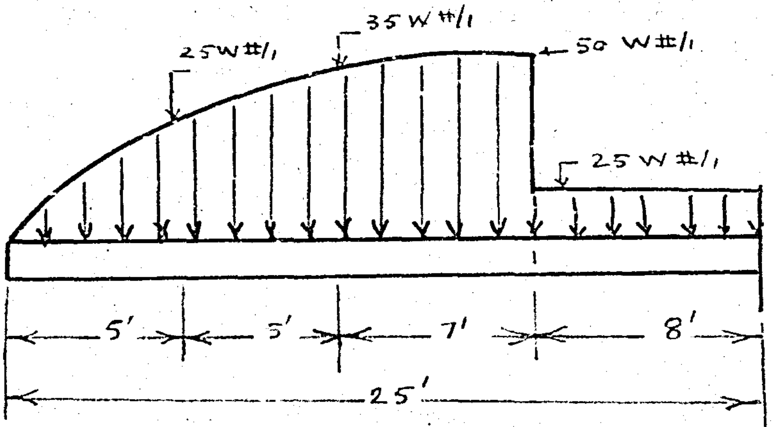
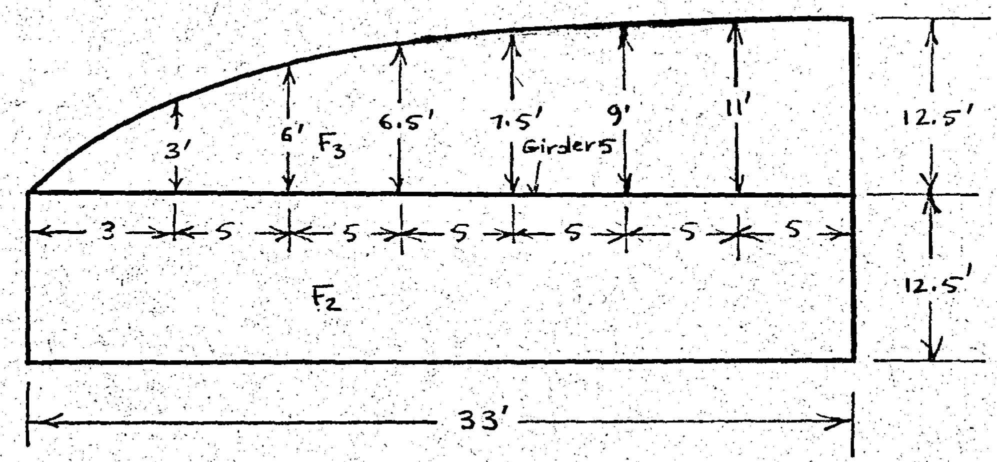


Fig IV-26 Loads on Girder 4



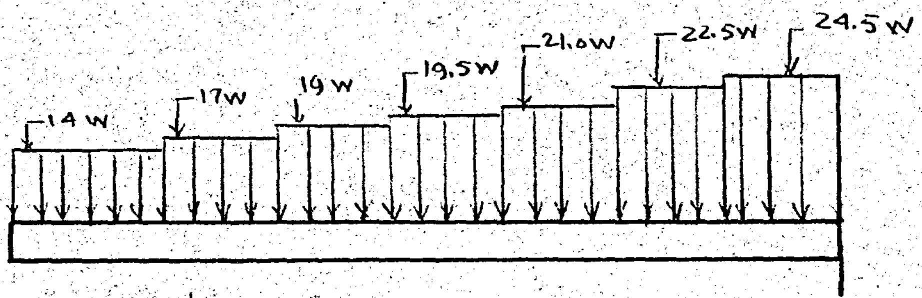


Fig. IV-27 Loads on Girder 5.

Bill of Materials of Casino

Bill of Materials of Floors

Steel & Concrete

Floor	No. of Parts in 3 slabs.	T-beams/ part	Bars	Shape	Length ft.	Wt./ft. lbs.	Wt. lbs.	Concrete ft^3.
Floor1	6	24	1 # 6	Straight	25	1.502	5400	
	6	24	1 # 6	Bent	26	1.502	5640	
	6	24	2 # 6	Straight	2 X 6	1.502	5200	
	6	24	1 # 3	Stirrup	2 X 3	0.376	325	
Floor2	12	17	1 # 6	Straight	25	1.502	7680	
	12	17	1 # 6	Bent	2 26	1.502	8000	
	12	17	2 # 6	Straight	2 X 6	1.502	74000	
	12	17	1 # 3	Stirrup	2 X 3	0.376	462	
	12	3	2 # 8	Straight	23	2.670	4430	
	12	3	2 # 9	"	15	3.400	3680	
	12	3	2 # 9	"	6	3.400	1470	
	12	3	12#33	Stirrup	3	0.376	486	
	12	3	2 # 8	Straight	17	2.670	3270	
	12	3	2 # 8	"	7	2.670	1340	
	12	3	4 # 3	Stirrup	3	0.376	163	
Floor3	12	14	1 # 8	Straight	15	2.670	6750	
	12	14	1 # 8	"	10	2.670	4500	
	12	14	1 # 8	"	5	2.670	2250	
	12	14	3 # 3	Stirrup	3	0.376	570	
	12	14	1 # 8	Straight	12	2.670	5400	
	12	14	1 # 8	"	6	2.670	2700	
	12	14	1 # 3	Stirrup	3	0.376	190	

Bill of Materials of Floors (Con't)

Floor	No.of Parts in 3 Slabs.	T—beams/ part.	Bars	Shape	Length ft	Wt./ft lbs	Wt. lbs	Concrete ft^3
Slab Lateral Reinforcement	3		# 2	Straight	16000	0.376	18100	12900
Slab Belt	3		4 # 3	Straight	400	0.376	1810	1200
					Total:		97,216	14100

Steel for Girders

Girder No.	No. of Gird. in 3 Slabs.	Girder (kind)	Plates (length X width). in.	Wt./ft. of length. lbs.	Length ft	Wt. kips.
1	3	36WF 16 1/2	—	260	100	78.00
	3	—	1/2X16 1/2	38.05	2 X 10	2.28
2	6	36WF 16 1/2	—	260	100	156.0
3	6	36WF 16 1/2	—	260	50	78.00
4	6	36WF 16 1/2	—	260	25	39.00
	6	—	1/8X16 1/2	6.8	5	0.21
5	12	36WF 16 1/2	—	260	25	78.00
	12	—	1 X 16 1/2	76.1	15	13.70
					Wt. =	44519

V (concrete) = 13,000 ft^3.

Failure of Choice of Proposal No.2

The bill of materials gives a sufficient evidence to the failure of choice of Proposal No.2.

If we are to compare the materials (steel and concrete) used in both proposals we find that the difference is great.

The amount of concrete necessary in Proposal No.2 is about 14000 cubic feet versus 11500 cubic feet in Proposal No.5. Hence the difference is not so great compared to that of steel where 543 kips of steel are needed for Proposal No.2 versus 233 kips for Proposal No.5 Although 20 ksi steel is used in the design of proposal No.2 and 30 ksi steel in Proposal No.5 , the difference is still great, and the price will be almost double. Roughly, the difference in steel cost is about three hundred thousand T.L.

This is not the only reason for the failure of this proposal, other factors count as well.

WF of such size 36 WF 16 1/2 might not be available in the market. Even if these were available, their transportation to the rock is very difficult and very costly.

The difficulty in construction is an important factor. Workmanship, including punching of webs for floor-rib reinforcement, cutting of plates, welding or riveting of plates, and connecting of beams to obtain longer spans all are serious obstacles in the construction and cost.

Furthermore, Proposal No.2 being too heavy structure it applies more load to the columns and hence their design will be more expensive.

This design (Proposal No.2) seems to be out of question.

The result derived here is based on comparison with the following proposal.

It will be demonstrated that Proposal No.5 is more reasonable, more economical, and more workable. It will be worked out in more detail as it is the final decision the design of the casino floors.

Proposal No.5

All major beams have a cross sectional area of 2ft. by 3 ft., and all minor beams have cross sectional area of 2ft. 6 in. by 1 ft. 6 in.

All beam dimensions are shown in Table IV - 1 .

Steel in beams is designed ~~in~~ such a manner so as to be symmetrical, so that bars will extend from one beam to another along the same axis. By doing this more steel is going to be employed sometimes, but this will add to the safety of the structure.

Design of Floor Ribbs

* All panels are going to constitute one way ribbed floors.
* All joists are going to have the same lateral dimensions shown in Fig. IV-3a.
* Loads:

Ribbed Floor D.L. = 54 lbs./sq. ft. Ref. 1

L.L. = 100 lbs./ sq. ft.

Partitions = 10 lbs./sq.ft.

Plastered Ceiling = 10 lbs./sq.ft.

Floor finish = 15 lbs./sq.ft.

Total load = 189 lbs./sq.ft.

* w/ linear foot on ribbs:

$$w = 189 \times \frac{25}{12} = 394 \text{ lbs./lin. ft.}$$

Design for 400 lbs./lin. ft. (Constant for all T-beams).

Table IV_1

Beams of Floors of Proposal No.2

No.	Beam		Length	Width	Depth	No. of beams
1	(Minor)	BQ	21' 6"	1' 6"	2' 6"	2
2	"	QS	35' 4"	1' 6"	2' 6"	2
3	"	SL	35' 4"	1' 6"	2' 6"	2
4	"	DN	21' 6"	1' 6"	2' 6"	2
5	"	NF	35' 4"	1' 6"	2' 6"	2
6	(Major)	MTV	70' 0"	2'	3'	1
7	"	RTK	70' 0"	2'	3'	1
8	"	AR &KJ	15' 0"	2'	3'	2
9	"	CP	24' 6"	2'	3'	2
10	"	PM	35' 4"	2'	3'	2
11	"	MG	21' 6"	2'	3'	2
12	"	RP	35' 0"	2'	3'	2
13	"	PE	24' 6"	2'	3'	2
14	"	MF	14'	2'	3'	2
15	"	KL	17' 6"	2'	3'	2

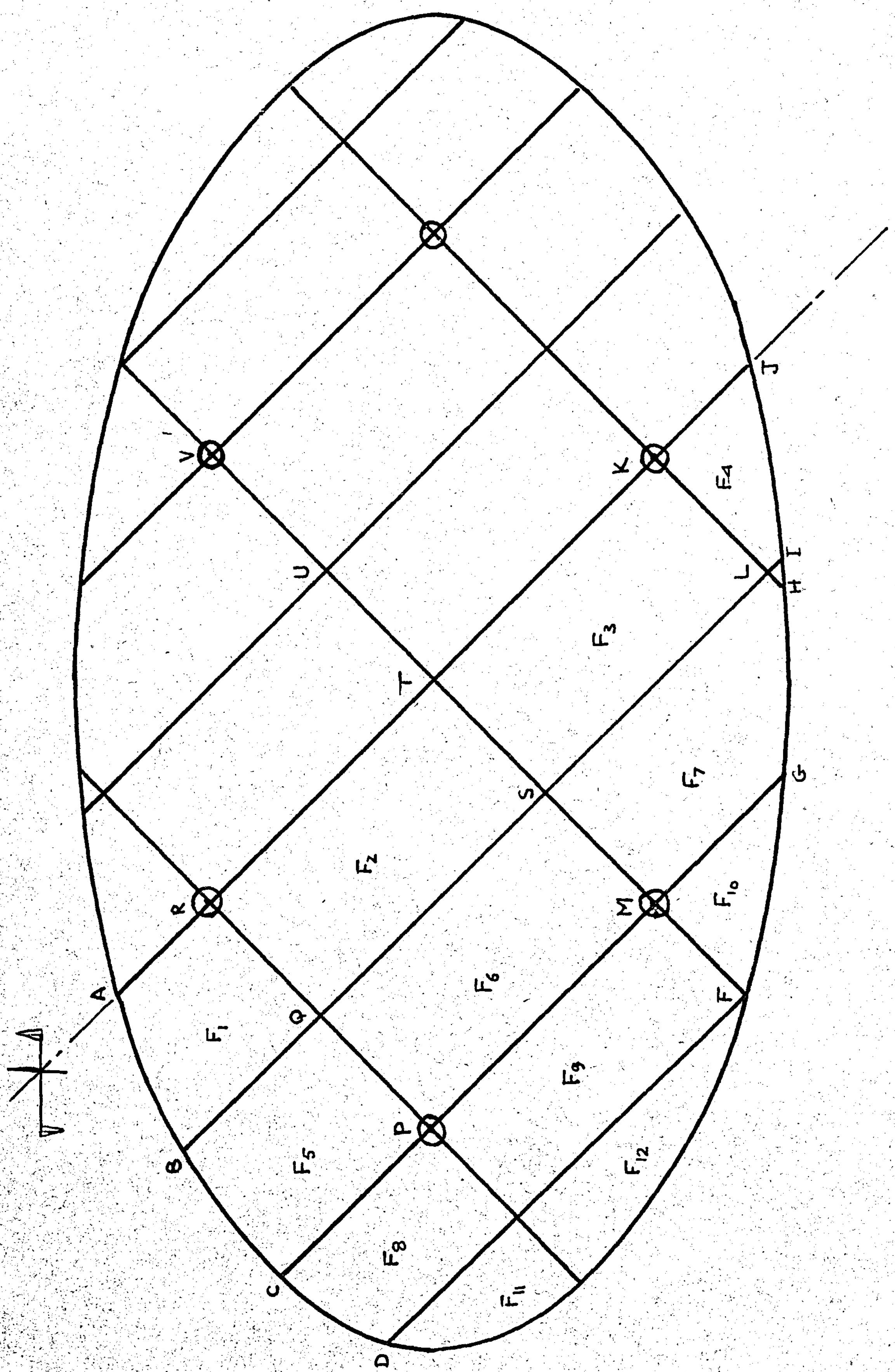


Fig. IV-28 Plan of Casino Floor Beams

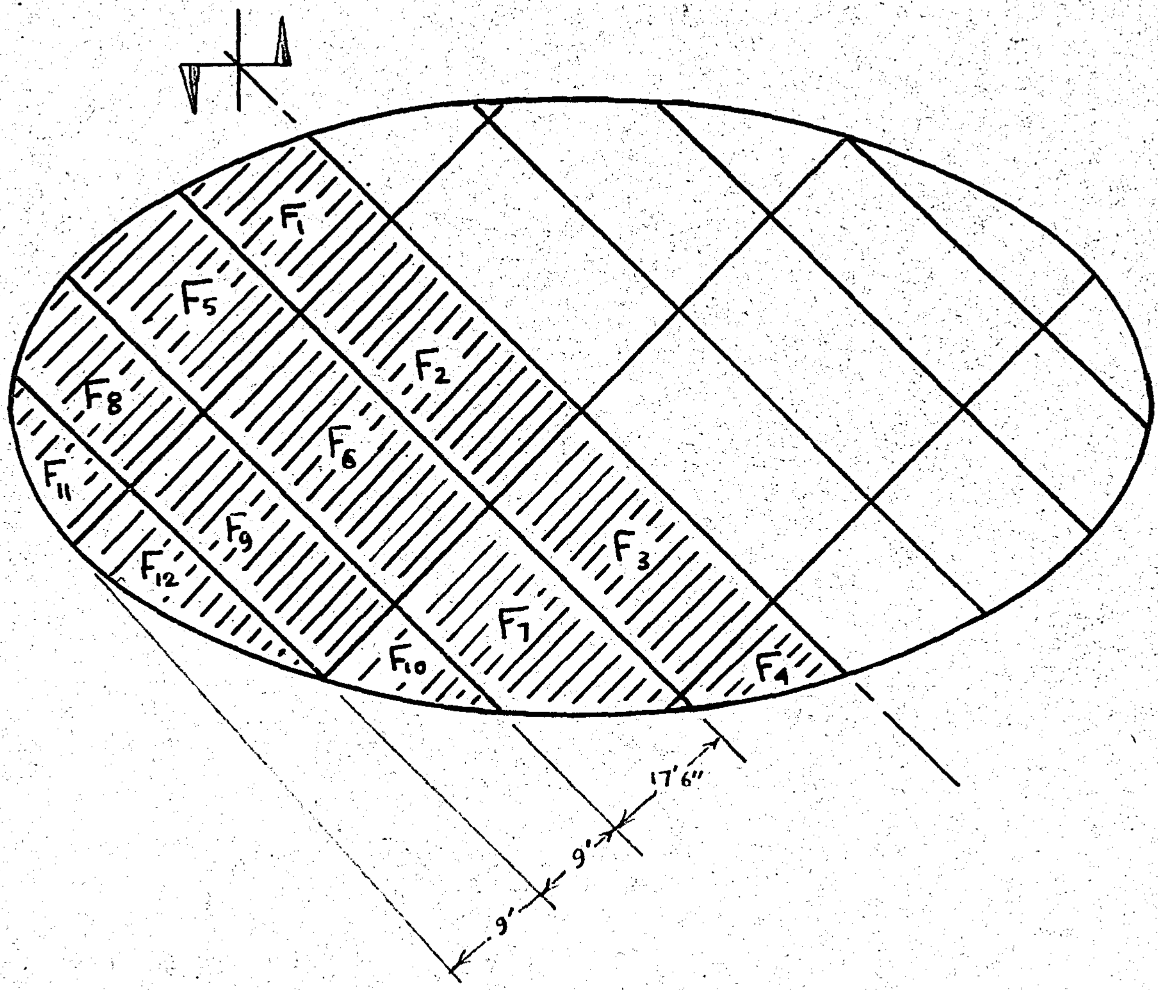


Fig. IV-29 Plan of Floor System of Proposal No. 5

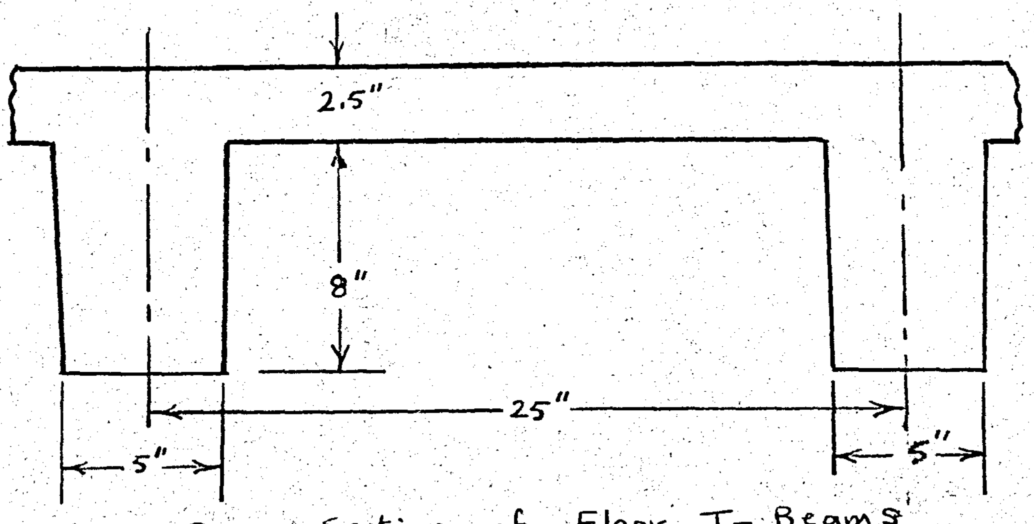


Fig. IV-30 Cross Section of Floor T-Beams

1. Fixed-Fixed T-beams

a- Long Beams: L = 17 ft. 6 in.

w = 0.4 kips/lin.ft.

$$(-)M = \frac{wl^2}{12} = \frac{0.4(17.5)^2}{12} = 10.2 \text{ kip-ft.}$$

$$(+)M = \frac{wl^2}{16} = \frac{0.4(17.5)^2}{16} = 7.7 \text{ kip-ft.}$$

$$(-)As = \frac{M}{Jdf} = \frac{10.2 \times 12}{7/8 \times 9 \times 30} = 0.52 \text{ sq. in.}$$

$$(+)As = \frac{M}{Jdf} = \frac{7.7 \times 12}{7/8 \times 9 \times 30} = 0.39 \text{ sq. in.}$$

(+)As .... 1 No.4 str.bott.
1 No.4 bent.

(-)As ....1 No.4 bent.
2 No.4 str.top.

Spacing: 1.5 X 2 = 3 in.
0.5 X 2 = 1 in.
1 X 1 = 1 in.
Total = 5 in. O.K.

Shear: $V = \frac{wl}{2} = \frac{0.4\ (17.5)}{2} = 3.5$ kips.

$v = \frac{V}{bJd} = \frac{3500}{5 \times 7/8 \times 9} = 89 \text{ psi} < 90 \text{ psi}$ O.K.

Bond: $u = \frac{V}{0Jd} = \frac{3500}{4.7 \times 7/8 \times 9} = 95 \text{ psi} < 210 \text{ psi}$ O.K.

b- <u>Short Beams:</u> $L = 14$ ft.

$w = 0.4$ kips/lin.ft.

$$(-)M = \frac{wl^2}{12} = \frac{0.4\,(14)^2}{12} = 6.5 \text{ kip-ft.}$$

$$(+)M = \frac{wl^2}{16} = \frac{0.4(14)^2}{16} = 4.9 \text{ kip-ft.}$$

$$(-)As = \frac{M}{Jdf} = \frac{6.5 X 12}{7/8 X 9 X 30} = 0.33 \text{ sq. in.}$$

$$(+)As = \frac{M}{Jdf} = \frac{4.9 X 12}{7/8 X 9 X 30} = 0.25 \text{ sq. in.}$$

(+)As .... 1 No. 4 Str.bott.
1 No. 4 bent.
(-)As .... 1 No. 4 bent.
1 No. 4 str.top.

Spacing: (similar to "a") Checks O.K.

Shear: (similar to "a") Checks O.K.

Bond : Checks O.K.

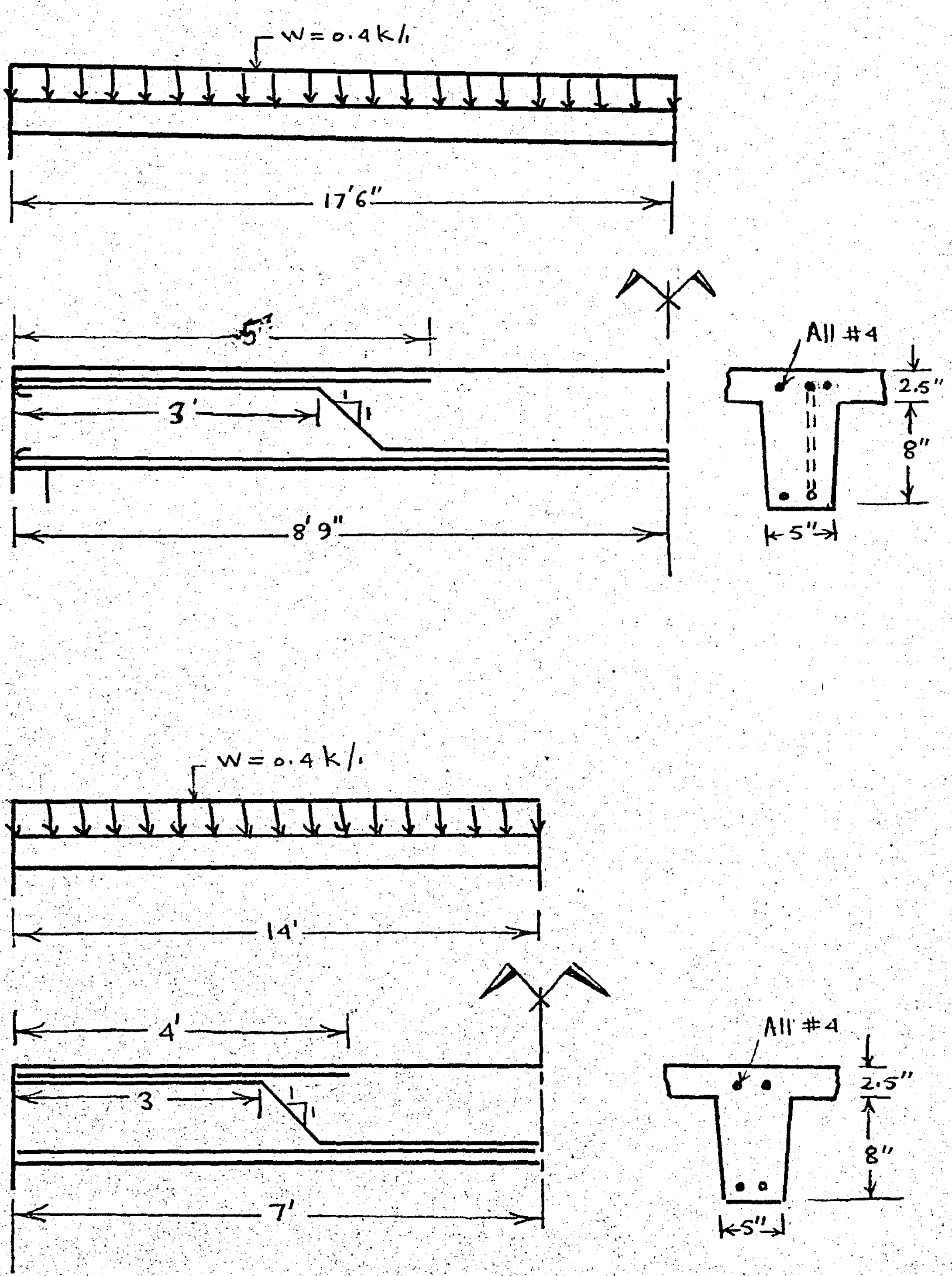


Fig. IV-31 Fixed - Fixed T-Beams, Loads & Steel Reinforcement.

2. Cantilever T-beams:

All the T-beams here have the same cross sectional dimensions and the same loading per linear foot. The only thing that differs is the length, so only two T-beams will be designed.

a- Long Beams: $9 \text{ ft.} \leq L \leq 17 \text{ ft.}$

$w = 0.4$ kips/lin.ft.

$$(-)M = \frac{wl^2}{2} = \frac{0.4\,(17)^2}{2} = 58 \text{ kip-ft.}$$

$$(-)As = \frac{M}{Jdf} = \frac{5.8 \times 12}{7/8 \times 8 \times 30} = 3.3 \text{ sq. in.}$$

(-)As .... 2 #9 .... L = 20 ft.

2 #9 .... L = 10 ft.

Shear: $V = wl = 0.4 \times 17 = 6.8$ kips.

$$v = \frac{V}{bJd} = 195 \text{ psi} > 90 \text{ psi.}$$

use stirrups, Fig. IV-31.

$$x = 17 \times \frac{105}{195} = 9.2 \text{ ft.}$$

Stirrups No.3 bars:

$$S = \frac{A_v f_v}{v' b} = \frac{0.22 \times 30{,}000}{5 \times 105} = 12.6 \text{ in} \;......\; 12 \text{ in.}$$

use .... 10 at 12 in.

Spacing: (similar to 1-a) Checks O.K.

Bond: $$u = \frac{V}{OJd} = \frac{6800}{14.2 \times 7/8 \times 8} = 69 \text{ psi} < 210 \text{ psi} \qquad \text{O.K.}$$

b- **Short Beams:** $1 \text{ ft.} \leq L \leq 9 \text{ ft.}$

$w = 0.4$ kips/ lin.ft.

$$(-)M = \frac{wl^2}{2} = \frac{0.4\ (9)^2}{2} = 16.2 \text{ kip-ft.}$$

$$(-)As = \frac{M}{Jdf} = \frac{16.2 \times 12}{7/8 \times 8 \times 30} = 0.94 \text{ sq.in.}$$

(−)As .... (i) 3#4 .....L = 9 ft.

(ii) 2#4 .....L = 6 ft.

Shear: $V = wl = 0.4 \times 9 = 3.6$ kips

$$v = \frac{V}{bJd} = \frac{3600}{5 \times 7/8 \times 8} = 104 \text{ psi} > 90 \text{ psi.}$$

Use stirrups, Fig. .

$$X = 9 \times \frac{14}{104} = 1.22 \text{ ft.}$$

Stirrups No.3 bars

$$S = \frac{A_v f_v}{v' b} = \frac{0.22 \times 30{,}000}{5 \times 14} = 94 \text{ in.} \ldots\ldots 12 \text{ in.}$$

use .... 2 at 12 in.

Spacing: (similar to 1-a) Checks O.K.

Bond: Checks O.K.

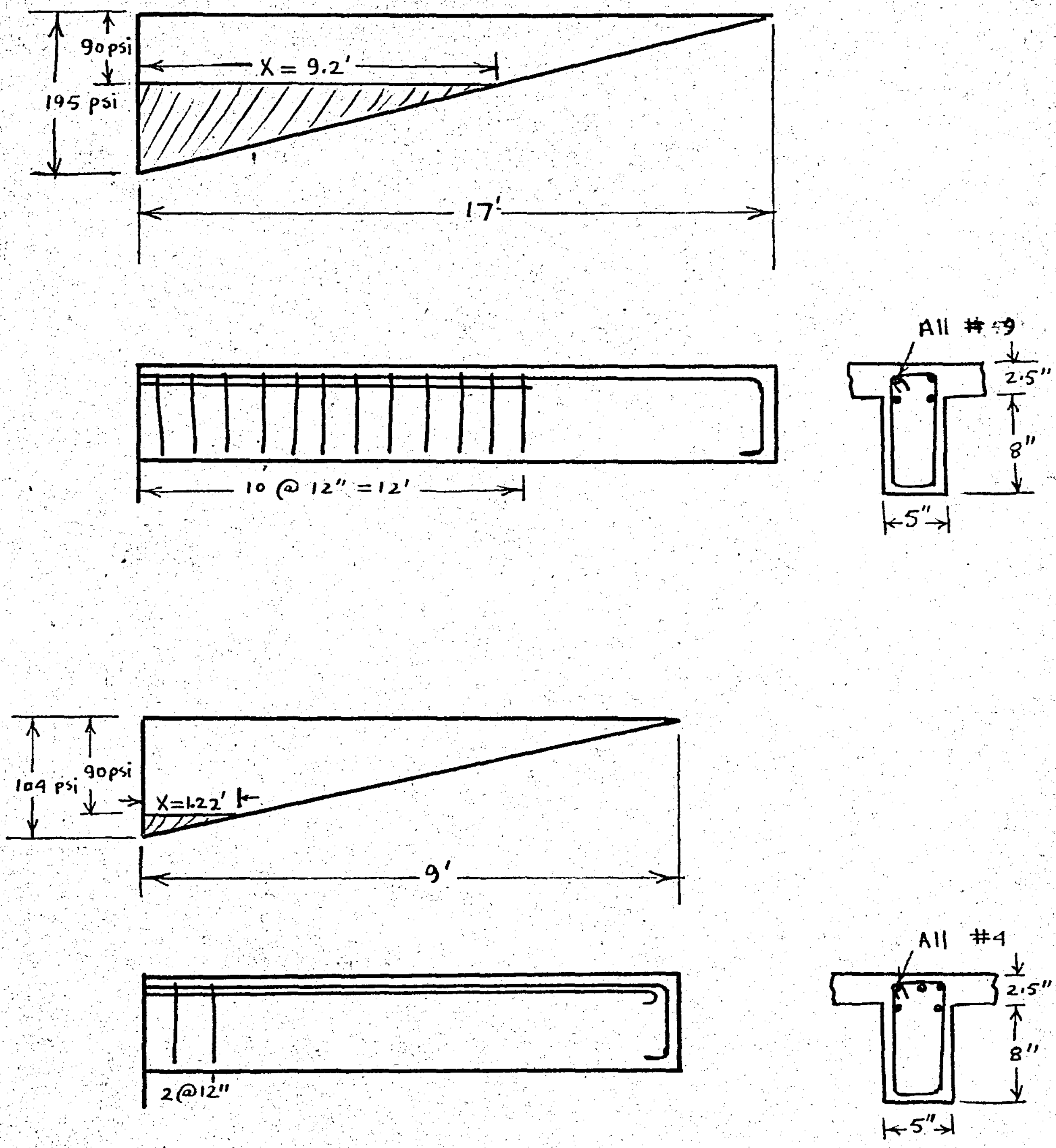


Fig. IV-32 Cantilever T-Beams ; Loads & Steel Reinforcement

Table IV-2

T-beams of Floors

Panel	Panels per floor	No. of beams per panel	Fixed - Fixed beams No.	Length -ft.	Cantilever bea No.	Av.L = 5ft	Av.L = 13
F	6	10	7	17.5	3	1	2
F	6	17	17	17.5	—	—	—
F	6	17	17	17.5	—	—	—
F	6	7	1	17.5	6	3	3
F	6	12	10	17.5	2	—	2
F	6	17	17	17.5	—	—	—
F	6	17	10	17.5	7	3	4
F	6	12	10	14	2	1	1
F	6	17	17	14	—	—	—
F	6	10	—	—	10	5	5
F	6	10	—	—	10	5	5
F	6	17	—	—	17	10	7
	Total:	79+27				28	29

Lateral Reinforcement:

Total w = 189 lbs./sq.ft.

$(-)\ M = \frac{wl^2}{12} = \frac{189 \times (2)^2}{12} = 63$ ft-lbs.

$(+)\ M = \frac{wl^2}{16} = \frac{189 \times (2)^2}{16} = 47$ ft-lbs.

$(-)As = \frac{M}{Jdf} = \frac{63 \times 12}{7/8 \times 2 \times 30{,}000} = 0.0144$ sq.in/ft.

$(+)As = \frac{M}{Jdf} = \frac{47 \times 12}{7/8 \times 2 \times 30000} = 0.0134$ sq.in/ft.

(-)As .... # 2 at 12 in. top.

(+)As .... # 2 at 12 in. bott.

Sum of Lateral Steel:

Area of Floor (approx.) $= \frac{2}{3}$ X length X width.

$A = \frac{2}{3} \times 150 \times 80 = 8000\ ft^2$.

L (Lateral Steel) = 8000 X 2 = 16,000ft.

Sum of Concrete for Ribbs:

$V = \frac{54 \times 8000}{150} = 2870\ ft^3$.

Steel and concrete for Floor Belt:

Width = 4 in.

Depth = 3 ft.

Steel = 4 #3.

L (belt's steel) = 400 X 4 = 1600 ft.

V (belt's concrete) = 400 ft^3.

B ill of Materials of Casino Floor Panels

Steel & Concrete

Member	No. of Parts	Bars	Shape	Length ft.	Wt/ft. lbs.	Total Wt lbs.	Total concrete ft
T-beams	3	1 No.2	Straight	16000	0.167	8000	8600
Fixed-Fixed	6 X79	1 No.4	"	18	0.668	5700	
	6 X 79	2 No.4	"	2 X 55	0.668	6300	
	6 X 79	1 No.4	bent	20	0.668	6300	
	6 X 27	1 No.4	straight	15	0.668	1620	
	6 X 27	1 No.4	"	2 X 4	0.668	860	
	6 X 27	1 No.4	bent	16	0.668	1720	
Cantilever	6 X 29	4 No.4	straight	14	0.668	6500	
	6 X 29	10No.3	stirrups	2.5	0.376	1640	
	6 X 28	5 No.4	straight	6	0.668	3350	
	6 X 28	2 No.3	stirrup	2.5	0.376	320	
Slab Belt	3	4 No.3	straight	400	0.376	1760	1200
					Total:	44070	9800

Beam Analysis and Design

Effect of Minor Beams on Major Beams

In the design of major beams, the torsional shears caused by minor beams should be taken into consideration.

A torsional moment caused by a minor beam results in flexual stresses in the continuous minor beams on one hand and in torsional stresses in the supporting major beam o n the other. As a result of the torsional moment the major beam is going to rotate which further causes other major beams and columns to rotate. Therefore the moments should be distributed to the members of the space frame taking into consideration the different flexural and torsional stiffness factors in the distribution.

This procedure is not going to be followed to avoid complexity first, and second because even by doing that we are not sure of the result.

Therefore in case of torsional moment distribution, the moment at the joint is going to be proportioned accoding to the stiffness of the resisting members. This in fact is the first distribution of moments.

Gordon Fisher and Paul Zia in their paper "Review of Code Requirements for Torsion Design" published in the Journal of the American Concrete Institute of January 1964 compare different methods for torsional reinforcement followed in different countries.

As we are following ACI specifications in this design, the American method is going to be employed here as it is in the (GSA Handbook).

This method will be used where torsional effects are considerable.

****

Torsion Effects

Flexural Stiffness

Fixed-Fixed beam $K_{FF} = \frac{4EI}{L}$ Ref. 12 p.77

Torsional Rigidity

$T = kG\theta\, a^3 b$ per inch of length when $\theta = 1$ Ref. 3 p. 337

$$K = \frac{Gk\, a^3 b}{L}$$

Ratios of Stiffness:

$$\frac{K_F}{K_T} = \frac{4EI/L_1}{Gka^3 B/L_2}$$

$$I(\text{minor beam}) = \frac{bh^3}{12} = \frac{18\ (30)^3}{12} = 4.05 \times 10^4\ \text{in.}^4$$

E = Modulus of Elasticity

$$G = \frac{E}{2(1+\ )} = \text{Shearing modulus of elasticity}$$

(concrete) = 0.15 Ref. 4 P. 262

$G = E/2.3$

$k = 0.196$

L_1 = length of minor beam.

L_2 = length of major beam.

a = 24 in. width of major beam.

b = 36 in. depth of major beam.

$$\frac{K_F}{K_T} = \frac{4E\,(4.05 \times 10^4)}{L_1} \times \frac{L_2}{(E/2.3)(0.196)(24)^3(36)}$$

$$\frac{K_F}{K_T} = 3.8\,\frac{L_2}{L_1}$$

The following two tables will help us in the design of the major and minor beams.

The tables were formulated after calculating the positive and negative moments on the beams due to the dead loads plus live loads.

The torsional effects of floors on beams are ignored as they are of minor importance.

Table IV-3 shows the values of beam moments as calculated in the following pages. From these values the net moments are obtained.

Table IV-4 shows the net moment at a joint and the distribution of that moment to the corresponding members according to the ratio $\frac{K_F}{K_T} = 3.8\,\frac{L_2}{L_1}$.

The torsional moments are designed for according to the GSA Handbook method, while the additional flexural moment is added to the L.L.&D.L. moment in the design of the corresponding minor beam.

Table IV-3

Torsional Moments

Effect of Beam Moments in Torsion

No.	Beam	Influence of	+M	Influence of	-M	Net
1	BQ	——	——	——	——	——
2	QS	——	——	——	——	——
3	SL	——	——	——	——	——
4	DN	——	——	——	——	——
5	NF	——	——	——	——	——
6	RTK	——	——	——	——	——
7	MTV	——	——	——	——	——
8	AR	——	——	——	——	——
9	CP	——	——	——	——	——
10	PM	——	——	——	——	——
11	MG	——	——	——	——	——
12	RP	BQ	905	QS	409	496
13	PE	ND	745	NF	335	410
14	MF	NF	335	——	——	335
15	KL	SL	409	——	——	409

Table IV-4

Net Moment Distribution to Torsion and Bending

No.	Joint	Net M. kip-ft.	Direction	Torsion		Flexure	
				Beam	M kip-ft	Beam	M kip-ft
1	Q	496	QB	PR	100	QS	396
2	N	410	ND	PE	112	NF	298
3	F	335	FN	MF	133	FN	202
4	L	409	LS	KL	140	LS	269

Torsional Shear (Major Beams):

$\tau_{max} = \frac{T}{\alpha b h^2}$ ....Ref. 4 p.270

$b = 3$ft. depth of beam.

$h = 2$ ft. width of beam.

T = Torsional moment

$\alpha = f\ (b/h)$ ....Ref. 4 p.271

For $b/h = 1.5$ .... $= 0.231$

$$\tau_{max} = \frac{T}{0.231\ (36)(24)^2} = \frac{T}{4800}$$

$$\tau_{max} = \frac{T}{4800}$$

From this relation the different torsional stresses are calculated and tabulated as follows:

Table IV - 5

No.	Beam	Beam No.	T kip-ft	$= \frac{T}{.4800}$ psi
1	PR	12	100	250
2	PE	13	112	280
3	MF	14	133	332
4	KL	15	140	350

Design for maximum torsional shear and use the same torsional reinforcement for the foun different beams.

max = 350 psi $\angle$ 360 psi. O.K.

The section is acceptable and required reinforcement for torsional shear which will be added to that of load shear.

$$A_v = \frac{350\ t}{30{,}000} = 0.0117\ t$$

For hoops of No. 5 bars

$$t = \frac{0.31}{0.0117} = 26.5 \text{ in} \ldots\ldots\ldots 2 \text{ ft.}$$

For longitudinal steel use minimum four No.5 bars arraged with the other flexural reinforcement so as to have a bar at each corner.

* Minor Beams *

1. **Beam BQ:** Fig. IV-33.

$L = 21.5$ ft.

$w = 0.19 \times 17.6 = 3.35$ kips/lin.ft.

w (D.L.) $= 2.5 \times 1.5 \times 0.150 = 0.56$ kips/lin.ft.

Total $w = 3.91$ kips/lin.ft.

$$(-)M = \frac{wl^2}{2} = \frac{3.91 \quad (21.5)^2}{2} = 0.905 \text{ kip-ft.}$$

$$(-)As = \frac{-M}{Jdf} = \frac{905 \times 12}{7/8 \times 24 \times 30} = 17.24 \text{ sq.in.}$$

(-)As 6No.10

6No.10

2No.10

Spacing:

$6 \times 1.27 = 7.62$ in.

$5 \times 1.27 = 6.35$ in.

$2 \times 1.5 = 3.00$ in.

Total $= 16.97$ in 18 in. O.K.

Shear:

$V = wl = 3.91 \times 21.5 = 84$ kips

$$v = \frac{V}{bJd} = \frac{84{,}000}{18 \times 7/8 \times 24} = 222 \text{ psi} > 90 \text{ psi}$$

Use stirrups, Fig. IV-33.

$$x = 21.5 \times \frac{132}{222} = 12.8 \text{ ft.}$$

Stirrup No.5 bars

$$S = \frac{A_v f_v}{v' \, b} = \frac{0.61 \times 30{,}000}{18 \times 132} = 7.8 \text{ in} \,\ldots\ldots\ldots\, 7 \text{ in.}$$

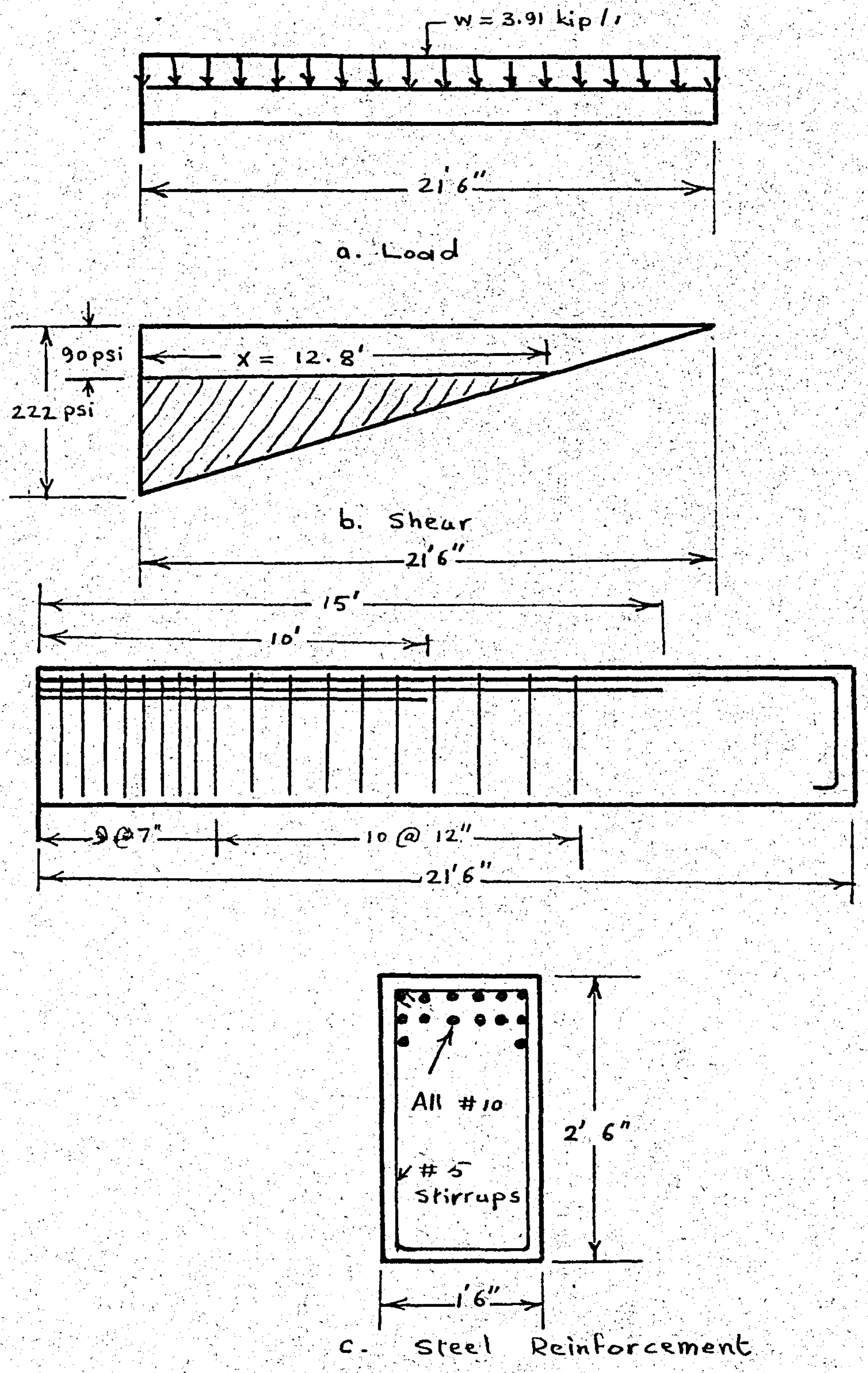


Fig. IV-33 Beam (1) BQ of Casino Floor - Proposal No. 5

use ...... 9 at 7 in.

10 at 12 in.

Bond:

$$u = \frac{V}{OJd} = \frac{84000}{55.9 X 7/8 X 24} = 72 \text{ psi} \angle 210 \text{ psi} \qquad \text{O.K.}$$

2. <u>Beam QS</u> Fig. IV 34.

$L = 35.3$ ft.

Total w = 3.91 kips (similar to beam BQ)

$$(-)M = \frac{wl^2}{12} = \frac{3.91\ (35.3)^2}{12} = 409+396 = 805 \text{ kip-ft.}$$

$$(+)M = \frac{wl^2}{16} = \frac{3.91\ (35.3)^2}{16} = 306 \text{ kip-ft.}$$

$$(-)As = \frac{-M}{Jdf} = \frac{805 X 12}{7/8 X 28 X 30} = 13.2 \text{ sq. in.}$$

$$(+)As = \frac{M}{Jdf} = \frac{306 X 12}{7/8 X 28 X 30} = 5.0 \text{ sq in.}$$

(-) As .... 2# 10 str. bott.

2# 10 bent.

(+) As .... 2# 10 bent.

4# 10 str. top.

6# 10 str. top.

Spacing:

Checks O.K. (similar to Beam BQ)

Shear: $V = \frac{wl}{2} = \frac{3.91 X 35.2}{2} = 69$ kips

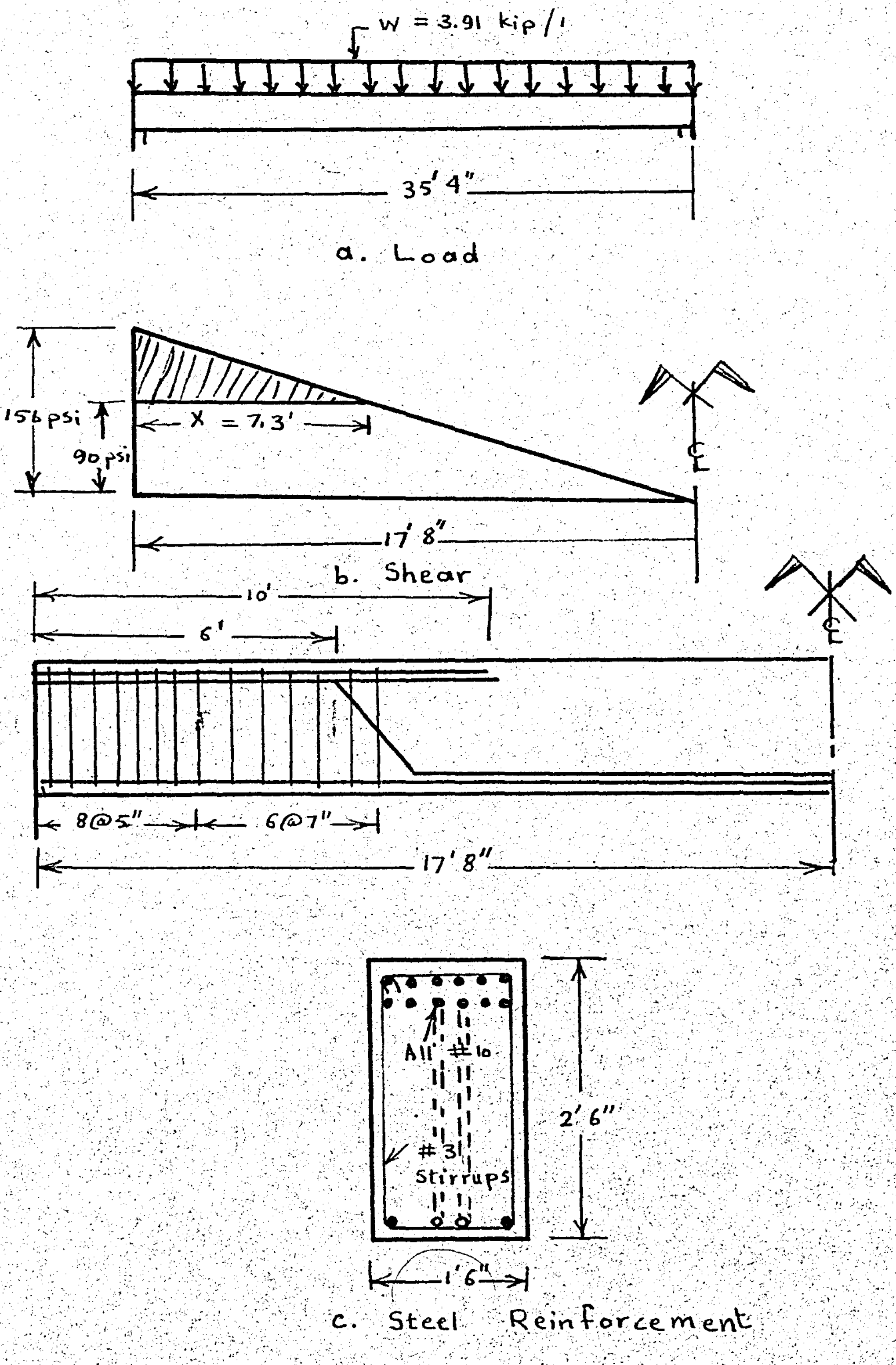


Fig. IV-34 Beam (2) QS of Casino Floor - Proposal No. 5

$v = \frac{V}{bJd} = \frac{69000}{18 \times 7/8 \times 28} = 156$ psi $>$ 90 psi

use stirrups, Fig.IV_34.

$x = 17.2 \times \frac{66}{156} = 7.3$ ft.

Stirrup No.3 bars

$S = \frac{A_v f_v}{v' b} = \frac{0.22 \times 30{,}000}{66 \times 18} = 5.5$ in. ...... 5 in.

Use ...... 6 at 5 in.

6 at 7 in.

Bond: $u = \frac{V}{OJd} = \frac{69000}{23.9 \times 718 \times 28} = 118$ psi $\angle$ 210 psi O.K.

3. <u>Beam SL</u>

Design similar to Beam QS.

4. <u>Beam DN</u> Fig.IV_35 .

$L = 21.5$ ft.

w (from F) $= 7 \times 0.19 = 1.33$ kips/lin.ft.

w (from S) $= 7 \times 0.19 = 1.33$ kips/lin.ft.

w (D.L.) $= 0.56$ kips/lin.ft.

Total $= 3.22$ kips/lin.ft.

$(-)M = \frac{wl^2}{2} = \frac{3.22\ (21.5)^2}{2} = 745$ kip-ft.

$(-)As = \frac{-M}{Jdf} = \frac{745 \times 12}{7/8 \times 24 \times 30} = 14.2$ sq.in.

(–) As 6 No.10

6 No.10

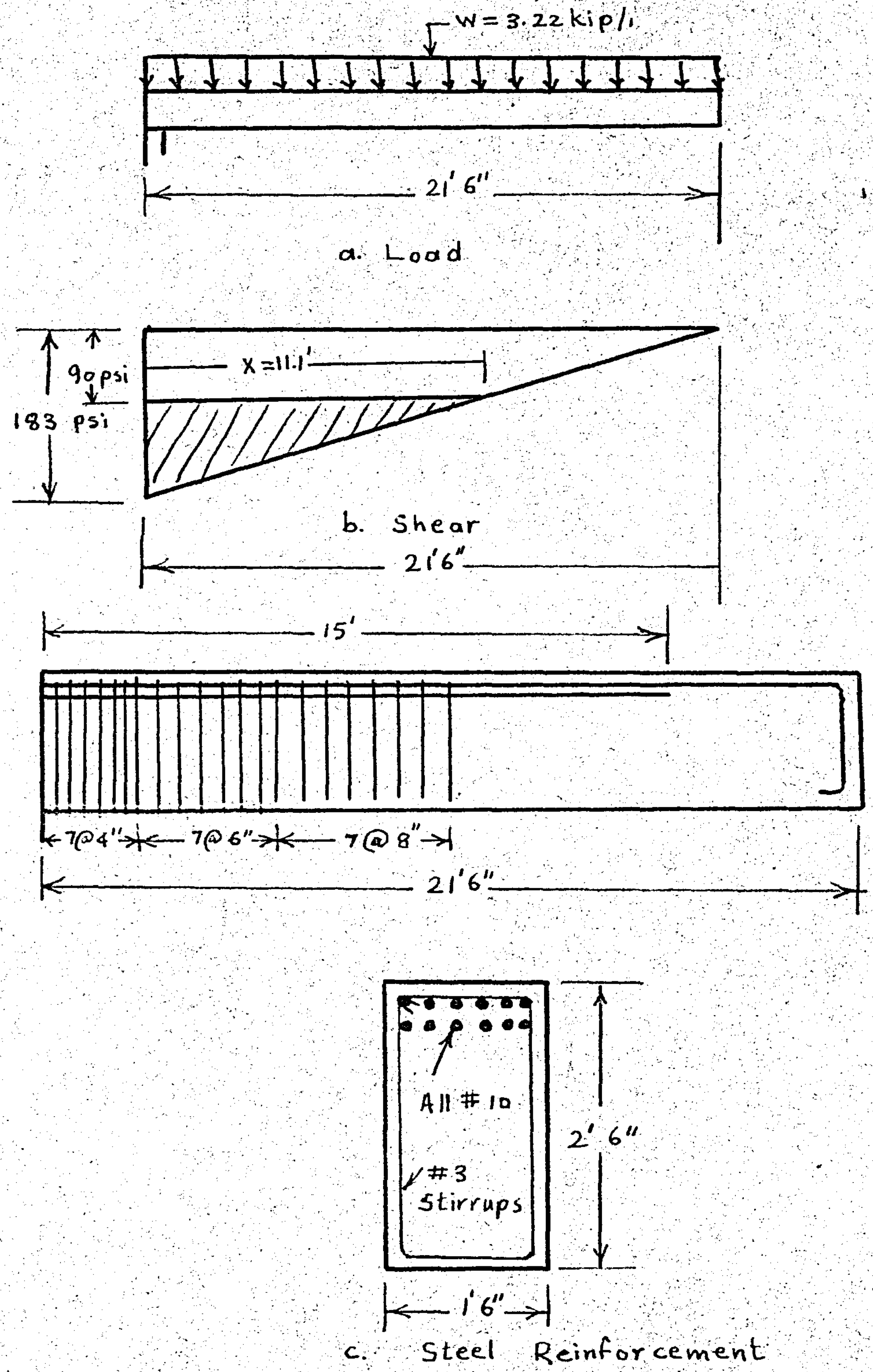


Fig. IV-35 Beam (4) DN of Casino Floor - Proposal No. 5

Spacing: Checks O.K. (similar to Beam BQ)

Shear:

$V = wl = 3.22 \times 21.5 = 69$ kips.

$v = \dfrac{V}{bJd} = \dfrac{69000}{18 \times 7/8 \times 24} = 183$ psi > 90 psi.

use stirrups, Fig. IV-35.

$x = 21.5 \times \dfrac{93}{180} = 11.1$ ft.

$S = \dfrac{A_v f_v}{v' b} = \dfrac{0.22 \times 30{,}000}{90 \times 18} = 41$ in. 4 in.

use .... 7 at 4 in.

7 at 6 in.

7 at 8 in.

Bond: Checks O.K. (Reference- previous beams).

5. Beam NF Fig. IV-36.

$L = 35.3$ ft.

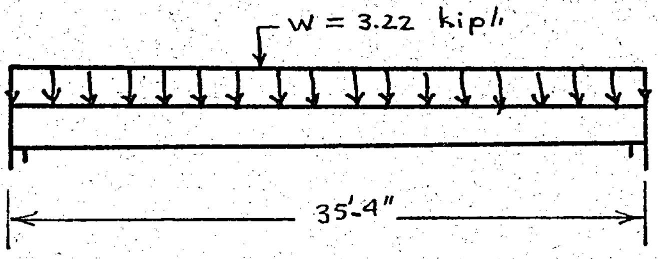
Total w = 3.22 kips/lin.ft. (similar to Beam DN)

$(-)M = \dfrac{wl^2}{12} = \dfrac{3.22\,(35.3)^2}{12} = 335 + 298 = 633$ kip-ft.

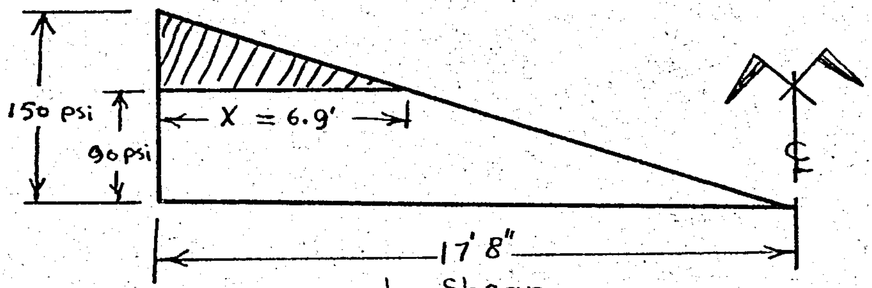
$(+)M = \dfrac{wl^2}{16} = \dfrac{3.22\,(35.3)^2}{16} = 251$ kip-ft.

$(-)As = \dfrac{-M}{Jdf} = \dfrac{633 \times 12}{7/8 \times 28 \times 30} = 10.03$ sq.in.

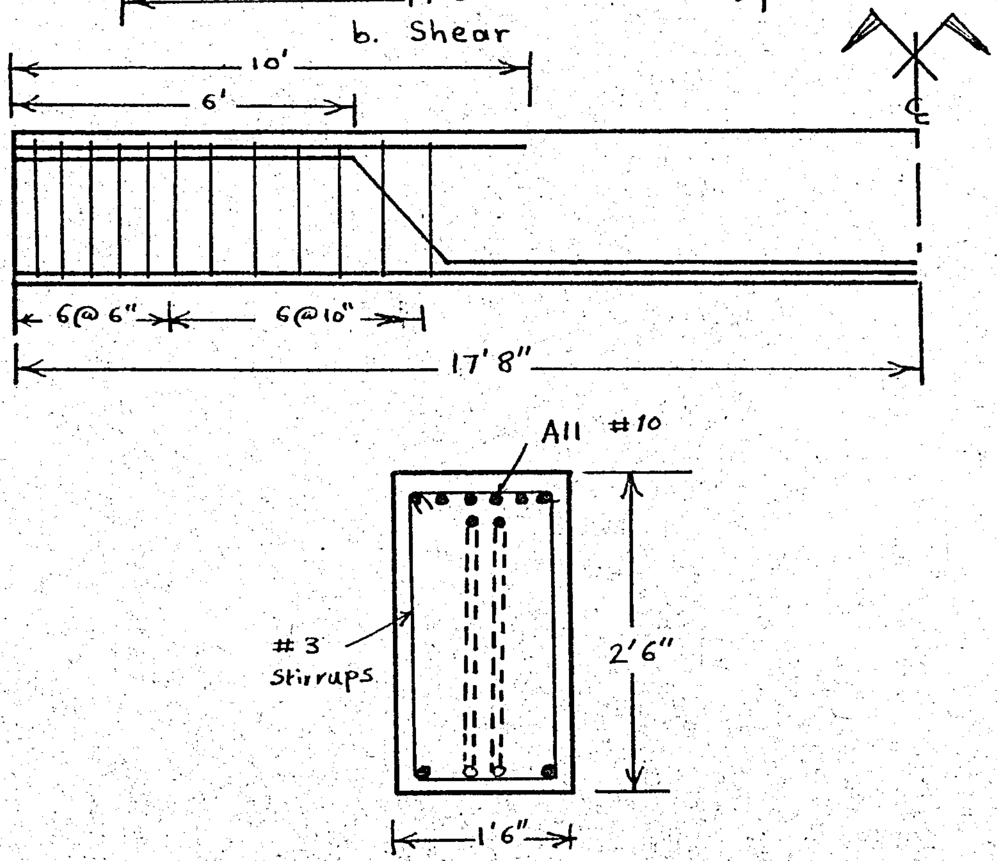
$(+)As = \dfrac{M}{Jdf} = \dfrac{251 \times 12}{7/8 \times 28 \times 30} = 4.1$ sq.in.



a. Load



b. Shear



c. Steel Reinforcement

Fig. IV-36 Beam (5) NF of Casino Floor - Proposal No. 5

(+) As .... 2 No.10 str.bott.

2 No.10 bent.

(-) As .... 2 No.10 bent.

6 No.10 str.top.

Spacing:

Checks O.K. (similar to Beam BQ)

Shear: $V = \frac{wl}{2} = \frac{3.22 \times 35.2}{2} = 57$ kips.

$v = \frac{V}{bJd} = \frac{57000}{18 \times 7/8 \times 24} = 150$ psi $>$ 90 psi.

Use stirrups. Fig. IV-36.

$x = 17.2 \times \frac{60}{150} = 6.9$ ft.

Stirrups No.3 bars

$S = \frac{A_v f_v}{v' b} = \frac{0.22 \times 30{,}000}{60 \times 18} = 6.1$ in ...... 6 in.

Use .... 6 at 6".

6 at 10".

Bond:

$u = \frac{V}{0Jd} = \frac{57000}{20.0 \times 7/8 \times 28} = 117$ psi $\angle$ 210 psi O.K.

* Major Beams *

Analysis of Beams RTK and MTV

Beam RTK Fig. IV - 37

$w = 0.190 \times 17.6 = 3.35$ kips/lin.ft.

w (D.L.) $= 2 \times 3 \times 0.15 = 0.9$ kips/lin.ft.

Total $w = 4.25$ kips/lin.ft.

$$\Delta_1 = \frac{w_1 L_1^4}{384\, E_1 I_1}$$

Beam MTV Fig. IV - 37

$$\Delta_2 = 2\left[\frac{Pb^2}{48\, E_2 I_2}(3L - 4b)\right]$$

$$b = \frac{L}{4}$$

$$a = \frac{3L}{4}$$

$$P = \frac{w_2 L_2}{2}$$

$$\Delta_2 = 2\left[\frac{w_2 L_2}{2E_2 I_2} \times \frac{L_2^2}{16(48)}(3L_2 - L_2)\right]$$

$$= \frac{w_2 L_2^4}{384\, E_2 I_2}$$

$$w_1 = w_2$$

$$L_1 = L_2$$

$$E_1 = E_2$$

$$I_1 = I_2$$

Therefore $\Delta_1 = \Delta_2$

Reaction force R between the two beams is zero.

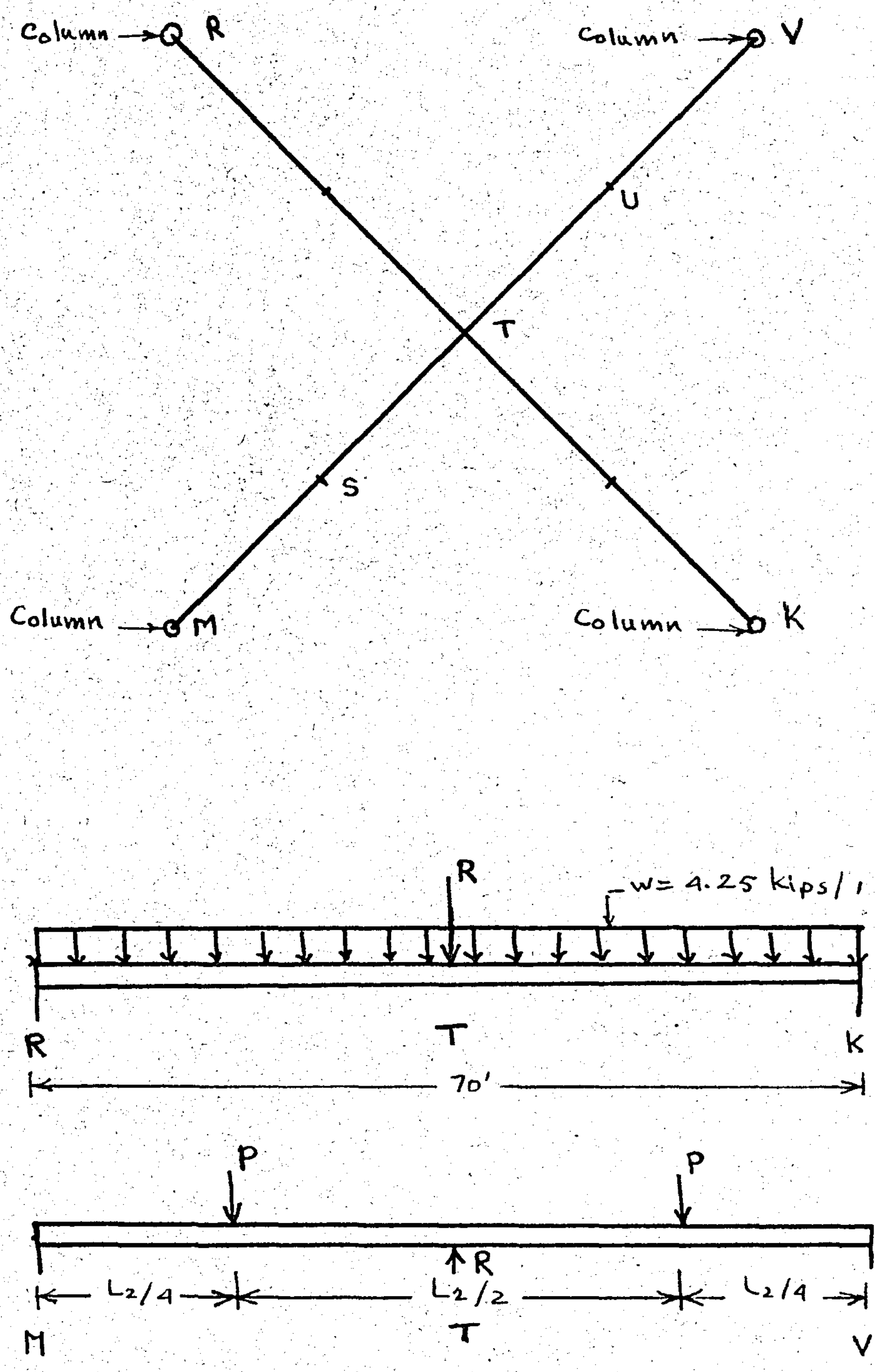


Fig. IV-37 Beams RTK & MTV - Loads & Reactions

6. Beam MTV   Fig. IV- 38

$L = 70$ ft.

$P = \frac{wl}{2} = \frac{4.25\ (70)}{2} = 148$ kips.

(w D.L. is accounted for in w = 4.25)

$(-)\ M = \frac{Pab^2}{L^2} + \frac{Pa^2 b}{L^2} = \frac{Pab}{L^2}(b + a)$

$a = 3L/4$

$b = L/4$

$(-)M = \frac{P}{L^2} \times \frac{3L}{4} \times \frac{L}{4}\left[\frac{L}{4} + \frac{3L}{4}\right] = \frac{3PL}{16}$

$= \frac{3\ (148)(70)}{16} = 1940$ kip-ft.

$(+)M = \frac{PL}{4} - \frac{3PL}{16} = \frac{PL}{16}$

Assuming full fixity at the support doesn't exist, (+)M is increased from PL/16 to PL/12

$(+)M = \frac{148(70)}{12} = 862$ kip.ft.

$(+)M$ (F.S.) $= 862 \times 1.5 = 1293$ kip-ft.

$(-)As = \frac{-M}{Jdf} = \frac{1940 \times 12}{7/8\ (30)\ (30)} = 29.5$ sq.in.

$(+)As = \frac{+M}{Jdf} = \frac{1293 \times 12}{7/8\ (30)(30)} = 17.8$ sq.in.

(+)As .... 8 No.10 str. bott.
8 No.10 bent.

(-)As .... 8 No.10 str. top.
8 No.10 bent.
6 No.10 str. top.

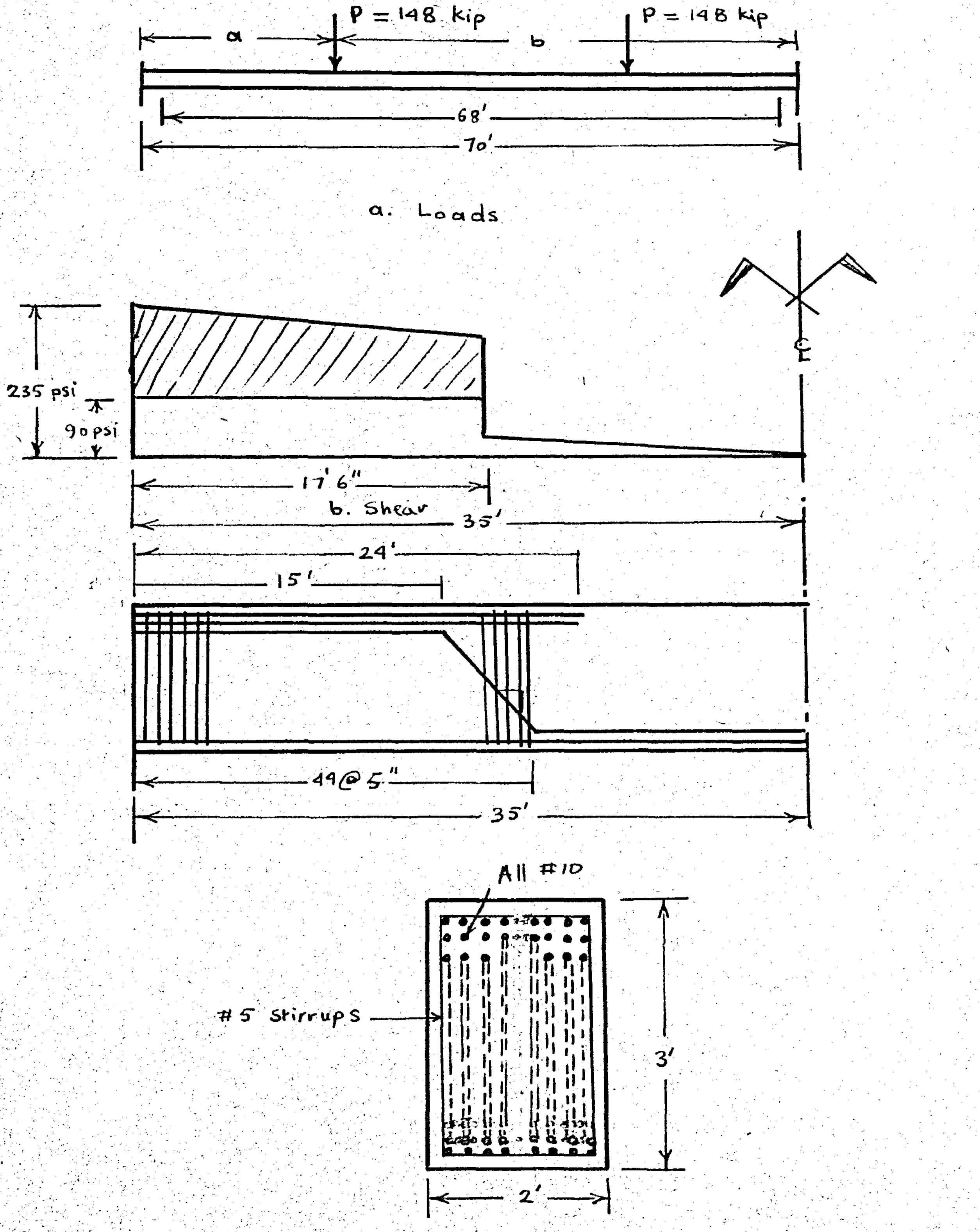


Fig. IV_38 Beam (6) MTV of Casino Floor - Proposal No. 5

Spacing: 8 X 1.27 = 10.16 in.

7 X 1.27 = 8.89 in.

2 X 2 = 4.00 in.

Total = 23.05 in 24 in. O.K.

Shear:

$V = P = 148$ kips.

$$v = \frac{V}{bJd} = \frac{148000}{24 \times 7/8 \times 30} = 235 \text{ psi} > 90 \text{ psi}$$

use stirrups, Fig. IV - 38

$x = 17.5$ ft.

Stirrup No.5 bars

$$S = \frac{A_v f_v}{v' b} = \frac{0.61 \times 30{,}000}{145 \times 24} = 5.2 \text{ in.}\ldots 5 \text{ in.}$$

use .... 44 at 5 in.

Bond: $$u = \frac{V}{OJd} = \frac{148000}{7/8 \times 87.8 \times 30} = 64 \text{ psi} \angle 210 \text{ psi} \qquad \text{O.K.}$$

* The long spans and heavy loads on the beam required the use of big amount of steel. However this design is within the ACI allowables. Here, this is preferable to the use of WF beams as was shown in Proposal No.2 . Big WF beams might not be available in the market, and even if they were available their transportation to the required position on top of the columns on the rock is very difficult and very costly.

7. Beam RTK   Fig. IV - 39

$L = 70$ ft.

$w = 4.25$ kips/lin.ft.

$(-)M = \frac{wl^2}{12} = \frac{4.25\ (70)^2}{12} = 1740$ kip-ft.

$(+)M = \frac{wl}{16} = \frac{4.25\ (70)}{16} \doteq 1300$ kip-ft.

$(-)As = \frac{-M}{Jdf} = \frac{1740 \times 12}{7/8 \times 30 \times 30} = 26.5$ sq.in.

$(+)As = \frac{+M}{Jdf} = \frac{1300 \times 12}{7/8 \times 30 \times 30} = 19.8$ sq. in.

(+)As .... 8 No.10 str.bott.
8 No.10 bent.

(−)As .... 8 No.10 str. top.
8 No.10 bent.
6 No.10 str. top.

Spacing:

Checks O.K. (similar to beam MTV)

Shear: $V = \frac{wl}{2} = \frac{4.25\ (20)}{2} = 148$ kips.

$v = \frac{V}{bJd} = \frac{148000}{24 \times 7/8 \times 30} = 235 \text{ psi} > 90$ psi.

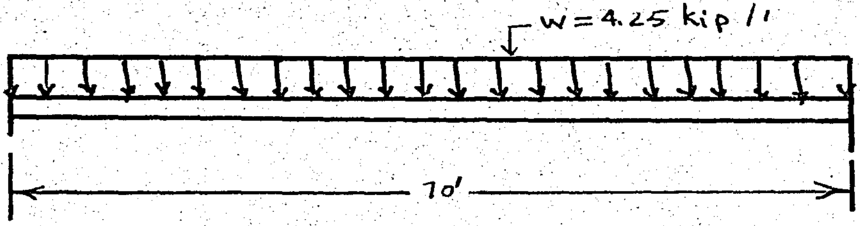
use stirrups, Fig. IV - 39

$x = 35 \times \frac{145}{235} = 21.5$ ft.

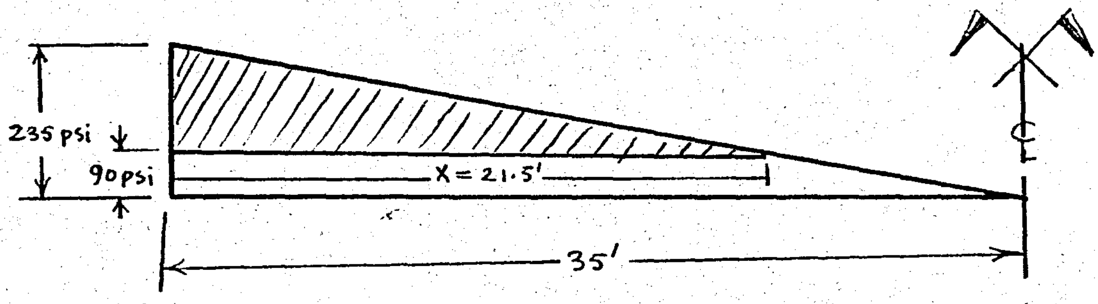
Stirrup No. 5 bars

$S = \frac{A_v f_y}{v'b} = \frac{0.61 \times 30,000}{145 \times 24} = 5.2$ in....... 5 in.

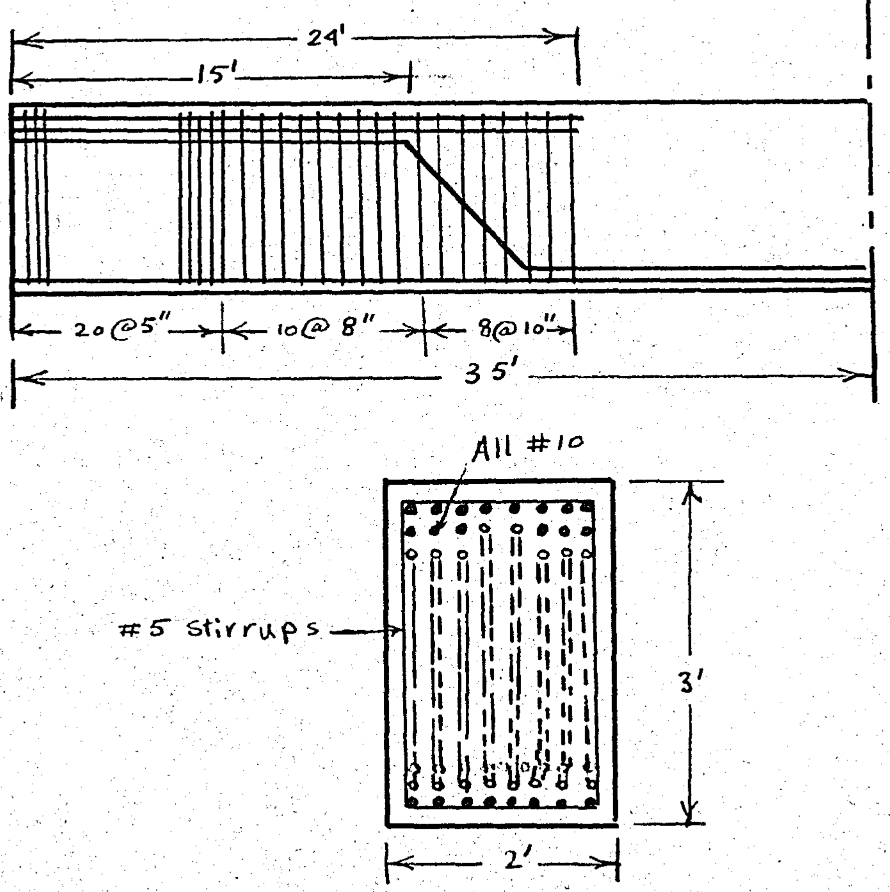
use .... 20 at 5 in.



a. Load



b. Shear



c. Steel Reinforcement

Fig. IV-39 Beam (7) RTK of Casino Floor – Proposal No. 5

10 at 8 in.

8 at 10 in.

Bond:

$$u = \frac{V}{0Jd} = \frac{148000}{7/8X87.8X30} = 64 \text{ psi} < 210 \text{ psi} \qquad \text{O.K.}$$

8. Beam AR Fig. IV-40

$L = 15$ ft.

$w = 0.190 \text{ X } 17.6 = 3.35$ kips/lin.ft.

w (D.L.) $= 0.15 \text{ X } 3 \text{ X } 2 = 0.9$ kips/lin.ft.

Total $w = 4.25$ kips/lin.ft.

$$(-)M = \frac{wl^2}{2} = \frac{4.25\ (15)^2}{2} = 480 \text{ kip-ft.}$$

$$(-)As = \frac{-M}{Jdf} = \frac{480 \text{ X } 12}{7/8 \text{ X } 30 \text{ X } 30} = 7.3 \text{ sq.in.}$$

(-)As 6 No.10

Spacing: Checks O.K. (Reference - Beam RTK)

Shear:

$V = wL = 4.25 \text{ X } 15 = 64$ kips.

$$v = \frac{V}{bJd} = \frac{64000}{24X7/8X30} = 101 \text{ psi} > 90 \text{ psi.}$$

Use stirrups, Fig. IV-40

$$x = 15 \text{ X } \frac{11}{101} = 1.63 \text{ ft.}$$

Stirrups No.5 bars

$$S = \frac{A_v f_v}{v'b} = \frac{0.61 \text{ X } 30{,}000}{24 \text{ X } 11} = 7 \text{ in.}$$

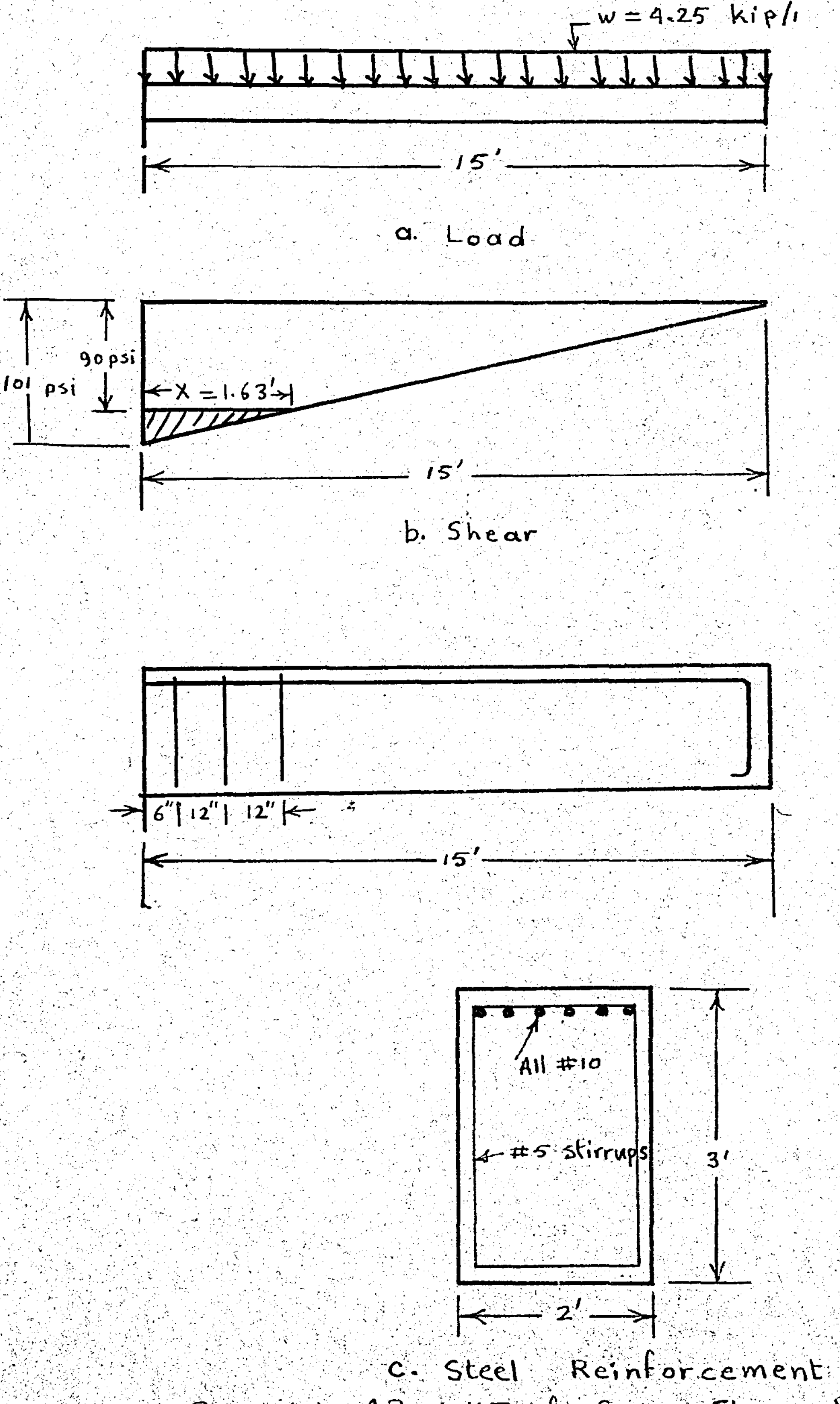


Fig. IV.40 Beam (8) AR & KJ of Casino Floor - Proposal No.5

use .... 1 at 6 in.

2 at 12 in.

Bond:

$$u = \frac{V}{OJd} = \frac{6400}{27.9X7/8X30} = 88 \text{ psi} < 210 \text{ psi} \qquad O.K.$$

9. Beam CP Fig. IV - 41

$L = 24.5$ ft.

w (from F_8) = 7 X 0.19 = 1.33 kip/lin.ft.

w (from F_5) = 8.8X 0.19 = 1.68 kip/lin.ft.

w (D.L.) = 3 X 2 X 0.15 1 0.90 kip/lin.ft.

Total w = 3.91 kip/lin.ft.

$$(-)M = \frac{wl^2}{2} = \frac{3.91\ (24.5)^2}{2} = 1180 \text{ kip-ft.}$$

$$(-)As = \frac{-M}{Jdf} = \frac{1180 \text{ X } 12}{7/8X30 \text{ X } 30} = 18 \text{ sq.in.}$$

(-)As 8 No.10

8 No.10

Spacing: Checks O.K. (similar to Beam MTV)

Shear:

$V = wL = 3.91 \text{ X } 24.5 = 96$ kips.

$$v = \frac{V}{bJd} = \frac{96000}{24 \text{ X } 7/8 \text{ X } 30} = 152 \text{ psi} > 90 \text{ psi.}$$

Use stirrups, Fig. IV - 41

$$x = 24.5 \text{ X } \frac{62}{152} = 10 \text{ ft.}$$

Stirrup No.5 bars

$$S = \frac{A_v f_v}{v'b} = \frac{0.61 \text{ X } 30{,}000}{24 \text{ X } 62} = 12.3 \text{ in....12 in.}$$

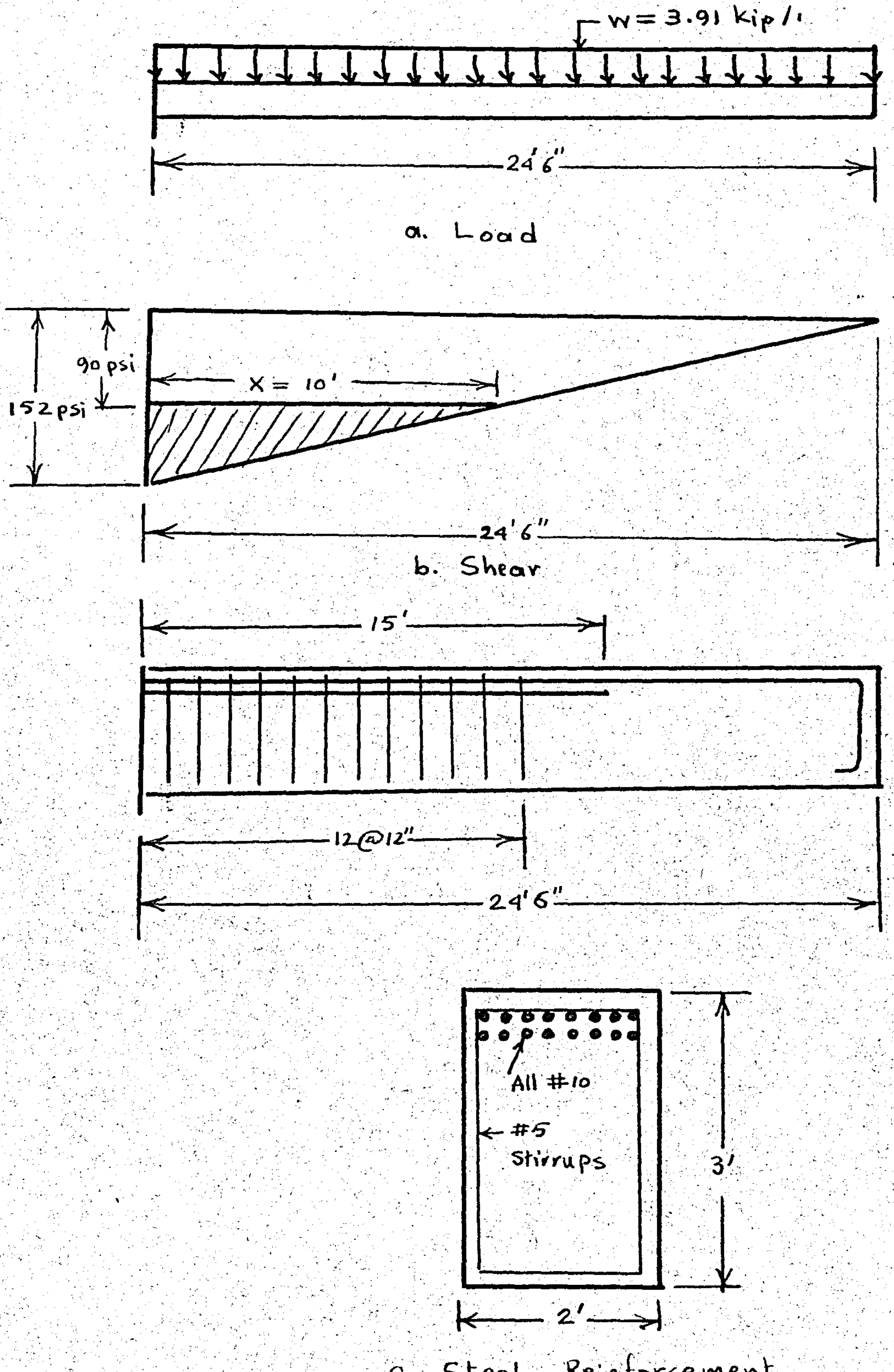


Fig. IV-41 Beam (9) CP of Casino Floor - Proposal No. 5

Use .... 12 at 12 in.

Bond: Checks O.K. (Reference- previous beams)

10. Beam PM Fig. IV - 42

$L = 35.3$ ft.

$w = 3.91$ kip/lin.ft. (similar to Beam CP).

$(-)M = \frac{wl^2}{12} = \frac{3.91\ (35.3)^2}{12} = 409$ kip-ft.

$(+)M = \frac{wl^2}{16} = \frac{3.91(35.3)^2}{16} = 306$ kip-ft.

$(-)As = \frac{-M}{Jdf} = \frac{409 \times 12}{7/8 \times 30 \times 30} = 6.25$ sq.in.

$(+)As = \frac{+M}{Jdf} = \frac{306 \times 12}{7/8 \times 30 \times 30} = 4.68$ sq.in.

(+) As .... 2 No.10 str. bott.

2 No.10 bent.

(-)As .... 3 No.10 str. top.

2 No.10 bent.

Spacing: Checks O.K. (Reference - Beam MTV)

Shear:

$Vz = \frac{wl}{2} = \frac{3.91 \times 35.2}{2} = 69$ kips.

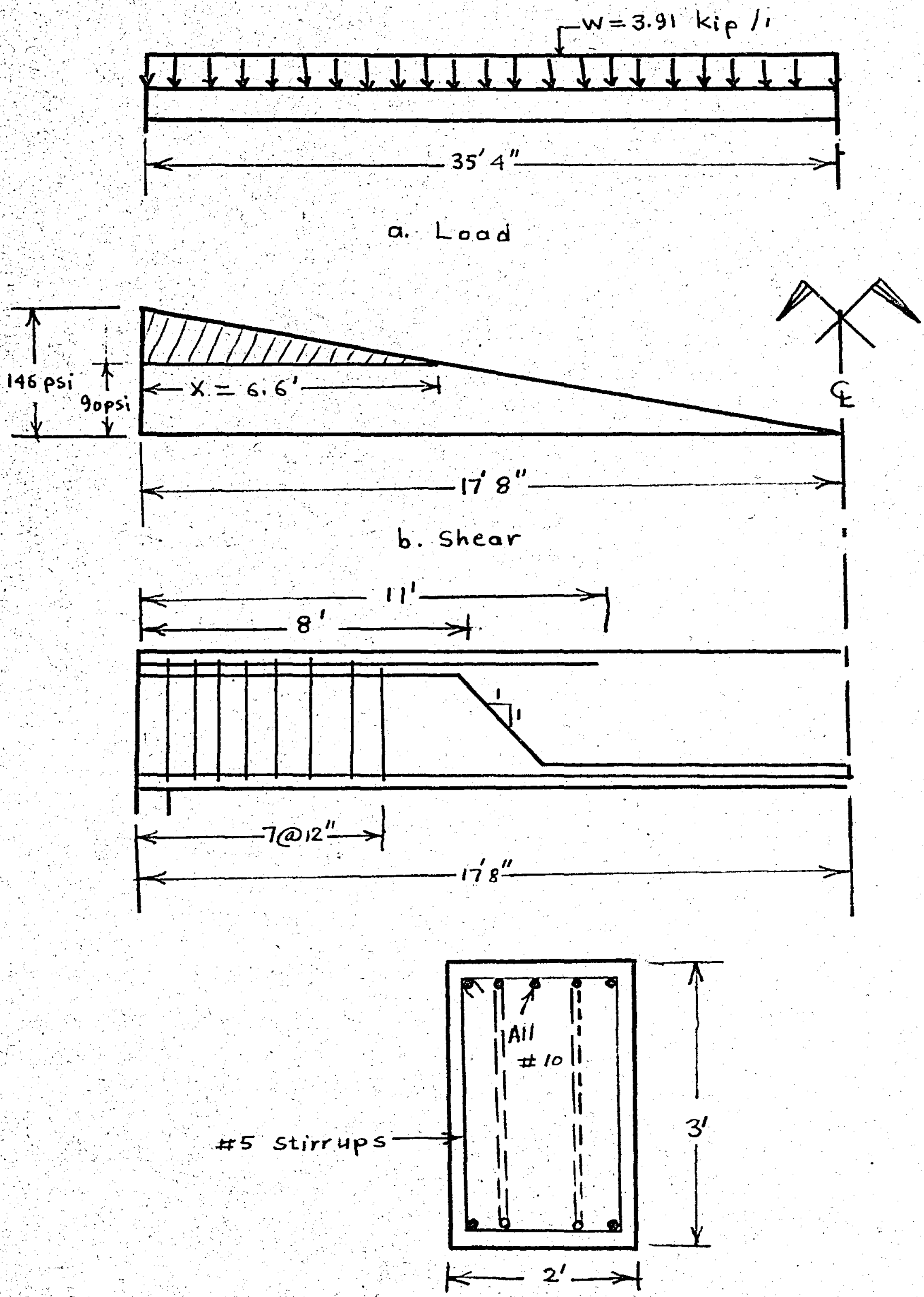
$v = \frac{V}{bJd} = \frac{69{,}000}{18 \times 7/8 \times 30} = 146$ psi $>$ 90 psi.

Use stirrups, Fig. IV - 42

$x = 17.2 \times \frac{56}{146} = 6.6$ ft.

Stirrups No.5 bars

$S = \frac{A_v f_v}{v' b} = \frac{0.61 \times 30{,}000}{56 \times 24} = 13.6$ in.... 12in.



c. Steel Reinforcement

Fig. IV-42 Beam (10) PM of Casino Floor – Proposal No. 5

use .... 7 at 12 ".

Bond:

$$u = \frac{V}{OJd} = \frac{69000}{20 \times 7/8 \times 30} = 132 \text{ psi} < 210 \text{ psi.}$$

11. Beam MG Fig. IV - 43

$L = 21.5$ ft.

$w = 3.91$ kips/lin.ft. (similar to Beam CP).

$$(-)M = \frac{wl^2}{2} = \frac{3.91\,(21.5)^2}{2} = 905 \text{ kip-ft.}$$

$$(-)As = \frac{-M}{Jdf} = \frac{905 \times 12}{7/8 \times 30 \times 30} = 13.8 \text{ sq.in.}$$

(-)As 7 No.10

4 No.10

Spacing:

Checks O.K. (reference beam MTV)

Shear:

$V = wl = 3.91 \times 21.5 = 84$ kips.

$$v = \frac{V}{Jbd} = \frac{84000}{24 \times 7/8 \times 30} = 133 \text{ psi} > 90 \text{ psi.}$$

Use stirrups, Fig. IV - 43.

$$x = \frac{A_v f_v}{v'b} = \frac{0.61 \times 30{,}000}{24 \times 43} = 17.7 \text{ in.} \; .... \; 12 \text{ in.}$$

Use 8 at 12 in.

Bond: Checks O.K. (Reference - previous beams).

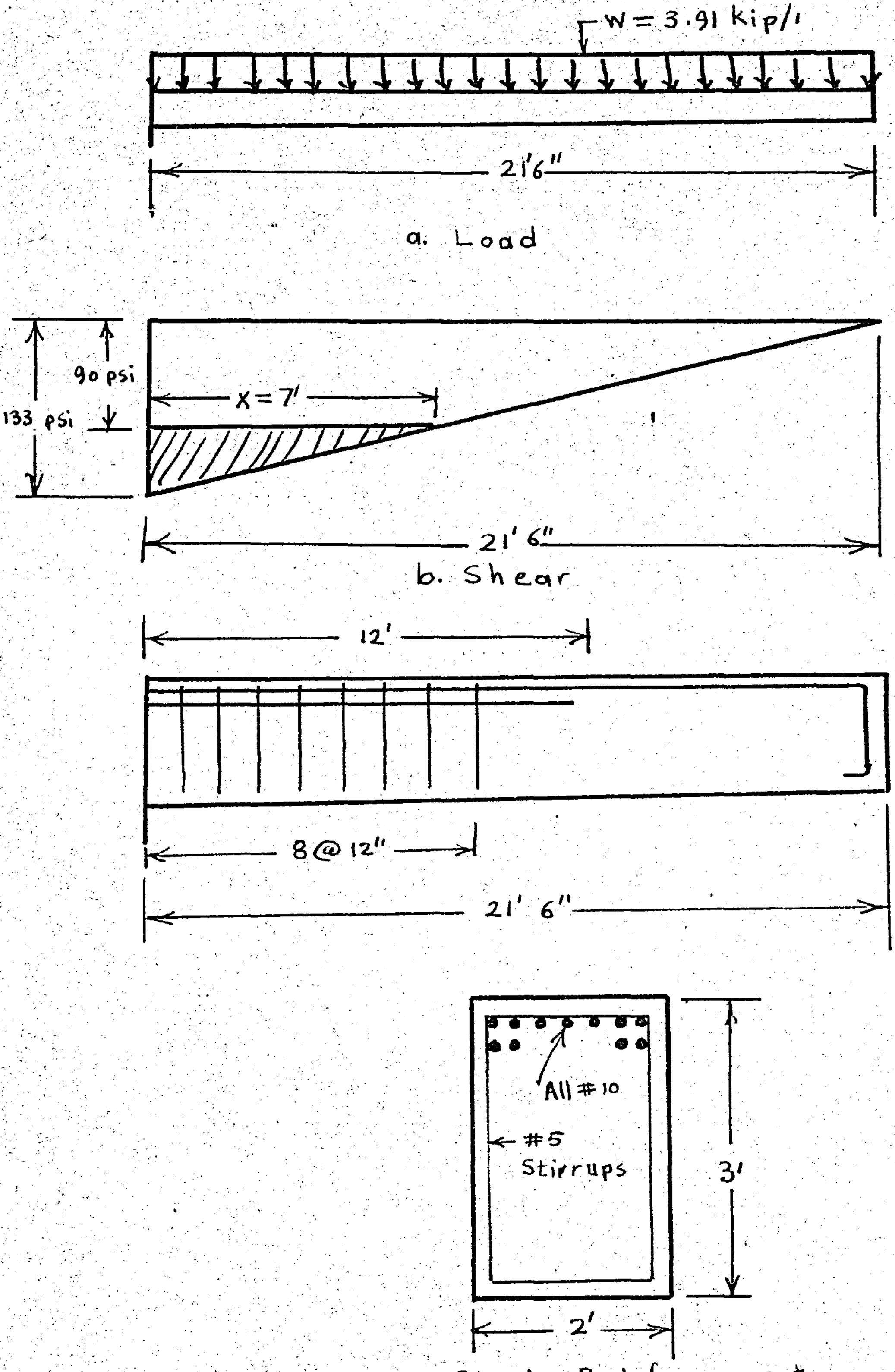


Fig. IV-43 Beam (11) MG of Casino Floor – Proposal No. 5

12. <u>Beam RP</u> Fig. IV-44.

$L = 35.3$ ft.

w (D.L.) $= 3 \times 2 \times 0.15 = 0.9$ kips/lin.ft.

P (concentrated load) = Max. shear of Beam QS +
Max. shear of Beam QB
$= 69 + 84 = 153$ kips.

Torsion exists (Table. IV - 3 .)

$$(-)M = \frac{wl^2}{12} + \frac{PL}{8} = \frac{0.9\,(35)^2}{12} + \frac{153\,(35)}{8}$$

$$= 762 \text{ kip-ft.}$$

$$(+)M = \frac{wl^2}{16} + \frac{PL}{8} = \frac{0.9\,(35)}{16} + \frac{153\,(35)}{8}$$

$$= 739 \text{ kip-ft.}$$

$$(+)M \text{ (F.S.)} = 69 + 670\,(1.5) = 1070 \text{ kip-ft.}$$

$$(-)As = \frac{-M}{Jf\,d} = \frac{762 \times 12}{7/8 \times 30 \times 30} = 114 \text{ sq.in.}$$

$$(+)As = \frac{+M}{Jdf} = \frac{1070 \times 12}{7/8 \times 30 \times 30} = 16.8 \text{ sq.in.}$$

(+)As .... 8 No.10 str. bott.
8 No.10 bent.
(-)As .... 8 No. 10bent.
4 No. 10 str. top.
(including 1 No. 1 for torsion).

Spacing:

Checks O.K. (similar to beam MTV).

Shear:

$$V = \frac{wl}{2} + \frac{P}{2} = \frac{0.9 \times 35}{2} + \frac{153}{2} = 92.3$$

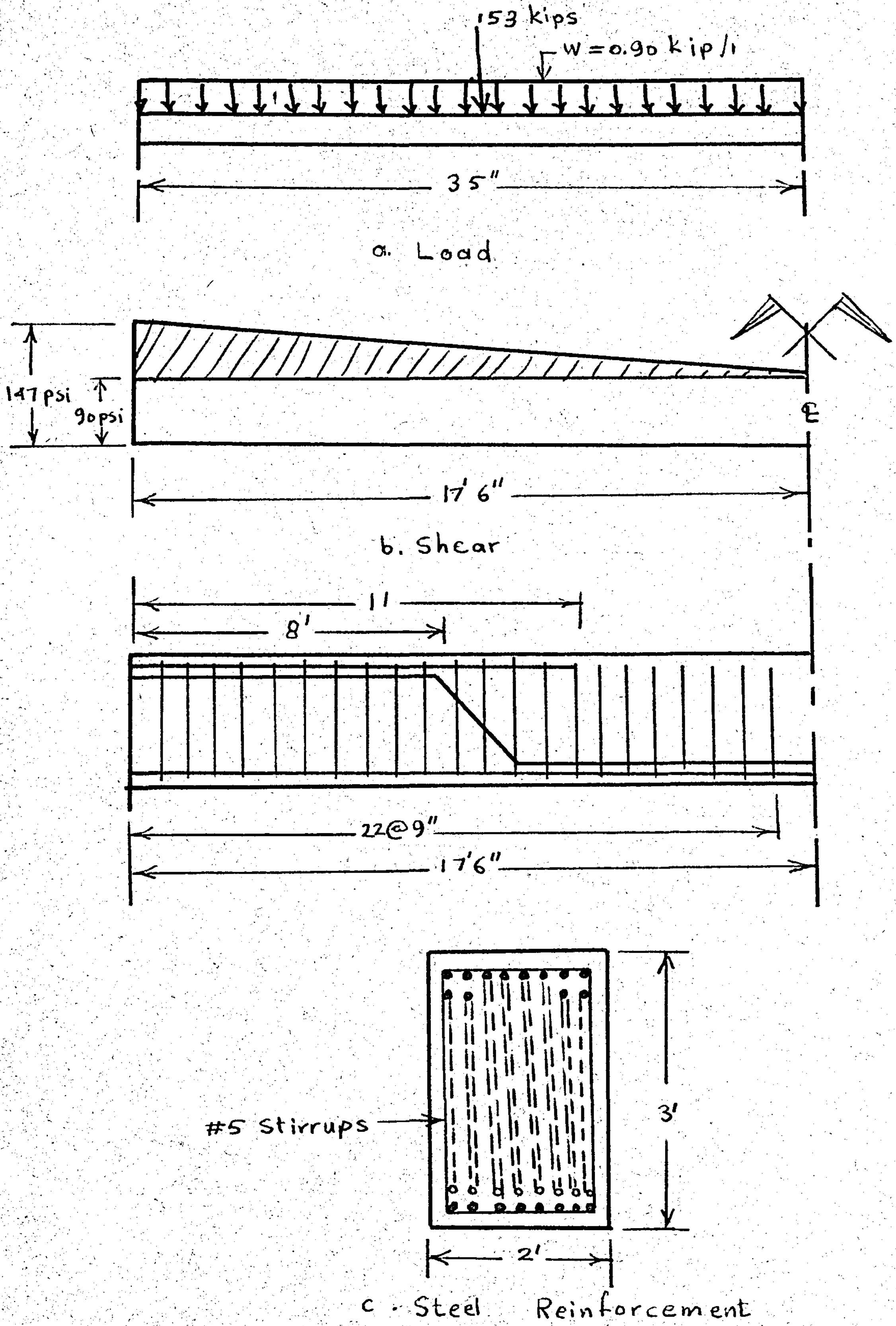


Fig. IV-44 Beam (12) RP of Casino Floor - Proposal No. 5

$$v = \frac{V}{bJd} = \frac{92300}{24 \times 7/8 \times 30} = 147 \text{ psi} > 90 \text{ psi.}$$

Use stirrups, Fig. IV-44.

$x = 17.5$ ft.

Stirrup No.5 bars.

$$S = \frac{A_v f_v}{v' b} = \frac{0.61 \times 30{,}000}{57 \times 24} = 13.4 \text{ in.}$$

Plus 1 No.5 at 24 in. for torsion.

Use 44 at 9 in.

Bond:

Check O.K. (Reference previous beams).

13. Beam PE Fig. IV-45.

$L = 24.5$ ft.

w (D.L.) $= 3 \times 2 \times 0.150 = 0.9$ kip/lin.ft.

P (concentrated load) at 14 ft. from support

= Max. shear from Beam ND +

Max. shear from Beam NF

$= 69 + 57 = 126$ kips.

Torsion exists (Table IV-3.)

$$(-)M = \frac{wl^2}{2} + PL' = \frac{0.9\,(24.5)^2}{2} + 126\,(14) = 2030 \text{ kip ft.}$$

$$(-)As = \frac{-M}{Jdf} = \frac{2030 \times 12}{7/8 \times 30 \times 30} = 31 \text{ sq.in.}$$

(-)As 8 No.10

8 No.10

8 No.10

2 No.10

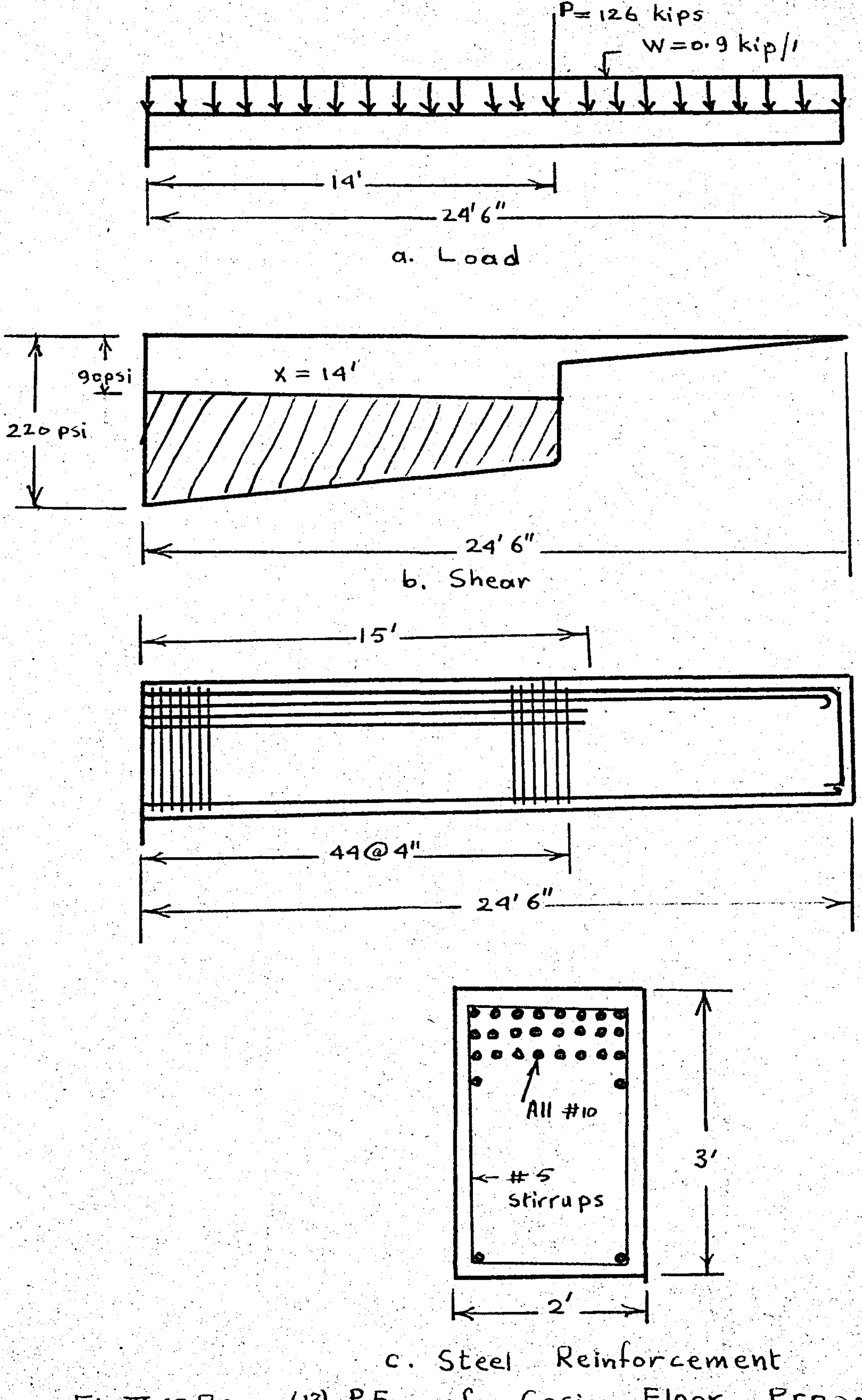


Fig. IV-45 Beam (13) P E of Casino Floor – Proposal No. 5

(including 1 No.10 for torsion).

+ 2 No.10 for torsion at bottom corners.

Spacing:

Checks O.K. (similar to Beam MTV).

Shear:

$V = P + wl = 126 + 0.9\ (24.5) = 138$ kips.

$v = \dfrac{V}{bJd} = \dfrac{138000}{24 \times 7/8 \times 30} = 220$ psi > 90 psi.

Use stirrups, Fig. IV-45.

$x = 14$ ft.

Stirrups No.5 bars.

$S = \dfrac{A_v f_y}{v' b} = \dfrac{0.61 \times 30{,}000}{24 \times 130} = 5.8$ in.

plus 1 No.5 at 24 in. for torsion.

use .... 44 at 4 in.

Bond:

Checks O.K. (Reference - previous beams).

14. Beam MF Fig. IV-46.

$L = 14$ ft.

w (D.L.) $= 3 \times 2 \times 0.15 = 0.9$ kips/lin.ft.

P(concentrated load) = max. shear in beam MF = 57 kips.

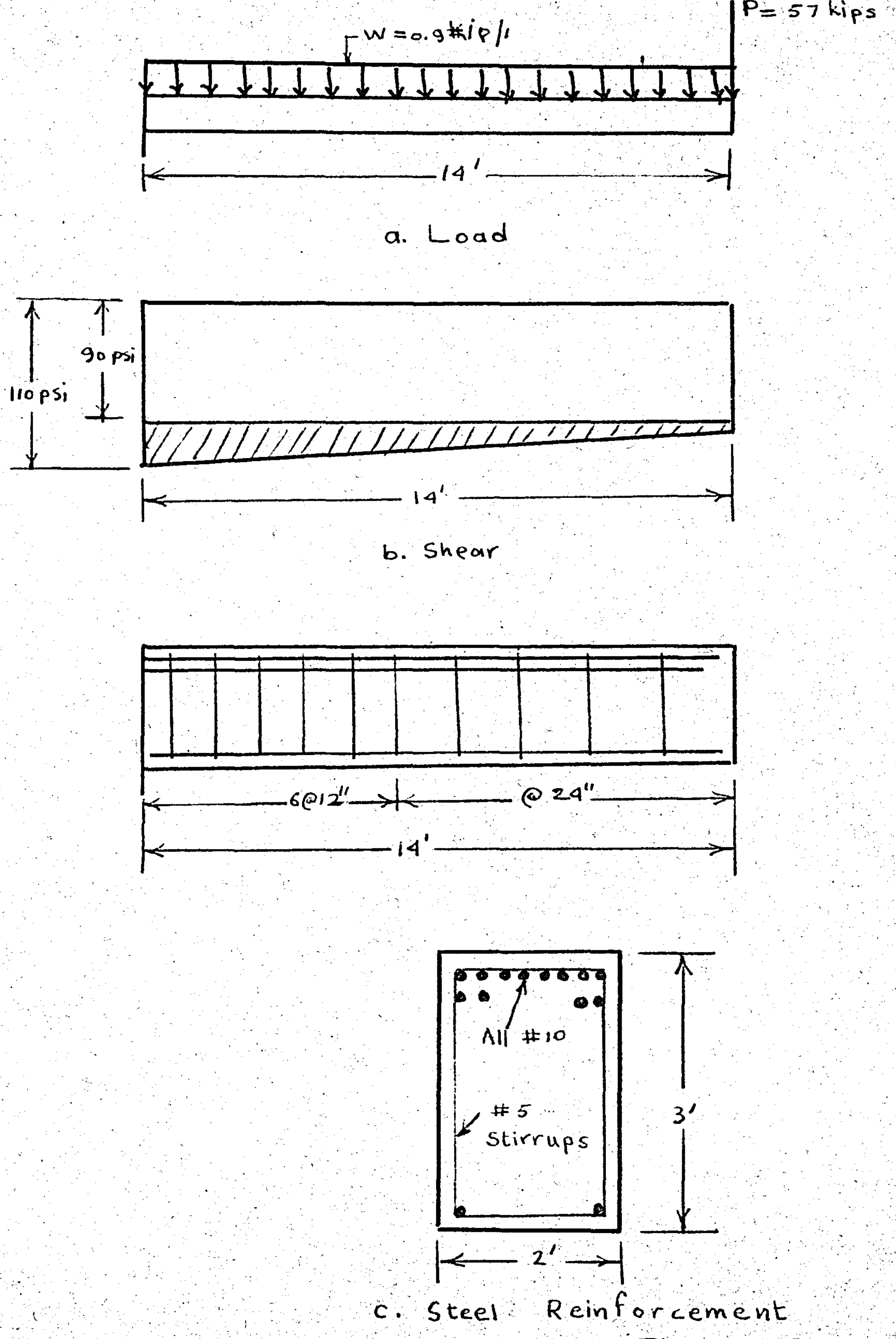


Fig. IV-46 Beam (14) MF of Casino Floor - Proposal No. 5

Torsion exists (Table IV - 3 .)

$$(-)M = \frac{wl^2}{2} + PL$$

$$= \frac{0.9\ (14)}{2} + 57\ (14) = 888 \text{ kip-ft.}$$

$$(-)As = \frac{-M}{Jf\ d} = \frac{888 \times 12}{7/8 \times 30 \times 30} = 13.6 \text{ sq.in.}$$

(-)As =.......... 8 No.10

4 No.10

including 1 No.10 for torsion.

plus 2 No.10 for torsion at bottom corners.

Spacing:

Checks O.K. (similar to Beam MTV).

Shear:

$$V = P + wl = 57 + 0.9\ (14) = 69.6 \text{ kips.}$$

$$v = \frac{V}{bJd} = \frac{69600}{24 \times 7/8 \times 30} = 110 \text{ psi} > 90 \text{ psi.}$$

Use stirrups, Fig. IV - 46

$$x = 14 \times \frac{20}{110} = 2.6 \text{ ft.}$$

Stirrups No. 5 bars

$$S = \frac{A_v f_v}{v'b} = \frac{0.61 \times 30{,}000}{24 \times 20} = 38 \text{ in.} \ .... \ 12 \text{ in.}$$

Plus 1 No.5 at 24 in. for torsion.

use ....4 at 12 in.

5 at 24 in.

Bond: Checks O.K. (Reference - previous beams).

15. Beam KL Fig. IV 47.

$L = 17.5$ ft.

w (D.L.) $= 3 \times 2 \times 0.15 = 0.9$ kips/linft.

P (concentrated load) = max. shear in Beam LS = 69 kips.

Torsion exists

$$(-)M = \frac{wl^2}{2} + Pl = \frac{0.9\ (17.5)}{2} + 69\ (17.5) = 1348 \text{ kip-ft.}$$

$$(-)As = \frac{M}{Jdf} = \frac{1348 \times 12}{7/8 \times 30 \times 30} = 20.6 \text{ sq.in.}$$

(-)As 8 No.10

8 No.10

2 No.10

including 1 No.10 for torsion.

plus 2 No.10 for torsion at bottom corners.

Spacing: Checks O.K. (similar to Beam MTV)

Shear: $V = P + wl = 69 + 0.9\ (17.5) = 84.8$ kips.

$$v = \frac{V}{bJd} = \frac{84{,}8000}{24 \times 7/8 \times 30} = 134 \text{ psi} > 90 \text{ psi.}$$

Use stirrups Fig. IV 47

$$x = 17.5 \times \frac{44}{134} = 5.8 \text{ ft.}$$

stirrups No.5 bars

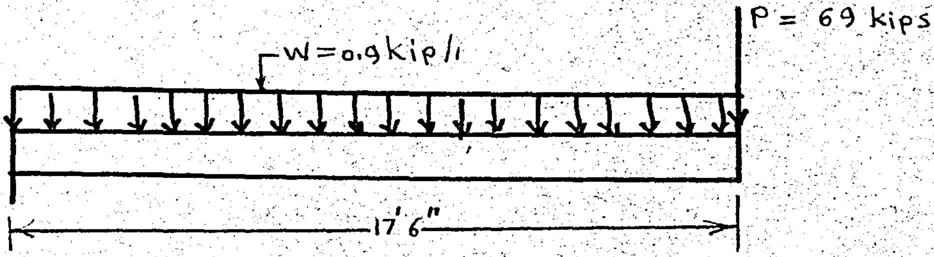
$$S = \frac{A_v f_v}{v' b} = \frac{0.61 \times 30{,}000}{24 \times 44} = 17.3 \text{ in.}$$

plus 1 No.5 at 24" for torsion.

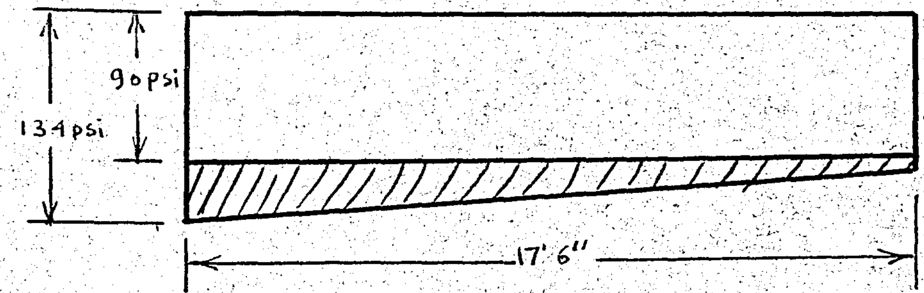
use .... 8 at 10 in.

5 at 24 in.

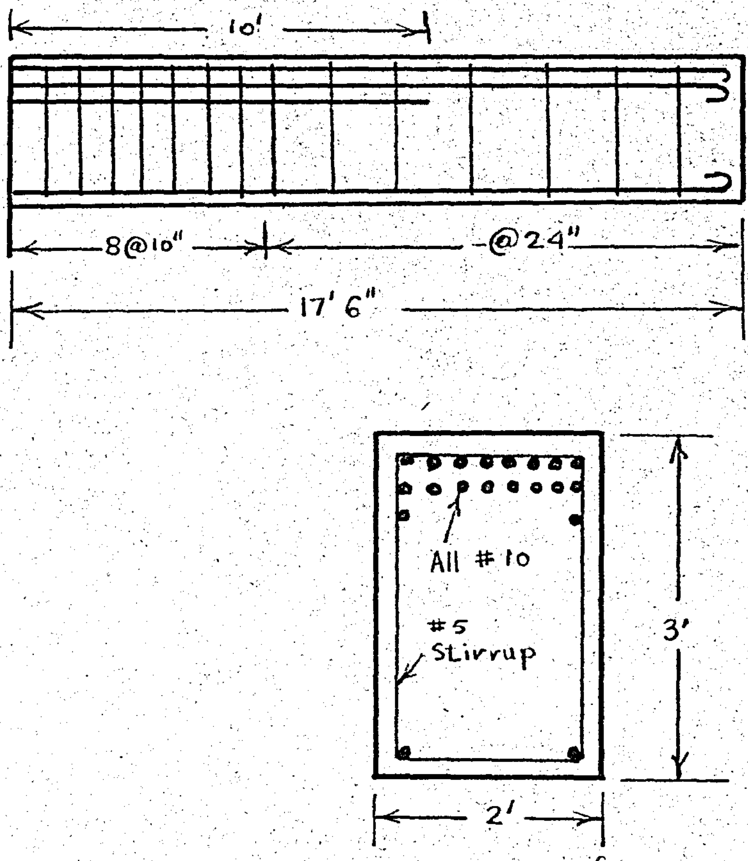
Bond: Checks O.K. (Reference - previous beams).



a. Load



b. Shear



c. Steel Reinforcement

Fig. IV-47 Beam (15) KL of Casino Floor - Proposal No. 5.

Bill of Materials of Casino Floor Beams

Steel & Concrete

Member	No. of Parts	Bars	Shape	Length ft	Wt./ft lbs.	Wt. lbs.	Concrete ft^3
Beam 1	6	12 # 10	Straight	23	4.30	7120	71
	6	2 # 10	Straight	10	4.30	520	
	6	19 # 5	Stirrup	8	1.04	950	
Beam 2	6	2 # 10	Straight	36	4.30	1860	120
	6	2 # 10	Bent	40	4.30	2060	
	6	10 # 10	Straight	2 X 12	4.30	6200	
	6	14 # 3	Stirrup	2 X 8	0.380	510	
Beam 3	6	2 # 10	Straight	36	4.30	1860	120
	6	2 # 10	Bent	40	4.30	2060	
	6	10 # 10	Straight	2 X 12	4.30	6200	
	6	14 # 3	Stirrup	2 X 8	0.38	510	
Beam 4	6	6 # 10	Straight	23	4.30	3560	71
	6	6 # 10	Straight	14	4.30	2180	
	6	21 # 3	Stirrup	8	0.380	380	
Beam 5	6	2 # 10	Straight	36	4.30	1860	120
	6	2 # 10	Bent	40	4.30	2060	
	6	6 # 10	Straight	2 X 12	4.30	3720	
	6	12 # 10	Stirrups	2 X 8	4.30	4950	
Beam 6	3	8 # 10	Straight	72	4.30	7400	200
	3	8 # 10	Bent	78	4.30	7850	
	3	14 # 10	Straight	2 X 20	4.30	7250	
	3	40 # 5	Stirrups	2 X 10	4.04	2600	

Bill of Materials of Casino Floor Beams (Con't.)

Member	No.of Parts	Bars	Shape	Length ft	Wt./ft lbs.	Wt. lbs	Concrete ft^3
Beam 7	3	8 # 10	Straight	72	4.30	7400	200
	3	8 # 10	Bent	76	4.30	7850	
	3	12 # 10	Straight	2 X 20	4.30	6200	
	3	38 # 5	Stirrup	2 X 10	1.04	2370	
Beam 8	6	6 # 10	Straight	16	4.30	2480	114
	6	3 # 5	Stirrup	10	1.04	190	
Beam 9	6	8 # 10	Straight	28	4.30	5800	130
	6	8 # 10	Straight	16	4.30	3310	
	6	12 # 5	Stirrups	10	1.04	760	
Beam 10	6	10 # 10	Straight	36	4.30	9320	120
	6	2 # 10	Bent	40	4.30	2070	
	6	3 # 10	Straight	2 X 12	4.30	1860	
	6	7 # 5	Stirrups	2 X 10	1.04	870	
Beam 11	6	7 # 10	Straight	22	4.30	3980	71
	6	4 # 10	Straight	12	4.30	1240	
	6	8 # 5	Stirrups	10	1.04	500	
Beam 12	6	8 # 10	Straight	36	4.30	7420	120
	6	8 # 10	Bent	40	4.30	8220	
	6	4 # 10	Straight	2 X 12	4.30	2480	
	6	44 # 5	Stirrups	2 X 10	1.04	5500	
Beam 13	6	8 # 10	Straight	28	4.30	5800	130
	6	8 # 10	Straight	22	4.30	4550	
	6	8 # 10	Straight	16	4.30	3310	
	6	2 # 10	Straight	12	4.30	570	

Bill Of Materials of Casino Floor Beams (Con't.)

Member	No. of Parts	Bars	Shape	Length ft	Wt./ft lbs.	Wt. lbs	Concrete ft^3
	6	2 # 10	Straight	28	4.30	1450	
	6	44 # 5	Stirrups	2 X 10	1.04	5500	
Beam 14	6	10 # 10	Straight	15	4.30	5880	76
	6	4 # 10	Straight	10	4.30	1040	
	6	10 # 5	Stirrup	10	1.04	630	
Beam 15	6	10 # 10	Straight	20	4.30	5180	92
	6	10 # 10	Straight	12	4.30	3100	
	6	12 # 5	Stirrups	10	1.04	750	
					Total:	189,240	1755

* Estimated Value of blasted rock = leveling + basement blasting

Leveling = Av. area X av. depth.

= 100 X 50 X 8 = 40,000 ft^3.

Basement = Area of polygon X depth.

= 3750 X 10 = 37,500 ft^3.

Total Volume = 77,500 ft^3.

DESIGN OF CASINO COLUMNS

Loads and Moments on Columns.

Stiffness Factors:

Beams:

All the major beams have the same cross-sectional area of 2 ft. by 3 ft.

K of cantilever beams: Stiffness of cantilever end at the support is taken as zero as the carry-over factor to the free end is zero.

K of fixed-fixed beams:

$$I = \frac{bh^3}{12} = \frac{2(3)^3}{12} = 4.5 \text{ ft}^4.$$

$$L = 35 \text{ ft.}$$

$$K = \frac{I}{L} = \frac{4.5}{35} = 0.128$$

Columns:

All columns have the same length L = 15 ft. and the same outside diameter of 3 ft.

K of columns:

$$I = \frac{\pi R^4}{4} = \frac{\pi (3/2)^4}{4}$$

$$K = \frac{I}{L} = 0.265$$

$$k \text{ (column)} = \frac{K \text{ (column)}}{\text{sum of K at joint}} = \frac{0.265}{0.393} = 0.675$$

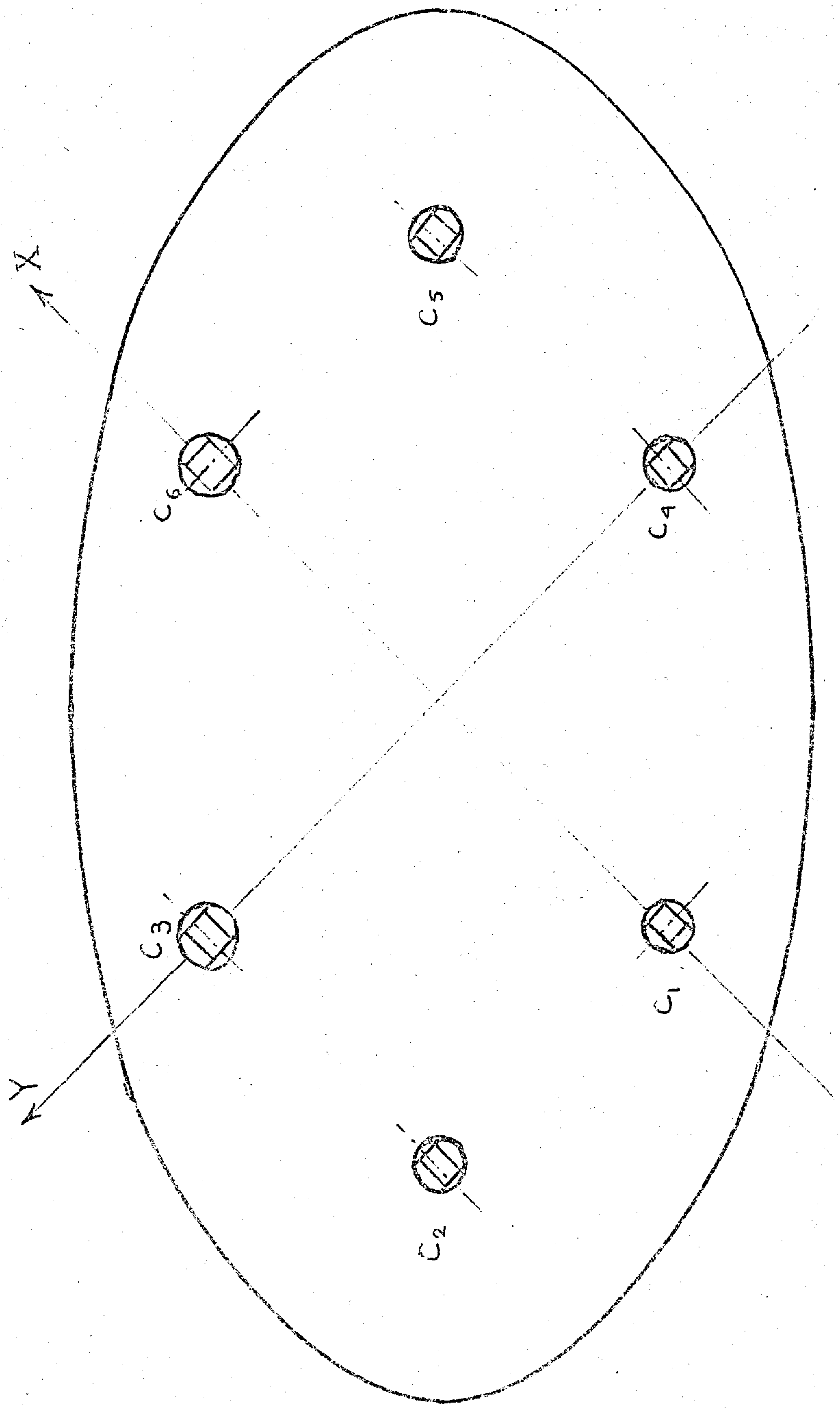


Fig IV-48 Plan of Casino Columns; Orientation of Steel Cores

Table IV - 6 Moments Resisted by Columns kip-ft.

Floor	Column	M_x	M_y	M	k	k M
Top Floor	1	586	1260	1390	0.675	940
"	2	1268	771	1540	0.675	1040
"	3	1052	496	1160	0.675	780
"	4	1052	496	1160	0.675	780
"	5	1268	771	1540	0.675	1040
"	6	586	1260	1390	0.675	940
First Floor	1	586	1260	1390	0.675	940
"	2	1268	771	1540	0.675	1040
"	3	1052	496	1160	0.675	780
"	4	1052	496	1160	0.675	780
"	5	1268	771	1540	0.675	1040
"	6	586	1260	1390	0.675	940
Basement	1	586	1260	1390	0.675	940
"	2	1268	771	1540	0.675	1040
"	3	1052	496	1160	0.675	780
"	4	1052	496	1160	0.675	780
"	5	1268	771	1540	0.675	1040
"	6	586	1260	1390	0.675	940

Table IV-7 Moments at Joints (Kip-ft.)

Joint	M_x	(-) M_x	M_x	M_y	(-) M_y	M_y
1	1348	-762	586	480	-1740	-1260
2	762	-2030	-1268	1180	-409	771
3	1940	-888	1052	409	-905	-496
4	888	-1940	-1052	905	409	496
5	2030	-762	1268	409	-1180	-771
6	762	-1348	-586	1740	-480	1260

Table IV - 8 Loads On Columns (kips)

Floor	Column	P_1	P_2	P_3	P_4	P_5	P_6	P
Top Floor	1	85	92	64	148	6	452	847
"	2	92	138	96	69	0	0	395
"	3	148	70	69	84	0	0	371
"	4	148	70	69	84	6	452	830
"	5	92	138	96	69	0	0	395
"	6	85	92	64	148	0	0	389
First Floor	1	85	92	64	148	13	847	1249
"	2	92	138	96	69	13	395	803
"	3	148	70	69	84	13	371	755
"	4	148	70	69	84	13	830	1224
"	5	92	138	96	69	13	395	803
"	6	85	92	64	148	13	389	791
Basement	1	85	92	64	148	13	1249	1651
"	2	92	138	96	69	13	803	1211
"	3	148	70	69	84	13	755	1139
"	4	148	70	69	84	13	224	1608
"	5	92	138	96	69	13	803	1211
"	6	85	92	64	148	13	791	1193

* P_1, P_2, P_3, & P_4 are the loads due to max. shears of the supported beams.
* P_5 is dead load of column.
* P_6 direct loads on columns from columns above.

As the heavy structure is supported by six columns only, the values of the axial loads and moments on the columns are quite as shown in Table .

It is preferable in this case to design steel columns encased in concrete where the steel is designed to carry the whole load on a column.

The steel core is going to be encased in concrete to have a total diameter of three feet as was originally planned. The encasement will serve a further function and that is it prevents corrosion of the steel core. The concrete encasement will be reinforced with a wire mesh.

The steel column will be designed to carry the whde load on it prior to construction of concrete encasement.

The cores will be chosen to have the same cross section for simplicity of construction and for perfect alingment of cores on top of each otherş provided that economy still prevails.

Design of Columns 1,3,4 and 6 of Floors 1, 2, and 3

M (max) = 940 kip-ft.

N (max) =1651 kips.

Try 2 - 14 WF 287 plus 3/4 in. cover plates, Fig IV-49.

A = 84.37

Depth = 16.81

Width = 16.13

$I = 3912 \text{ in.}^4$

$x = \sqrt{r^2 - (d/2)^2} = \sqrt{(18)^2 - (8.07)^2}$

x = 16.1 in. approx. = 16.13 in. O.K.

Total cross sectional area A:

A = 2(84.37) + 30 X 3/4 X 2

$A = 214 \text{ in}^2.$

$\text{Max.f(axial)} = \frac{N(max)}{A} = \frac{1651}{214} = 7.72 \text{ ksi.}$

$\text{Max.f(bending)} = \frac{M(max)C}{I}$

M(max) = 940 kip-ft.

C = 8.41 + 0.75 = 9.16

I = 2(3912) + 45(8.41+0.37)

$= 11{,}284 \text{ in}^4.$

$\text{Max.f(bending)} = \frac{940 \times 12 \times 9.16}{11284} = 9.4 \text{ ksi}$

Max. Stress = 9.4 + 7.72 = 17.12 ksi $<$ 18 ksi O.K.

Design of Columns 2 and 5 for Floors 1, 2, and 3:

M (max.) = 1040 kip-ft.

N (max.) = 1211 kips.

Try the same design of column 1

Max.f(axial) $= \frac{1211}{214} = 5.68$ ksi

Max.f(bending) $= \frac{1040 \times 12 \times 9.16}{11,284} = 10.2$ ksi

Max. stress $= 10.2 + 5.68 = 15.88$ ksi 18 ksi O.K.

Reinforcement of Concrete Encasement

The concrete shall be reinforced with welded wire mesh having wires of No. 10 AS and W gage or its equivalent.

The wires encircling the column being spaced not more than 4 in. apart and those parallel to the column axis not moe than 8 in. apart . This mesh shall extend entirely around the column at a distance of one inch inside the outer concrete surface (except at the four corners where the distance will not exceed half an inch.)

The space inside the steel column will be filled with concrete.

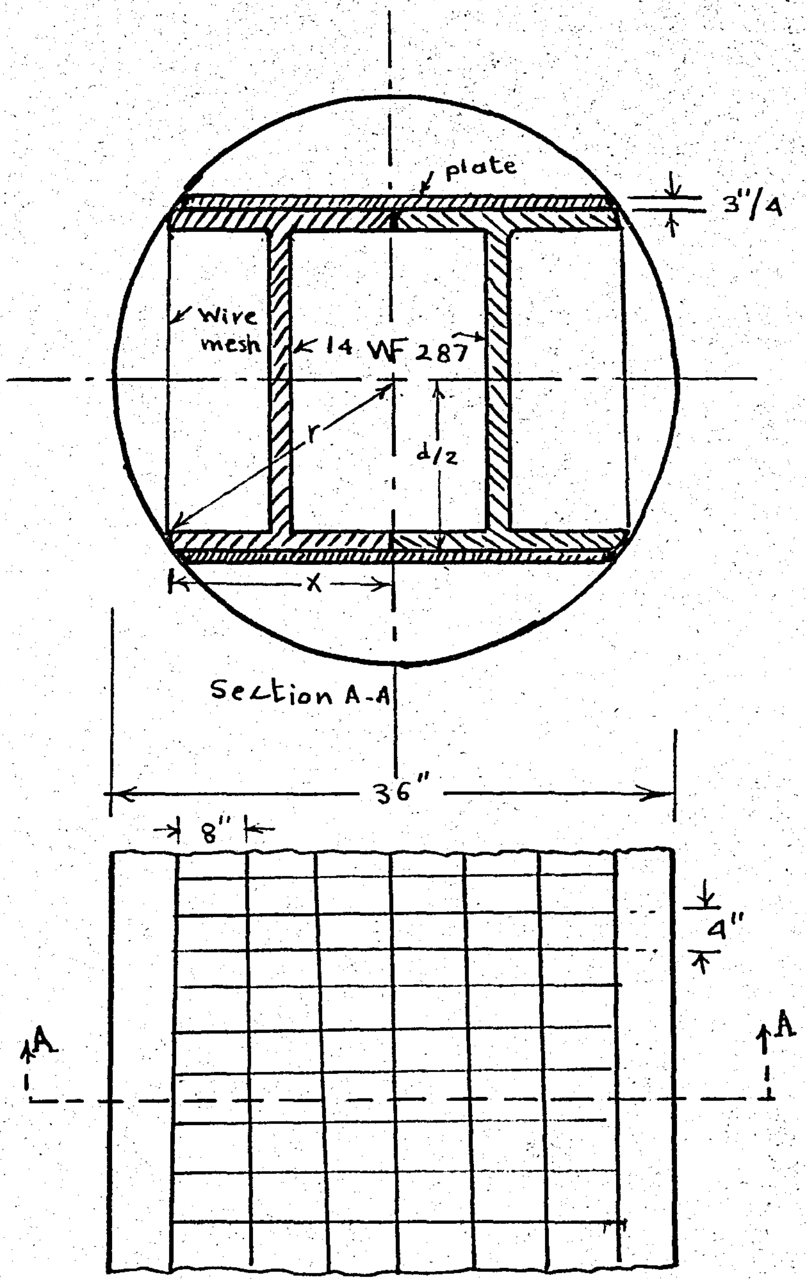


Fig. IV-49 Casino Columns - Steel Reinforcement.

Construction of Columns:

Each two 14 WF 287 of a column will be welded together along the outside edges.

The corner plates will be welded to the column at the edges and along the entire length of the column to provide a homogeneous cross section.

Special brackets shall be used to receive the entire floor load at each floor level.

The columns will be oriented in the direction of maximum moments as shown in Fig. .

Footings:

These will consist of plain conrete square blacks of 6 ft. X 3 ft. 10 #9 dowels each five feet long will be used to connect with the supporting footing.

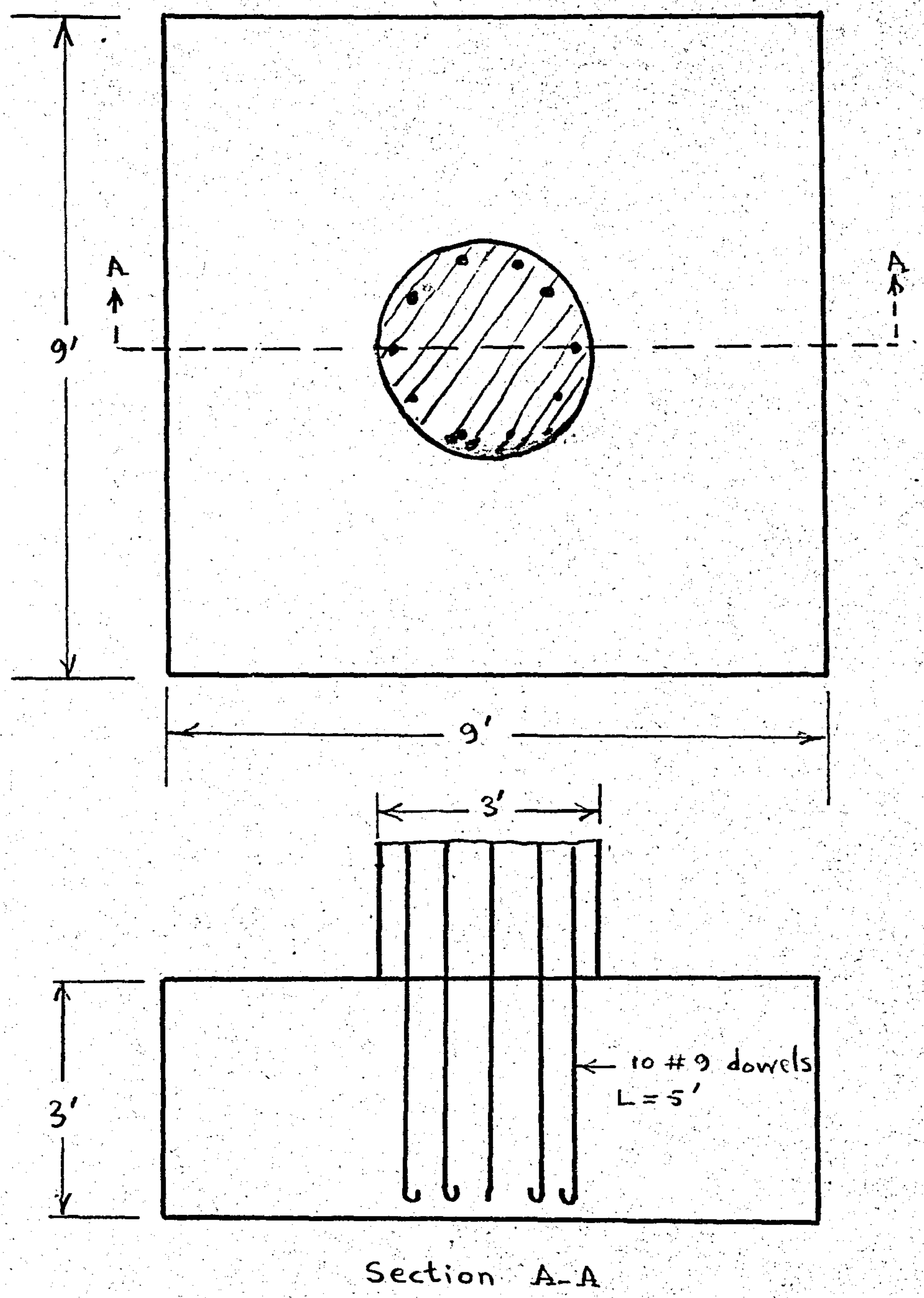


Fig. IV-50 Casino Footings.

Bill of Materials of Casino Columns

Steel and Concrete

Material	No. of Parts	Material by part	Total	Total Wt.
Concrete	18	(76 + 75) ft	2718 ft	
Cover Plates	18	3/4 in. 60 ft	3/4in. 1080 ft	33 kips.
WF Beams	18	24 ft. 14WF 287	432ft.14WF 287	124 kips.
Wire Mesh	18	440 ft.No.10AS&W	7920ft.No.10AS&W	
Dowels	6	10 #9 . 5 ft.	300 ft. #9	1020 lbs.

Chapter V

DESIGN OF THE SHORE BUILDING

General Illustrations

Architectural:

The fact that the area of the casino is limited, the space over the rock should not be misused as was mentioned in Chapter IV. Things which are not directly related to business at the casino should be placed in the shore building.

The shore building consists of a small structure. Fig. It includes entrance to the bridge, kitchens, store, staff room, and administration.

On the southern side of the building there willbe a parking place big enough to accomodate one hundred to two hundred cars. Architectural design and dimensions are shown in the following pages.

Structural: f_s = 20000 psi tensile strength (structural steel)

* f'_c = 2000 psi at 28 days.
* live load assumed = 50 lbs/sq.ft.
* Foundation is laid on solid rock of 9000 psi crushing strength.

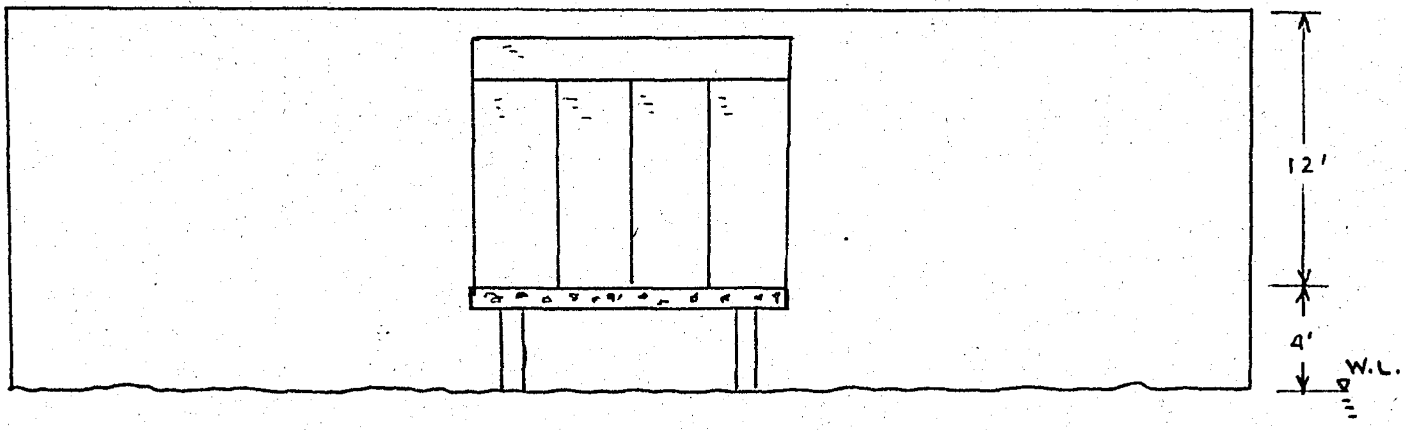


Fig. V-1 Shore Building - North View (Scale 1/100)

Pigeon Rock Casino

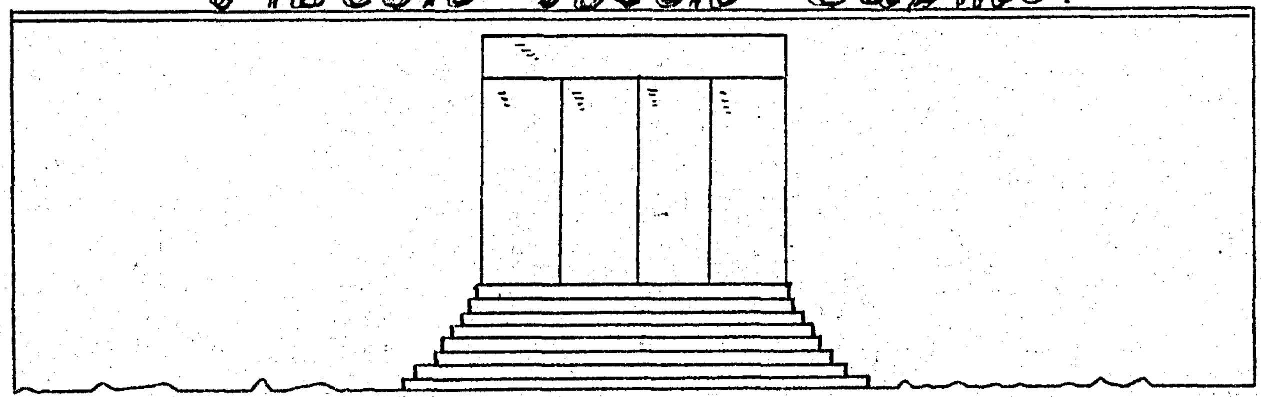


Fig V-2 Shore Building South View (Scale 1/100)

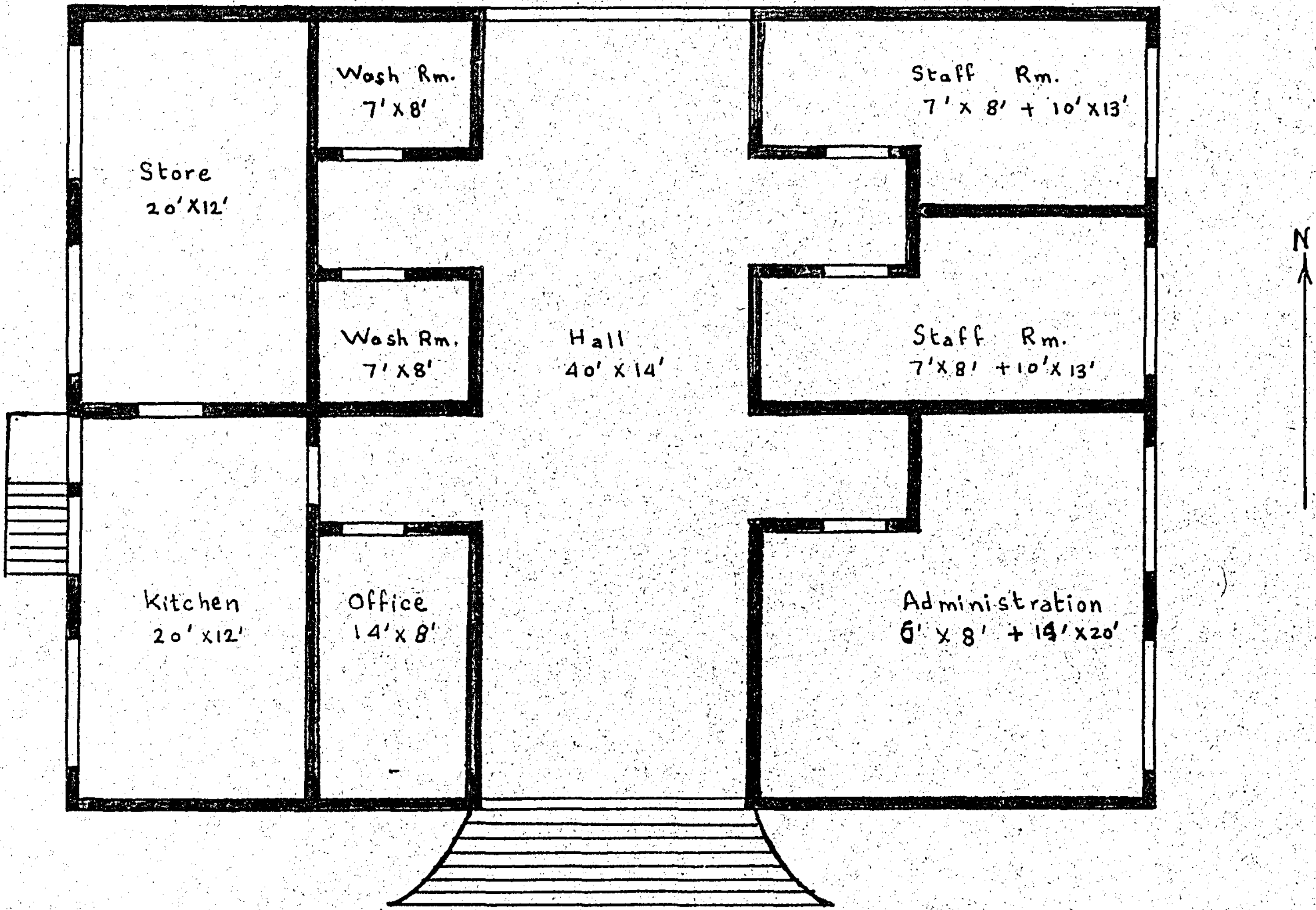


Fig. V-3 Shore Building Plan (Scale 1/100)

Structural Design

Design of Slabs

Design of Slab S_1: Fig. V - 4 .

Type: Two way slab.

Thickness: t = 6 in.

Perimeter/180 = (80 X 12) / 180 = 5.33 in. 6 in. O.K.

Load: w = 0.125 kips/ sq.ft.

Moments: Short span = long span (Two edges disc.)

1. (-)M at cont. edge = 0.049 X 0.125 X (20) = -2.45 kip-ft.
2. (-)M at disc. edge = 0.025 X 0.125 X (20) = -1.25 kip-ft.
3. (+)M at midspan = 0.037 X 0.125 X (20) = +1.85 kip-ft.

Steel: $As = \frac{M}{fsJd} = \frac{M}{20\ (7/8)\frac{(6)}{12}} = \frac{M}{8.8}$

1. (-)As = -0.278 sq.in. /ft. strip.
2. (-)As = -0.141 sq.in. /ft. strip.
3. (+)As = +0.210 sq.in. /ft. strip.

(Arrange steel with adjacent slabs) Fig. V-5,6 .

Design of Slab S_2: Fig. V - 4

Type: Two way slab.

Thickness: t = 6 in. (similar to S_1).

Load: w = 0.125 kip/sq.ft.

Moments: similar to S_5 (one edge disc.)

1. (-)M at cont. edge = 0.041X0.125X(20) = -2.05 kip-ft.
2. (-)M at disc. edge = 0.021X0.125X(20) = -1.05 kip-ft.
3. (+)M at midspan = 0.031X0.125X (20) = +1.55 kip-ft.

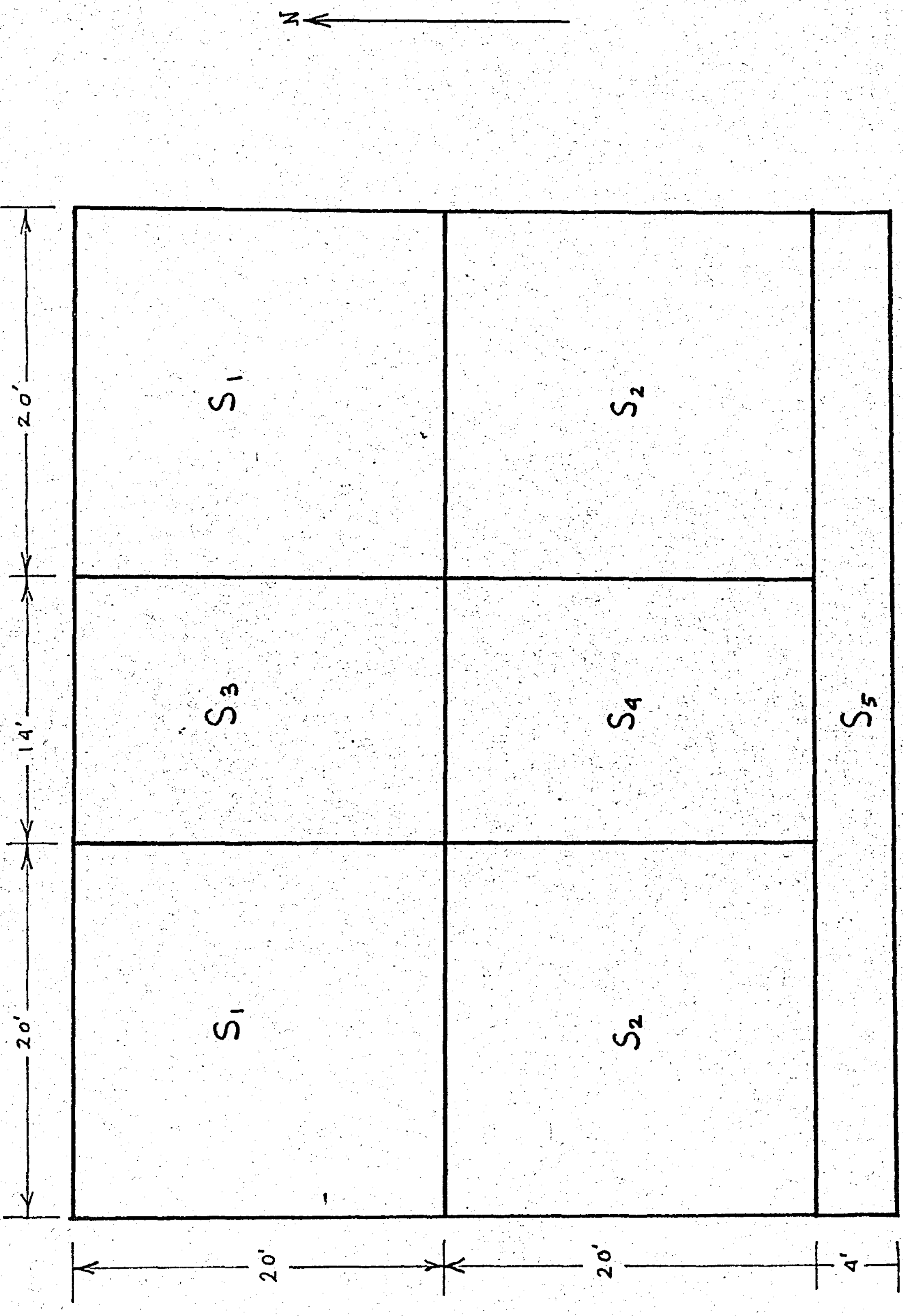


Fig. V-4 Shore Building - Slabs (Scale 1/100)

Steel: $As = \frac{M}{8.8}$ (similar to S_1)

1. (-)As = -0.233 sq.in./ft. strip
2. (-)As = -0.119 sq.in./ft. strip
3. (+)As = +0.176 sq.in./ft. strip

(Arrange steel with adjacent slabs) Fig. V - 5,7

Design of Slab S_3: Fig. V - 4.

Type: Two way slab.

Thickness: t = 6 in. (ref. S_1).

Load: w = 0.125 kips/sq.ft.

Moments: m = 14/20 = 0.7. (one edge disc.)

Short span:

1. (-)M at cont. edge = 0.062 (0.125)(14) = 1.52 kip-ft.
3. (+)M at midspan = 0.047 (0.125)(14) = 1.15 kip-ft.

Long Span:

4. (-)M at comt.edge = 0.041(0.125)(14) = 101 kip-ft.
5. (-)M at disc. edge = 0.021 (0.125)(14) = 0.51 kip-ft.
6. (+)M at disc. edge = 0.031 (0.125)(14) = 0.76 kip-ft.

Steel: $As = \frac{M}{8.8}$ (similar to S_1).

1. (-)As = -0.173 sq. in./ft. strip.
3. (+)As = +0.130 sq.in./ft. strip.
4. (-)As = -0.114 sq.in./ft. strip.
5. (-)As = -0.058 sq.in./ft. strip.
6. (+)As = +0.087 sq.in./ft. strip.

(Arrange steel with adjacent slabs) Fig. V - 6,8

Design of Slab S_3: Fig. VI-4.

Type: Two way slab.

Thickness: $t = 6$ in. (ref. S_1).

Load: $w = 0.125$ kips/sq.ft.

Moments: $m = 14/20 = 0.7$. (one edge disc.)

Short Span:

1.(–)M at cont. edge = 0.062 (0.125)(14) = 1.52 kip-ft.

3.(+)M at midspan = 0.047 (0.125)(14) = 1.15 kip-ft.

Long span:

4.(–)M at cont. edge = 0.041 (0.125)(14) = 1.01 kip-ft.

5.(–)M at disc. edge = 0.021 (0.125)(14) = 0.51 kip-ft.

6.(+)M at disc. edge = 0.031 (0.125)(14) = 0.76 kip-ft.

Steel: $As = \dfrac{M}{8.8}$ (similar to S_1).

1. (–)As = –0.173 sq.in./ft. strip.

3. (+)As = +0.130 sq.in./ft. strip.

4. (–)As = –0.114 sq.in./ft. strip.

5. (–)As = –0.058 sq.in./ft. strip.

6. (+)As = +0.087 sq.in./ft. strip.

(Arrange steel with adjacent slabs) Fig.V-6, 8 .

Design of Slab S_4: Fig. VI-4

Type: Two way slab.

Thickness: $t = 6$in. (ref. S_1).

Load: $w = 0.125$ kips/sq.ft.

Moments: $m = 14/20 = 0.7$ (interior panel).

Short Span:

1.(-)M at cont.edge = 0.055 (0.125)(14) = 1.35 kip-ft.

3.(+)M at midspan = 0.041 (0.125)(14) = 1.01 kip-ft.

Long Span:

4.(-)M at cont. edge = 0.033 (0.125)(14) = 0.81 kip-ft.

6.(+)M at midspan = 0.025 (0.125)(14) = 0.66 kip-ft.

Steel:

$As = \frac{M}{8.8}$ (similar to S_1).

1. (-)As = -0.153 sq.in./ft. strip.

3. (+)As = +0.115 sq.in./ft. strip.

4. (-)As = -0.092 sq.in./ft. strip.

6. (+)As = +0.075 sq.in./ft. strip.

(Arrange steel with adjacent slabs) Fig. V-7,8.

Design of Slab S_5: Fig. V-4.

Type: Cantilever. One way slab.

Thickness: t (fixed end) = 6 in.

t (free end) = 4 in.

Load: $w = 0.05 + \frac{(4+6)}{2 \times 12}(0.150) = 0.1125$

$w = 0.125$ lbs/sq.ft.

(extra load due to sign on cantilever).

Moments: $M = \frac{wl^2}{2} = \frac{0.125\,(4)^2}{2} = 1.0$ kip-ft.

Steel: $As = \frac{M}{8.8}$ (Ref. S_1).

$$As = \frac{1}{8.8} = 0.114 \text{ sq.in./ft. strip.}$$

(Arrange steel with adjacent slabs). Fig . V-5,8.

Check for Shear and Bond S S5

Shear: Check for maximum shear.

$$V\ (max) = \frac{wl}{2} = \frac{125 \times 20}{2} = 1250 \text{ lbs./ft. strip.}$$

$$v = \frac{V}{bJd} = \frac{1250}{12X7/8X5} = 24 \text{ psi} \angle 60 \text{ psi} \quad O.K.$$

Bond: Check for minimum bond:

$$u = \frac{V}{OJd}$$

$$O\ (Min) = 1.77 \text{ in./ft. strip.}$$

$$V = \frac{wl}{2} = \frac{125 \times 14}{2} = 875 \text{ lbs/ft. strip.}$$

$$u = \frac{875}{1.77X7/8X5} = 112 \text{ psi} \angle 140 \text{ psi} \quad O.K.$$

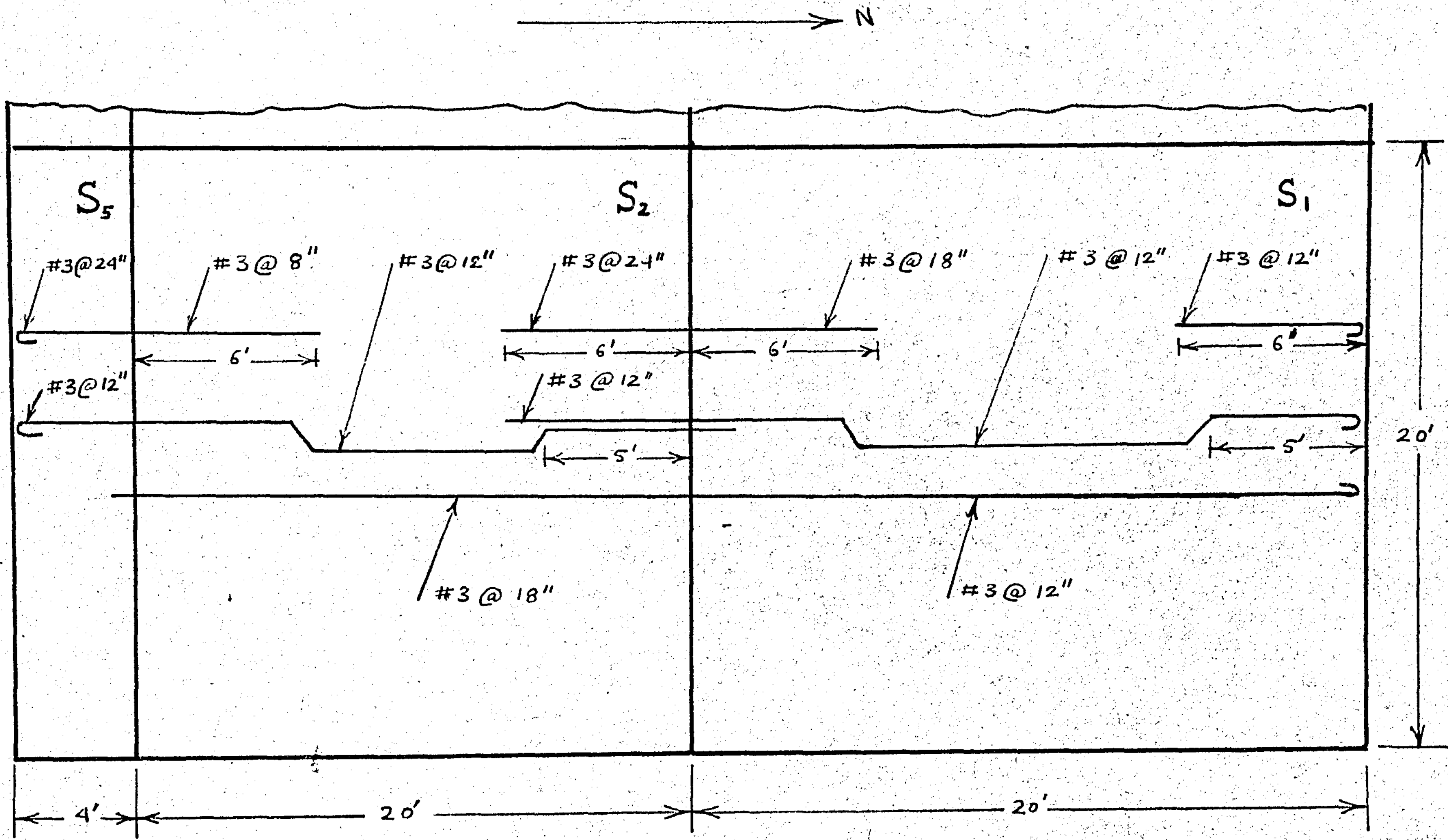


Fig. V-5 S_1, S_2, and S_5 Steel Reinforcement

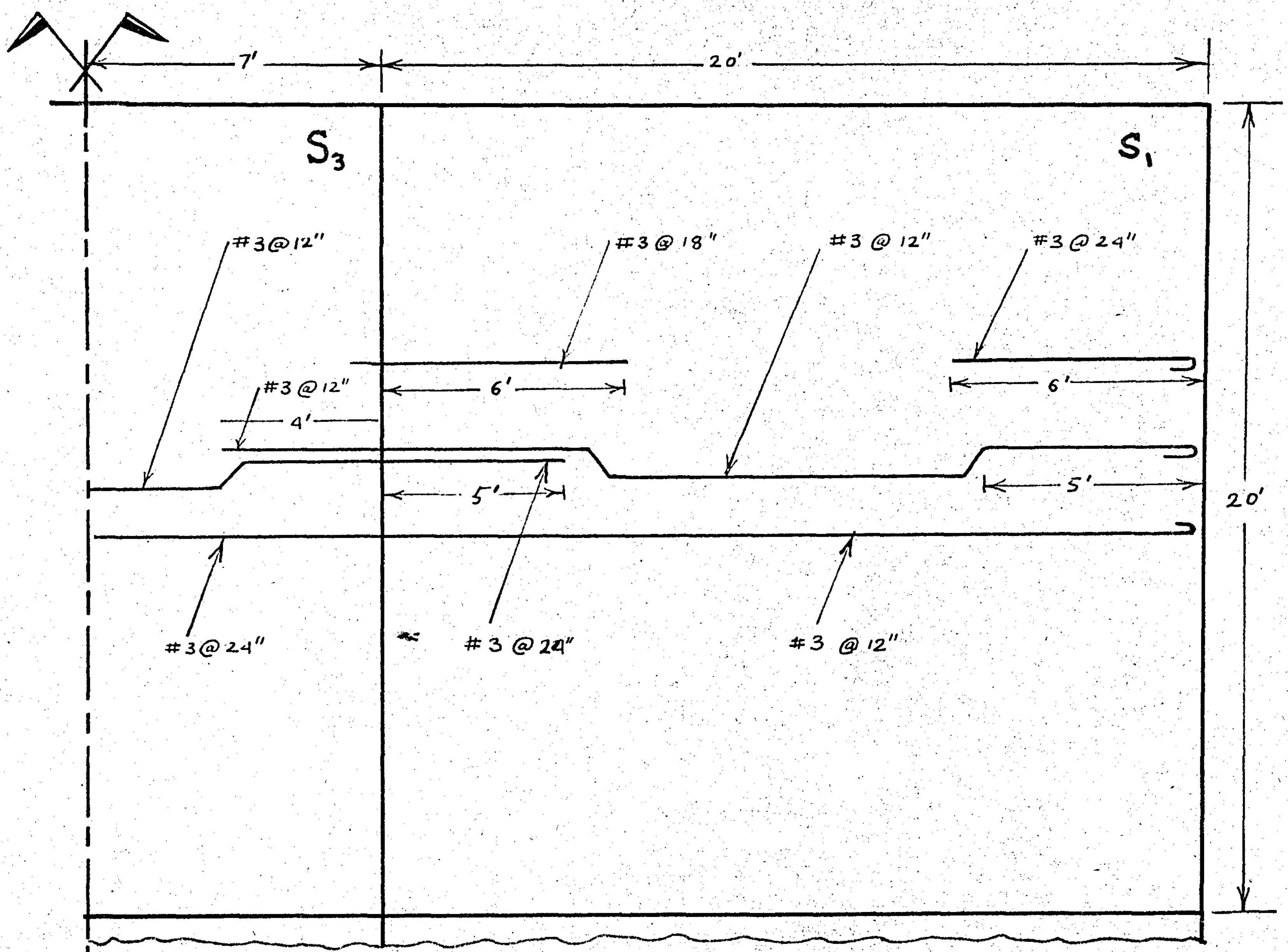


Fig. V-6 S_1 and S_3 Steel Reinforcement

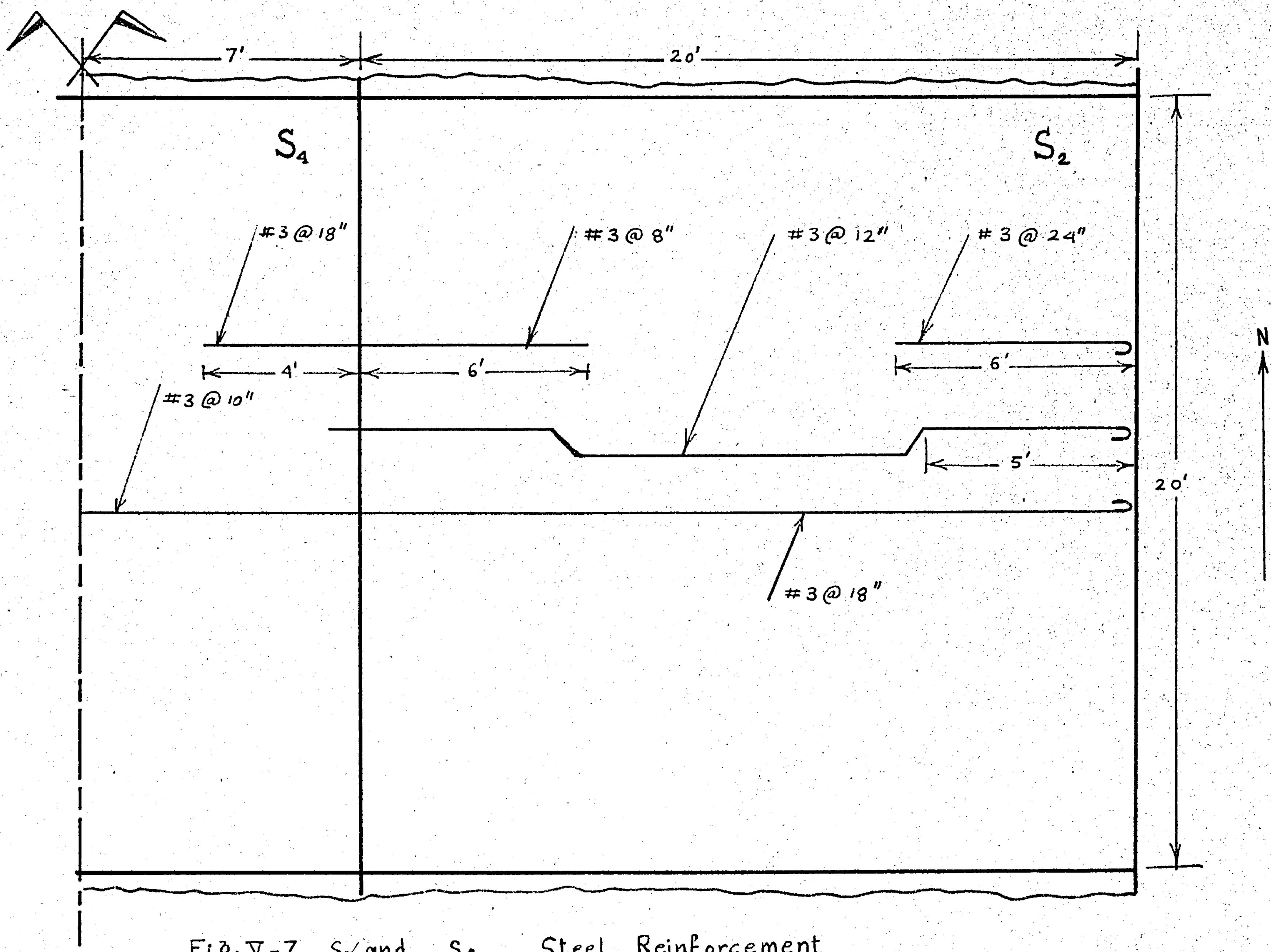


Fig. V-7 S_2 and S_4 Steel Reinforcement

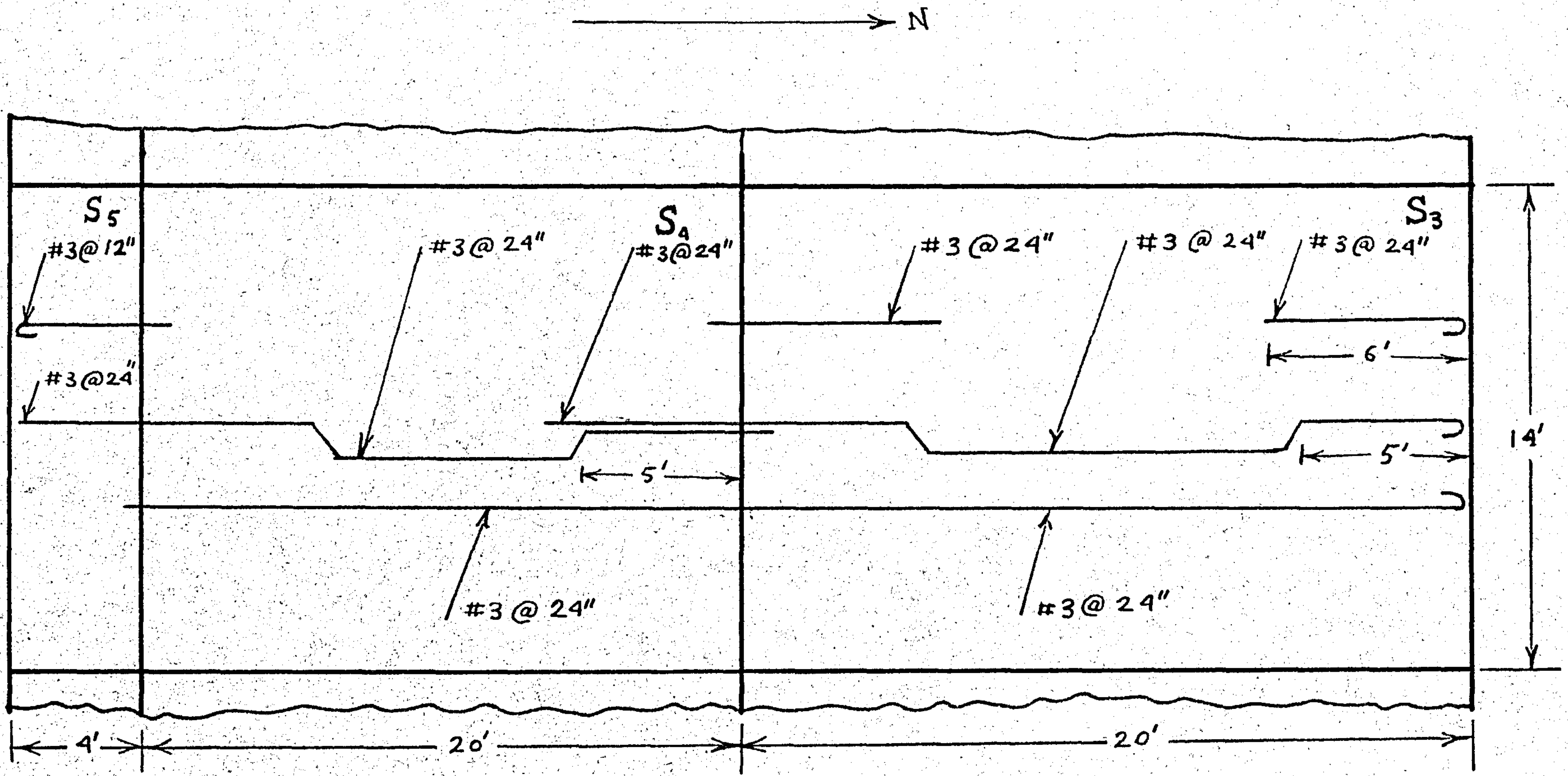


Fig. V-8 S_3, S_4, and S_5 Steel Reinforcement

Design of Beams

Loads: w (slab) = 125 lbs/sq.ft.

Bending moments may be determined approximately by using an equivalent uniform load per linear foot of beam for each panel supported as follows:

For the short span: $\frac{wS}{3}$

For the long span: $\frac{ws}{3} \frac{(3 - m^2)}{2}$

Where S is the short span, and m is the ratio of short span to long span.

Moments:* $(+)M = \frac{wl^2}{14}$

* $(+)M = \frac{wl^2}{16}$ for center (short) beams.

$(-)M = \frac{wl^2}{12}$ (Code).

* $(-)M = \frac{wl^2}{10}$ This value for negative end moment is taken as some end supports may drive more negative moment from adjacent beams (as in the case of spans of unequal length).

Design of Beam 1 Fig. V-10.

Load: $w = \frac{125\ (20)}{3} =$ 835 lbs./lin.ft. of beam.

w (D.L.) = 2 X 15 = 300 lbs./lin.ft. of beam.

Total w = 835 + 300 = 1135 lbs./lin.ft. of beam.

Moments: $(+)M = \frac{wl^2}{14} = \frac{1\ (1135)(19)^2}{14} = 29.2$ kip-ft.

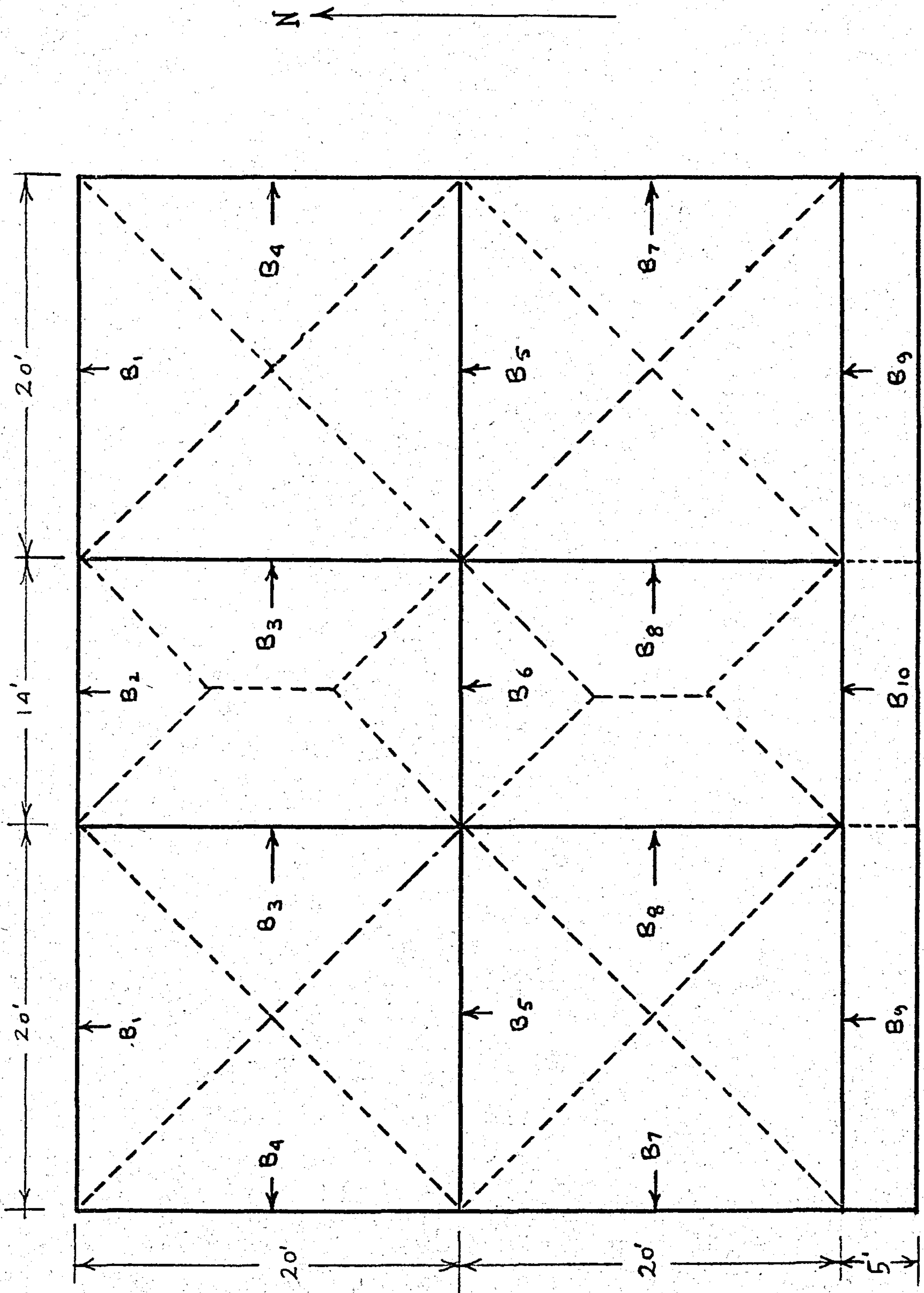


Fig. V - 9 Shore Building - Beams (Scale 1/100)

$$(-)M = \frac{wl^2}{10} = \frac{1}{10}(1135)(19)^2 = 40.9 \text{ kip-ft.}$$

Steel: $(+)As = \frac{(+)Ms}{Jdf} = \frac{29.2 \times 12}{7/8 \times 22 \times 20} = 0.91$ sq.in.

$(-)As = \frac{(-)Ms}{Jdf} = \frac{40.9 \times 12}{7/8 \times 22 \times 20} = 1.28$ sq.in.

(+)As .... 1 No.5 str. bott.

2 No.5 bent.

(-)As .... 2 No.5 bent.

3 No.5 str. top.

Spacing: 5 X 0.625 = 3.125 in.

1 X 4 = 4.000 in.

1.5 X 2 = 3.000 in.

Total = 10.125 in. 12 in. O.K.

Shear: $V = \frac{wl}{2} = \frac{1.135(19)}{2} = 10.8$ kips, $v = \frac{10,800}{12 \times 7/8 \times 24} = 43$ psi < 60 psi O.K.

Bond: $u = V/oJd = 10800/9.8 \times 7/8 \times 22 = 57$ psi < 140 psi ... O.K.

Design of Beam 2 Fig. V-11.

Load: $w = \frac{125\ (14)}{3} = 570$ lbs./lin.ft. of beam.

w (D.L.) = 2 X 150 = 300 lbs./lin.ft. of beam.

Total w = 570 + 300 = 870 lbs/ lin.ft. of beam.

Moments: $(+)M = \frac{wl^2}{16} = \frac{1}{16}(870)(13)^2 = 9.2$ kip-ft.

$(-)M = \frac{wl^2}{10} = \frac{1}{10}(870)(13)^2 = 14.8$ kip-ft.

Steel: $(+)As = \frac{(+)M}{Jdf} = \frac{9.2 \times 12}{7/8 \times 22 \times 20} = 0.28$ sq.in.

$(-)As = \frac{(-)M}{Jdf} = \frac{14.8 \times 12}{7/8 \times 22 \times 20} = 0.46$ sq.in.

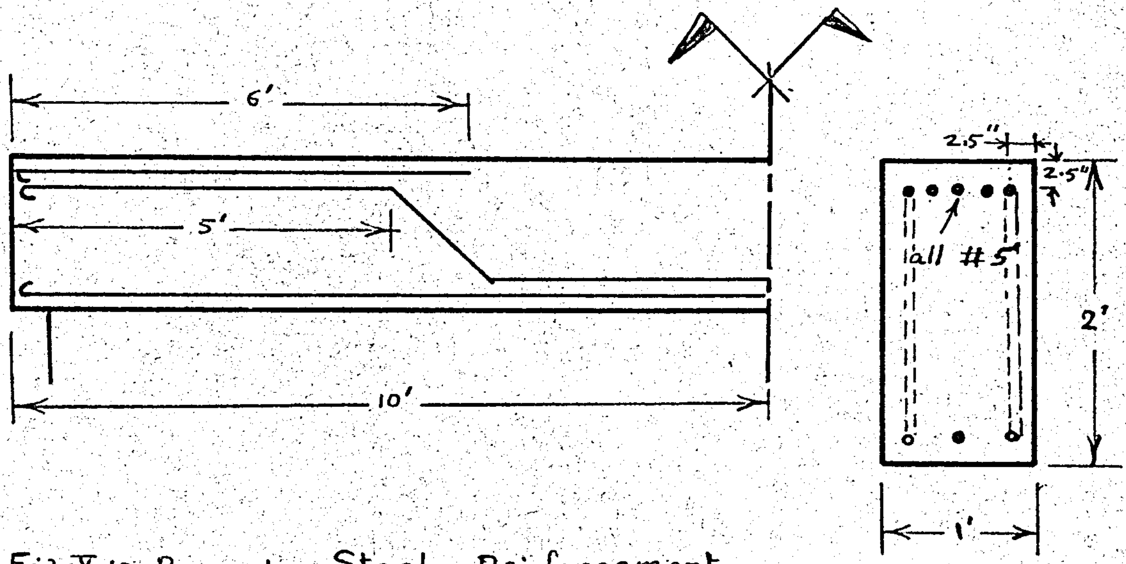


Fig. V-10 Beam 1 Steel Reinforcement

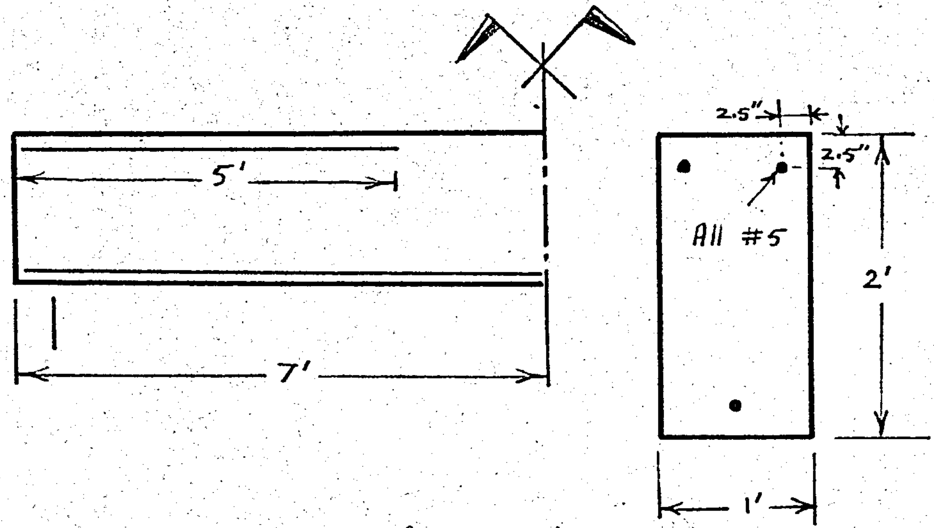


Fig. V-11 Beam 2 Steel Reinforcement

(+)As .... 1 No.5 str. bott.

(-)As .... 2 No.5 str. top.

Spacing: (Refer. Beam 1) Checks O.K.

Shear:

$$V = \frac{wl}{2} = \frac{0.870\ (13)}{2} = 5.65 \text{ kips.}$$

$$v = \frac{V}{bJd} = \frac{5650}{12 \times 7/8 \times 24} = 22.5 \text{ psi} < 60 \text{ psi} \quad \text{O.K.}$$

Bond: $$u = \frac{V}{OJd} = \frac{5650}{3.9 \times 7/8 \times 22} = 75 \text{ psi} < 140 \text{ psi} \quad \text{O.K.}$$

Design of Beam 3 Fig. V-12.

Load: $$w \text{ (from square slab)} = \frac{125\ (20)}{3} = 835 \text{ lbs/lin.ft.}$$

$$w \text{ (from small slab)} = \frac{125\ (14)}{3} \quad \frac{3-(0.7)}{2} = 730 \text{ lbs/lin.ft.}$$

$$w \text{ (D.L.)} = 300 \text{ lbs/lin.ft.}$$

$$\text{Total } w = 835 + 730 + 300 = 1865 \text{ lbs/lin.ft.}$$

Moments: $$(+)M = \frac{wl^2}{14} = \frac{1}{14}(1865)(19)^2 = 48.1 \text{ kip-ft.}$$

$$(-)M = \frac{wl^2}{10} = \frac{1}{10}(1865)(19)^2 = 67.2 \text{ kip-ft.}$$

Steel: $$(+)As = \frac{(+)M}{Jdfs} = \frac{48.1 \times 12}{7/8 \times 22 \times 20} = 1.5 \text{ sq.in.}$$

$$(-)As = \frac{(-)M}{Jdfs} = \frac{67.2 \times 12}{7/8 \times 22 \times 20} = 2.1 \text{ sq.in.}$$

(+)As .... 2 No.6 str. bott.

2 No.6 bent.

(-)As .... 2 No.6 bent.

3 No.6 str. top.

Spacing: $5 \times 0.75 = 3.75$ in.

$2 \times 0.375 = 0.75$ in.

$4 \times 1 = 4.00$ in.

$1.5 \times 2 = 3.00$ in.

Total $= 11.50$ in 12 in. O.K.

Shear: $V = \frac{wl}{2} = \frac{1.865\ (19)}{2} = 17.8$ kips.

$v = \frac{V}{bJd} = \frac{17800}{12 \times 7/8 \times 22} = 77$ psi $>$ 60 psi

Use stirrups: Fig. V – 12.

$x = 9.5 \times \frac{17}{77} = 2.1$ ft.

Stirrups: No.3 bars.

$S = \frac{A_v f_v}{v' b} = \frac{0.22 \times 20{,}000}{17 \times 12} = 21.6$ in.

use .... 2 No.3 at 12 in.

Bond: $u = \frac{V}{OJd} = \frac{17800}{9.4 \times 7/8 \times 22} = 98$ psi $\angle$ 140 psi O.K.

<u>Design of Beam 4</u> Fig. V – 10.

Design the same as Beam 1.

<u>Design of Beam 5</u> Fig. V – 13.

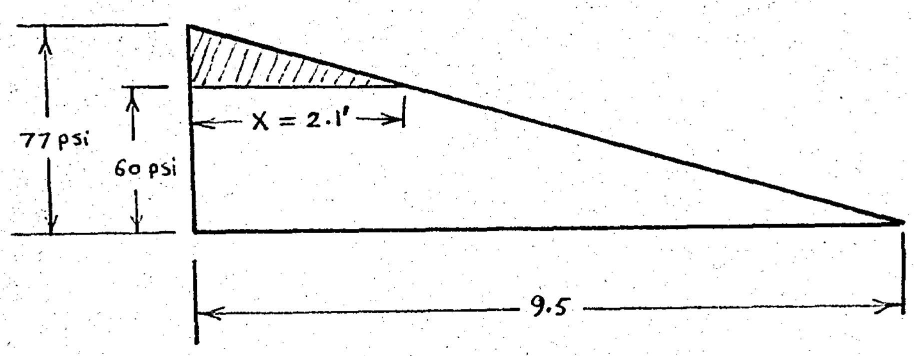
Load: $w = 2\ \frac{125 \times 20}{3} = 1670$ lbs/lin.ft.

w (D.L.) $= 300$ lbs/lin.ft.

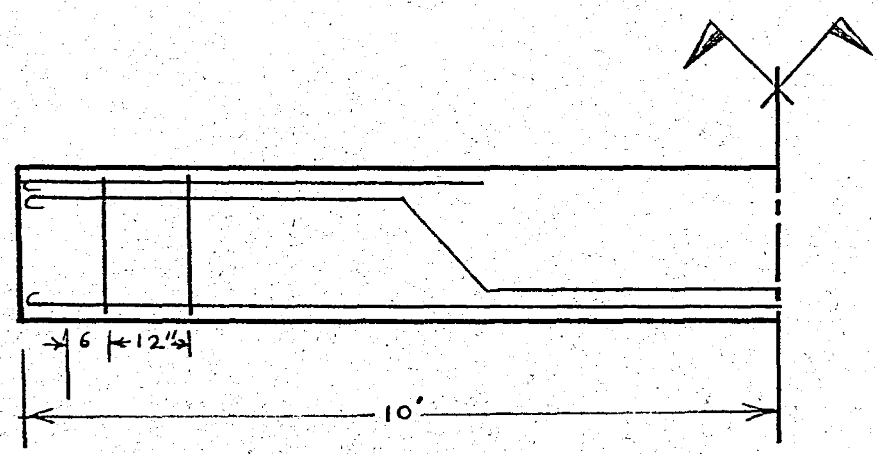
Total $w = 1970$ lbs/lin.ft.

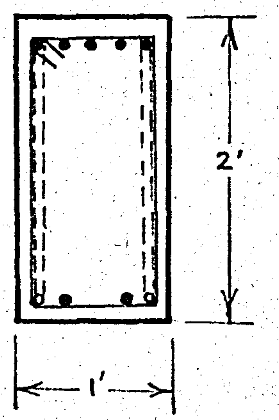
Moment: $(+)M = \frac{wl^2}{14} = \frac{1}{14}(1970)(19)^2 = 51.0$ kip-ft.

$(-)M = \frac{wl^2}{10} = \frac{1}{10}(1970)(19)^2 = 71.0$ kip-ft.



a. Shear





b. Steel

Fig. V - 12 Beam 3

Steel: $(+)As = \frac{51.0 \times 12}{7/8 \times 22 \times 20} = 1.58$ sq.in.

$(-)As = \frac{71.0 \times 12}{7/8 \times 22 \times 20} = 2.20$ sq.in.

(+)As .... 2 No.6 str.bott.
2 No.6 bent.
(-)As .... 2 No.6 bent.
3 No.6 str. top.

Spacing: (similar to Beam 3) Checks O.K.

Shear: $V = \frac{wl}{2} = \frac{1.970\ (19)}{2} = 18.7$ kips.

$v = \frac{V}{bJd} = \frac{18700}{12 \times 7/8 \times 22} = 81$ psi $>$ 60 psi.

Use stirrups: Fig. V-13.

$x = 9.5 \times \frac{21}{81} = 2.45$ ft.

Stirrups No.3 bars,

$S = \frac{Avfv}{v'b} = \frac{0.22 \times 20{,}000}{21 \times 12} = 17.4$ in.

use .... 2 No.3 at 12in.

Bond: $u = \frac{V}{OJd} = \frac{18700}{11.8 \times 7/8 \times 22} = 82$ psi $<$ 140 psi O.K.

Design of Beam 6 Fig. V-15.

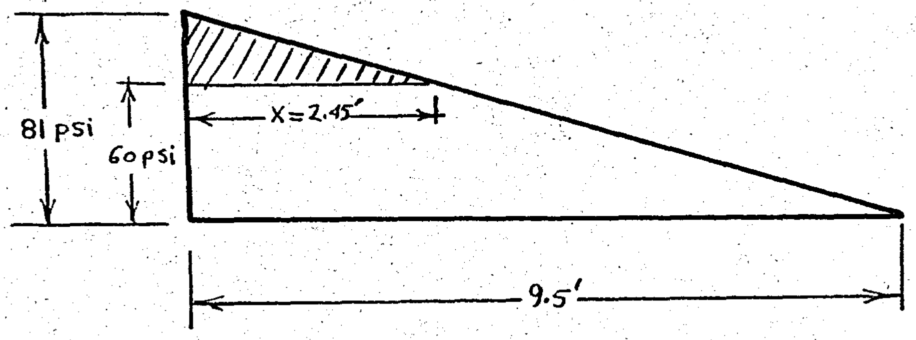
Load: $w = 2 \times \left(\frac{125 \times 14}{3}\right) \times \left(\frac{3 - (0.7)}{2}\right) = 1460$ lbs/ln-ft.

$w = (D.L.) = 300$ lbs./ lin.ft.

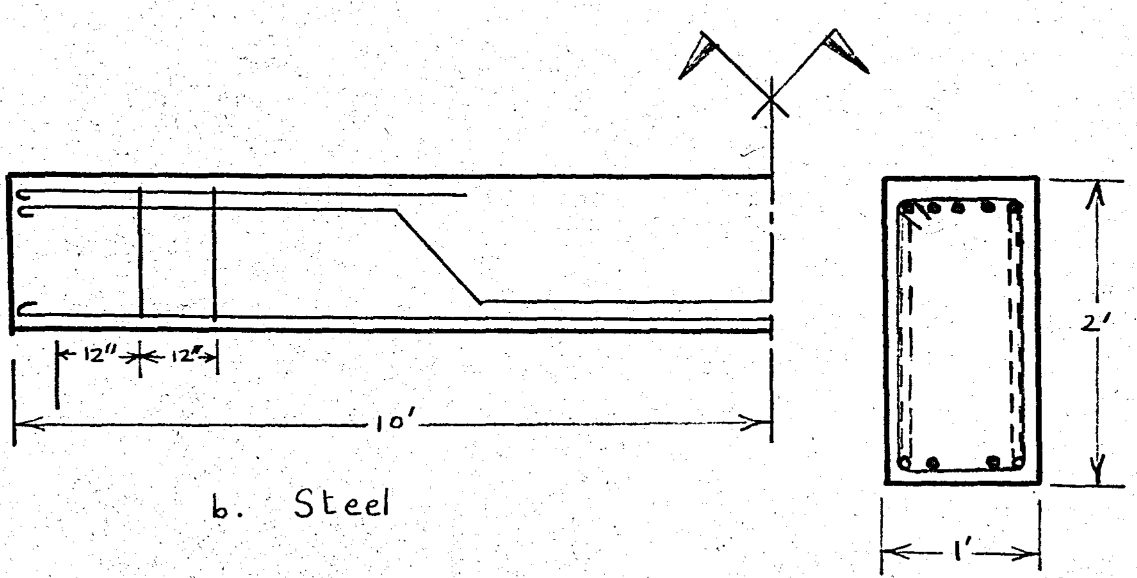
Total $w = 1460 + 300 = 1760$ lbs/ sq.ft.

Moments: $(+)M = \frac{wl^2}{16} = \frac{1}{16}(1760)(13)^2 = 18.6$ kip-ft.

$(-)M = \frac{wl^2}{10} = \frac{1}{10}(1760)(13)^2 = 29.6$ kip-ft.



a. Shear



b. Steel

Fig. V - 13 Beam 5.

Steel: $(+)As = \frac{(+)M}{Jdfs} = \frac{18.6 \times 12}{7/8 \times 22 \times 20} = 0.58$ sq.in.

$(-)As = \frac{(-)M}{Jdfs} = \frac{29.6 \times 12}{7/8 \times 22 \times 20} = 0.93$ sq.in.

(+)As .... 2No.6 str. bott.

(-)As .... 3 No.6 str. top.

Spacing: (ref. Beam 1) Checks O.K.

Shear: $V = \frac{wl}{2} = \frac{1.760\ (13)}{2} = 11.4$ kips.

$v = \frac{V}{bJd} = \frac{11400}{12 \times 7/8 \times 22} = 49.5$ psi $\angle$ 60 psi O.K.

Bond: $u = \frac{V}{OJd} = \frac{11400}{7.1 \times 7/8 \times 22} = 83.5$ psi $\angle$ 140 psi O.K.

Design of Beam 7 Fig. V-10.

Design the same as Beam 1.

Design of Beam 8 Fig. V-12.

Design the same as Beam 3.

Design of Beam 9 Fig. V-14.

Load: $w = \frac{125 \times 20}{3} = 833$ lbs/lin.ft.

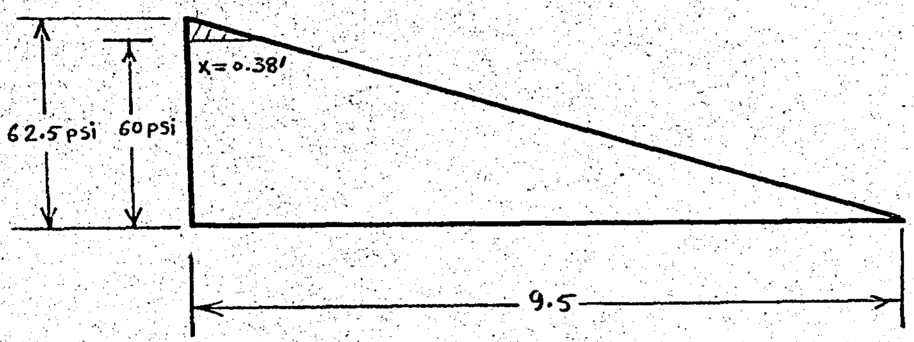
w (cantilever) = 375 lbs/lin.ft.

w (D.L.) = 300 lbs/lin.ft.

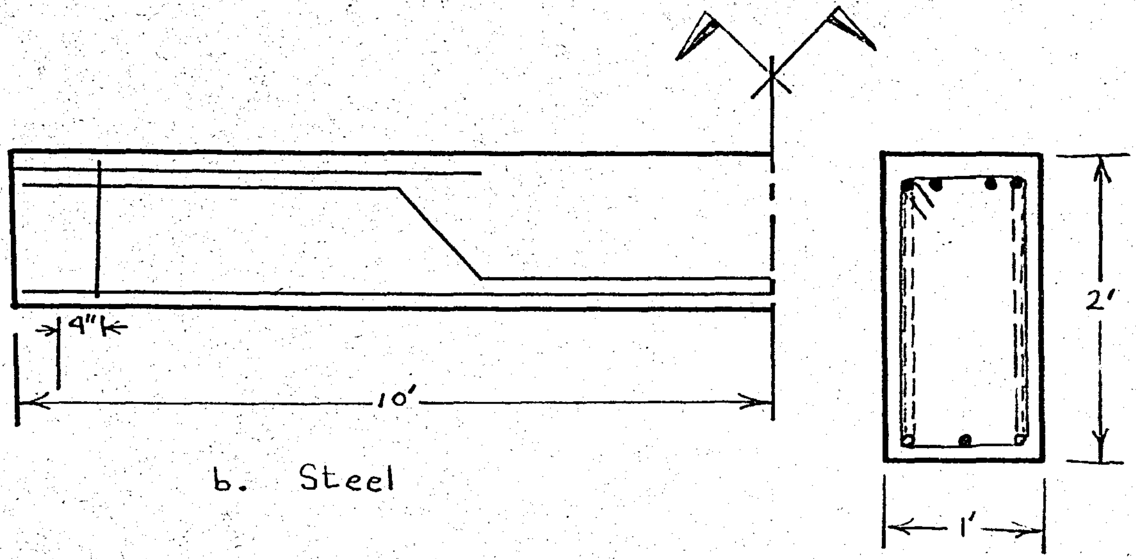
Total w = 833 + 375 + 300 = 1508 lbs/lin.ft.

Moment: $(+)M = \frac{wl^2}{14} = \frac{1}{14}(1508)(19)^2 = 39.0$ kip-ft.

$(-)M = \frac{wl^2}{10} = \frac{1}{10}(1508)(19)^2 = 54.2$ kip-ft.



a. Shear



b. Steel

Fig. V-14 Beam 9

Steel: $(+)As = \frac{39.0 \times 12}{7/8 \times 22 \times 20} = 1.22$ sq.in.

$(-)As = \frac{54.2 \times 12}{7/8 \times 22 \times 20} = 1.69$ sq.in.

(+)As .... 1 No.6 str. bott.

2 No.6 bent.

(-)As .... 2 No.6 bent.

2 No.6 str. top.

Shear: $V = \frac{wl}{2} = \frac{1.508 \quad (19)}{2} = 14.4$ kips.

$v = \frac{V\ T}{bJd} = \frac{14400}{12 \times 7/8 \times 22} = 62.2 \text{ psi} > 60$ psi.

use stirrups: Fig. V-14.

$x = 9.5 \times \frac{2.5}{62.5} = 0.38$ ft.

Stirrups No.3 bars.

Use .... 1 No.3 at 6 in.

Bond: (Ref. Beam 1) Checks O.K.

Design of Beam 10 Fig. V-16

Load: $w = \left(\frac{125 \times 14}{4}\right) \left(3 - \frac{(0.7)}{2}\right) = 730$ lbs./lin.ft.

w (cantilever) .... = 375 lbs./lin.ft.

w (D.L.) = 300 lbs./lin.ft.

Total w = 730 + 375 + 300 = 1405 lbs./lin.ft.

Moment: $(+)M = \frac{wl^2}{16} = \frac{1}{16}(1405)(13)^2 = 14.8$ kip-ft.

$(-)M = \frac{wl^2}{10} = \frac{1}{10}(1405)(13)^2 = 23.8$ kip-ft.

Steel: $(+)As = \frac{(+)M}{Jdfs} = \frac{14.8 \times 12}{7/8 \times 22 \times 20} = 0.46$ sq.in.

$$(-)As = \frac{(-)M}{Jdfs} = \frac{23.8 \times 12}{7/8 \times 22 \times 20} = 0.74 \text{ sq.in.}$$

(+)As .... 3 No.6 str. bott.

(-)As .... 2 No.6 str. top.

Spacing: (Ref. Beam 1) Checks O.K.

Shear: $V = \frac{wl}{2} = \frac{1405\ (13)}{2} = 9.3$ kips.

$$v = \frac{V}{bJd} = \frac{9300}{12 \times 7/8 \times 22} = 40 \text{ psi} \angle 60 \text{ psi} \qquad \text{O.K.}$$

Bond: $u = \frac{V}{OJd} = \frac{9300}{4.7 \times 7/8 \times 22} = 103$ psi 140 psi O.K.

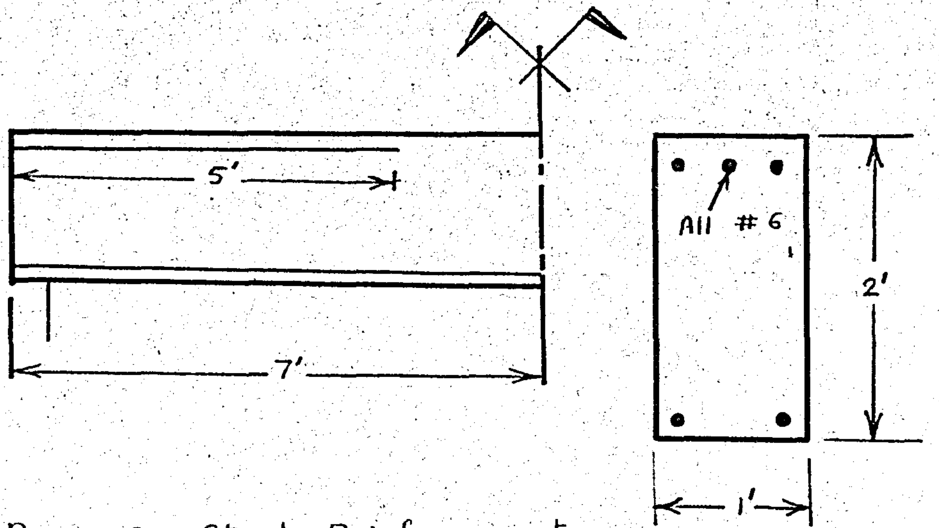


Fig. V-15 Beam 6 Steel Reinforcement

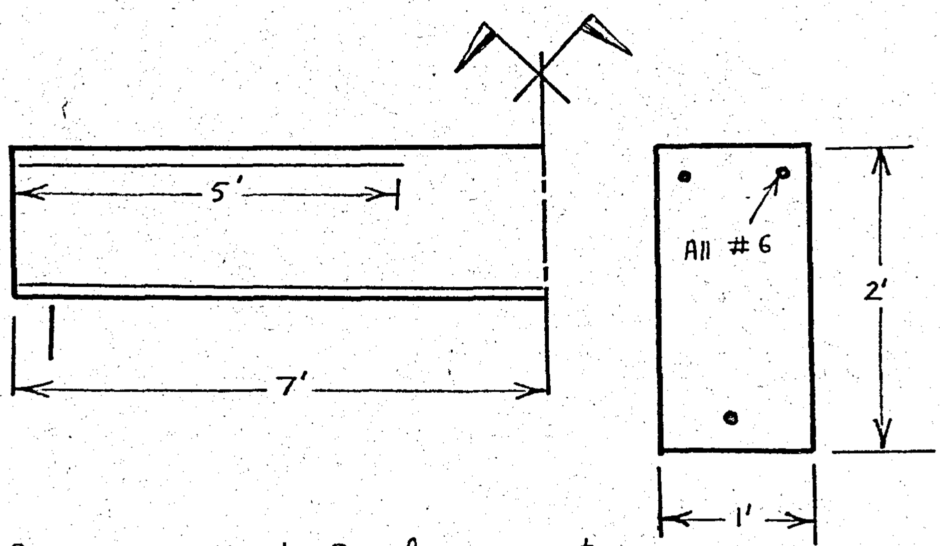


Fig. V-16 Beam 10 Steel Reinforcement

Design of Columns

* All beams have a depth of 2 ft.
* All columns have a length of 12 ft.
* All columns cross section = 1 ft X 2 ft.
* All unsupported length of columns is equal to 10 ft.
* All unsupported length of columns to depth = 10 ft. (short colm.
* All columns will have the same design, the design will be with respect to maximum load and maximum moment.
* Moment carried by column $= \dfrac{kM}{k \text{ (at Joint)}}$
* Relative stiffness; $K = \dfrac{bh^3/12}{L}$.

$$K \text{ (colimn around x-axis)} = \frac{8/12}{12} \longrightarrow 40$$

$$K \text{ (column around y-axis)} = \frac{2/12}{12} \longrightarrow 10$$

$$K \text{ (long beam)} = \frac{8/12}{20} = \longrightarrow 24$$

$$K \text{ (short beam)} = \frac{8/12}{14} = \longrightarrow 34$$

$$k_x = \frac{40}{64} = 0.63$$

$$k_{y_1} = \frac{10}{34} = 0.30$$

$$k_{y_2} = \frac{10}{68} = 0.15$$

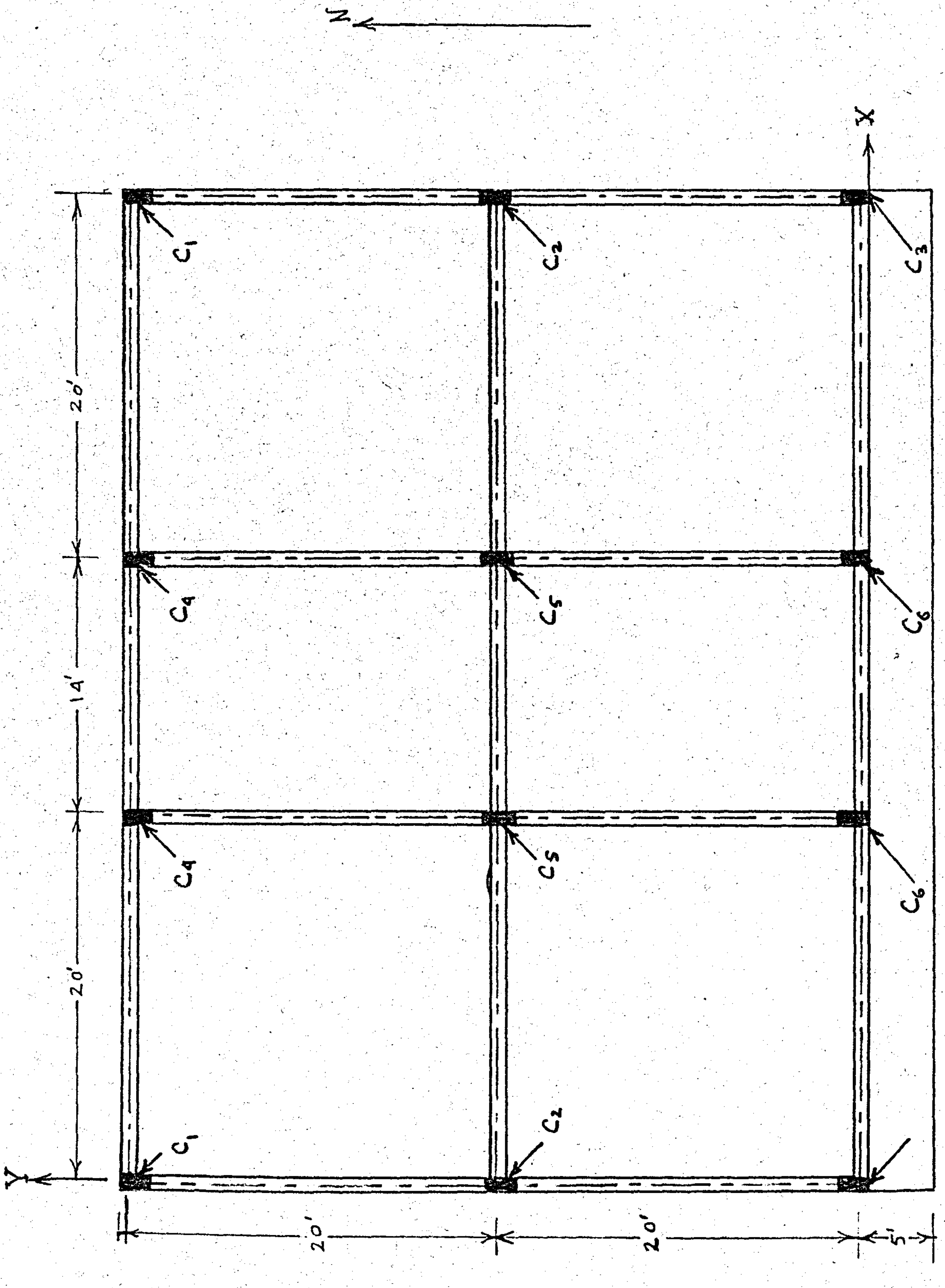


Fig. VI-17 Shore Building - Columns (Scale 1/100)

Table V-1

Moments and Loads on Columns

Column	M_x	M_y	kM_x	kM_y	Load
C	40.9	40.9	12	26	22.7
C	71.0	0	21	0	42.4
C	54.2	40.9	16	26	26.4
C	26.1	67.2	4	43	36.1
C	41.4	0	6	0	69.3
C	30.4	67.2	5	43	43.6

Design of a Column:

M_x (max.) = 21 kip-ft.

M_y (max.) = 43 kip-ft.

N (max.) = 70 kips.

$$e_1 = M_1/N = 12 \times \frac{21}{70} = 3.6 \text{ in.}$$

$$e_2 = M_2/N = 12 \times \frac{43}{70} = 7.3 \text{ in.}$$

$$e_1/t = \frac{3.6}{12} \qquad 2/3$$

$$e_2/t = \frac{7.3}{24} \qquad 2/3$$

$$P = N\left(1+\frac{Be}{t}\right) = N\left[1 + B\left(\frac{e_1}{t_1} + \frac{e_2}{t_2}\right)\right]$$

$$P = 70 \left[1 + 3.5\,(0.3+0.3)\right] = 70(1+2.1) = 217 \text{ kips.}$$

Steel: $P = 0.18 f'_c A_g + A_s f_s$

$$217 = 0.18\,(2)\,(288) + A_s\,(20)$$

$$A_s = \frac{217 - 103}{20} = \frac{11.4}{20} = 5.7 \text{ sq.in.} \qquad 6 \text{ sq.in.}$$

$$P_g = \frac{6}{288} = 2.1 \text{ percent}$$ ..... Fig. V-18

Stress: $$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b} \leq 1$$

$$F_a = 0.18 \times 2000 + 20{,}000\,(0.021) = 780 \text{ psi.}$$

$$F_b = 0.45 \times 2000 = 900 \text{ psi.}$$

$$f_a = 7000 / 288 = 243 \text{ psi.}$$

$$f_b = \frac{M}{S}$$

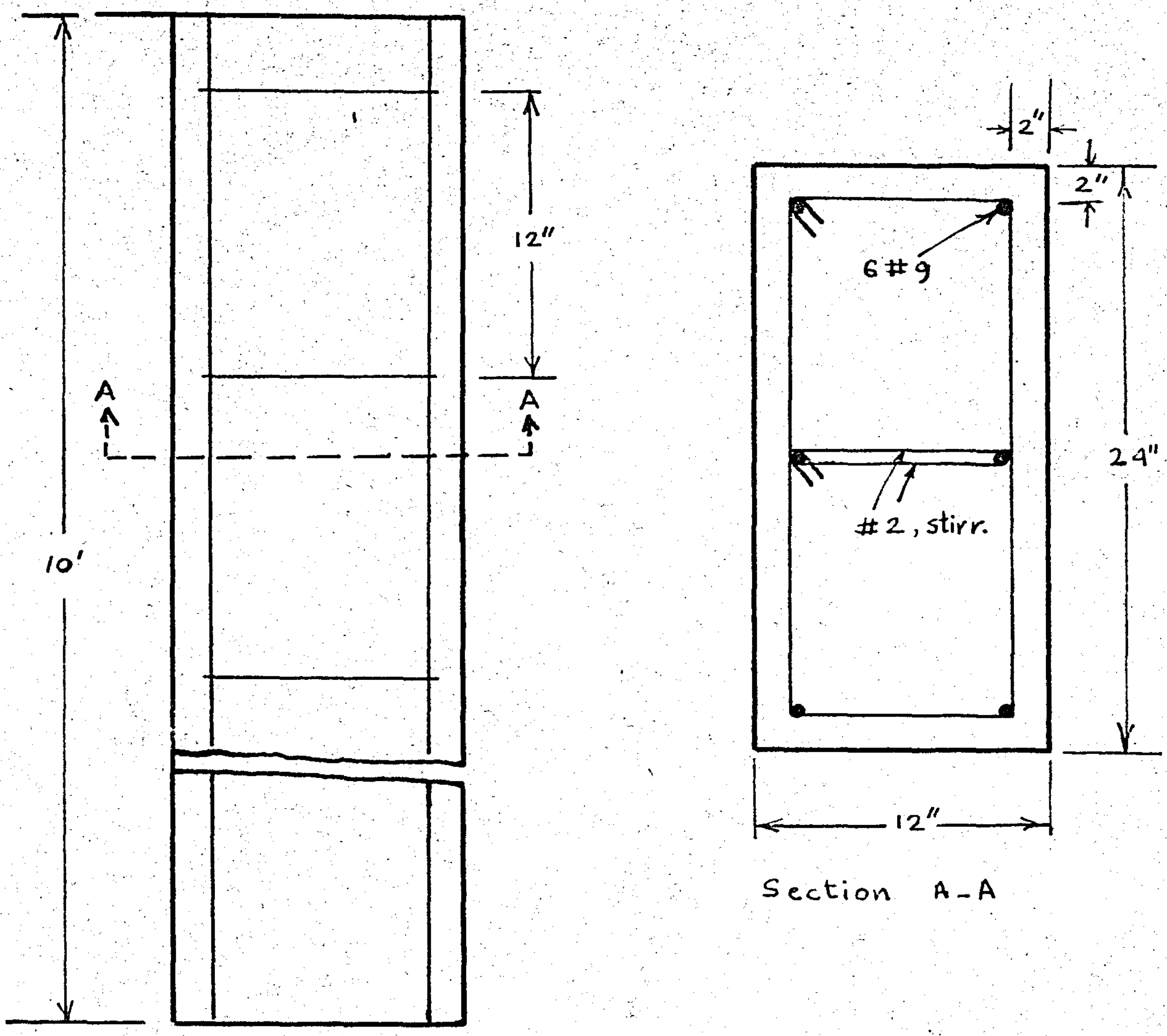


Fig. V-18 Column Steel Reinforcement

$$S = \frac{bh^3/12 + (n-1)\ As\ d^2}{C}$$

$$S_{xx} = \frac{(24)^3 + 9\ (4)\ (20)^2}{12} = 2360 \text{ in}^3.$$

$$S_{yy} = \frac{2\ (12)^3 + 9\ (6)\ (8)^2}{6} = 1152 \text{ in}^3.$$

$$f_{bx} = \frac{43000 \times 12}{2360} = 218 \text{ psi.}$$

$$f_{by} = \frac{21000 \times 12}{1152} = 218 \text{ psi.}$$

$$\frac{243}{780} + \frac{218}{900} + \frac{218}{900} \angle 1 \qquad \text{O.K.}$$

Ties: $48 \times 1/4 = 12$ ".

Use 10 No.2 at 12 in.

Design of Footings: Fig VI - 19

The foundation is laid on solid rock of minimum crushing strength of 9000 psi.

Plain concrete footings of two feet by three feet with a depth of 18 in. will be employed to avoid sliding of foundation. Borings for the footings are blasted in rock.

In each footing there will be 6 No.9 bar dowels each three feet long.

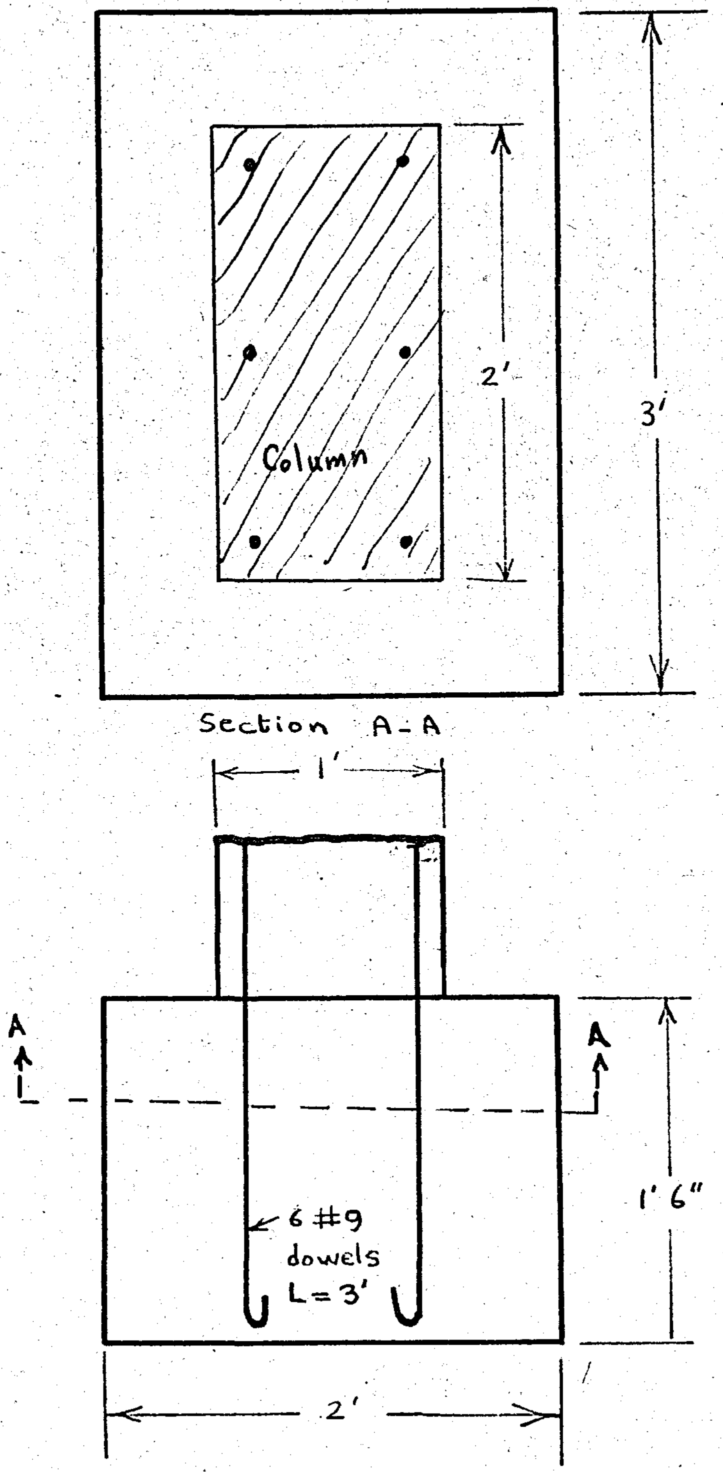


Fig. V-19 Footings & Dowels

Bill of Materials of the Shore Building

Steel & Concrete

Member	No. of Parts	Bars	Shape	Length	Wt./ft. lbs.	Wt. lbs.	Concrete ft³.
S_1	2	24 # 3	Straight	6	0.376	106.0	400
	2	20 # 3	Bent	22	"	323.0	
	2	20 # 3	Straight	6	"	86.0	
	2	20 # 3	"	21	"	309.0	
	2	20 # 3	"	6	"	86.0	
	2	14 # 3	"	6	"	62.0	
	2	20 # 3	Bent	22	"	323.0	
	2	20 # 3	Straight	21	"	309.0	
S_2	2	10 # 3	"	6	"	43.0	400
	2	20 # 3	Bent	22	"	323.0	
	2	14 # 3	Straight	21	"	222.0	
	2	30 # 3	"	6	"	132.0	
	2	10 # 3	"	6	"	43.0	
	2	20 # 3	"	6	"	86.0	
	2	20 # 3	Bent	22	"	323.0	
	2	14 # 3	Straight	21	"	222.0	
	2	14 # 3	"	6	"	62.0	
S_3	1	20 # 3	"	2 X 4	"	59.0	140
	1	20 # 3	Bent	16	"	118.0	
	1	10 # 3	Straight	15	"	55.0	
	1	7 # 3	"	2 X 6	"	31.0	
	1	7 # 3	Bent	22	"	57.0	

Bill of Materials of the Shore Building (Con't)

Member	No. of Parts	Bars	Shape	Length	Wt./ft lbs.	Wt. lbs.	Concrete ft^3.
	1	7 # 3	Straight	21	0.376	56.0	
	1	7 # 3	"	6	"	16.0	
S_4	1	14 #3	Straight	2 X 4	"	41.0	140
	1	24 # 3	"	15	"	132.0	
	1	7 # 3	"	6	"	16.0	
	1	7 # 3	Bent	22	"	57.0	
	1	7 # 3	Straight	21	"	54.0	
S_5	1	54# 3	"	6	"	119.0	135
	1	27# 3	"	5	"		
B_1, B_4, B_7	6	3 # 5	"	2 X 6	1.0043	225.0	180
	6	2 # 5	Bent	24	"	300.0	
	6	1 # 5	Straight	21	"	131.0	
B_2	1	2 # 5	"	2 X 4	"	17.0	21
	1	1 # 5	"	14	"	15.0	
B_3, B_8	4	3 #5	"	2 X 6	"	150.0	120
	2	2 # 5	Bent	24	"	100.0	
	2	2 # 5	Straight	21	"	87.0	
	2	2 # 3	Stirrup	2 X 6	0.376	18.0	
B_5	2	3 # 6	Straight	2 X 6	1.502	110.0	60
	2	2 # 6	Bent	24	"	146.0	
	2	2 # 6	Straight	21	"	128.0	
	2	2 # 3	Stirrup	2 X 6	0.376	18.0	

B ill of Materials of the Shore Building (Con't)

Member	No. of Parts	Bars	Shape	Length ft	Wt./Ft. lbs	Wt. lbs.	Concrete ft³.
B_6	1	3 # 6	Straight	2 X 4	1.502	32.0	21
	1	2 # 6	"	14	"	42.0	
B_9	2	2 # 6	"	2 X 6	"	72.0	60
	2	2 # 6	Bent	24	"	144.0	
	2	2 # 6	Straight	21	"	126.0	
	2	2 # 3	Stirrup	2 X 6	0.376	18.0	
B_{10}	1	2 # 6	Straight	2 X 4	1.502	24.0	21
	1	3 # 6	"	14	"	63.0	
C	12	6 # 9	Straight	12	3.900	2920	240
	12	20#2	Ties	4	0.167	160.0	
Footings	12	6 # 9	Straight	3	3.900	730.0	108
					Total:	9697	1698

Volume of Blasted Rock:

$$V = \text{Area} \times \text{av. depth}$$

$$= (54 \times 45) \times 6 = 14,580 \text{ ft}^3.$$

Chapter VI

DESIGN OF THE STAIRWAY

General Illustrations

* General specific views of the stairway structure are shown in the following pages.
* The space inside the center wall is big enough for two elevators. Sanitary pipes, water pipes, and electric wire extensions may pass through that space.
* All columns have 1ft X 1ft. cross dimension.
* All beams have 1ft X 2 ft. cross dimension.
* All steps have a rise = 6 in. and a run = 10 in.
* Side walls are to be built of concrete hollow blocks 6 inches deep. W/sq.ft. of wall including window openings etc. is assumed to equal to 30 lbs./sq.ft.
* f_s = 20,000 psi structural steel.
* f'_c = 2000 psi at 28 days.
* Foundation is laid on solid rock of 9000 psi crushing strength.

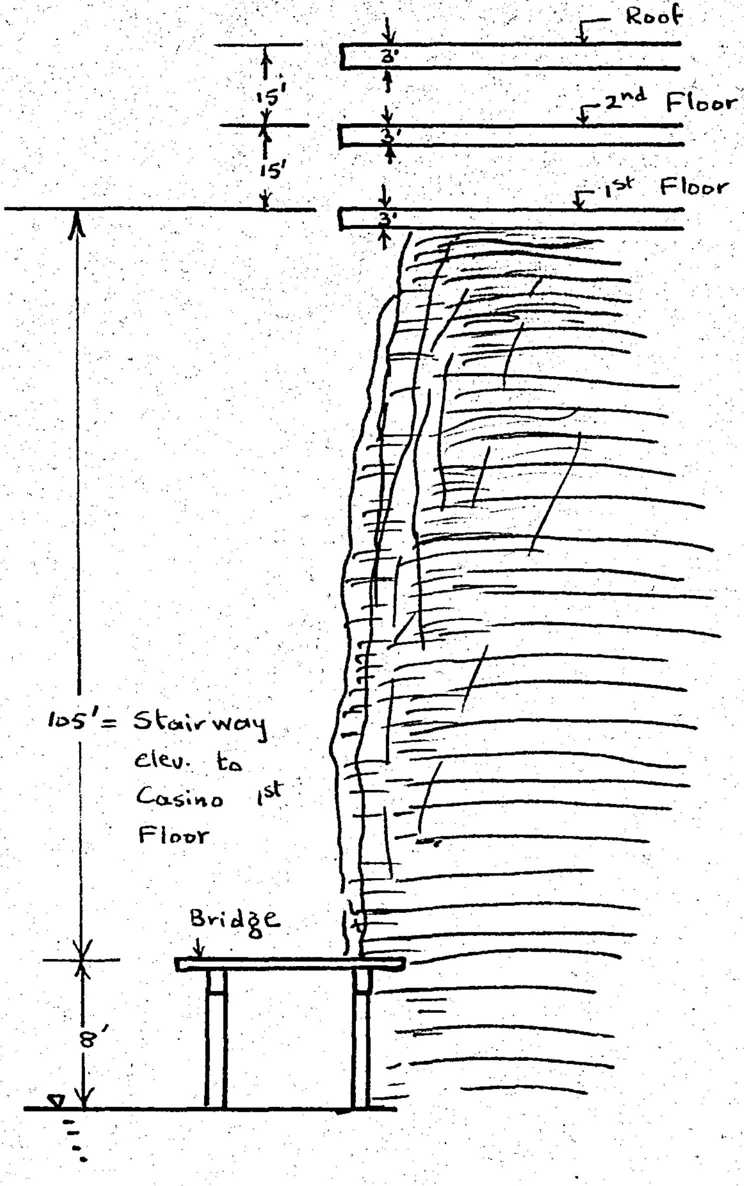


Fig. VI-1 Elevations of Stairway & Floors

Structural Design

Design of Top Slab

Design of S_1 Fig. VI - 2 .

$L = 12.5$ ft. $s = 8$ ft.

Loads: This slab is going to carry the elevator's dead load of motor, passenger cars, live load, plus impact load.

An over safe approximation for all this would be 3 tons per square meter.

3 tons/sq.meter $= \frac{3 \times 2200}{12} = 550$ lbs/sq.ft.

w (D.L.) $= \frac{6}{12} \times 150 = 75$ lbs./sq.ft.

Total $w = 625$ lbs./sq.ft.

Design- two way slab:

For edges continuous:

$m = 8/12.5 = 0.64$

Short span:

1. (-)M at cont. edge $= 0.060\ (0.625)(8)) = 2.4$ kip-ft.
2. (+)M at mid-span $= 0.044\ (0.625)(8) = 1.76$ kip-ft.

Long span:

3. (-)M at cont. edge $= 0.033\ (0.625)(12.5) = 3.22$ kip-ft.
4. (+)M at mid-span $= 0.025\ (0.625)(12.5) = 2.44$ kip-ft.

Steel: $A_s = \frac{M}{f_s J d} = \frac{M\ 12}{20 \times 7/8 \times 5.5} = \frac{M}{8}$

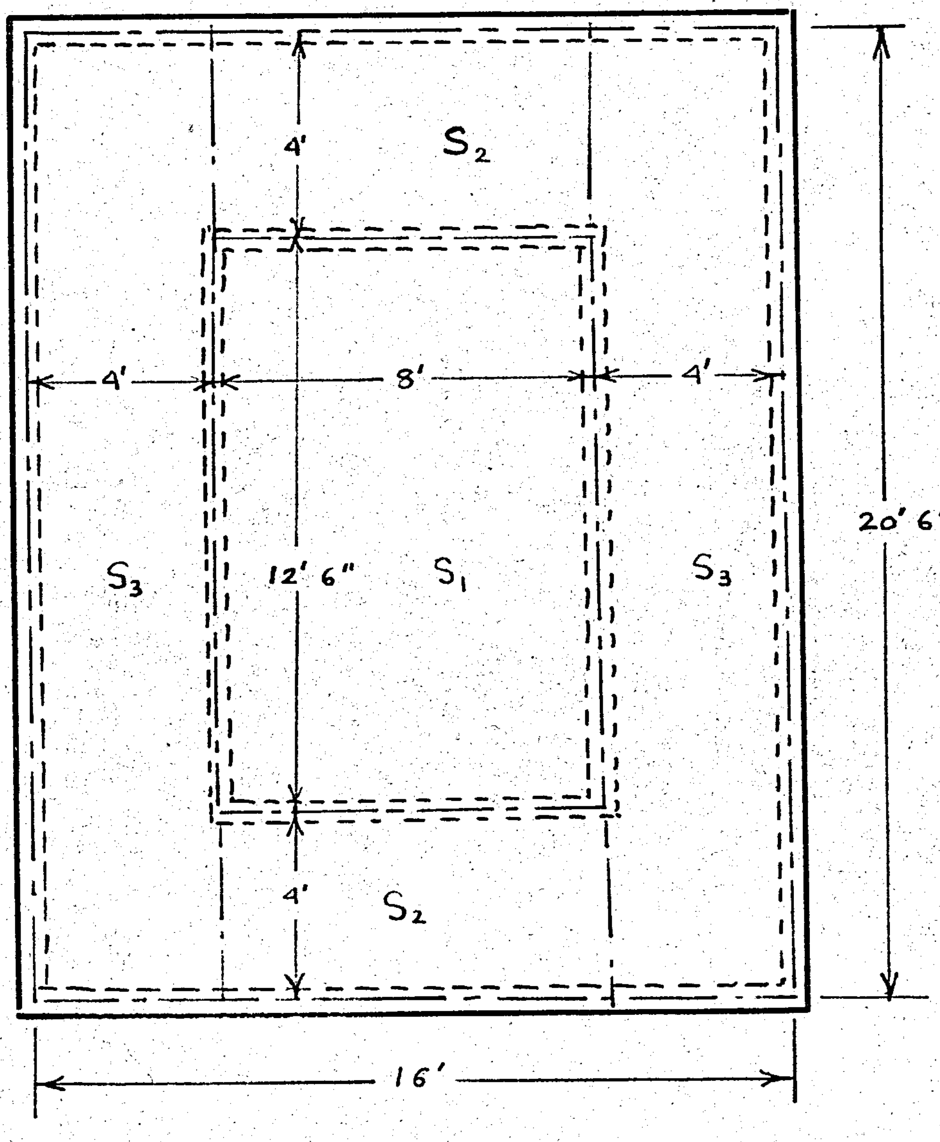


Fig. VI-2 Plan of Top Slab of Stairway

1. (-)As = 0.300 sq.in./lin.ft.
2. (+)As = 0.220 sq.in./lin.ft.
3. (-)As = 0.400 sq.in./lin.ft.
4. (+)As = 0.300 sq.in./lin.ft.

Arrange steel with adjacent slabs. Fig.VI 3&4.

<u>Design of S_2</u> Fig. VI-2 .

L = 8 ft. s = 4 ft.

Load: w (L.L.) = 100 lbs/sq.ft.

w (D.L.) = $\frac{6}{12}$ X 150 = 75 lbs/sq.ft.

Total w = 175 lbs./sq.ft.

Design two way slab:

One edge discontinuous:

m = 4/8 = 0.5

Moments: Short span:

1. (-)M at cont. edge = 0.083 (0.175)(4) = 0.232 kip-ft.
2. (+)M at mid span = 0.062 (0.175)(4) = 0.174 kip-ft.

Long span:

3. (-)M at cont. edge = 0.033 (0.175)(8) = 0.28 kip-ft.
4. (+)M at mid span = 0.025 (0.175)(8) = 0.37 kip-ft.

Steel: As = $\frac{M}{8}$ (similar to slab S_1).

1. (-)As = 0.029 sq.in./lin.ft.
2. (+)As = 0.022 sq.in./lin.ft.

3. (-)As = 0.035 sq.in./lin.ft.

4. (+)As = 0.046 sq.in./lin.ft.

Arrange steel with adjacent slabs. Fig. VI 3 & 4.

Design of S   Fig. VI - 2.

Load = 20.5 ft.   S = 4 ft.

Load: Total w = 175 lbs./sq.ft.

Design two way slab:

One edge discontinuous:

m = 4/20.5 = 0.2 ∠ 0.5

Moments: Short span:

1. (-)M at cont. edge = 0.083 (0.175)(4) = 0.232 kip-ft.

2. (+)M at mid span = 0.062 (0.175)(4) = 0.174 kip-ft.

Long span:

3. (-)M at cont. edge = 0.033 (0.175)(20.5) = 2.43 kip-ft.

4. (+)M at mid span = 0.025 (0.175)(20.5) = 1.84 kip-ft.

Steel: $As = \frac{M}{8}$ (similar to slab S_1).

1. (-)As = 0.029 sq.in/linft.

2. (+)As = 0.022 sq.in/lin.ft.

3. (-)As = 0.30 sq.in/lin.ft.

4. (+)As = 0.23 sq.in/lin.ft.

Arrange steel with adjacent slabs. Fig. VI 3 & 4.

From S_1, S_2, and S_3 shear and bond Checks O.K.

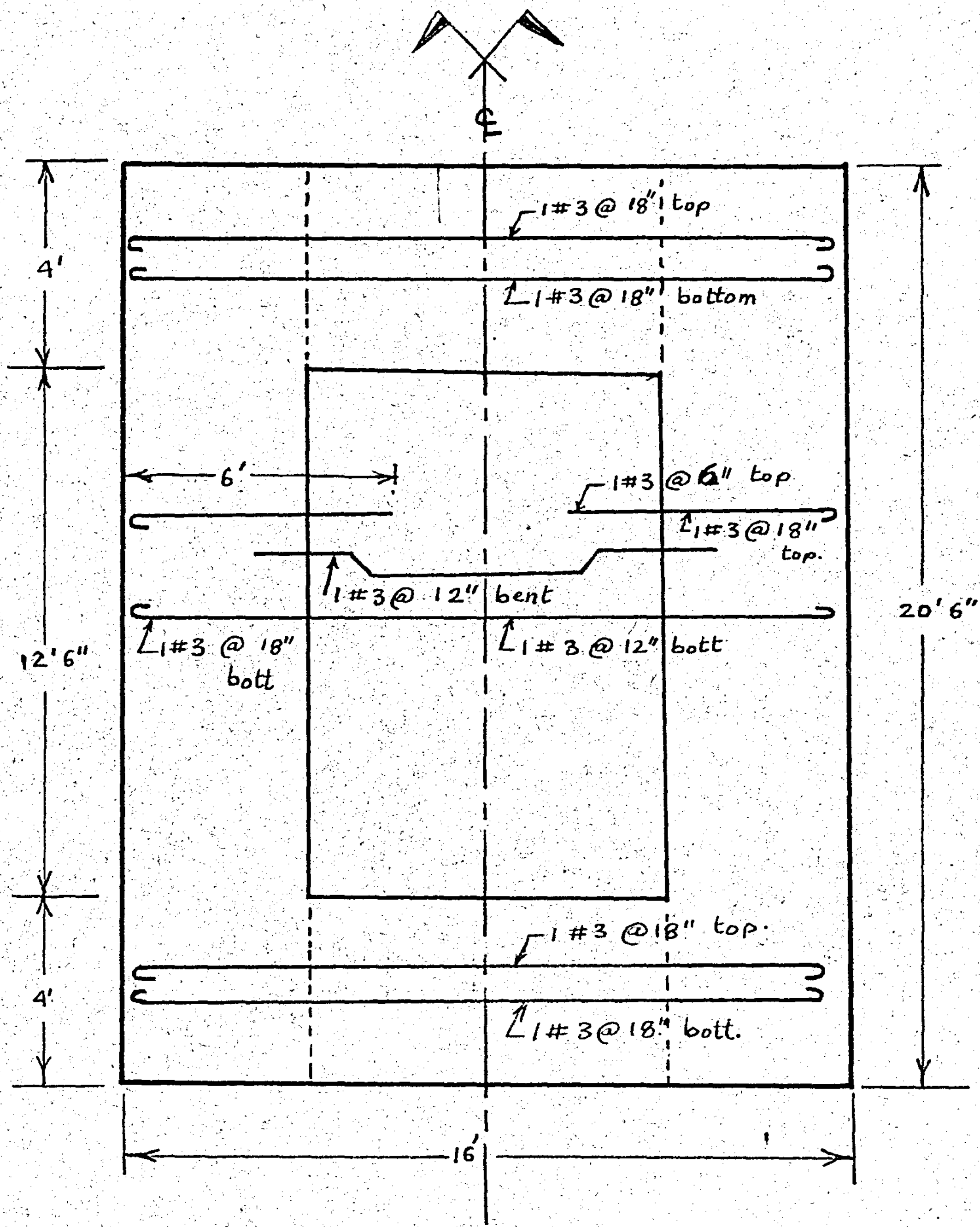


Fig. VI-3 Top Slab of Stairway - Steel Reinforcement.

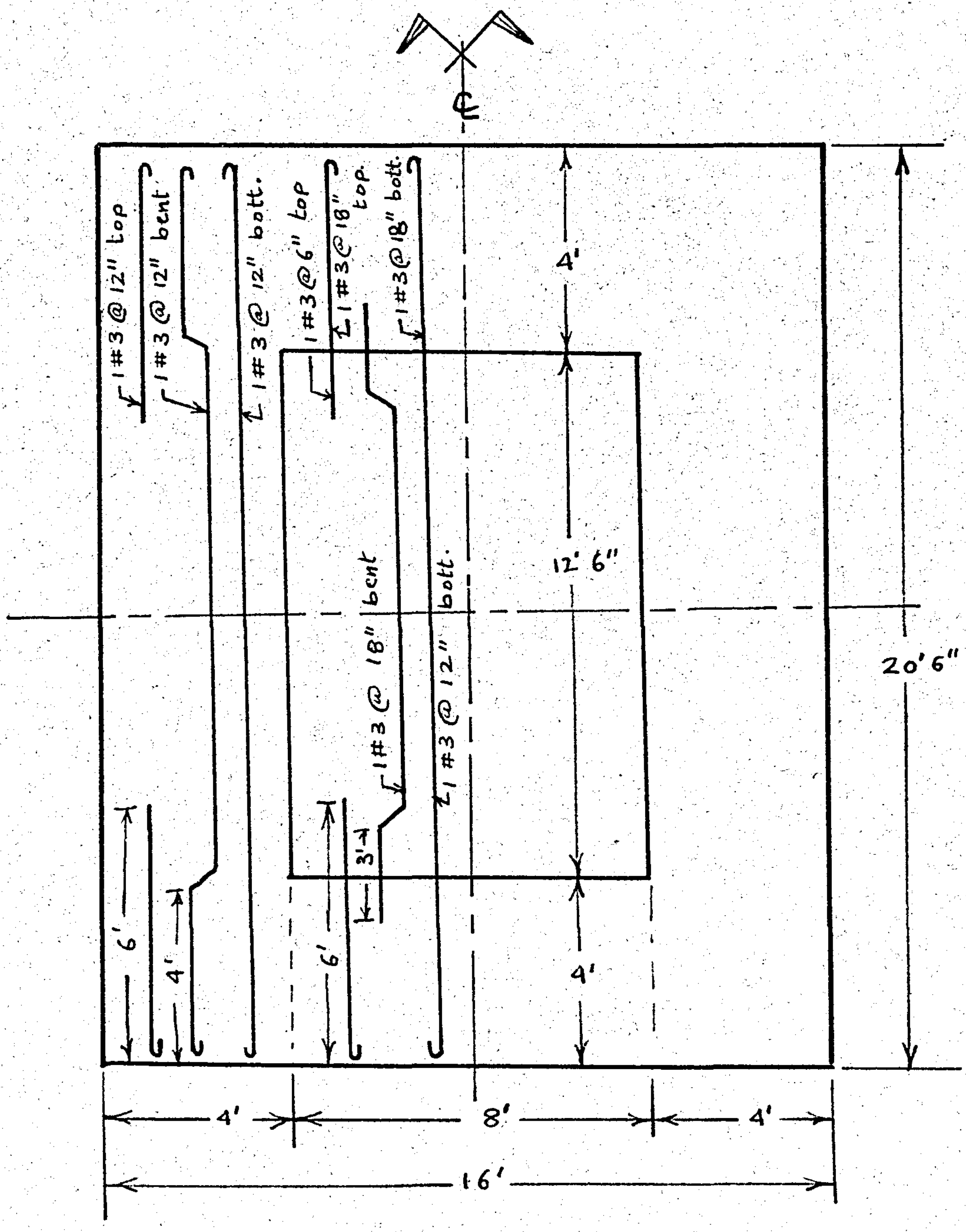


Fig. VI-4 Top slab of Stairway - Steel Reinforcement.

Design of Supporting Beams of Top Slab

Design of Beam B. Fig. VI - 5 & 6

$L = 20.5$ ft.

Load: w (from S) $= 2 \times 1\ (0.175) = 0.35$ kip/lin.ft.

w (D.L.) $= 2 \times 1\ (0.150) = 0.30$ kip/lin.ft.

Total $w = 0.65$ kip/lin.ft.

Moments: $(-)M = \dfrac{wl^2}{12} = \dfrac{0.65\ (20.5)^2}{12} = 22.8$ kip-ft.

$(+)M = \dfrac{wl^2}{16} = \dfrac{0.65\ (20.5)^2}{16} = 17.0$ kip-ft.

Steel: $(-)As = \dfrac{-M}{Jdf} = \dfrac{22.8 \times 12}{7/8 \times 22 \times 20} = 0.71$ sq.in.

$(+)As = \dfrac{+M}{Jdf} = \dfrac{17.0 \times 12}{7/8 \times 22 \times 20} = 0.53$ sq.in.

(+)As .... 1 No.4 str.bott.
2 No.4 bent.

(-)As .... 2 No.4 bent.
2 No.4 str. top.

Spacing:

$4 \times 0.5 = 2.0$ in.

$3 \times 1 = 3.0$ in.

$2 \times 1.5 = 3.0$ in.

Total $= 8.0$ in 12.0 in. O.K.

Shear: $V = \dfrac{wl}{2} = \dfrac{0.65 \times 20.5}{2} = 6.7$ kips.

$v = \dfrac{V}{bJd} = \dfrac{6700}{12 \times 7/8 \times 22} = 29$ psi $\angle$ 60 psi O.K.

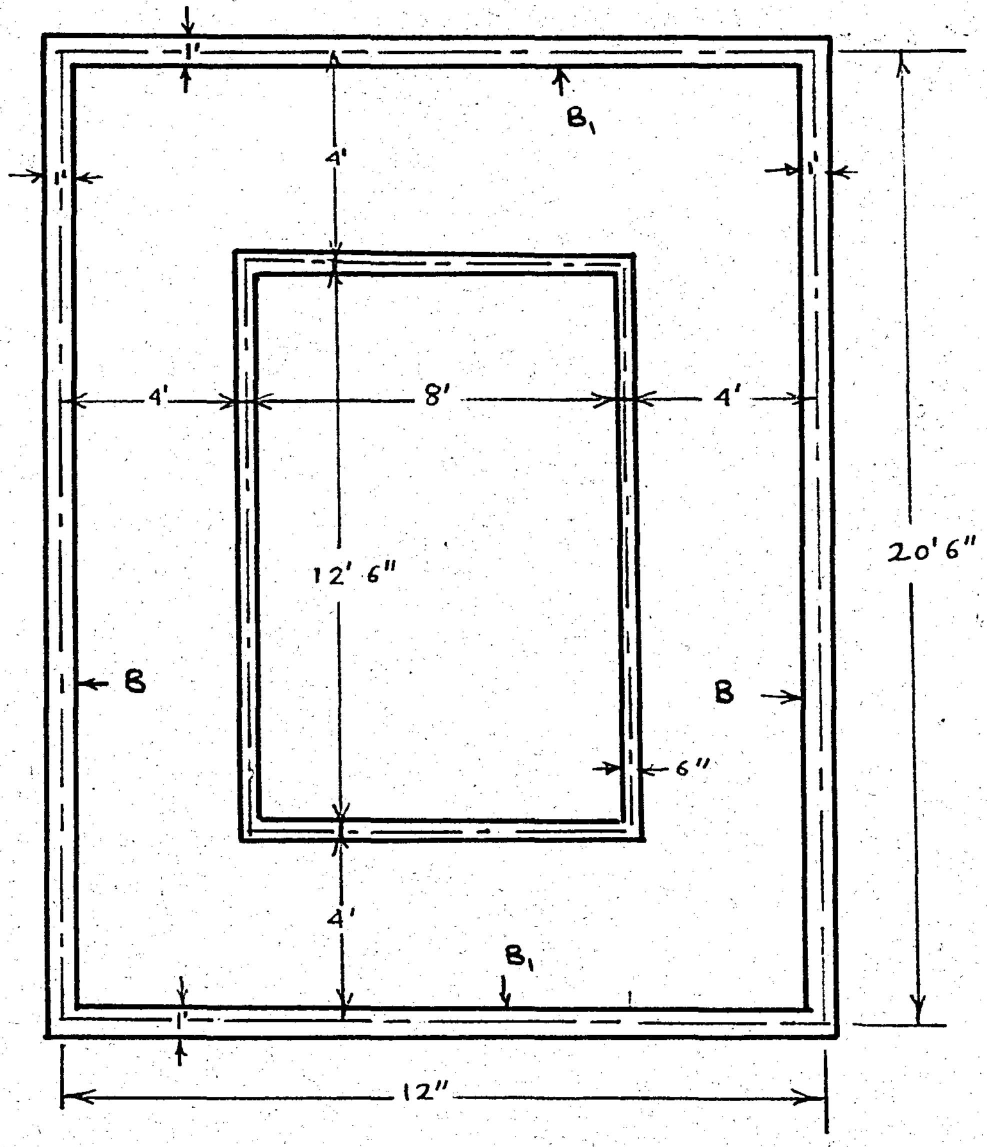


Fig. VI-5 Plan of Supporting Beams of Top Slab of Stairway

Bond: $u = \frac{V}{0jd} = \frac{6700}{6.2 \times 7/8 \times 22} = 56$ psi < 140 psi O.K.

Design of Beam B_1 Fig. VI - 5 47

$L = 16$ ft.

Load: Total $w = 0.65$ kip/lin.ft. (similar to Beam B).

Moment: $(-)M = \frac{wl^2}{12} = \frac{0.65\ (16)^2}{12} = 14.0$ kip-ft.

$(+)M = \frac{wl^2}{16} = \frac{0.65\ (16)^2}{16} = 10.4$ kip-ft.

Steel : $(-)As = \frac{-M}{Jdf} = \frac{14.0 \times 12}{7/8 \times 22 \times 20} = 0.44$ sq.in.

$(+)As = \frac{+M}{Jdf} = \frac{10.4 \times 12}{7/8 \times 22 \times 20} = 0.32$ sq.in.

(+)As .... 1 No.4 ~~bent~~ str. bott.
2 No.4 bent.

(-)As .... 2 No.4 bent.
1 No.4 str. top.

Spacing: Checks O.K. (similar to Beam B).

Shear: Checks O.K. (Reference - Beam B).

Bond: Checks O.K. (Reference - Beam B).

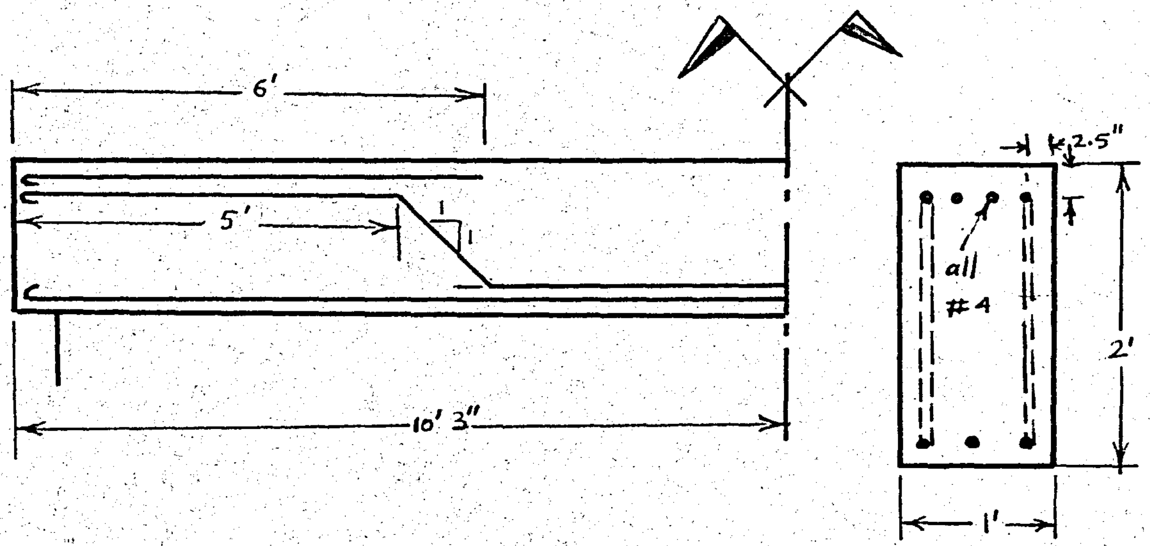


Fig. VI - 6 Beam B , Steel Reinforcement

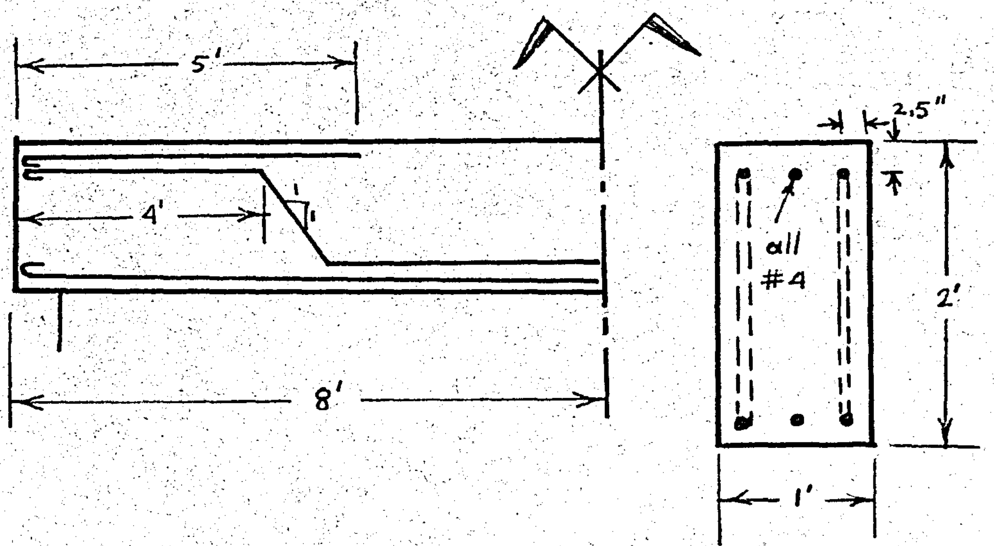


Fig. VI-7 Beam B_1 Steel Reinforcement

Design of Stairs

Design of One way slab. Fig. VI - 9 & 10

$L = 20.5$ ft.

Loads: $w\ (D.L.) = 150 \times \frac{1}{2} \times \frac{(12.5) + (7.5)}{12} + 150 \times \frac{1}{2} \times \frac{1}{2}$

$= 91 + 38 = 129$ lbs./sqft.

$w\ (L.L.) = 71$ lbs/sq.ft.

Total $w = 200$ lbs./sq.ft.

Moments: $(+)M = \frac{wl^2}{16} = \frac{0.2\ (20.5)^2}{16} = 5.3$ kip-ft.

$(-)M = \frac{wl^2}{12} = \frac{0.2\ (20.5)^2}{12} = 7.0$ kip-ft.

Steel: $(+)As = \frac{+M}{Jdf} = \frac{5.3 \times 12}{7/8 \times 5 \times 20} = 0.73$ sq in/ft-strip.

$(-)As = \frac{-M}{Jdf} = \frac{7.0 \times 12}{7/8 \times 5 \times 20} = 0.96$ sq.in/ft-strip.

(+)As .... 1 No.7 at 18 in. bent.

(-)As .... 1 No.7 at 12 in. str. bott.

Lateral Steel:

Use 1 No.5 bar, 4 ft. 6 in long under each tread and extend it to the center wall.

Design of S_2 (connecting fly overs of two stair panels)

Design similar to S_2 of top slab.

Steel: Short span:
1. 1 No.3 at 18 in. top.

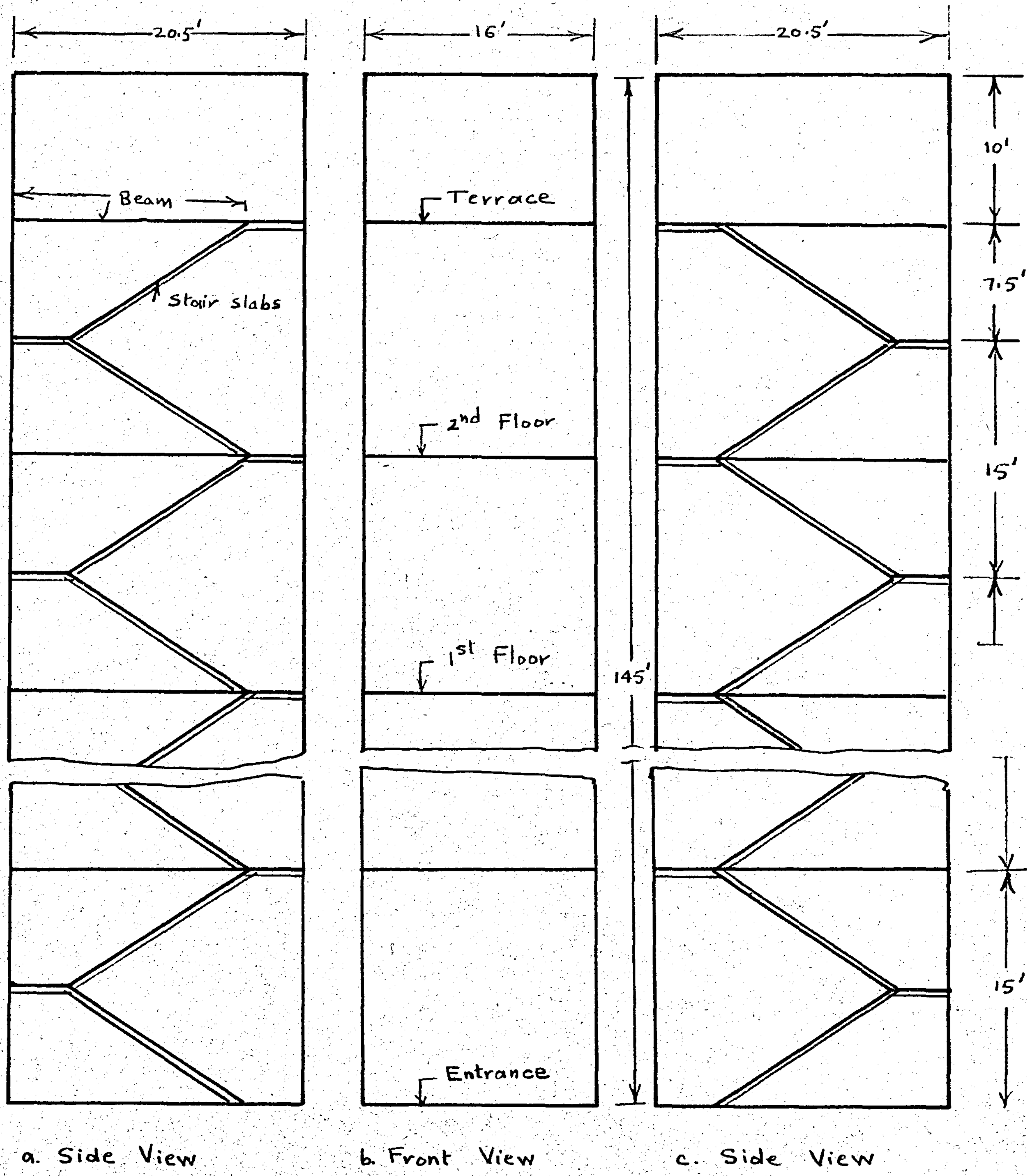
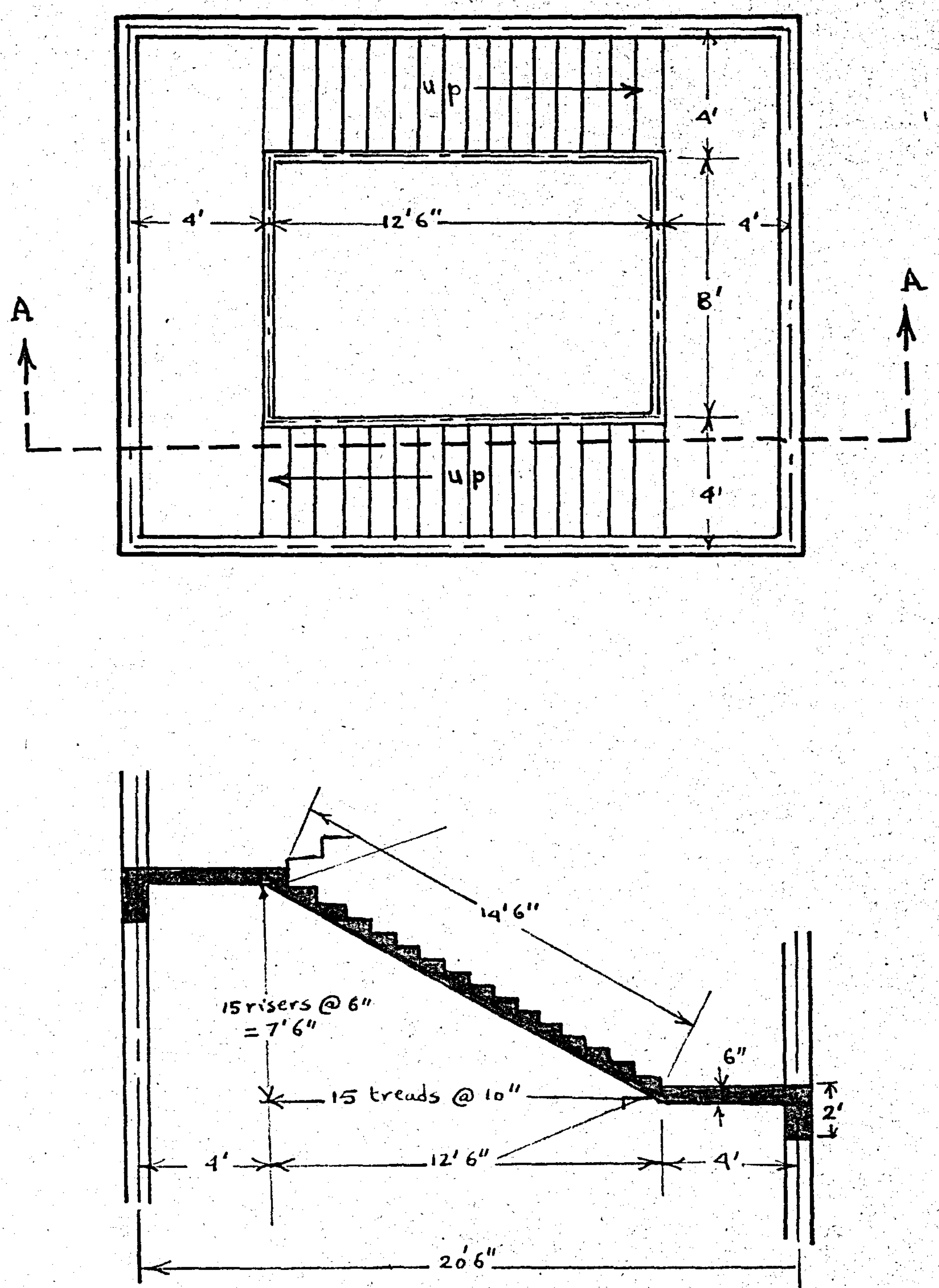


Fig. VI-8 Stairway Elevations & Stair Slabs.



Section A-A

Fig. VI-9 Plan of Stairs

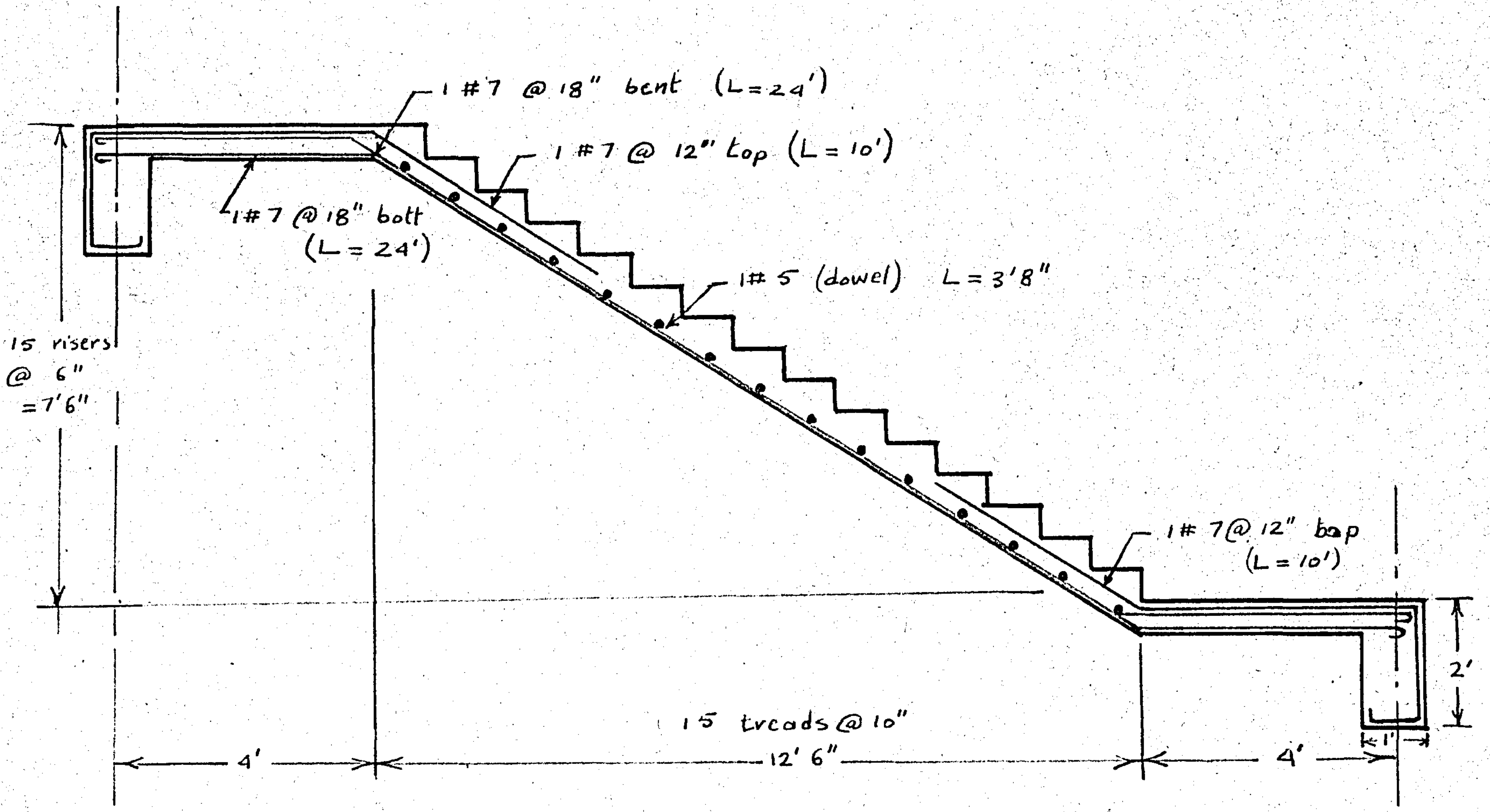


Fig. VI - 10 Stair Slab - Steel Reinforcement (Scale 1 ft = 1 cm.)

2. 1 No.3 at 18 in. bott.

Long span:

3. 1 No.3 at 18 in. top.

4. 1 No.3 at 18 in. bott.

Design of Beams

Design of Beams $B_2 - B_{10}$ and $B_2' - B_{10}'$ Fig. VII-11 & 12

$L = 16$ ft.

Load: w_1 (from stair slabs) $= \frac{200 \times 20.5}{2} = 2050$ lbs/lin.ft.

w_2 (from slab S) $= 175 \times 2 = 305$ lbs/lin.ft.

w_3 (from supported wall) $= 30 \times 13 = 390$ lbs/lin.ft.

w_4 (D.L. of beam) $= 2 \times 1 \times 150 = 300$ lbs/lin.ft.

Total wt. 0 — 4 ft. = 3090 lbs./lin-ft.

4ft. — 12 ft. = 1040 lbs./lin-ft.

12 ft. — 16 ft. = 3090 lbs./lin-ft.

A safe assumption at average w for moment and shear calculations:

$$w \text{ (average)} = \frac{3090 + 1040}{2} = 2065 \text{ lbs/lin-ft.}$$

$$\text{Moments: } (-)M = \frac{wl^2}{12} = \frac{2.065\,(16)^2}{12} = 44.2 \text{ kip-ft.}$$

$$(+)M = \frac{wl^2}{16} = \frac{2.065\,(16)^2}{16} = 33 \text{ kip-ft.}$$

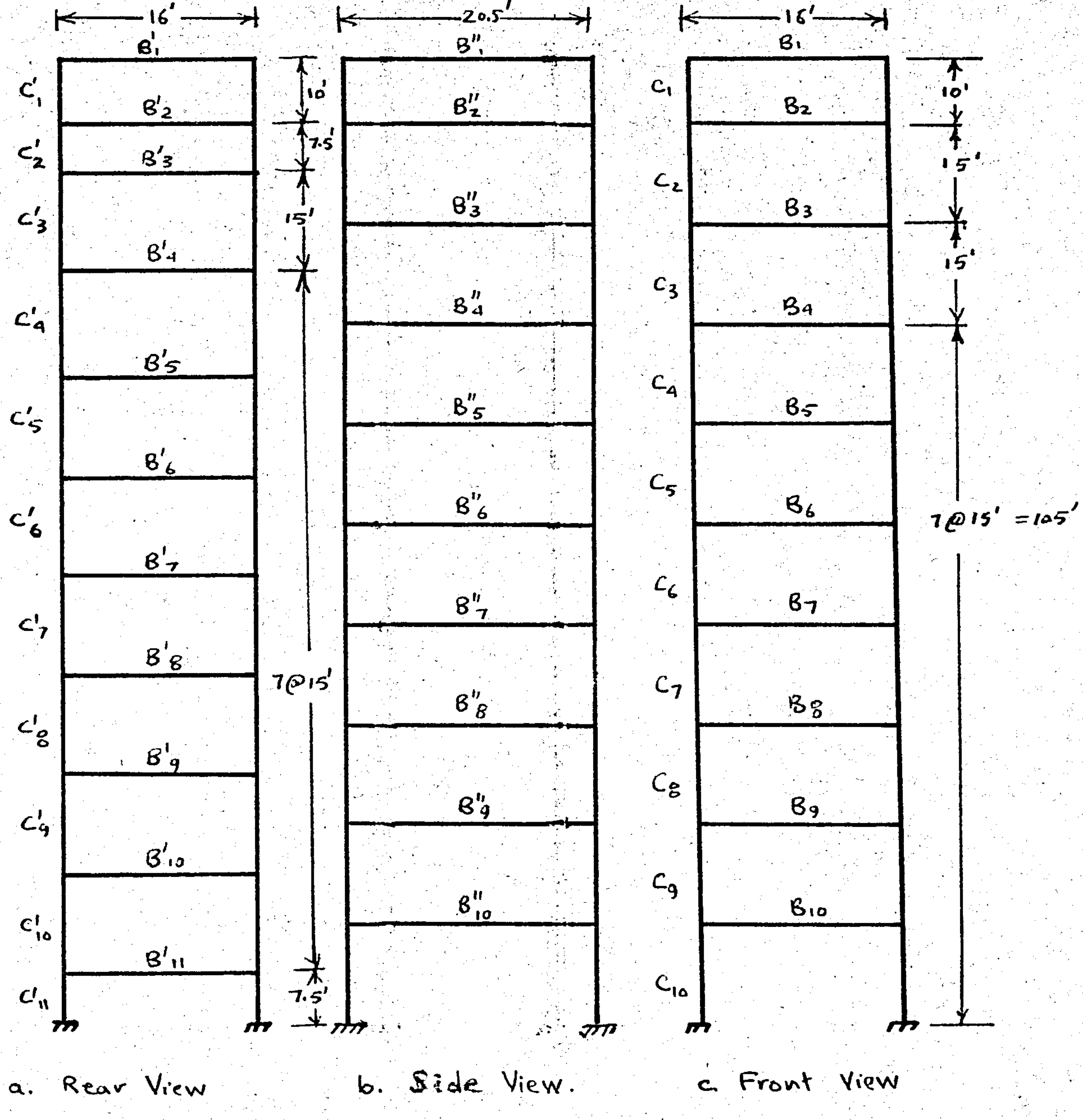


Fig. VI-11 Stairway - Beams & Columns.

Steel: $(-)As = \dfrac{-M}{Jdf} = \dfrac{44.2 \times 12}{7/8 \times 20 \times 20} = 1.52$ sq. in.

$(+)As = \dfrac{+M}{Jdf} = \dfrac{33 \times 12}{7/8 \times 20 \times 20} = 1.14$ sq. in.

(+)As .... 2 No.5 str. bott.

2 No.5 bent.

(-)As .... 2 No.5 bent.

3 No.5 str. top.

Spacing: 5 X 0.625 = 3.13 in.

4 X 1.00 = 4.00 in.

2 X 1.5 = 3.00 in.

Total = 10.13 in 12 in.

Shear: $V = w_1(4) + w_2(4) + w_3(8) + w_4(8)$

$= 2050\ (4) + 650(4) + 300\ (8) + 390\ (8)$

$= 16{,}320$ lbs.

$v = \dfrac{V}{bJd} = \dfrac{16320}{12 \times 7/8 \times 22} = 70$ psi $>$ 60 psi O.K.

use stirrups: Fig. VI-12.

$x = 8 \times \dfrac{10}{70} = 1.2$ ft.

Stirrups No.3 bars

$S = \dfrac{AvFv}{v'b} = \dfrac{0.22\ (20{,}000)}{10 \times 12} = 37$ in.

use 2 at 8 in.

Bond: Checks O.K.

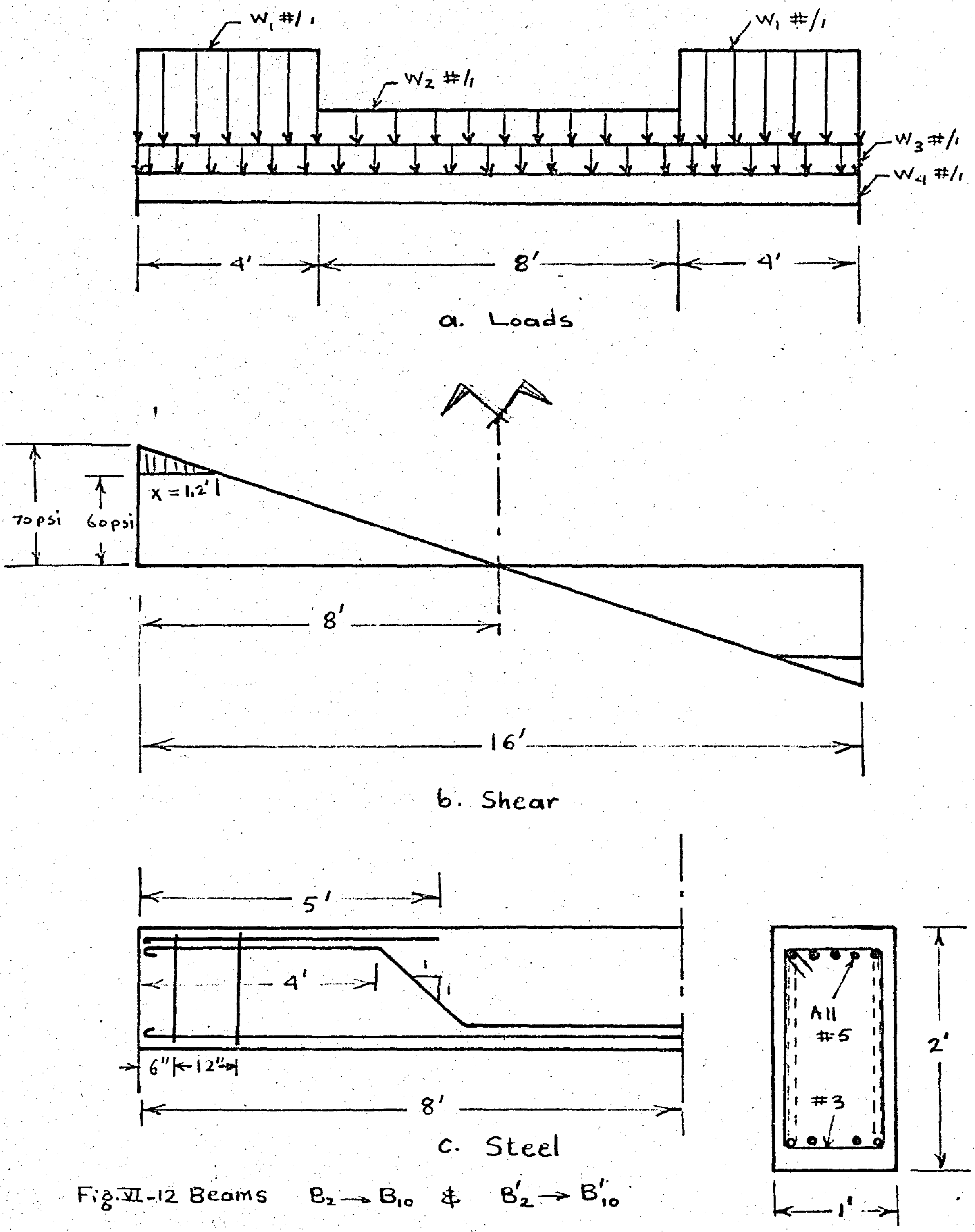


Fig. VI-12 Beams $B_2 \rightarrow B_{10}$ & $B'_2 \rightarrow B'_{10}$

Design of Beams B" - B" Fig. VI 11 & 13

These beams serve two purposes:

1. To give further rigidity to the structure to resist horizntal loads as wind, bomb, or earth quake.
2. They carry the concrete hollow blocks which close the sides of the stairway structure.

$L = 20.5$ ft.

Load: w (from wall) = 30 X 13 = 390 lbs./lin.ft.

w (D.L.) = 2 X 1 X 150 = 300 lbs./lin.ft.

Total w = 690 lbs./lin.ft.

Moments: $(-)M = \frac{wl^2}{12} = \frac{0.690 \ (20.5)^2}{12} = 24.2$ kipft.

$(+)M = \frac{wl^2}{16} = \frac{0.690 \ (20.5)^2}{16} = 18.2$ kip-ft.

Steel: $(-)\ As = \frac{-M}{Jdf} = \frac{24.2 \times 12}{7/8 \times 20 \times 20} = 0.83$ sq.in.

$(+)As = \frac{+M}{Jdf} = \frac{18.2 \times 12}{7/8 \times 20 \times 20} = 0.62$ sq.in.

(+)As .... 2 No.5 str.bott.

(-)As .... 3 No.5 str.top.

Spacing: Checks O.K.

Shear: $V = \frac{wl}{2} = \frac{0.69 \times 20.5}{2} = 7.1$ kips

$v = \frac{V}{bJd} = \frac{7100}{12 \times 7/8 \times 22} = 31$ psi $\angle$ 60 psi O.K.

Bond: $u = \frac{V}{OJd} = \frac{7100}{5.9 \times 7/8 \times 22} = 63$ psi $\angle$ 140 psi O.K.

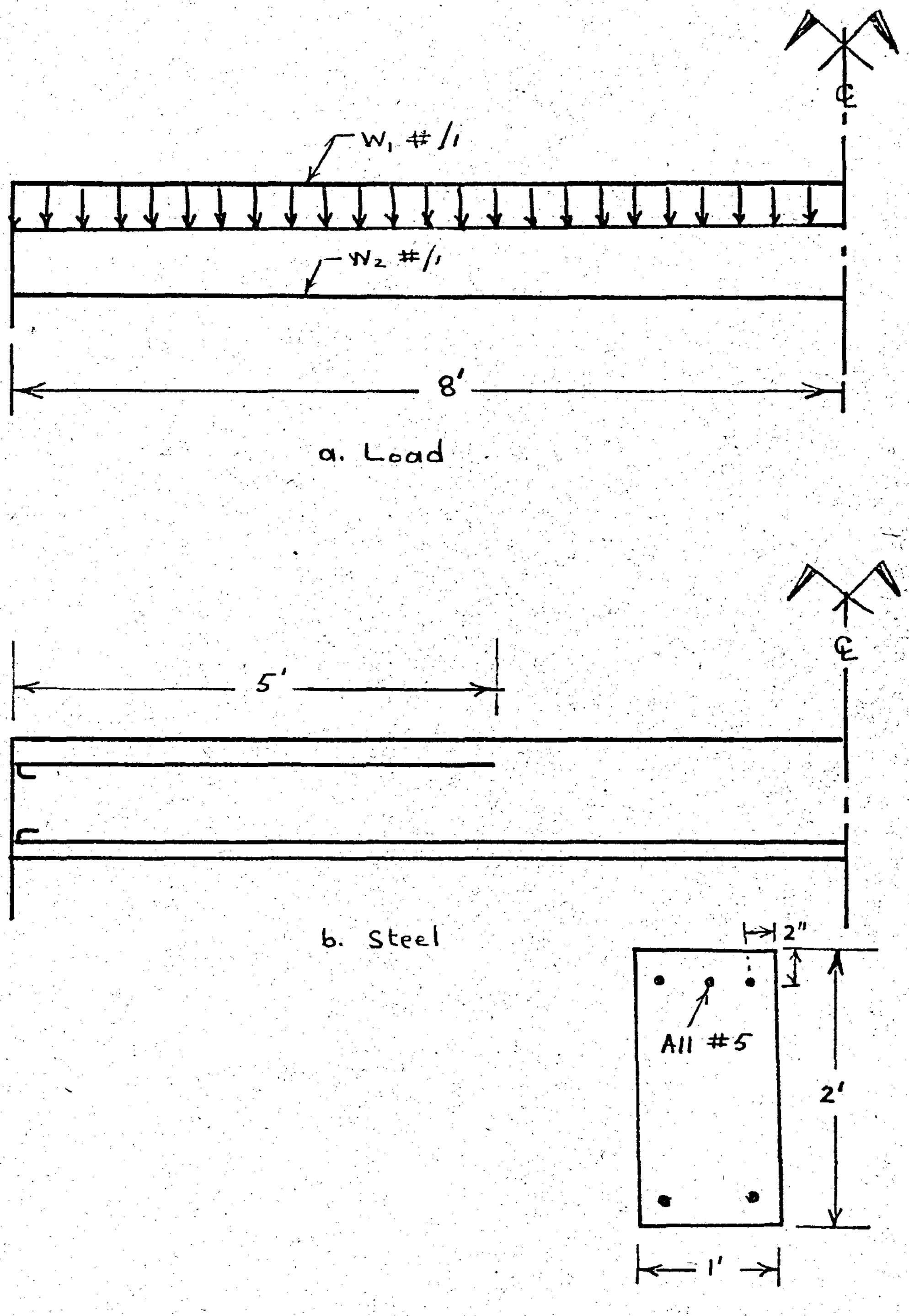


Fig. VI-13 Beams $B''_2 \rightarrow B''_{10}$

Design of Center Wall

The center wall will serve more than one purpose. It gives further rigidity to the slender structure and this will help in resisting horizontal loads. It will support slabs S at different levels and these in turn cut the unsupported length of the wall.

The wall is going to be 6 inches thick for the uppermost 15 feet, and for each successive 25 feet downward the minimum thickness shall be increased one inch. Fig. VI-15.

The wall will have opening for elevators' doors at the entrance of the stairway and at the threee levels of the casino structure. More openings for doors can be made at other levels of the stairway fly-overs.

Load on the Wall: Fig. VI-14.

w (from top slab)

$$w = \frac{w_1 s}{3} + 2w_2 = \frac{625 \times 8}{3} + 2(175) = 2020 \text{ lbs/lin.ft.}$$

w on AD or BC = w(from S_1) + w (from S_2)

$$w = \frac{w_1 s}{3} \left(\frac{3 - m}{2}\right) + 2w_2$$

$$= \frac{625\ (8)}{3} \ \frac{(3 - 0.41)}{2} + 2\ (175) = 2500 \text{ lbs/lin.ft.}$$

use w = 2500 lbs/lin.ft. at top level.

w (load from S at each level or possible load from stairs =

2 X 200 = 400 lbs/lin.ft.

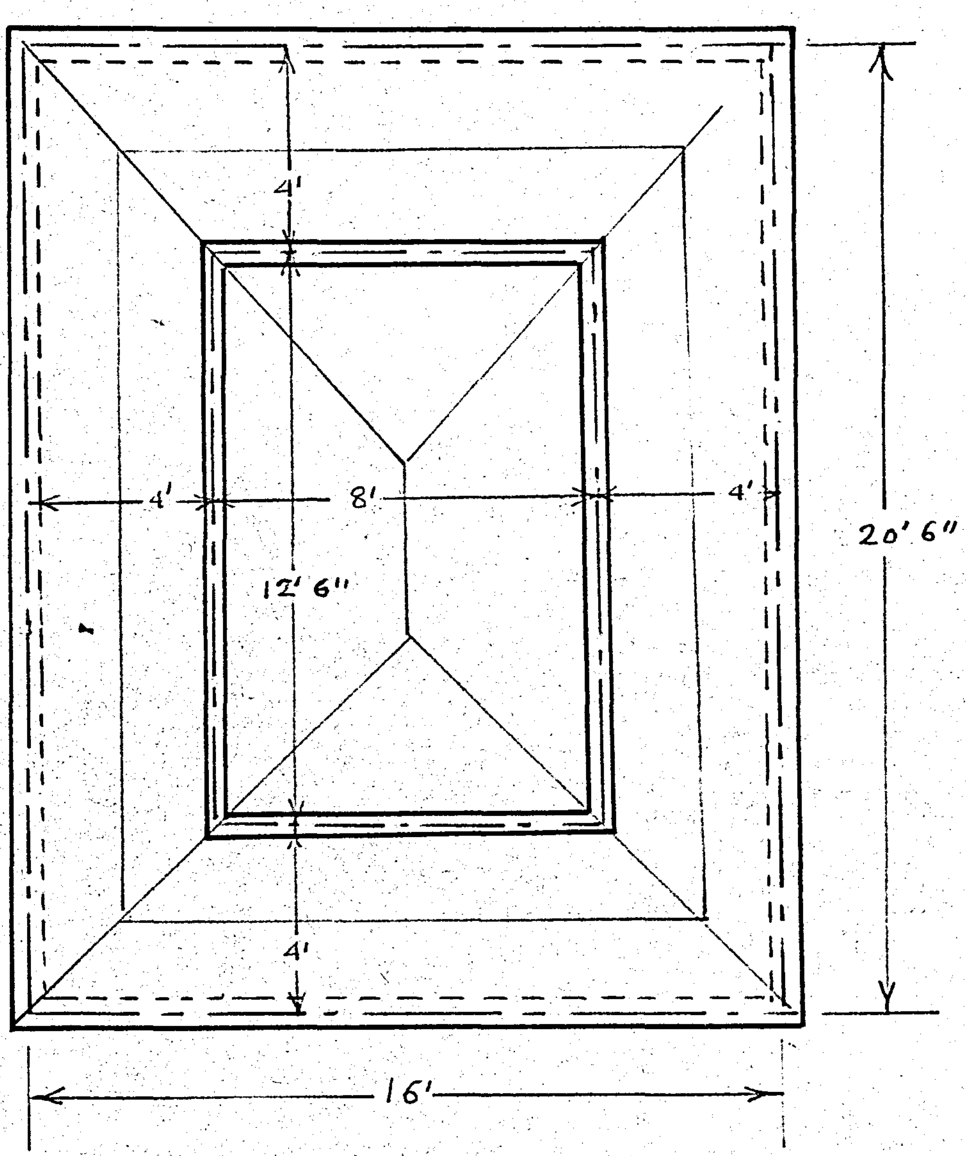


Fig. VI-14 Plan of Stairway - Loads on Center-Wall.

Design of Sections:

The following formula is going to be used in the design for fc.

$$fc = 0.225\, f'c \left[1 - \left(\frac{h}{40t}\right)^3\right].$$

$$f'c = 2000 \text{ psi}$$

$$fc = 675 \left[1 - \left(\frac{h}{40t}\right)^3\right].$$

Area of horizontal reinforcement shall be not less than 0.0025 and that of the vertical reinforcementnot less than 0.0015 times the area of the reinforced section of the wall.

For anchorage, 1 No.5 , 4 ft. 6 in. under each tread of the stairs will be extended to the wall and bent up at 3 inches.

There shall be two No.5 bars around all window and door openings. Such bars shall extend 24 inxhes beyond the corner of the openings.

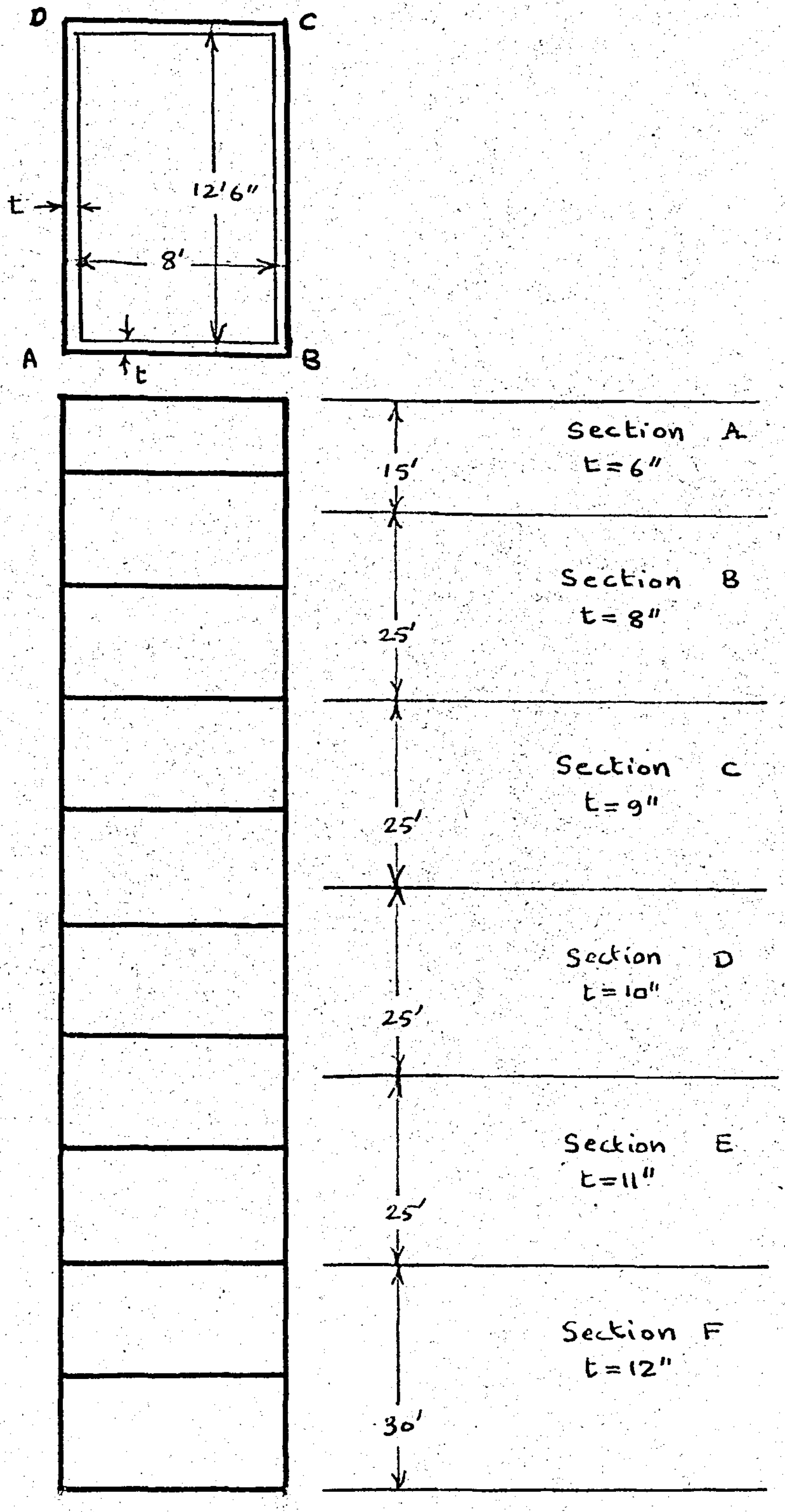


Fig VI-15 Center Wall Sections & Thicknesses

Design of Section A Fig. VI - 15.

$t = 6$ in.

$L = 15$ ft.

$w = 2.5 + 15\ (0.15)(0.5) + 0.40 = 4.02$ kip/lin.ft.

$$fc = 675 \left[1 - \left(\frac{15 \times 12}{40 \times 6}\right)^3\right] = 392 \text{ psi}$$

$$f = \frac{4020}{6 \times 12} = 56 \text{ psi} < 392 \text{ psi} \quad \text{O.K.}$$

Horizontal reinforcement:

$As = 0.0025 \times 12 \times 6 = 0.18$ sq.in/lin.ft.

As .... No.3 at 6 in.

Vertical reinforcement:

$As = 0.0015 \times 12 \times 6 = 0.11$ sq.in/lin.ft.

As .... No.3 at 12 in.

Design of Section B Fig. VI - 15

$t = 8$ in.

$L = 25$ ft.

$w = 4.02 + 25(0.15) \times 8/12 + 0.4 = 6.92$ kip/lin.ft.

$$fc = 675 \left[1 - \left(\frac{25 \times 12}{40 \times 8}\right)^3\right] = 136 \text{ psi.}$$

$$f = \frac{6920}{8 \times 12} = 72 \text{ psi} < 136 \text{ psi} \quad \text{O.K.}$$

Horizontal reinforcement:

$As = 0.0025 \times 12 \times 8 = 0.24$ sq.in/lin.ft.

As .... No.3 at 4 in.

Vertical reinforcement:

As = 0.0015 X 12 X 8 = 0.15 sq.in/lin.ft.

As ....No.3 at 8 in.

<u>Design of Section C</u> Fig. VI _ 15.

t = 9 in.

L = 25 ft.

w = 6.92 + 25 (0.15)(9/12) + 0.4 X 2 = 10.53 kips/lin.ft.

$$fc = 675 \left[1 - \left(\frac{25 \times 12}{40 \times 9}\right)^3\right] = 285 \text{ psi}$$

$$f = \frac{10530}{12 \times 9} = 97 \text{ psi} \angle 285 \text{ psi} \qquad \text{O.K.}$$

Horizontal reinforcement:

As = 0.0025 X 12 X 9 = 0.27 sq.in/linft.

As .... No.3 at 4 in.

Vertical reinforcement:

As = 0.0015 X 12 X 9 = 0.16 sq.in/lin.ft.

As .... No.3 at 8 in.

<u>Design of Section D</u> Fig. VI - 15

t = 10 in.

L = 25 ft.

w = 10.53 + 25 X 0.15X 10/12 + 0.4 X2 = 14.45 kips/lin.ft.

$$fc = 675 \left[1 - \left(\frac{25 \times 12}{40 \times 10}\right)^3\right] = 392 \text{ psi}$$

$$f = \frac{14450}{10 \times 12} = 121 \text{ psi} \quad 392 \text{ psi} \qquad \text{O.K.}$$

Horizontal reinforcement:

As = 0.0025 X 12 X 10 = 0.30 sq.in/lin.ft.

As .... No.3 at 4 in.

Vertical reinforcement:

As = 0.0015 X 12 X 10 = 0.18 sq.in/lin.ft.

As .... No.3 at 6 in.

<u>Design of Section E</u> Fig. VI - 15.

t = 11 in.

L = 25 ft.

w = 14.45 + 25 X 0.15 X 11/12 + 0.4 = 18.30 kips/lin.ft.

$$fc = 675 \left[1 - \left(\frac{25 \times 12}{40 \times 11}\right)^3\right] = 460 \text{ psi.}$$

$$f = \frac{18300}{12 \times 11} = 151 \text{ psi} \quad 460 \text{ psi} \qquad \text{O.K.}$$

Horizontal reinforcement:

As = 0.0015 X 12 X 11 = 0.18 sq.in/lin.ft.

in two layers.

As .... No.3 at 12in. each layer.

Design of Section F Fig. VI - 15

$t = 12$ in.

$L = 25$ ft.

$w = 18.30 + 25 \times 0.15 \times 12/12 + 0.4 \times 2 = 22.85$ kips/linft.

$$f_c = 675 \left[1 - \left(\frac{25 \times 12}{40 \times 2}\right)^3\right] = 505 \text{ psi.}$$

$$f = \frac{22850}{12 \times 12} = 158 \text{ psi} \angle 505 \text{ psi} \qquad \text{O.K.}$$

Horizontal reinforcement:

$A_s = 0.0025 \times 12 \times 12 = 0.36$ sq.in/lin.ft.

in two layers.

As .... No.3 at 6 in each layer.

Vertical reinforcement:

$A_s = 0.0025 \times 12 \times 12 = 0.22$ sq.in/lin.ft.

in two layers.

As .... No.3 at 12 in. each layer.

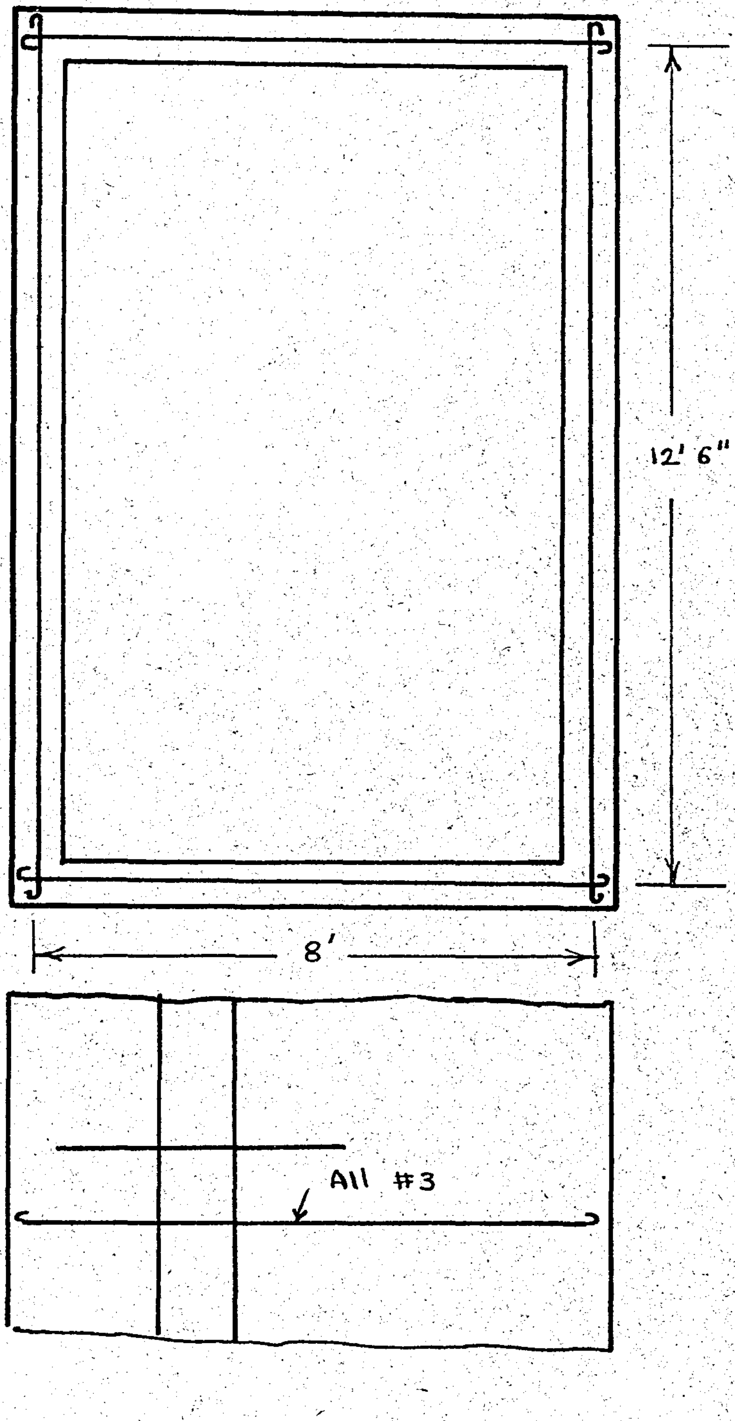


Fig. VI-16 Center Wall, Steel Reinforcement

Design of Columns

The reinforced concrete design of columns is going to consist of three different farms:

1- Columns $C_1 \rightarrow C_3$ and $C'_1 \longrightarrow C'_3$

2- Columns $C_4 \rightarrow C_7$ and $C'_4 \longrightarrow C'_7$

3- Columns $C_8 \rightarrow C_{10}$ and $C'_8 \longrightarrow C'_{11}$

In each of these designs the maximum concentrated load and the maximum bending momentsin the X and Y directions are considered.

This is done here to simplify the work on the situ and reduce the amount of labor. Another important thing here is that the quantity of steel is not going to vary considerably if ten different designs are made for the ten different columns.

Relative stiffness of columns $K = I/L$

$$K\ (\text{long beam}) = \frac{1\ (8)}{12\ (20.5)} = \underline{0.0325} \qquad 325$$

$$K\ (\text{short beam}) = \frac{1\ (8)}{12\ (16)} = 0.0415 \qquad 415$$

$$K\ (\text{column}) = \frac{1\ (1)}{12\ (15)} = 0.0056 \qquad 56$$

$$k_y = \frac{K\ (\text{column})}{\Sigma K_y} = \frac{56}{381} = 0.146$$

$$k_x = \frac{K\ (\text{column})}{\Sigma K_x} = \frac{56}{471} = 0.118$$

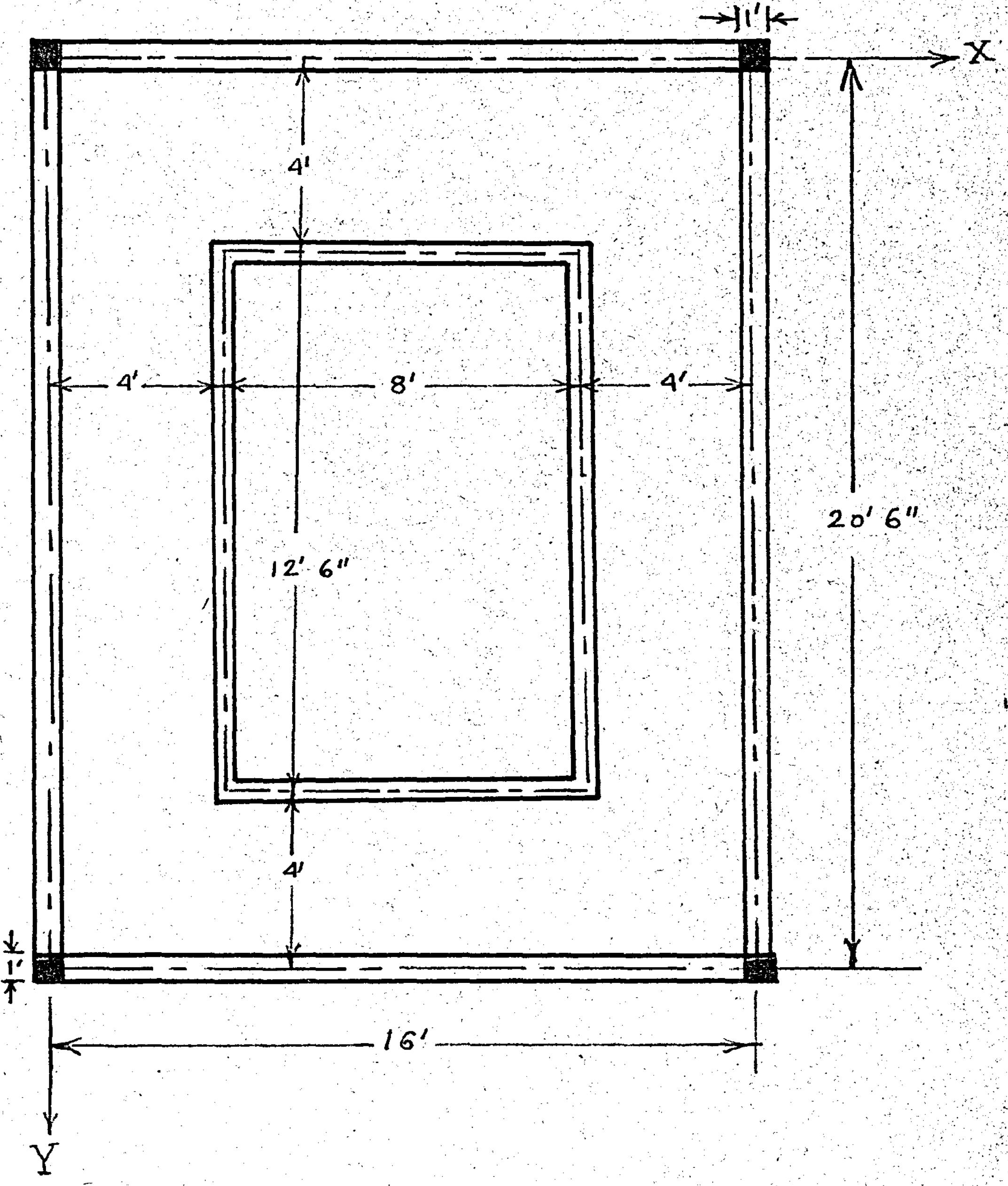


Fig VI-17 Plan of Stairway - Showing Columns & Direction

Table VI-1

Moments on Columns

Column	M_x	M_y	k_x	k_y	kM_x	kM_y
C & C'	14.0	22.8	0.118	0.146	1.65	3.34
C – C & C'– E'	44.2	24.2	0.118	0.146	5.20	3.53

Table VI-2

Loads on Columns

Column	L	h/t	P kips	$\triangle$P kips	Σ P kips
C	10	8	11.9	—	11.9
C	15	13	11.9	24.9	36.8
C	"	"	36.8	25.0	61.8
C	"	"	61.8	"	86.8
C	"	"	86.8	"	111.8
C	"	"	112	"	137
C	"	"	137	"	162
C	"	"	162	"	187
C	"	"	187	"	212
C	"	"	212	"	237

* L = length of Column.
* h = unsupported length of column.
* t = min. thickness of column = 1 ft.

Design of Columns $C_1 \rightarrow C_4$ & $C'_1 \rightarrow C'_4$ Fig. VI - 18 - a

At C_4: $M_x = 5.20$ kip-ft.

$M_y = 3.53$ kip-ft.

$N = 86.8$ kips.

(Bending effect is small, it will be accounted for by increasing percentage of steel).

$N = P\ (1.3 - 0.03\ h/t)$

$N = P\ (1.3 - 0.03 \times 13)$

$P = 86.8 / 0.91 = 95$ kips.

Steel: $P = 0.18\ f'cAg + fsAs$

$95 = 0.18\ (2)(144) + 20\ As$

As = 2.2 sq.in.

As 4 No.9

Tie bars: $\frac{1}{4} \times 48 = 12$ in.

As .... No.2 at 12in.

Design of Columns $C_5 \rightarrow C_8$ & $C'_5 \rightarrow C'_8$ Fig. VI - 18 - b

At C_8 $M_x = 5.20$ kip-ft.

$M_y = 3.53$ kip-ft.

$N = 187$ kips.

(Bending effect is neglected, it will be accounted for, by increasing percentage of steel).

$N = P\ (1.3 - 0.03\ h/t)$

$N = P\ (1.3 - 0.03 \times 13)$

$P = 187 / 0.91 = 205$ kips

Steel: $P = 0.18\ f'c\ Ag + Fs\ As$

$205 = 0.18\ (2)(144) + 20\ As$

$As = 7.2$ sq.in.

As 8 No.9

Tie bars: $\frac{1}{4} \times 48 = 12$ in.

As .... No.2 at 12 in.

Design of Columns $C_9 \rightarrow C_{10}$ & $C'_9 \rightarrow C'_{10}$ Fig. VI-18-c

At C_{10} $M_x = 5.2$ kip-ft.

$M_y = 3.53$ kip-ft.

$N = 237$ kips.

(Bending effect is neglected).

$N = P\ (1.3 - 0.03\ h/t)$

$N = P\ (1.3 - 0.03 \times 13)$

$P = 237 / 0.91 = 270$ kips.

Steel: $P = 0.18\ f'c\ Ag + fs\ As$

$270 = 0.18\ (2)(144) + 20\ As$

$As = 10.9$ sq.in.

As 12 No.9

Tie bars: $\frac{1}{4} \times 48 = 12$ in.

As .... No. at 12 in.

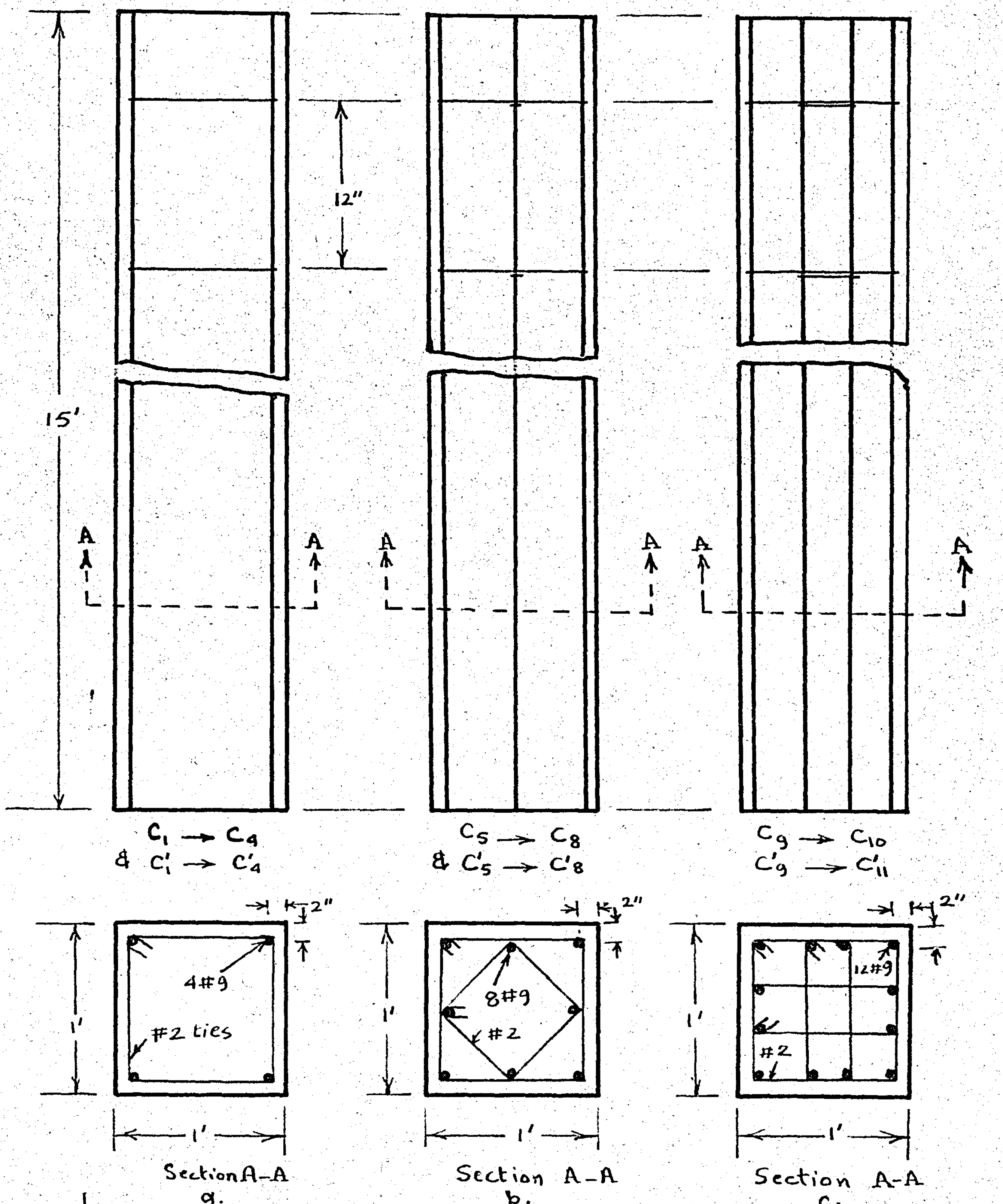


Fig. VI-18 Stairway Columns _ Steel Reinforcement (Scale 1'=3cm)

Design of Footings

There will be one square footing under each column 3 ft X 3 ft and 2 ft. deep. Form No.9 dowels each 4 ft. long will be used between each column and the supporting footing.

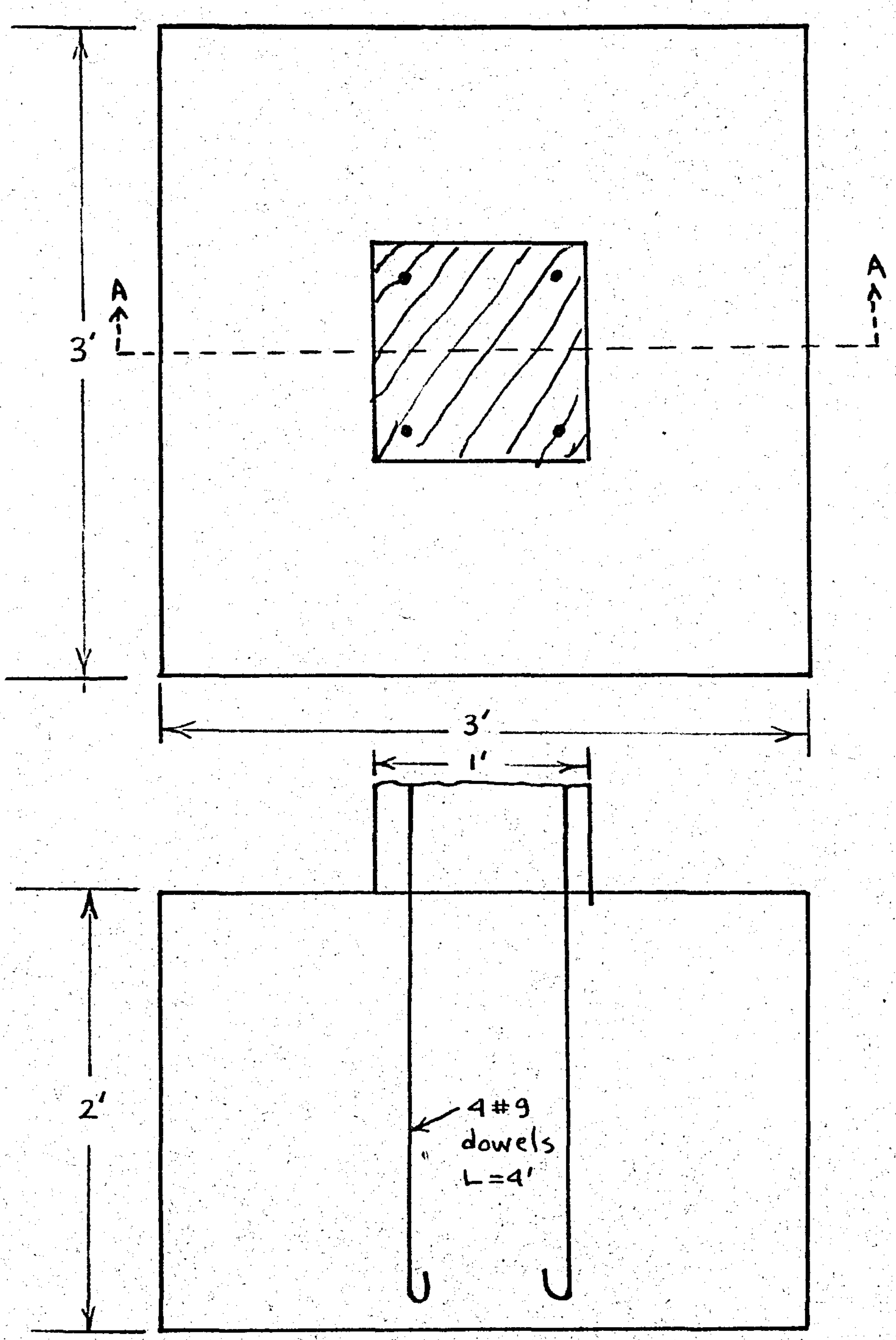


Fig. VI-19 Stairway Footings.

Bill of Materials of the Stairway

Steel and Concrete

Member	No. of Parts	Bars	Shape	Length ft	Wt/ft. lbs.	Wt. lbs.	Concrete ft^3
Top Slab	1	8 # 3	Straight	16	0.376	48.4	164
"	1	4 # 3	"	9	0.376	13.6	
"	1	12 # 3	Bent	10	0.376	45.2	
"	2	8 # 3	Straight	2 X 7	0.376	84.0	
"	2	16 # 3	"	2 X 4	0.376	97.0	
"	2	3 # 3	"	16	0.376	36.0	
"	2	3 # 3	"	16	0.376	36.0	
Supporting beams	2	1 # 4	"	21	0.668	29.8	82
"	2	2 # 4	Bent	24	0.668	66.0	
"	2	2 # 4	Straight	6	0.668	16.5	
" B_1	2	1 # 4	"	17	0.668	23.4	64
"	2	2 # 4	Bent	19	0.668	52.1	
"	2	1 # 4	Straight	5	0.668	6.7	
Stairs	18	3 # 7	Bent	24	2.044	2650.0	1280
	18	3 # 7	"	24	2.044	2650.0	
	18	4 # 7	"	10	2.044	1460.0	
	18	4 # 7	"	10	2.044	1460.0	
	18	5 # 5	Straight	4'6"	1.043	1279.0	
Beams B & B'	18	2 # 5	"	17	1.043	640.0	586

Bill of Materials of the Stairway (con't)

Member	No. of Parts	Bars	Shape	Length	Wt/ft. lbs.	Wt. lbs.	Concrete ft^3
	18	2 # 5	Bent	20	1.043	750.0	
	18	3 #5	Straight	2 X 5	1.043	562.0	
	18	2 #3	Bent	2 X 6	0.376	1622.0	
Slab S	18	10 #3	Straight	5	0.376	340.0	288
	18	6 #3	"	10	0.376	408.0	
Beams B"	18	2 #5	"	21	1.043	790.0	720
"	18	3 #5	"	2 X 6	1.043	680.0	
Columns	16	4 #9	"	17	3.400	3700.0	520
	16	12 #2	Ties	4	0.167	128.0	
	16	8 #9	Straight	17	3.400	7400.0	
	16	12 #2	Ties	4	0.167	128.0	
	16	12 #2	"	3	0.167	96.0	
	8	12 #9	Straight	17	3.400	64.0	
	8	12 #2	Ties	4	0.167	64.0	
	8	12 #2	"	2 X 3	0.167	128.0	
Footings	4	4	Dowels	4	3.400	218.0	72
Center Wall	2	430#3	Straight	16	0.376	5190.0	4573.
	2	430#3	"	21	0.376	6800.0	
	1	74 #3	"	16	0.376	446.0	
	1	111#3	"	27	0.376	1130.0	
	1						

Bill of Materials of the Stairway (con't.)

Member	No. of Parts	Bars	Shape	Length	Wt/ft. lbs	Wt. lbs.	Concrete ft^3
	1	111#3	Straight	27	0.376	1130.0	
	1	148#3	"	27	0.376	1500.0	
	1	148#3	"	27	0.376	1500.0	
	1	148#3	"	27	0.376	1500.0	
	1	16 #5	"	2	1.043	33.4	
					Total:	51,071	8349

Estimated volume of Blasted Rock

$$V = \frac{\text{Volume of Stairway Structure}}{4}$$

$$= \frac{20 \times 16 \times 145}{4} = 11{,}600 \text{ ft}^3.$$

Chapter VII

DESIGN OF THE BRIDGE

General Illustrations

The design of the bridge is of minor architectural and structural importance compared to that of the casino. There are no restrictions with regard to length of spans, number of piles, or elevation above water level.

No ships or big boats are going to pass under the bridge. The water is comparatively shallow. The design of a number of piles does not constitute an economic or a structural problem. Furthermore the sea bed, most probably, consists of solid rock similar to the surrounding.

The bridge will be designed of five parts of equal dimensions as shown in the plan Fig. VII-1. Each of these parts will consist of four piles covered with a slab resting on two beams.

One reason for this design is that in case of settlement or failure of one of the five parts the other four will not be affected. Another reason is for temperature effects. The bridge as a whole being about 380 feet long is weak in resisting stresses due to temperature changes if it were made of one piece.

Every section will be an inch or so apart from the adjacent section from each side so as to allow expansions in hot weather.

In case the floor of the shore Building comes out to be few feet higher than the level of the bridge few steps will be constructed at the entrance of the bridge.

The bridge is going to be desigined so as to resist loads due to transportation of materials and equipment during the construction of the casino and stairway and to resist loads due to pedestrian after the construction.

The wave pressure on the bridge will be ignored because the exposed area of a pile is small so that the net wave effect on the bridge is not of considerable importance.

The bridge may be covered if desired with steelframes and glass or by any other material that is found to be suitable, and this additional load will be accounted for in the structural design.

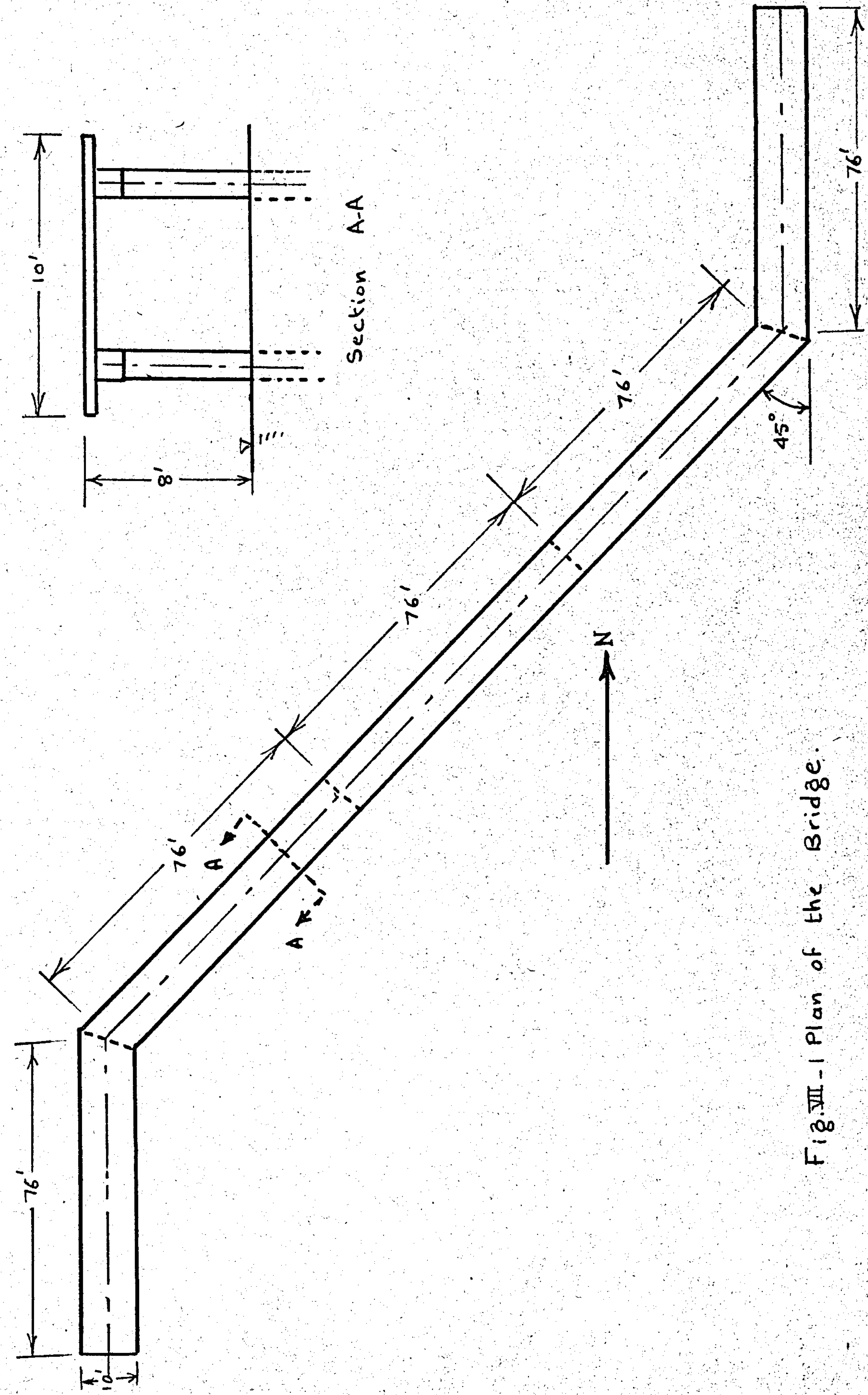


Fig. VII-1 Plan of the Bridge.

Structural Design

$f'c$ = 2000 psi at 28 days

fs = 20,000 psi (deformed bars).

L.L. = 100 lbs/sq.ft.

Proportioning of Beam Spans: Fig. .

The beams are proportioned in such a way so as to have the piles under pure compression. Net moment on a pile is zero.

$$\frac{wl_2^2}{2} = \frac{wl_1^2}{12}$$

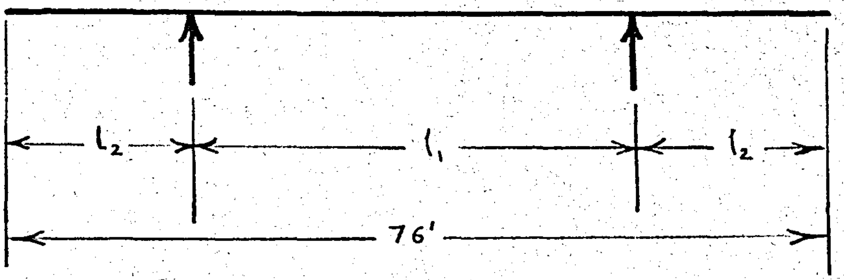
$$l_2 = l_1/\sqrt{6}$$

$$l_2 = (76 - l_1) / 2$$

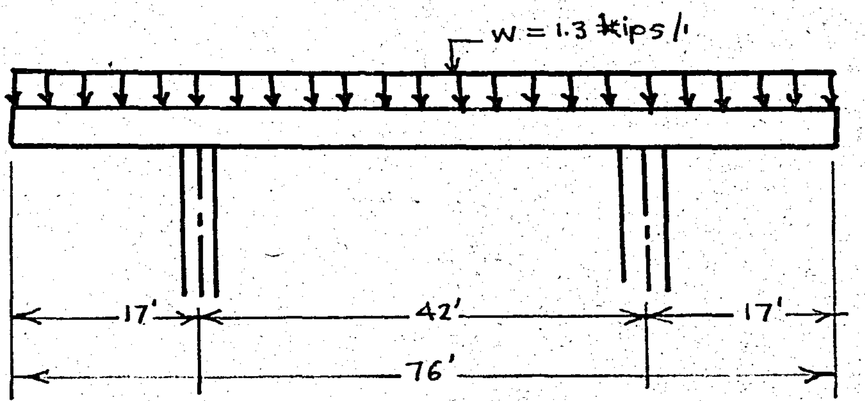
$$\frac{76 - l_1}{2} = \frac{l_1}{\sqrt{6}}$$

$$l_1 = 42 \text{ ft.}$$

$$l_2 = 17 \text{ ft.}$$



a. Proportioning of Spans



b. Beams- Lengths & Loads

Fig. VII-2 Bridege Beams

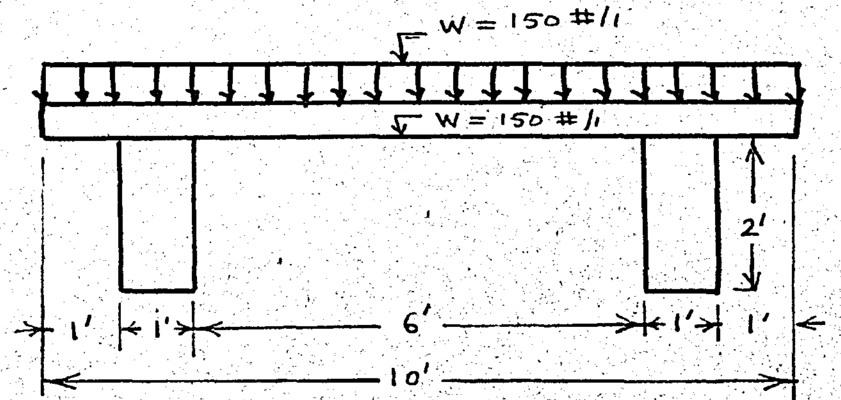


Fig. VII-3 Bridge Floor Slab.

Design of Slab: Fig. VII -3.

L.L. = 100 lbs/sq.ft.

D.L. = $\frac{4 \times 0.15}{12}$ = 50 lbs/sq.ft.

Partition and cover load = 50 lbs/sq.ft.

Total load = 200 lbs/sq.ft.

Design of one foot-strip: Fig. .

$$(-)M = \frac{wl^2}{12} = \frac{0.22\ (7)^2}{12} = 0.82 \text{ kip-ft.}$$

$$(+)M = \frac{wl^2}{16} = \frac{0.2(7)^2}{16} = 0.61 \text{ kip-ft.}$$

$$(-)As = \frac{0.82 \times 12}{7/8 \times 12 \times 30} = 0.047 \text{ sq.in./ft. strip}$$

$$(+)As = \frac{0.61 \times 12}{7/8 \times 12 \times 20} = 0.035 \text{ sq.in./ft. strip}$$

(-) As .... No.2 at 12 in.

(+) As .... No.2 at 12 in.

Longitudinal (temperature) reinforcement:

As = 0.002 X 12 X 4 = 0.096 sq.in./ft. strip

As .... No.2 at 6 in.

Design of Supporting Beams   Fig. VII - 2.

$w = 0.2 \times 5 = 1$ kip/ft.

w (D.L.) $= 2 \times 1 \times 0.15 = 0.3$ kip/ft.

Total $w = 1.3$ kip/ft.

Design of Beam L_2 Fig. VII - 4.

$$(-)M = \frac{wl_2^2}{2} = \frac{1.3\ (17)^2}{2} = 188 \text{ kip-ft.}$$

$$(-)A_s = \frac{-M}{Jdf} = \frac{188 \times 12}{7/8 \times 20 \times 20} = 6.45 \text{ sq.in.}$$

(-)As .... 4 No.9 ... L = 18 ft.

4 No.9 ... L = 10 ft.

Spacing and bond check. O.K.

Shear: $V = wl = 1.3 \times 12 = 15.6$ kips.

$$v = \frac{V}{bJd} = \frac{15600}{12 \times 7/8 \times 24} = 62 \text{ psi approx.} \cong 60 \text{ psi} \quad \text{O.K.}$$

Design of Beam L_1 Fig. VII - 5.

$$(-)M = \frac{wl_1^2}{12} = \frac{1.3\ (42)^2}{12} = 192 \text{ kip-ft.}$$

$$(+)M = \frac{wl_1^2}{16} = \frac{1.3\ (42)^2}{12} = 144 \text{ kip-ft.}$$

$(-)A_s = 8$ sq. in. (From design of beam L)

$$(+)A_s = \frac{M}{Jdf} = \frac{144 \times 12}{7/8 \times 20 \times 24} = 4.1 \text{ sq.in.}$$

(+)As .... 2 No.9 str. bott.

2 No.9 bent.

$(-)A_s$ .... 2 No.9 bent.

6 No.9 str. top.

Spacing and bond check O.K.

Shear:

$$V = \frac{wl}{2} = \frac{1.3\ (42)}{2} = 27.4 \text{ kips.}$$

$$v = \frac{27.4}{12 \times 7/8 \times 24} = 108 \text{ psi} > 60 \text{ psi}$$

Use stirrups, Fig. VII - 5.

$$x = 48 \times \frac{21}{108} = 9.3 \text{ ft.}$$

Stirrup No.3 bars

$$S = \frac{A_v f_v}{v' b} = \frac{0.22 \times 20{,}000}{12 \times 48} = 7.6 \text{ in} \;\ldots\ldots\; 7 \text{ in.}$$

use .... 10 at 7 in.

5 at 10 in.

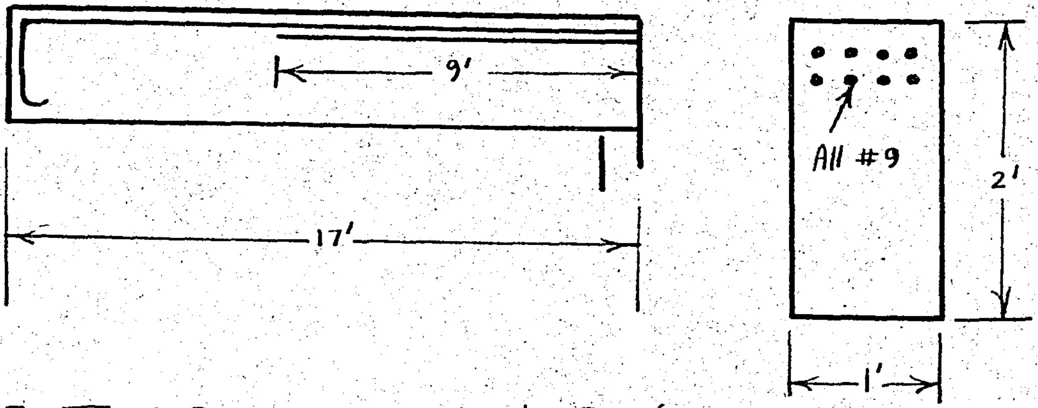


Fig. VII-4 Beam L_2 - Steel Reinforcement

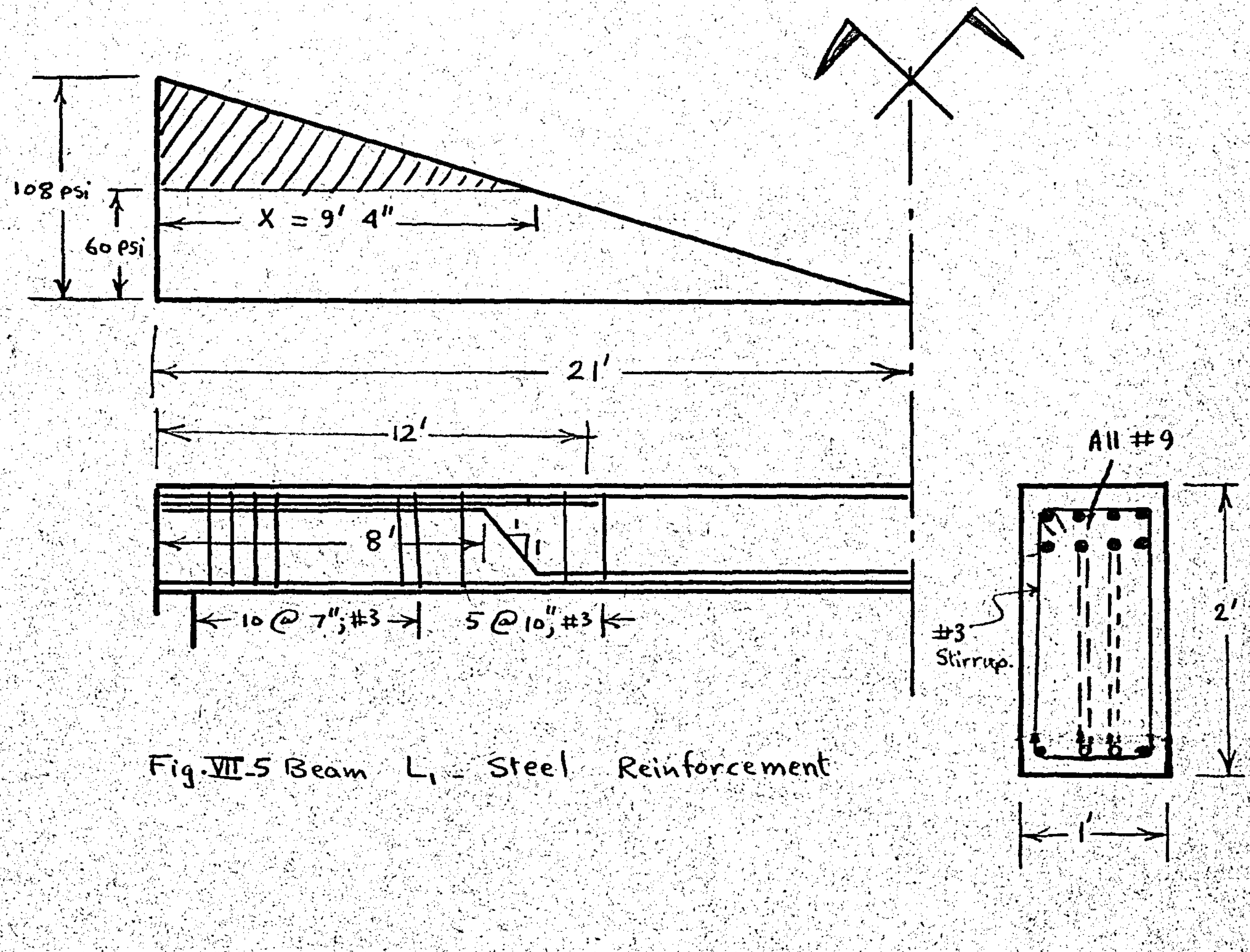


Fig. VII-5 Beam L_1 - Steel Reinforcement

<u>Pile Design</u>:

Since the depth of the water at different points is unknown, the design of piles will be left till a hydraulic survey is made and the different depths under the bridge are measured.

All piles have a concentrated load of the same value

$$P = V \text{ of } l_1 + V \text{ of } l_2$$

$$P = 15.6 + 27.4 = 43 \text{ kips.}$$

The pile is designed exactly like a spirally reinforced concrete column under a concentrated load.

The way of construction is as follows:

A 12 in pipe with sharpened edge is driven till it reaches the bed rock. Water is pumped out and the steel skeleton is placed inside the pipe and then concrete is poured and vibrated. When the concrete dries the steel pipe is driven out.

It is desired to use for the piles concrete made with special cement of high rate of reaction and high chemical resistance to avoid chemical attack of sea water. Alumina cement may be used.

The beams will be poured monolithitically with the floors but not with the piles. The longitudinal reinforcement of the piles will extend two feet for embedment in the beams.

Sample Design of a Pile:

Diameter $D = 12$ in (standard for all piles)

Length $L = 20$ ft (assumed for one pile)

$L/D = 20/1 = 20 > 10$ ...... long column.

P (allowable) $= P\left[1.3 - 0.03(20)\right] = 0.7P$

$P = 43/0.7 = 62$ kips.

Longitudinal Steel:

$P = 0.225 f'c\,Ag + fsAs$

$62 = 0.225\,(2)\,113 + 20\,As.$

$As = 0.55$ sq. in.

$Pg = 0.55/113 < 1$ percent .... use 1% steel.

$As = 1.13$ sq. in.

Use 6 No. 4 bars

Spiral Reinforcement: Fig. VII - 6.

$$Ac = \frac{\pi d^2}{4} = \frac{(12-3)^2\pi}{4} = 63.5 \text{ sq. in.}$$

$$P' = 0.45\left(\frac{Ag}{Ac} - 1\right)\frac{f'c}{f's}$$

$$= 0.45\left(\frac{113}{63.5} - 1\right)\frac{2}{20} = 0.035$$

Volume of spiral in 1-ft. length of pile.

$V = 12\,AcP' = 12\,(63.5)0.035 = 26.6\ \text{in}^3.$

Spacing: max. spacing $= 3$ in.

$$\frac{d}{6} = \frac{9}{6} = 1.5 \text{ in. governs.}$$

$$\pi\,(8.75) \times \frac{12}{1.5} = 220 \text{ in. per foot.}$$

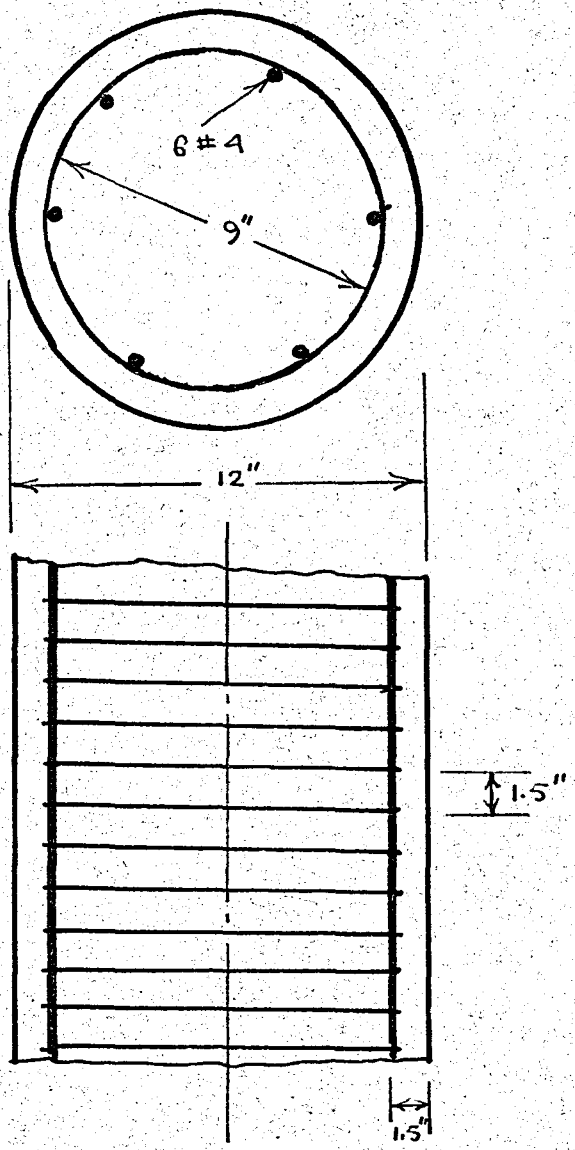


Fig. VII_6 Sample Design of a Pile _ Steel Reinforcement.

$$L = \frac{220}{12} \times 20 = 366 \text{ ft.}$$

$$\text{Cross-section} = \frac{26.6}{220} = 0.12 \text{ sq. in.}$$

Use No.3 bars.

Bill of Materials of the Bridge

Steel & Concrete

- Per part of five parts -

Member	No. of Parts	Bars	Shape	Length ft.	Wt./ft. lbs.	Total Wt. lbs.	Total Concrete ft^3
Slab	1	76 No.2	Straight	11	0.167	140	254
	1	76 No.2	"	11	0.167	140	
	1	20 No.2	"	76	0.167	260	
Beam L	4	4 No.9	"	18	3.40	980	136
	4	4 No.9	"	10	3.40	545	
Beam L	2	6 No.9	"	2 X 12	3.40	980	168
	2	2 No.9	Bent	46	3.40	625	
	2	2 No.9	Straight	46	3.40	625	
	2	15 No.3	Stirrup	2 X 6	0.376	136	
Pile	4	6 No.4	Straight	22	0.668	352	63
	4	1 No.4	Spiral	366	0.668	980	
					Total	5763	621

Total Steel = 5 X 5763 = 28,815 lbs. = 28.82 kips.

Total concrete = 5 X 621 = 3105 ft^3.

Chapter VIII

GENERAL BILL OF MATERIALS

AND COST ESTIMATION

This is the part of the project that can not be even roughly estimated for the time being.

The amounts of steel, cement, aggregate, and other building materials can be calculated accurately.

Other things which constitute a major part of the costs of the different items of the project cannot even be estimated at this time.

One of the reasons is that in Lebanon as well as in may other countries prices are always fluctuating. Another very important reason is the peculiarity of the location of the structure. Transportation of building material might be quite costly. The extension of the pulley system from the share to the rock might add considerably to the transportation expenses.

The cost of the land on the shore including the parking place can only roughly be estimated especially the prices of land in that area of the city are rising at a fast rate.

The government might ask for a big amount of money before giving permission for the project bo be worked out on the rock.

A rough estimation of overall costs is avoided here because it might be very far from reality.

The only things that can be estimated here are the amounts of building materials of steel and concrete. The amount of rock blasting is estimated too.

Just to give an idea of the costs of the materials required the prices in Turkey are employed for the purpose. The project itself might cost several times the cost of the structural materials that are needed.

Steel

Bars

fs = 20 ksi

Type	Weight lbs.	Cost per lbs. Turkey T.L.	Cost per lbs. Lebanon L.L.	Total Cost Turkey T.L.	Total Cost Lebanon L.L
No.2	1244				
No.3	25897				
No.4	1332				
No.5	5759				
No.6	887				
No.7	8220				
No.9	24273				
Total	67612	0.90		61,000	

fs = 30 ksi

No.2	8960
No.3	3720
No.4	32780
No.5	33752
No.6	3060

Steel Bars, Con't.

Type	Weight lbs.	Cost per lb. Turkey T.L.	Cost per lb. Lebanon L.L.	Total Cost Turkey T.L.	Total Cost Lebanon L.L.
No.9	7220				
No.10	173,392				
Total	262,884	0.9		237,000	

Other Steel

Type	Wt. lbs	Cost per lbs. Turkey T.L.	Cost per lbs. Lebanon L.L.	Total Cost. Turkey T.L.	Total Cost. Lebah L.L
t= 3/4" Plates	33000				
14 WF 287	124000				
Total	157000	0.9		142,000	

Concrete

f'c psi	Volume ft^3.	Volume $m^3 = \frac{ft^3}{36.5}$	Cost per m Turkey T.L.	Cost per m Lebanon L.L.	Total Cost Turkey T.L.	Total Cost Leba'n IL
2000	11016	302	75		22,6 00	
3000	21504	590	75		44500	
Total		892	75		71100	

Rock Excavation

Volume ft^3	Volume m^3	Cost per m Turkey T.L.	Cost per m Lebanon L.L.	Total Cost Turkey T.L.	Total Cost Leba'n L
103,680	2820	95		269,000	

Approx:

Total Cost of Concrete, steel, and Rock excavation =

61,000 + 237,000 + 142,000 + 71,000 + 269,000 =

780,000 T.L.

Chapter IX

PROCEDURE AND METHODS OF CONSTRUCTION

Steps in Construction

Opening of a new Road:

A road, starting at the main street and ending at the place where the shore building is going to be constructed, is going to be opened and leveled.

The road will be opened and leveled before the construction starts. This will serve for transporting the materials of construction to the site. At a later stage, and after the construction is over, the road will be paved to serve for the casino.

In this construction compressors and bulldozers will be used to cut the extra material and fill it where it is needed.

Construction of the Shore Building:

Rock blasting is going to be the first step before the construction.

The frames and slabs of the building will be constructed and at least half of the concret block walls will be built.

This part of the building will be used as a temporary construction office and temporary stores for construction materials such as cement, steel, floor tiles, pipes, and the sort.

Construction of the Bridge:

Before the construction takes place, a hydraulic survey for measuring water depth at different points is going to be made. The sea bed should be investigated for foundation purposes under each pile to be constructed.

A revised design of the piles will be made taking into consideration the length of the pile and the foundation conditions.

The method of pile construction shown in Chapter VII will be followed.

For driving the piles a temporary wooden raft may be constructed and pulled by a motor boat.

Beams and floor for each section will be constructed as soon as the supporting four piles are ready.

Construction of the Stairway:

As soon as the construction of the bridge is finished the

the rock can be blasted for preparing the required space for the stairway. The blasting is going to be done mainly by employing the necessary number of compressors. The power generators will be placed on the shore and the rubber pipes of the compressors will be extended over the bridge.

The construction of the stairway will start whenever possible. Further blasting of rock at higher levels will take plae gradually as the structure proceeds. This is done so as to make ~~the~~ rock to be blasted within an easy reach.

The blasted rock will be transported over the bridge and dumped somewhere else. A reasonable amount of rock can be dumped in the water along the sides of the bridge.

Construction of the Casino:

When the top of the rock is easily reached by the stairs a transport cable is going to be extended between the top of the rock and the shore. The shortest distance between the Rock and the shore is preferable to minimize transportation costs. The approximate distance as shown in Fig.IX-1 is about two hundred feet.

The rubber pipes of the compressors will be extended from the shore to be rock along the cable.

The pulley will have a capacity to transport material and

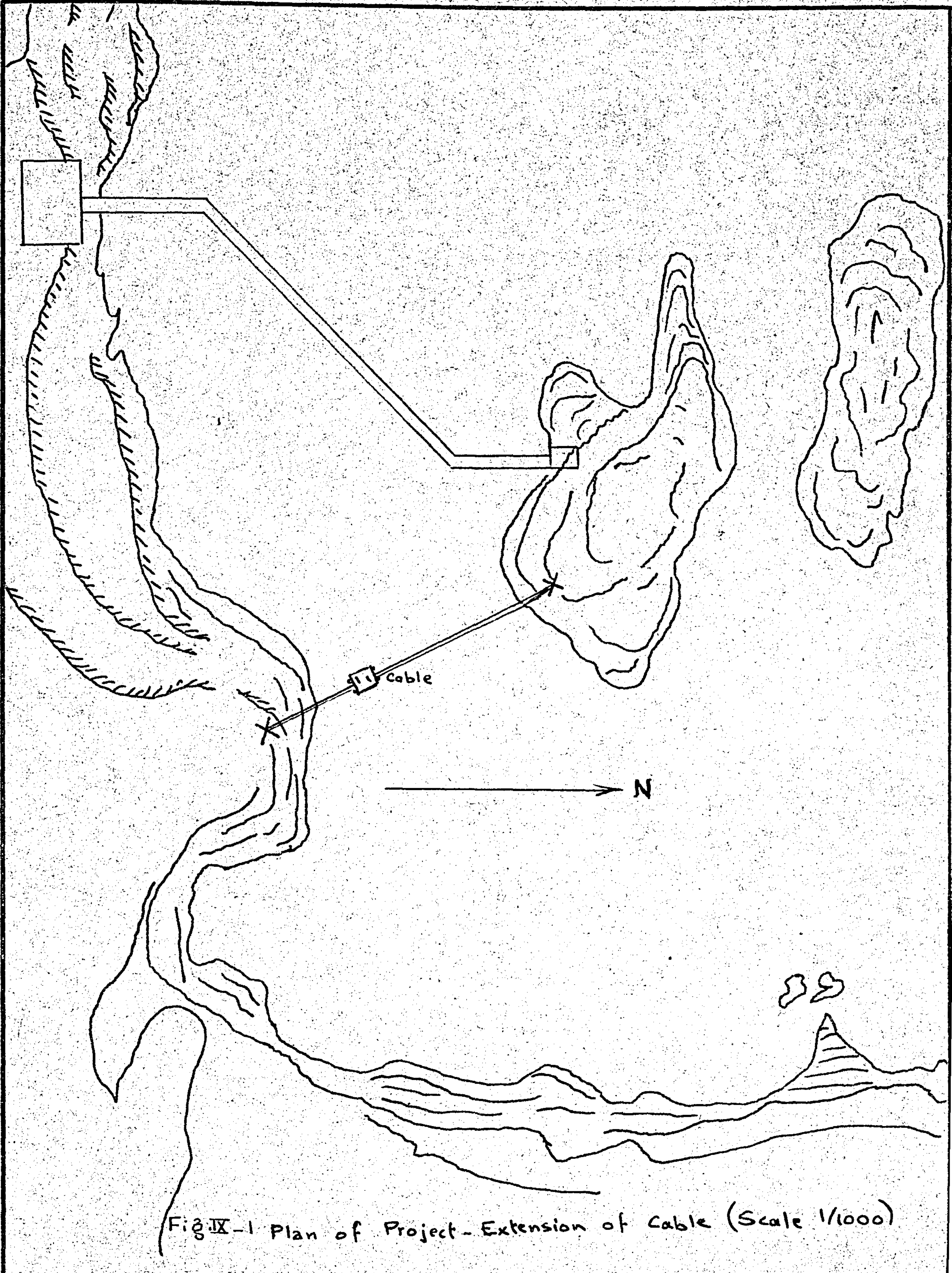


Fig. IX-1 Plan of Project - Extension of Cable (Scale 1/1000) equipment to the rock and to transport blasted rock back to the shore. This capacity should not be less than two thousand ponds.

The pulley will transport some of the blasted rock to the shore, while the rest of the rock will be dumped in the sea at different locations around the Rock.

The position of the pulley's end on the Rock may be changed when necessaey as the structure proceeds.

Illustrative points

* All concrete is going to be mixed by concrete mixers.
* The concrete for the stariway will be transported over the bridge and elevated by pulley to higher levels of the construction
* The conrete for the casino will be mixed on the shore near the pulley's end point and will be transported by the cable.
* All concrete should be compacted and vibrated by vibrators.
* All cracks in the rock will be fillwd with concrete as soon as the rock blasting is finished.
* Water pipe installations should be extended to the rock when the stairway structure is finished.

* Side openings in the rock for sanitary pipes, ventilation fans, or for other purposes such as doorway from the stairway to the basement of the casino can be made as the construction proceeds.

* Bending of steel bars and welding of steel columns will take place on the rock.

* Different steps in construction on the rock can be going on at the same time.

* Unless specified ACI Code should be followed.

* For any further doubtful or unillustrated points - construction the engineer or the authorized person in charge should be consulated.

Chapter X

GENERAL CONCLUSION

The idea to be fulfilled several factors should be considered. The Government's permission, the necessary capital, and the suitable design are the three main factors.

If these three points are fulfilled the structure can take place and if will no doubt be a success from all points of view.

People can come to the casino to dine at noon or in the evening. They can come to the terrace to enjoy watching the sun rise in the morning or the sun set in the afternoon. Others will come at night and watch the program at the night club.

The location of the casino is so unique, the views are so beautiful, so that it encourages pople including tourists to go there.

The warm weather in winter season and the cold sea breeze in hot summer nights of Beirut will attract people to the Pigeon Rock Casino the whole year round.

The isolated location of the casino will become a relieve to the people from the noisy life of the city.

The Lebanese government realizing the importance of tourism as a major source of national income encourages and supports constructive ideas.

The government most likely will not object the project if the idea and the design are convincing enough to lead to a higher step in the development of Tourism in Lebanon.

What will be left is the necessary capital !

Bibliography

1. Urquhart, L.C. , o'Rourke, C.E. , and Winter, G.
"Design of Concrete Structures"
Sixth ed. 1958 McGraw-Hill Book Company, Inc.

2. Seely F.B. and Smith, J.O.
"Advanced Mechanics of Materials"
2 nd ed. 8th printing Oct. 1963
John Wiley & Sons, Inc.

3. Michalos, J.
"Theory of Structural Analysis and Design"
Copyright c 1958 The Ronald Press Company.

4. Troxell, G.E. and Davis H.E.
"Composition and Properties of Concrete"
Copyright c 1956 McGraw-Hill Book Company Inc.

5. French, T.E. and Svensen, C.L.
Mechanical Drawing.
Copyright 1957 McGraw-Hill Book Company Inc.

6. Singer, Ferdinand L.
"Strength of Materials"
Copyright 1951 , Harper & Brothers, Publishers, N.Y.

7. Ries, H. and Watson, T.L.
"Engineering Geology"
5th ed., 9th printing, Sept. 1953.
John Wiley & Sons Inc. London.

8. Schultz, J.R. and Cleaves, A.B.
"Geology in Engineering"
3rd printing, Feb. 1958.
John Wiley & Sons Inc. N.Y.

9. American Institute of Steel Construction Manual
Copyright 1947. 5th Ed. , 29th printing 1962.
American Institute of Steel Constructions N.Y.

10. ACI Building Code.
ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318 - 63).
ACI publication. Michigan.

11. Reinforced Concrete Design Handbook. of the American Concrete Institute. Reported by Committee 317.
Published by ACI.
Portland Cement Association.
Concrete Reinforcing Steel Institute.
Rail Steel Bar Association.
First Edition.

12. Fisher, G. and Zia, P.
"Review of Code Requirement for Torsion Design"
Published by ACI Journal . Jan. 1964.

13. National Advertising System.
"Tourist & Hotel Guide for Lebanon".
Distributed by M.E.A.
Catholic Press - Beirut . 1962.

14. "Lebanon Fortnightly".
November 1963.
Published by M.E.A.

15. Yapi ve Ekonomisi.
Reşat Uluğ
Guzel Istanbul Matbaasi.
Ankara 1964.