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FACULTY OF ENGINEERING – AMERICAN UNIVERSITY OF BEIRUT

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S E N I O R   P R O J E C T

A BRIDGE ON NAHR ABU-ALI ON THE MAIN BOULEVARD OF TRIPOLI

Designed by

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Candidate to the degree of

B.C.E.

Submitted to

Faculty Of Engineering

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F O R W O R D

The design of a bridge was chosen because of the supplementary knowledge it would give the designer, who, being familiar with road construction in general, lacks a thorough perspective of all the problems to be met in designing major highways. In addition, it is purely structural.

The understanding of the moving loads and the arrangements of such loads to give the maximum moments and shears at any location along the span, are principal steps towards the design of railroad bridges and parking garages.

G.E.N.

A C K N O W L E D G M E N T

The candidate wishes to tender his warmest thanks to professors Jack Nasr and Raja Elia for their very useful and constructive suggestions and under whose supervision the design was carried out.

Indebtedness is felt to professors K. Yeramian and Fuad Khazen for their valuable help and important advice in various problems encountered in the design.

Also thanks are due to Mr. William Harold of the point 1V for the hydraulic data of the river.

Finally, mention should be made that it was professor Nicola Menessa who proposed to the candidate the location of the bridge.

G.E.N.

C O N T E N T S

I N T R O D U C T I O N

The bridge carries the main road from southern Tripoli to northern Syria. It falls on the continuation of the existing Boulevard that joins Tripoli with a 2 km. straight strip to the main Tripoli - Beirut road. It is 500 meters to the west of the existing 10 meters bridge of Bab Al - Tibbany, and is located to the west of the city.

The location of the Boulevard has been designed by the municipality of Tripoli in 1937, and in 1947, only a part was executed. At the time being, it is hoped that the remaining part will be finished within the coming five years.

Nahr Abu - Ali flows approximately from the east to the south and forms a natural barrier between the north and south sides of the town. At different distances upstream and within the city, there are three crossings at three bridges built very long ago. Only one of them serves for traffic purposes and the other two are meant for pedestrians. All these three bridges had been functioning allright until December 17, 1955, when a very high flood, discussed elsewhere, occured and destroyed the two pedestrians' bridges.

In the following design, the calculations have been checked as deligently as possible. Even that, it is too much to be hoped that the work is devoid of errors. The candidate will be very thankful for bringing his attention to any slips that may have occured in the design.

In case this study is considered as complete, special attention is called for the assumptions which were necessary to complete the design of the substructure.

G.E.N.

CHAPTER I

GENERAL

Analysis Of River Flow And Hydraulic Data.- Until Dec. 17, 1955, Nahr Abu-Ali was most of the time flowing uniformly with very light floods every ten years or so. Consequently, not much interest had been given to it, and were it not of the average monthly flow records measured by Kadisha Electric Company near Kosba, approximately 35 km. upstream, there would have been much doubt of knowing anything about its flow. A rough approximation, however, could have been obtained from old people living near the area or by studying the marks of the highest floods that ever existed on an existing bridge or along the banks of the river.

In December 17, 1955, the highest flood that Tripoli ever witnessed occured. " It is a very extraordinary flood " engineers said about it. Ksara observatory wrote, " The meteorological cause for the Tripoli flood is the meeting of hot air masses coming from the south-west (Africa), with the cold air masses coming from the north-west (Anatolia.) They also wrote, " The exceptional gravity of this phenomena seems due on one hand, to the intensity of hot winds of SW bringing a quantity of precipitable steam abundantly renewed, and, on the other, to the topography of the region of Tripoli which form a basin where a mass of cold air coming from the north west could be locked. "

Following is a map prepared by Ksara observatory showing the precipitation contours of the two hours duration flood.

Now, three questions present themselves :-

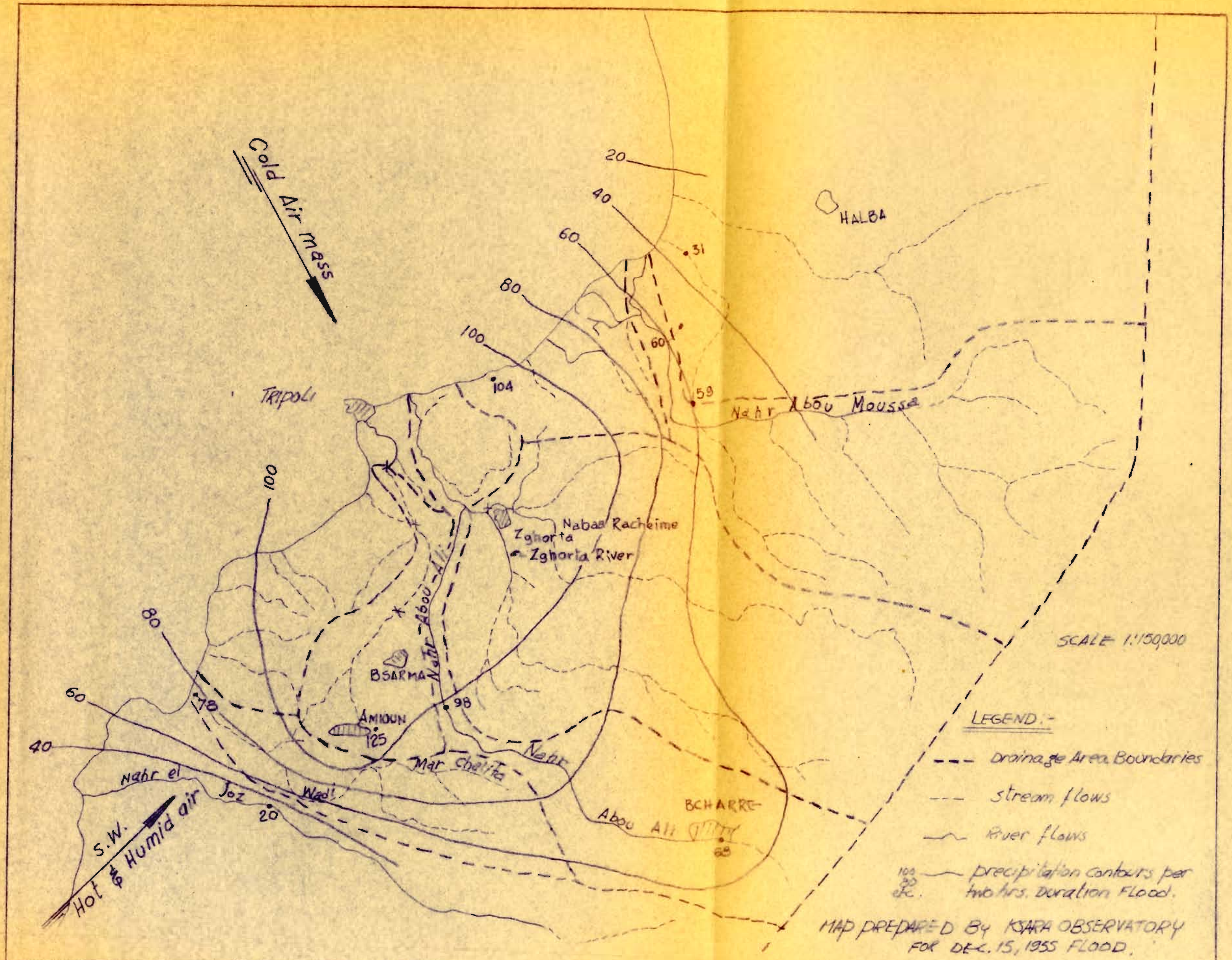
1) Do we design the bridge to take care of the ordinary floods ?

2) Do we have to count for the highest flood that took place in 1955 and design the bridge accordingly ? Or

3) Do we have to design it for a probable higher flood ?

Question 3, was raised because at any time, the hot and cold masses of air might meet somewhere else, shifting the 100 mm. precipitation contour (refer to map) a little upwards causing more precipitation on the water-shed area of Abu-Ali and thus increasing the probability of a higher flood. This consideration cannot at the time being exactly answered, as it needs thorough study and evaluation of the direction of previous storms and the topography of the whole district as well. The 100 mm. contour might be shifted way up to the cold region of the mountains and snow will precipitate instead, which, evidently will take more time to melt and hence there will be no flood.

In answer to question 1, the designer does not recommend a positive answer. Eventhough the frequency of the 1955 flood is one hundred years, there is the probability of having a similar flood the following month, probably two in one year, or none in one hundred years. To construct a bridge and have the probability of having it damaged the


Cold Air mass
20
40
60
80
100
HALBA
31
60
59
104
Tripoli
Nahr Abou Moussa
Nabaa Racheime
Zghorta
Zghorta River
100
80
Nahr Abou Ali
BSARMA
98
78
AMIOUN
125
60
40
Mar Chelita
Nahr
Nahr el Joz
Wadi
20
BCHARRE
Abou Ali
68
S.W.
Hot & Humid air
SCALE 1:150,000
LEGEND:-
Drainage Area Boundaries
stream flows
River flows
100 80 etc.
precipitation contours per two hrs. Duration Flood.
MAP PREPARED BY KSARA OBSERVATORY FOR DEC. 15, 1955 FLOOD.

following month or the following year of its construction, is as costly as constructing a bridge to last its whole life. After all, the question of how much money is available plus how badly the bridge is needed, is the decesive element in limiting its opening.

The designer henceforth assumes that the bridge shall be designed to take care of the maximum flood of 1955, the data of which was obtained from the point lV office in Ras Beirut, and was worked out by the same people few days after the occurance of the flood. Fig. 1. illustrates two sections taken at two ideal locations along the river in the city of Tripoli. As it may be noticed, a relatively high value of "n", the coefficient of roughness, has been taken. This is due to the many orange trees on the waterway. A better approach to the exact flow calculation would have been reached by dividing the cross section into two parts and taking two different values of "n", one being for the orange trees section and the other for the usual river bed.

DATA

Average slope of water surface = 0.00544

" cross sectional area = 2650 sq.ft.

" wetted perimeter = 231 ft.

$\therefore$ Hydraulic radius = 11.48 ft.

Assuming "n" = 0.05, and using Manning formula, we have,

$$V = \frac{1.486}{n} R^{2/3} S^{\frac{1}{2}}$$

$$\text{or } V = \frac{1.486}{0.05} (11.84)^{2/3} (0.00544)^{\frac{1}{2}}$$

$$= 11.18 \text{ '/sec}$$

$$\text{and } Q = 2650 \times 11.18$$

$$= 29{,}650 \text{ cfs} = 840 \text{ m}^3\text{/sec}$$

Assuming to have the same slope of the river bed, the required cross sectional area would be 2650 sq.ft.

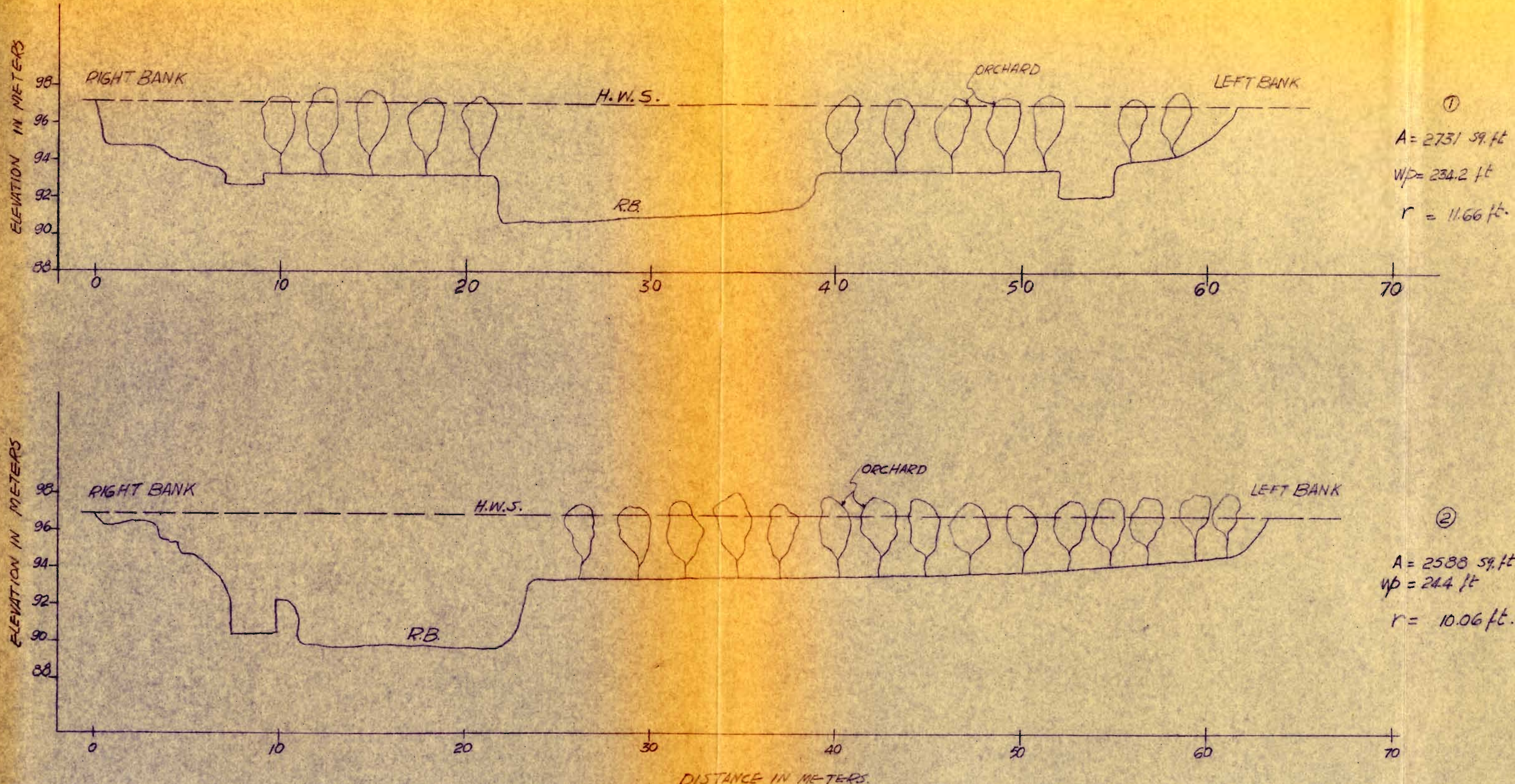


FIG. 1 TWO CROSS SECTIONS AT NAHR ABU-ALI SHOWING H.W.S. ATTAINED DURING DEC. 17, 1955 FLOOD.

General Considerations.- The 1955 flood has damaged a great deal of structures that were built alongside the banks including two pedestrians' crossings in the heart of the city. Becoming the interest of individuals, organizations and societies, engineers were appointed to carry on the future planning and design along the banks all through the city. Mr. Afif Silman, secretary of the engineers syndicate in Lebanon, was responsible for the new planning alongside the banks and for the reconstruction of the two bridges that were done with during the flood. It has been concluded, after an interview with him, that no final decision had been taken on future planning and designing of bridges, however, it has been proposed that a clear waterway width of 25 meters to 30 meters shall be provided, and that future building structures shall be 30 meters away from each side of the river in order to allow for a Boulevard to run parallel to the river on its two sides. " In case of any probable higher flood or a similar one, " he said, " such a cross section, including both ways of Boulevards, would be able to consume all the flow." The Boulevards shall have convenient slopes towards the river.

Design Considerations.- Following is a map furnished by the Municipality of Tripoli, showing the location of the Boulevard as designed in 1937. The yellow colored section represents the executed 2 km. straight strip that connects Tripoli with the main Tripoli-Beirut road. Because the designer was unable to get a profile of the center line of the Boulevard, he copied from the original maps, elevation points along the road, as marked, so that he may be able to decide on the elevation and slope of the bridge and consequently to design drainage facilities. The width of the river at the location of the bridge is 22 meters and the river bed is 2.5 meters,obtained by reconnaissance, below the natural soil surface. To provide for a clear opening of 2650 sq.ft., the following alternatives arise;

1) To design the bridge as one simply supported span,

2) As an arch, or

3) As a multispan bridge.

The first alternative is not recommended because:-

a. It will carry the Boulevard into a camel-like back in that location and consequently a very poor sight distance will result in both the vertical and horizontal directions.

b. It will turn out the Boulevard into a usual second or 3d. class highway.

c. It will dissatisfy the conditions for future extension of the city at that location, and

d. In case of any future construction of Boulevards alongsides the river, it will be impossible to execute such a project.

The second alternative shall not be discussed firstly, because it is not within the scope of this report and secondly, there is no proper data of the foundation which is the first factor to be considered in arch designs. Therefore, the bridge shall be designed as a multispan bridge with a level section all through except of cambering to get rid of precipitable water to the sides and in turn to the embankments.

There are three types of multispan bridges:-

1. Simply supported,
2. Contimous, and
3. Cantilever.

Weighing the advantages and disadvantages of each type, the designer decided to work out the bridge as a cantilever. The advantages of the cantilever design in comparison with the simply supported design are:-

a. Cantilever design requires less concrete, steel, and formwork for the main girders.

b. At each pier the cantilever design uses one beqring per line of girders, and, therefore, requires a smaller width of piers than the simply supported girders in which two bearings in a line are needed at each pier.

c. The reactions of the piers are always central.

The only disadvantage is that it requires more skill on the part of the designer and the arrangement ofthe steel is somewhat more complicated.

In comparison with continuous girder designs, the cantilever design has the advantage that they are statically determinate, and that, therefore the possibility of bad effects of unequal settlement of foundation is less. They also can be of longer spans. In this concern, it is assumed that no proper foundations for continuous spans is available at the site.

Referring again to the following map, it can be seen that the roadway width is 22 meters including a median strip of 4 meters, thus leaving a clear width of road of 9 meters, or 3 lanes - 3 meters per lane in each direction. Coming up to the bridge, it is decided that the median strip shall be continued along the bridge. This is for more safety of the people and vehicles, and for a better illumination of the roadway. However, the design ~~of the~~ of the slab and girders underneath shall be kept the same as those which are directly under the moving traffic.

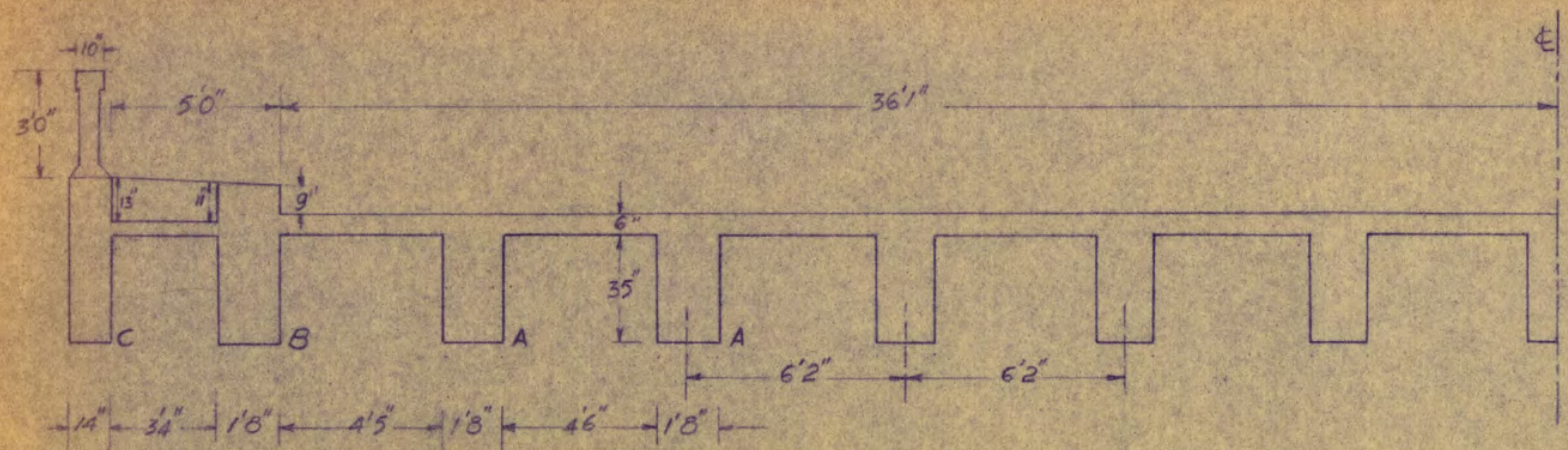
Due to its wideness, the bridge shall be designed as a Deck-Girder bridge with dimensions as shown in Fig. 2. with the following concrete and steel properties.

f'_c = 2,500 psi

f_c = 1,125 psi

f_s = 20,000 psi

These properties shall be used in both the superstructure and the substructure.



Section 1-1

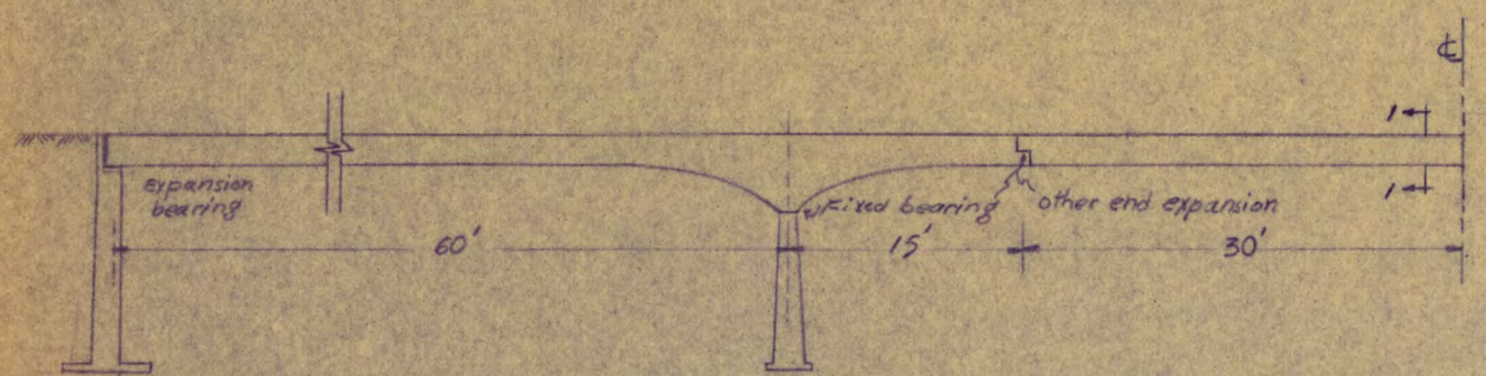
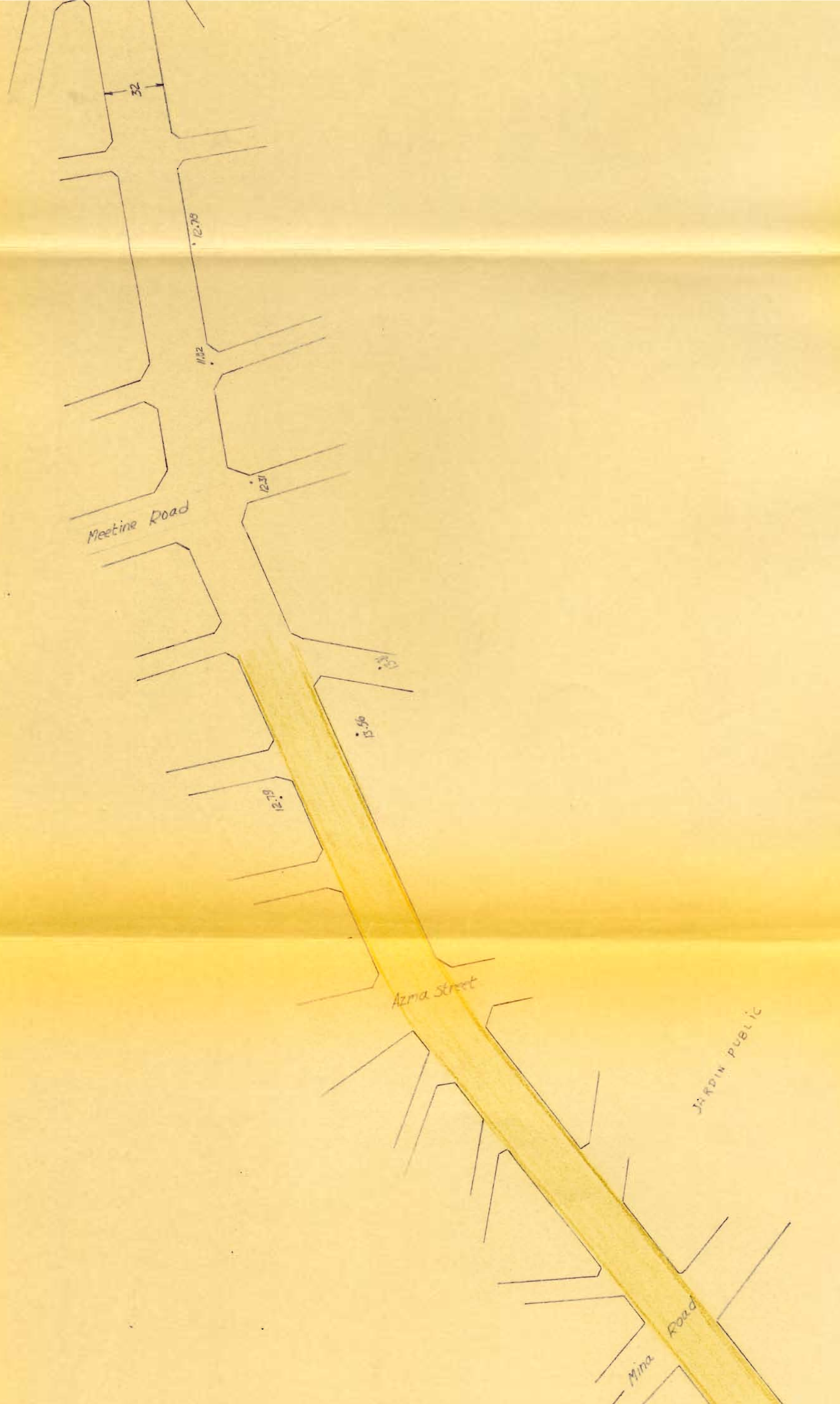


Fig. 2


32
12.78
11.32
12.31
Meetine Road
13.56
12.78
Azma Street
JARDIN PUBLIC
Mina Road

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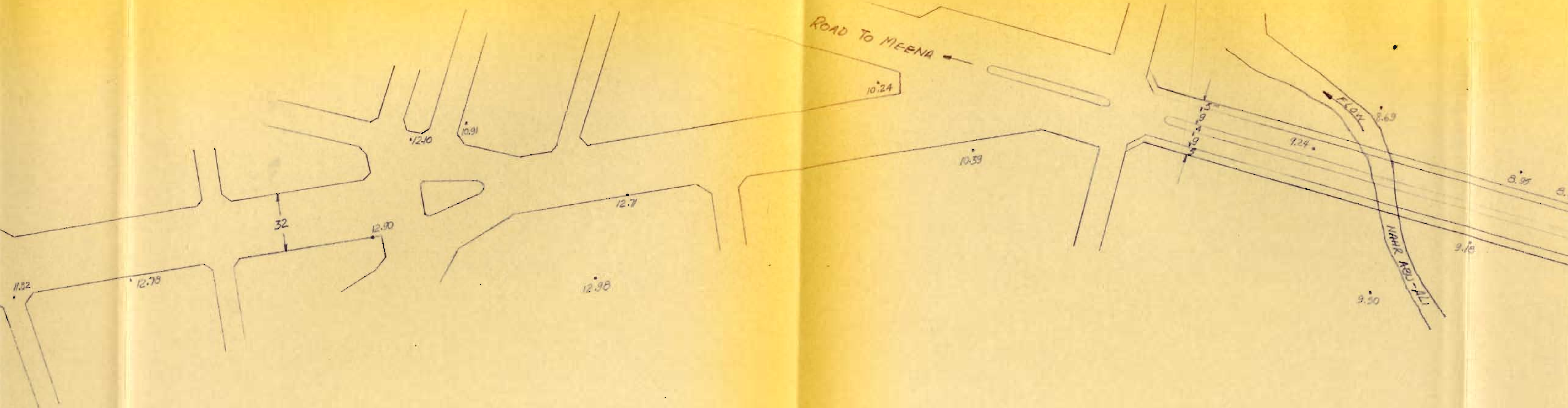

NOTE: BOULEVARD HAS A 10-meters
RIGHT OF WAY ON EACH
DIRECTION. NOT SHOWN HERE

" MU
"MAP (
NORTH

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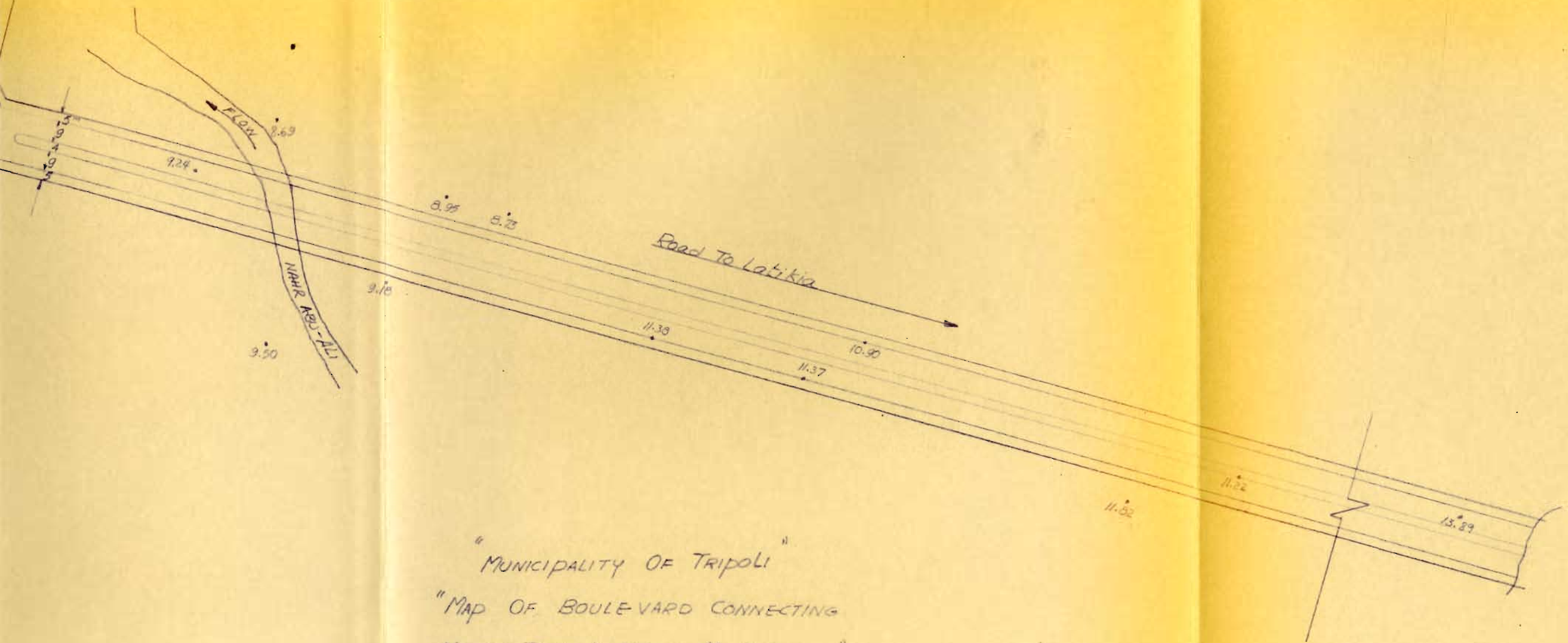

"MUNICIPALITY OF TRIPOLI"

"MAP OF BOULEVARD CONNECTING NORTH TRIPOLI WITH AL-KOBBY."

"Scale 1:2000"

CHAPTER II

SPECIFICATIONS

All the following specifications have been taken from the Standard Specifications for Highway Bridges by the American Association of State Highway Officials, unless otherwise stated. The following, however, are only those which were used in the design of this project.

1) Determination of waterway Area.- For the determination of the waterway area to be provided by any drainage structure, a careful study shall be made of local conditions, including flood height, flow and frequency, size and performance of other openings in the vicinity carrying the same stream, climatic conditions, characteristics of the channel and of the watershed area, available rainfall records and any other information pertinent to the problem and likely to effect the safety or economy of the structure.

The waterway provided shall be sufficient to insure the discharge of flood water without undue back water head and at a velocity which will not increase the erosive action of the stream to such an extent as to endanger the structure.

When it is necessary to restrict the waterway to such an extent that the stream will be discharged at erosive velocities, deep foundations should be provided.

2) Channel Openings.- The clear vertical distance between the highest flood and the superstructure, shall be sufficient for the passage, without damage to the structure, of the largest drift which may be expected.

3) Pier Spacing and Location.- Piers shall be located so as to afford the minimum restrictions of the waterway. They shall be placed as nearly parallel with the direction of the stream current as is practicable, due consideration being given to avoid deflections of the current as might prove destructive to the foundations of the structure or to the adjacent stream banks.

4) Width Of Roadway And Sidewalk.- The width of roadway shall be the clear width measured at right angles, to the longitudinal centerline of the bridge between the bottoms of curbs.

The width of the side walk shall be the clear width measured at right angles to the longitudinal centerline of the bridge of the extreme inside portions of the hand-rail to top of the face of curb.

5) Curbs.- In rural areas, curbs are usually not less than 9 inches above the adjacent finished surface of the roadway. That portion of a curb more than 10 inches above the roadway surface shall be stepped back or sloped back so that no part of vehicles except the tires may come in contact with it.

6) Railings.- Railing along each side of the bridge shall be provided for the protection of traffic. Consideration shall be given to the aesthetic features of the railing to obtain proper proportioning of its various members and harmony with the structure as a whole.

Side walk railings shall have a minimum height above the surface of sidewalk of 3 feet less one half the horizantal width of the top rail, but in no case shall the height be less than 2 feet 6 inches. Clear openings shall be proportioned with due regard for safety of persons using the structure.

7) Roadway Drainage.- The transverse drainage of roadways shall be secured by means of a suitable crown in the roadway surface and longitudinal drainage by camber or gradient.

8) Floor Surfaces.- All bridge floors shall have non-skid characteristics.

9) Underpass Vertical Clearance.- A vertical clearance for an underpass shall not be less than 14 feet.

10) Loads.- Structures shall be proportioned for the following loads and forces :-

a. Dead Load

b. Live Load

c. Impact

d. Wind Loads.

11) Dead Load.- The dead load shall consist of the weight of the structure complete, including the wearing surface, roadway, sidewalks, pipes, conduits, and other utility services.

12) Live Loads.- The live load shall consist of the weight of the applied movng load of vehicles and pedestrians.

13) Highway Loadings.-

a. General.- The highway live loading on the roadway of bridges shall consist of standard trucks of H loadings or H-S loadings.

b. H-S Loading.- The H-S loadings are illustrated in fig. 3. (H 20 - S 16 - 44 has been used in this design) They consist of tractor truck with semi trailer. The H-S loadings are designated by the letter H followed by number indicating the gross weight in tons of the tractor truck and the letter S followed by the gross weight in tons of the single axle of the semi-trailer.

14) Spacing Between Trucks.- The spacing between trucks along the bridge shall be 19 feet on centers of the rear wheel of the front truck to the front wheel of the following similar truck. (page 11 of Taylor Reinforced Concrete Bridges) For arrangement see Fig. 4.

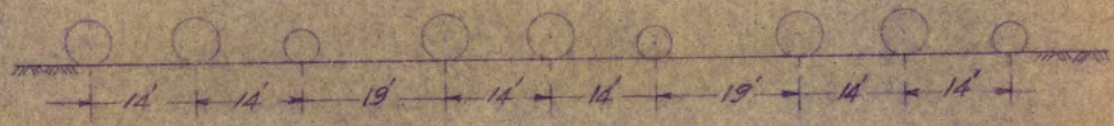

Fig. 4 Spacing of H20-S16 Trucks Along Roadway

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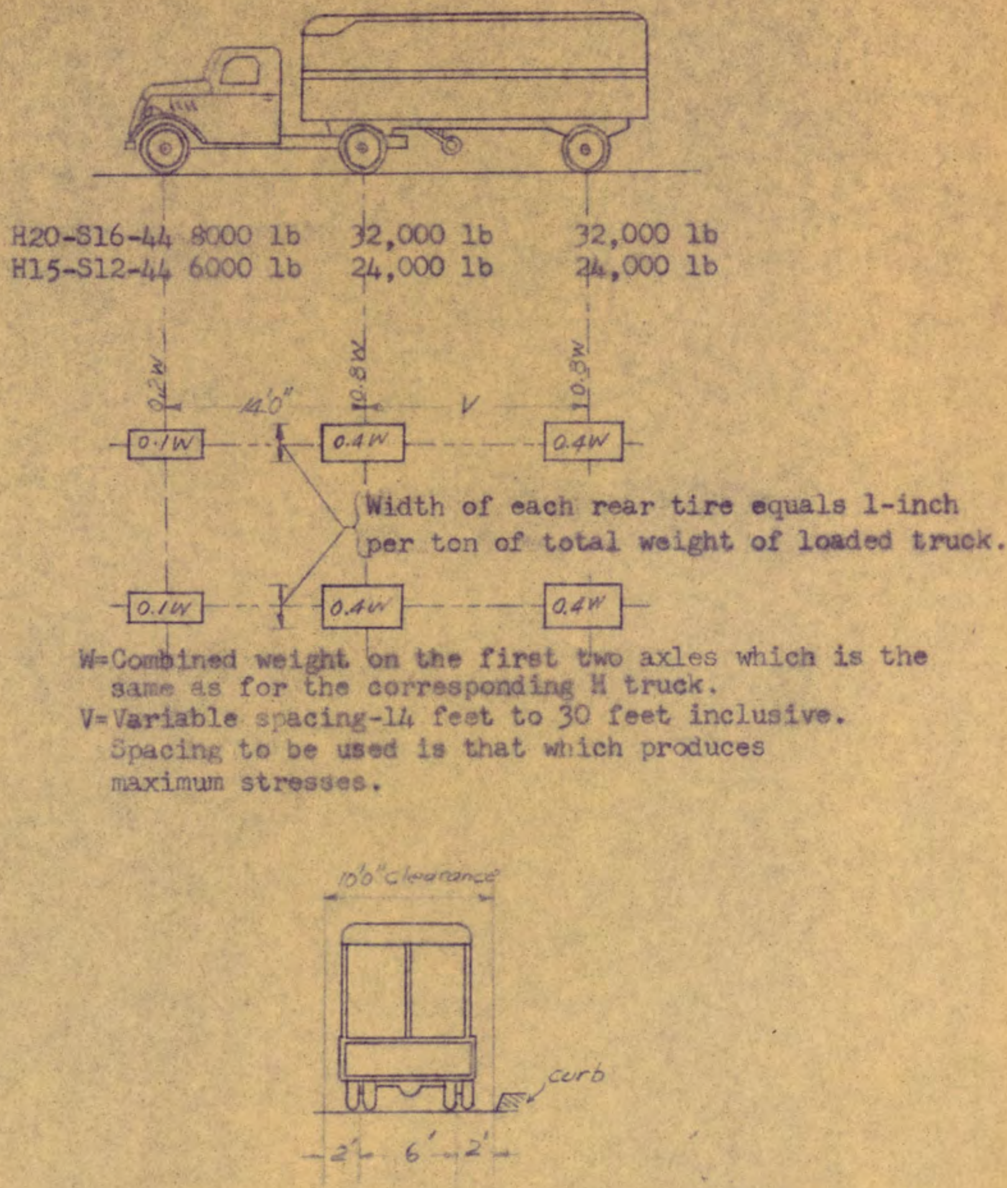

Fig. 3. Standard H-S truck loading.

In the design of floors (concrete slabs, steel grid floors, and timber floors) for H20 or H20-S16 loading, one axle load of 24,000 lb or two axle loads of 16,000 lb each, spaced four ft apart, may be used, whichever produces the greater stress, instead of the 32,000 lb axle shown.

For slab design the center line of wheel shall be assumed to be 1 ft from face of curb.

15) Reduction In Load Intensity.- Where maximum stresses are produced in any member by loading any number of traffic lanes simaltaneously, the following percentages of the resultant live load stress shall be used in view of improbable coincident maximum loading :-

Three lanes = 90 %

Over three lanes = 75 %

16) Sidewalk Loading.- Side walk floors, stringers and their immediate supports, shall be designed for a live load of 85 psf of sidewalk area.

17) Sidewalk Railings.- Side walk railings shall be designed to resist a lateral horizontal force of 300 pounds per linear foot together with a simaltaneous vertical force of 100 pounds per linear foot applied at the top of the railing.

18) Impact.- Live load stresses produced by H-S loadings shall be increased by an increment expressed as a function of live load stress, and shall be determined by the following formula :-

$$I = \frac{50}{L + 125}$$ in which,

I = Impact fraction (Maximum 30 %)

L = Loaded length of the span.

18) Wind Loads.- A transverse wind force of 50 pounds per square foot shall be considered to act on 1 1/2 times the area of the structure as seen in elevation.

19) Thermal Forces.- Provision shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be figured from an assumed temperature at the time of erection.

20) Force Of Stream Current.- All piers and other portions of structures which are subject to the force of flowing water,floating ice or drift shall be designed to resist the masimum stresses induced thereby.

Effect of flowing water on pier :-

P = KV , where,

P = Pressure in pounds per square foot.

V = Velocity of water in feet per second.

K = A constant, being 1 1/3 for square ends, 1/2 for angle ends where the angle is 30 degrees or less, and 2/3 for circular piers.

21) Distribution Of Loads And design Of Concrete Slab.-

a. Span Length.- For slabs monolothic with beams, S = Clear span.

b. Bending Moments.- The bending moment per foot width of slab shall be calculated by using the following formula for reinforcement perpendicular to traffic :-

$$M = \pm\ 0.2 \frac{P}{E} S \quad \text{in which,}$$

P = Load on one wheel of single axle = 12,000 lb. (In the design for H20 - S16, the single 24,000 lb. per axle governs for spans under 10.5 feet)

E = Width of slab over which a wheel load is distributed and is equal to 0.6S + 2.5 for S less than 7 ft. S being center distance of adjacent supporting beams.

c. Distribution Reinforcement.- Reinforcement shall be placed in all slabs transverse to the main steel reinforcement to provide fo lateral distribution of the concentrated live loads. The percentage of the main steel is $= 100 / (S)^{\frac{1}{2}}$. Maximum value 50 %
S,being the effective span of slab in feet.

d. Shear And Bond Stresses In Slab.- Slabs designed in accordance with the foregoing shall be considered satisfactory in bond and shear.

c. Cross Beams.- For cross beams built to provide rigidity for the structure, the cross sectional area of steel required shall be 0.3 percent of the effective cross section of the cross beam. (page 49 of Taylor Reinforced Concrete Bridges.)

22) Allowable Stresses.-

a. Flexure.-

Extreme fiber in compression $f_c = 0.4 f'_c$

Extreme fiber in tension,plain concrete, $= 0.03 f'_c$

b. Shear.-

Beams without web reinforcement $= 0.02\ f'_c$

" with " " $= 0.075\ f'_c$

CHAPTER III

DESIGN OF SIMPLY SUPPORTED MIDDLE SPAN

Adopt the cross section of the bridge shown in Fig. 2. In actual design the spacing of girders should be determined by comparative estimates of the costs of materials and of formwork for several arrangements of girders.

1) Design of Slab.- Assuming a 6" slab with a 2 1/2" wearing surface, the total weight due to dead load would be,

$$w = (6 + 2\ 1/2) \times 12 \times \frac{150}{144} = 105 \text{ lbs. per sq. ft.}$$

and the dead load moment is,

$$M_d = \frac{1}{10} w\, l^2 = \frac{1}{10} \times 105 \times (4.5)^2 \times 12$$

$$= 2,640 \text{ in-lb.}$$

A coefficient of $\frac{1}{10}$ for calculating dead positive and negative moments has been used due to abscence of defenite AASHO specifications. However, it could have been calcuted by the ACI code coefficients or by the moment distribution method.

For live load moment,

$$M_l = \pm 0.2 \frac{P}{E} \times S \ ; \quad P = 12,000 \text{ lbs.}$$

$$E = 0.6\ S + 2.5 = 5.2'$$

$$= \pm 0.2 \times \frac{12,000}{5.2} \times 4.5 \times 12 = 25,000 \text{ in-lb.}$$

Impact coefficient $I = \dfrac{50}{4.5 + 125} = 39\%$

$\therefore$ Use the maximum allowable of 30 %; the impact moment is therefore,

$$M_i = 25{,}000 \times 30\ \% = 7{,}500 \text{ in-lb.}$$

and the total moment, neglecting reduction, is,

$$M_t = 2{,}640 + 25{,}000 + 7{,}500 = 35{,}140 \text{ in-lb.}$$

hence, $d = \left(\dfrac{35{,}140}{12 \times 196}\right)^{\frac{1}{2}} = 3.9$ inches, say 4 in.

Allowing for 1 1/2 in. cover from center of bars, the total thickness would be 5 1/2 in. which is less than the assumed value. However, the total thickness shall be used and d = 4.5 in.

The steel required for slab reinforcement is,

$$A_s = \frac{M}{f_s\ j\ d} \quad \text{(Urquhart Design Of Concrete Structures)}$$

$$= \frac{35{,}140}{20{,}000 \times 0.866 \times 4.5} = 0.45 \text{ sq. in.}$$ This is furnished by No. 5 bars at 8 in on centers.

To satisfy the negative moment, the same number of steel with the same spacing should be furnished above the supports. This can be provided by bending up alternate bars at one fifth the span length from the support, and by supplementing the rest with No. 5 bars at 16 in. on centers. Theoratically, these supplements are not needed over the middle half of the span, however, they shall be

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provided all through, and any additional cost due to this extra provision, will be offbalanced by the savings in workmanship and wastages.

For transverse reinforcement, the area of steel required is $\sqrt{\frac{A_s}{S}} \times 100 = 0.22$ sq. in.

∴ Use No. 3 bars at 10 in. on centers in top and bottom.

Fig. 5 below shows a section for the reinforcement of the slab.

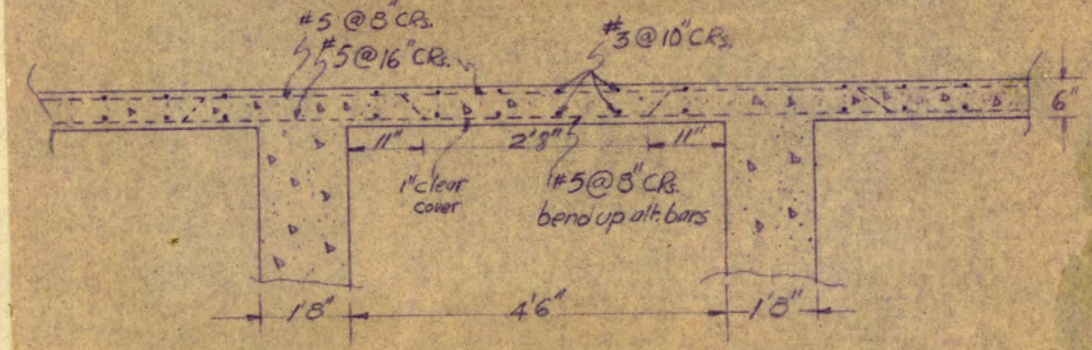

Fig. 5

2) Design Of Beams "A".- (Refer to Fig. 2.) Beams "A" are T beams with a flange width equal to the distance center to center of adjacent beams. Their required dimensions are either governed by the maximum shear or the maximum moment.

A) Calculations For the Maximum Shear.-

1. Dead Load Shear.- The dead load on the beam per foot length is,

from slab = 6.16 x 105 = 645 lbs.

from beam = 20 x 35 x 150/144 = 730 lbs.

TOTAL = 1,375 lbs.

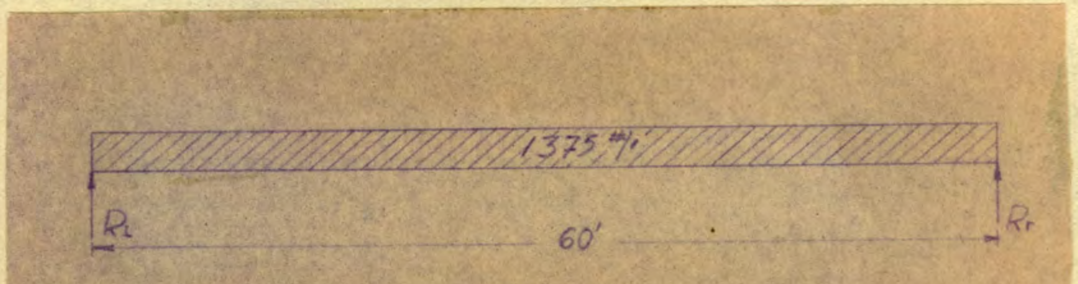

Fig. 6

∴ the end shear R_1 = 1,375 x 60/2 = 41,250 lbs.

2. Live Load Shear.- The maximum live load shear occurs with the truck-train positions as shown in Fig. 7. below.

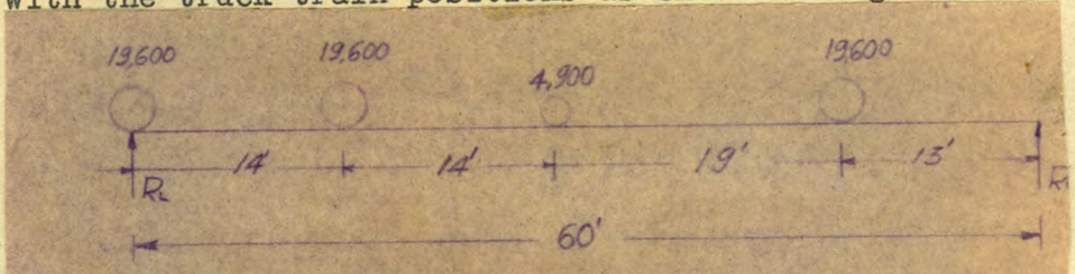

Fig. 7

Each beam supports $\frac{S}{5} = \frac{6.16}{5}$ = 1.23 wheel load per wheel. Therefore, the loads from the rear wheels are

$1.23 \times 16,000 = 19,600$ lbs., and from the front wheel

$1.23 \times 4,000 = 4,900$ lbs.

$$R_1 = 19,600 + 19,600 \frac{46+13}{60} + 4,900 \times 32/60 = 41,470 \text{ lbs.}$$

and impact coefficient $I = \frac{50}{60 + 125} = 27\ \%$

The impact shear therefore, is,

$$41,470 \times 27\ \% = 11,200 \text{ lbs.}$$

Total gross live load shear is,

$$41,470 + 11,200 = 52,670 \text{ lbs.}$$

Net live load shear is,

$$52,670 \times 75\ \% = 39,500 \text{ lbs.}$$

Hence, the total maximum end shear of the beam is, 80,750 lbs.

Therefore, the area b'd required to sustain the maximum shear is,

$$b'd = \frac{V_{max}}{j\ v}, \quad \text{where } V_{max} = \text{maximum shear}$$

j = constant = 7/8

v = $0.075\ f'_c$

$$= \frac{80,750}{7/8 \times 0.075 \times 2,500} = 500 \text{ sq. in.}$$

and d = 500/20 = 25 inches

Allowing for three rows of steel 3 in. center to center with 2 1/2" of clear insulation for the lowest row, the effective depth would be 41 - (3 + 3) = 35 in.

$\therefore$ Assumed depth of beam is O.K. with respect to shear and d = 35 in.

B) Calculations For The Maximum Moment.-

1. Dead Load Moment.- The dead load moment is,

$$M_d = 1/8\ w\ l^2$$

$$= 1/8 \times 1{,}375 \times (60)^2 \times 12 = 7{,}450{,}000 \text{ in-lb.}$$

2. Live Load Moment.- " The moment curve for a series of concentrated loads is a series of straight lines intersecting at the position of the loads, so that the absolute maximum live moment must occur directly beneath one of the loads. Also, the maximum moment, when the series of concentrated loads are applied to a simple end supported beam, occurs when the center of the span is halfway between that particular load and the resultant of the loads on the span."

The maximum moment occurs under the dashed wheel load with the truck-train position as shown in Fig. 8. below.

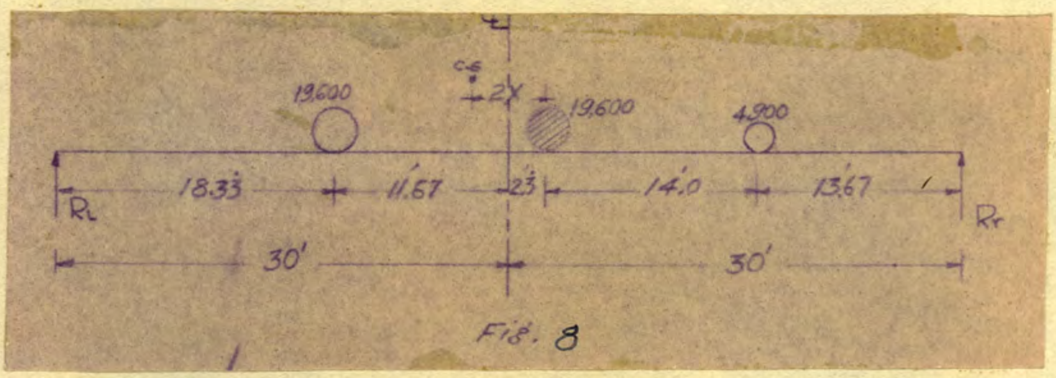

Fig. 8

$$4 \times 2x + 1(14 + 2x) = 4(14 - 2x)$$

$$\text{or} \quad 10x + 14 = 56 - 8x$$

$$\text{and} \quad x = 42/18 = 2.3\dot{3}$$

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$$\therefore \quad R_1 = 44,100 \times \frac{32.33}{60} = 23,700 \text{ lbs.}$$

and the maximum live load moment is,

$$M_1 = 23,700 \times 32.33 - 19,600 \times 14$$

$$= 491,000 \text{ ft-lb} = 5,900,000 \text{ in-lb.}$$

Impact moment = 5,900,000 x 27%= 1,650,000 in-lb.

Total gross live moment = 7,550,000 in-lb.

Total net live moment = 7,550,000 x 75 % = 5,660,000 in-lb.

and hence, the total maximum moment = 5,660,000 + 7,450,000

= 13,110,000 in-lb.

The steel area required is,

$$A_s = \frac{13,110,000}{20,000 \times (35 - 6/2)} = 20.6 \text{ sq. in.}$$

$$t/d = 6/35 = 0.17$$

$$pn = \frac{12A_s}{b\ d} = \frac{12 \times 20.6}{74 \times 35} = 0.095$$ in which

t, d, and b are as shown in Fig. 9 below.

Referring to diagram 3, appendix D of Urquhart Design of concrete structures, the beam is found to be a T beam with k = 0.401.

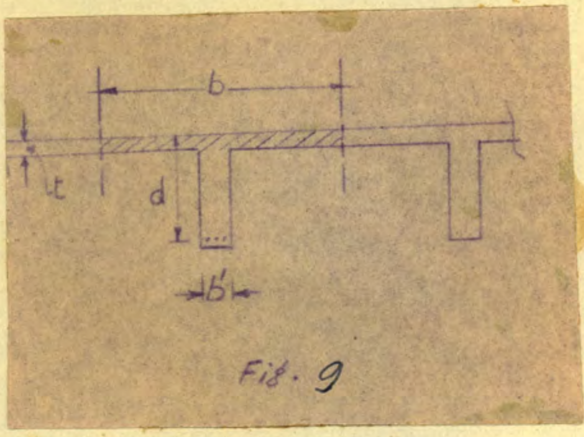

Fig. 9

$$\text{Also } f_c \equiv f_s\, k / n(1-k)$$

$$= \frac{20,000 \times 0.401}{12(1 - 0.401)} = 1,100 \text{ psi.}$$ which is

less than the allowable 1,125 psi

∴ the assumed section of the beam is O.K. with respect to moment.

As calculated before, the steel area required is 20.6 sq. in. This is furnished by fourteen No. 11 bars placed in three rows, the upper row being of four bars.

C) Calculations For Moments At Intermediate Points.-

1. Dead Load Moments.- At 10 ft. and 20 ft. from support, the dead load moments are respectively,

$$M_{10} = 12(41,250 \times 10) - 12(1,375 \times 10 \times 5)$$
$$= 4,120,000 \text{ in-lb.}$$

Similarly,

$$M_{20} = 6,600,000 \text{ in-lb.}$$

At midspan, the moment is zero.

2. Live Load Moment.- At 10 ft. from the left support, the maximum live load moment occurs with the truck-train position shown in Fig. 10. a. Taking moments about the right support, we find that,

$$R_1 = 30,800 \text{ lbs. and therefore,}$$

$$M_{10} = 30,800 \times 10 \times 12 = 3,700,000 \text{ in-lb.}$$

Similarly at 20 ft. from the left support, the maximum live load moment occurs with the truck-train position shown in Fig. 10.b.

$$M_{20} = 5,400,000 \text{ in-lb.}$$

3. Impact Moments.- The impact moments at 10 ft. and 20 ft. from left suppot are respectively,

$$M_{10} = 3,700,000 \times 27\ \% = 1,020,000 \text{ in-lb.}$$

$$M_{20} = 5,400,000 \times 27\ \% = 1,460,000 \text{ in-lb.}$$

4. Total Net Live Load Moments.- The total net live load moments after reduction due to the 6-lanes width roadway are,

$$M_{10} = (3,700,000 + 1,020,000)75\% = 3,550,000 \text{ in-lb}$$

Similarly,

$$M_{20} = 5,150,000 \text{ in-lb.}$$

5. Total Moments.- The total moments due to live and dead loads are,

$$M_{max} = 13,110,000 \text{ in-lb.}$$

$$M_{20} = 11,650,000 \text{ in-lb.}$$

$$M_{10} = 7,670,000 \text{ in-lb.}$$

Fig. 11.c. shows the total moment diagram and the points where bars can be bent up.

D) Bond.- The perimeter of steel bars required to satisfy the bond, is,

$$E_o = \frac{V_{max}}{u\ j\ d}$$ (After Urquhart) in which,

V_{max} = Maximum shear

$u = 0.045\ f_c$

$j = 7/8$

$$E_o = \frac{80,750}{0.045 \times 2,500 \times 35 \times 7/8} = 23.0 \text{ in.}$$

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This is furnished by five No. 11 bars. The lower row therefore of five bars, little over than one third of the total steel, (ACI Code specifications) will be continued straight into the support. The remaining two rows will be bent up to assist resisting the diagonal tension developed in the bracket and the ordinary shearing stresses in the whole beam. Fig. 11. a. shows the bent up bars of this steel.

E) Calculations for Shears At Intermediate Points.-

1. Dead Load Shears.- The dead load shear at 10 ft. and 20 ft. from the left support, are,

$$V_{10} = 41{,}250 - 10 \times 1{,}375 = 27{,}500 \text{ lbs.}$$

Similarly,

$$V_{20} = \qquad 13{,}750 \text{ lbs.}$$

2. Live Load Shears.- The maximum live load shear at 10 ft. 20 ft., and midspan, occurs with the truck-train in the position shown respectively in a, b, and c of Fig. 10. shown below.

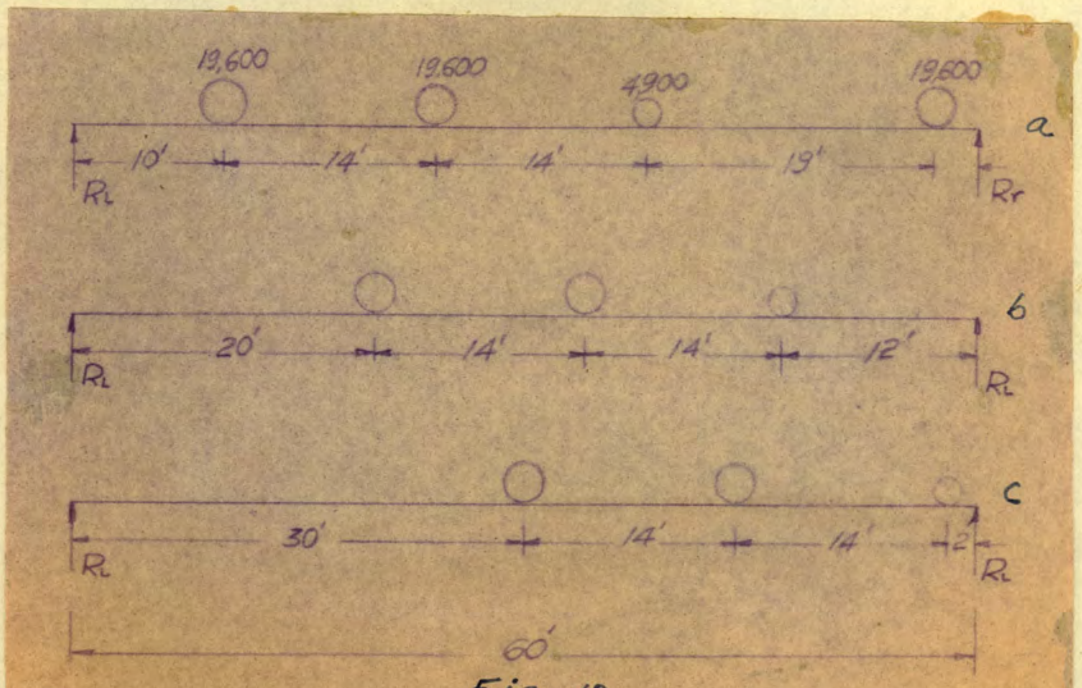

Fig. 10

Therefore,

$$R_{1-10} = 19{,}600\left(\frac{50 + 36 + 3}{60}\right) + 4{,}900 \times \frac{22}{60}$$

$$= 30{,}800 \text{ lbs.}$$

Similarly,

$$R_{1-20} = 22{,}480 \text{ lbs.}$$

and $R_{1-30} = 15{,}160$ lbs.

3. Impact Coefficients.- The loaded lengths for the above three cases are considered as 50 ft., 40 ft., and 30 ft. respectively.

$$\therefore \quad I_{10} = \frac{50}{50 + 125} = 28.6\ \%$$

Similarly,

$$I_{20} = 30.0\ \%$$

and $I_{30} = 32.3\ \%$, Use maximum 30 %.

4. Impact Shear.- The impact shear at 10 ft. 20 ft., and midspan are respectively 8,800 lbs., 6,750 lbs., and 4,500 lbs.

The following table on page 28, gives the total and unit shear values of four points along the beam. Fig. 11.b. shows the unit shear diagram .

E) Web Reinforcement.- The portion of the beam over which web reinforcement is required, is determined by computing the unit shears at various points along the beam. This is shown in Table 1, page 28, and graphically in Fig. 11.b.

TABLE I

TOTAL AND UNIT SHEAR

Location	Gross-Live lb.	Net-Live 25% Red. lb.	Total lb.	Unit psi
End			80,750	134
10' from L.S.	39,600	29,700	57,200	95
20' " " "	29,230	22,000	35,750	60
Midspan	19,660	14,700	14,700	25

The concrete resists a unit shear of $0.02\ f'_c = 50$ psi, (AASHO) and the remainder must be taken up by bent bars or stirrups. Nine bent bars were considered to resist the maximum portion of the shear and to help resisting the diagonal tension in the bracket, and the rest to be cared of by stirrups.

The maximum distance over which inclined bars may be considered effective in resisting shearing stresses is equal to the effective depth of the beam = 35 in. in accordance with the ACI Code. See Fig. 11.a.

At a distance of 8' 9" from the edge of the left support, the unit shear determined from the figure, is 100 psi. and the required spacing using two No. 3 U stirrups, is

$$s = \frac{A_v f_v}{(v_1 - v_2)b} \quad \text{in which,}$$

f_v = Allowable stress in steel = 20,000 psi

A_v = Cross section of steel.

v_1 & v_2 = Unit shears at points 1 & 2.

and therefore $s = \dfrac{20,000 \times 4 \times 0.11}{(100 - 50) \times 20} = 8.8$ inches.

The maximum allowable spacing is 1/2 x 35 = 17.5 in. Similarly, at 15 ft. from left support, the unit shear is 75 psi, and s = 17.5 in.

F) Arrangement Of Stirrups.- Use 6 @ 16" centers with the first stirrup placed 24" from the edge of the support, 11 @ 8", and 11 @ 16". Theoratically, stirrups are not needed over the first 8 ft. from support, where the inclined bars resist the diagonal tension, nor they are needed over the portion between 6 ft. from the center of the span, and the center of the span where the concrete resists the diagonal tension of that portion; but the arrangement herein given ensures the proper tying of the flange to the web and gives an additional safeguard against diagonal cracks.

For Stirrups details refer to Fig. 11.a.

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3) <u>Design Of Bracket At Hinges.</u>- Brackets at the hinges are vital parts of the construction and hence a great deal of care should be taken in their design. Over reinforcement of this part of the structure is recommended due to to the fact that the function of this extra reinforcement is not only to resist stresses due to the loads and to contraction, but in the first place, to resist the tendency to cracking caused by the sudden reduction in the cross section at the bracket. Although the existence of this tendency is known, its effect cannot be computed with any degree of accuracy. However, the excess steel used here forms only a very small portion of the total tonnage of the steel used in the structure.

The bracket itself should be considered as a short cantilever and investigated for bending stresses in the ordinary manner. No reliance on concrete should be made and vertical stirrups should be designed to resist all the diagonal tension developed. In addition, bars from the main reinforcement of the member should be bent at 45 degrees and should be spaced as near as possible. Some horizontal bars however, shall be provided to resist the tension produced by the contraction of the short span and caused by imperfect action of the expansion bearing. All reinforcement used to resist stresses in the bracket should not exceed an allowable stress more than half of that specified elsewhere in the structure. i.e. f_s = 10,000 psi.

The moment about the end of the bracket, assuming that all the load is concentrated at the farther end, is,

$$M = 80{,}750 \times 2 \times 12 = 1{,}940{,}000 \text{ in-lb.}$$

and the steel area required, is,

$$A_s = \frac{1{,}940{,}000}{10{,}000 \times 0.866 \times 13} = 17.0 \text{ sq.in.}$$ This

is furnished by 10 No. 11 bars arranged in two rows.

1. <u>Web Reinforcement</u>.- The spacing between vertical U stirrups, using five No. 5 bars, is,

$$s = \frac{A_v\, f_v\, j\, d}{V_{max}}$$

$$= \frac{5 \times 2 \times 0.31 \times 10{,}000 \times 0.866 \times 13}{80{,}750}$$

$$= 4.3 \text{ in.}$$

$\therefore$ use five No. 5 U stirrups at 4 in. on centers in both directions, up to a distance of 20" from the edge of the normal section.

Fig. 12. shows an enlarged section at bracket of the simply supported middle span, and section BB, a cross section through the bracket.

For the cantilever bracket, the same steel and arrange- shall be used, but in the opposite direction.

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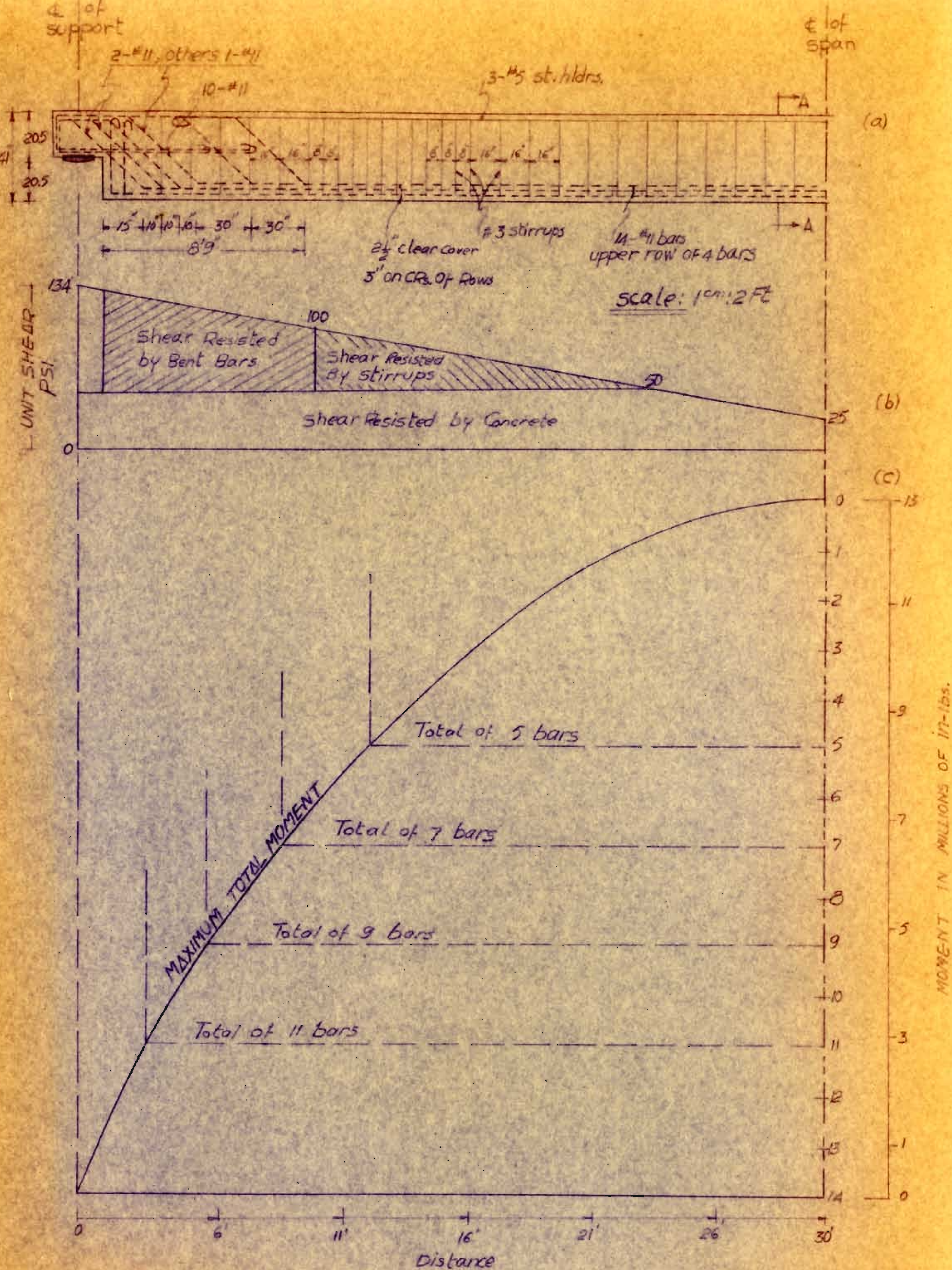
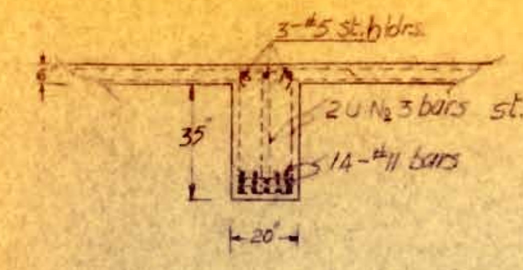


Fig. 11



SECTION A-A

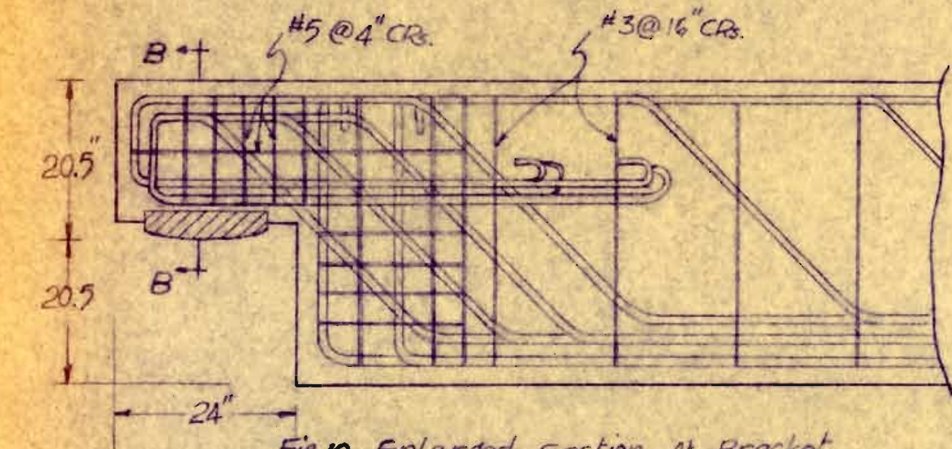
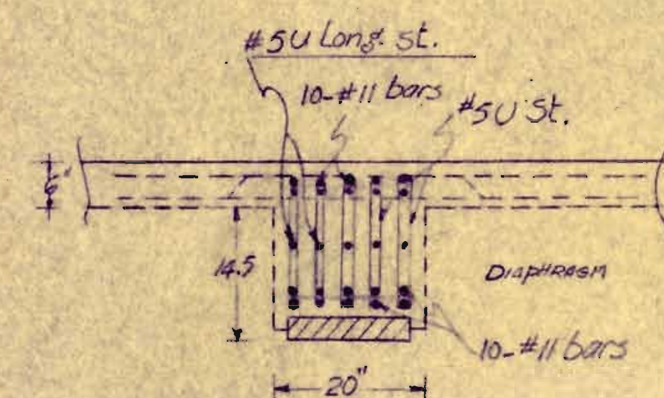


Fig. 12. Enlarged section at Bracket Showing Reinforcement

Scale :- 1cm : 10 in.



SECTION B-B

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4) Design Of Slab Under Footpath.-

Span = 4' 9"

Assume Slab= 4"

Weight due to slab = $12 \times 4 \times \frac{150}{144}$ = 50 psf

" " to sand = 1 x 120 = 120 psf

" " surfacing = $2\ 1/2 \times \frac{150}{12}$ = 85 psf

T O T A L = 200 psf

Live Load = 85 psf

Therefore, total load on slab = 285 psf

Assuming 1/10 moment coefficient due to partial restraint construction, the moment would be,

$$M = \frac{1}{10} \times 285 \times 5 \times 5 \times 12 = 8{,}600 \text{ in-lb.}$$

$$\text{and} \quad d = \left(\frac{8{,}600}{12 \times 196}\right)^{\frac{1}{2}} = 2 \text{ in.} + 1'' \text{ cover} = 3''$$

Therefore, use minimum allowable slab thickness of 4".

The steel area required is, $A_s = \dfrac{8{,}600}{20{,}000 \times 0.866 \times 3}$

= 0.165 sq. in. which is furnished by No. 3 bars placed at 8" on centers. Bend up alternate bars and supply the remaining required negative steel by No. 3 bars placed at 16 in. on centers.

For transverse reinforcement, use No. 3 bars at 12" on centers.

End shear = 285 x 3.33/2 = 475 lb./ft. of beam length.

Fig. 13 below, shows a cross section through the side-walk .

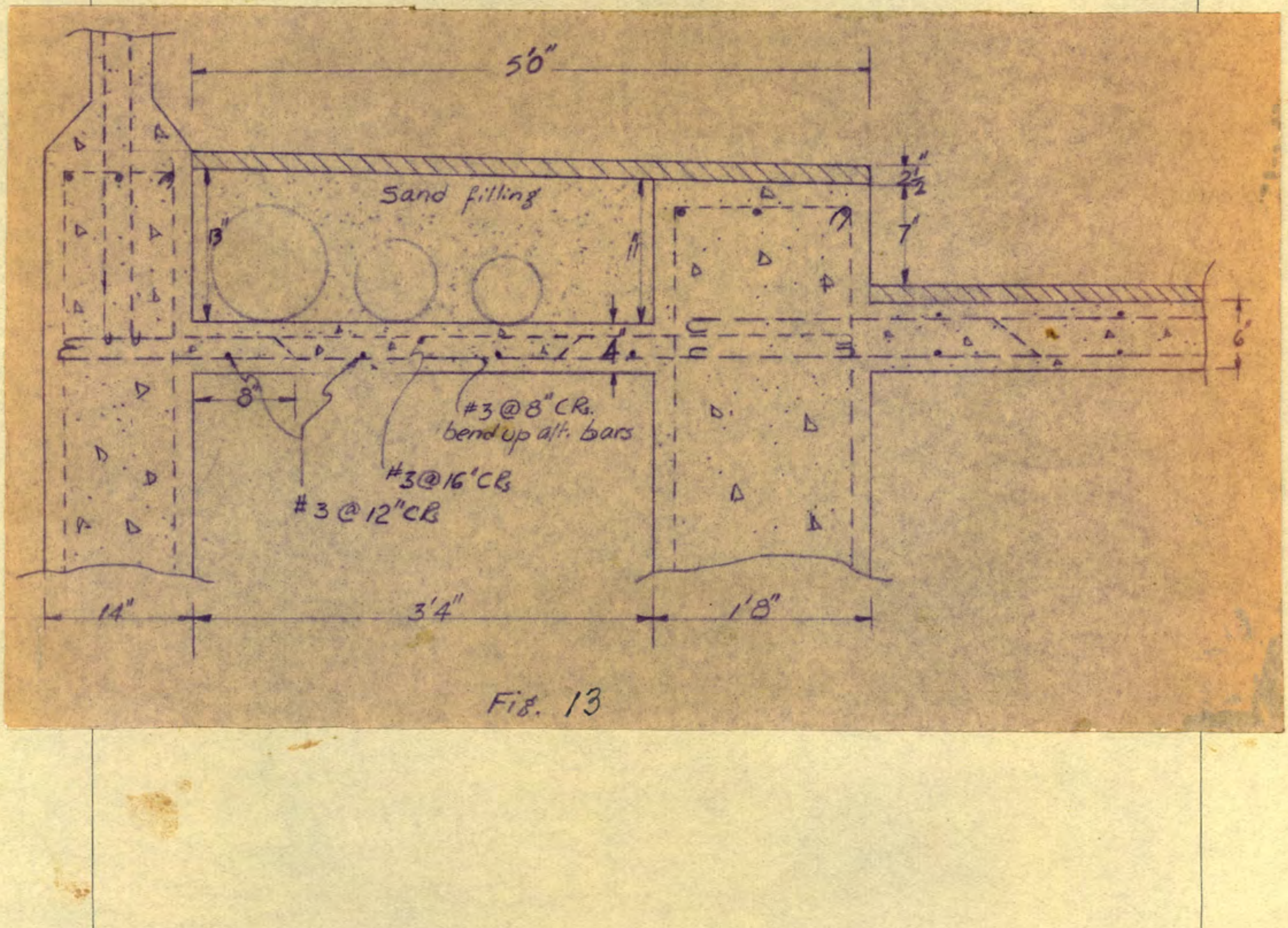

Fig. 13

5) Design Of Beams "B".- (Refer to Fig. 2) Beams "B" shall be designed as rectangular beams with a 20" x 50" section. The upper portion of the beams shall serve as a curb for the roadway.

A) Dead Loads.- The total dead load per foot of length acting on the beam, is,

From Roadway	=	105 x 2.25	=	237 lb.
" Sidewalk	=			475 lb.
From beam	=	20 x 50 x 150/144		
				=1,040 lb.
T O T A L	=			1,750 lb.

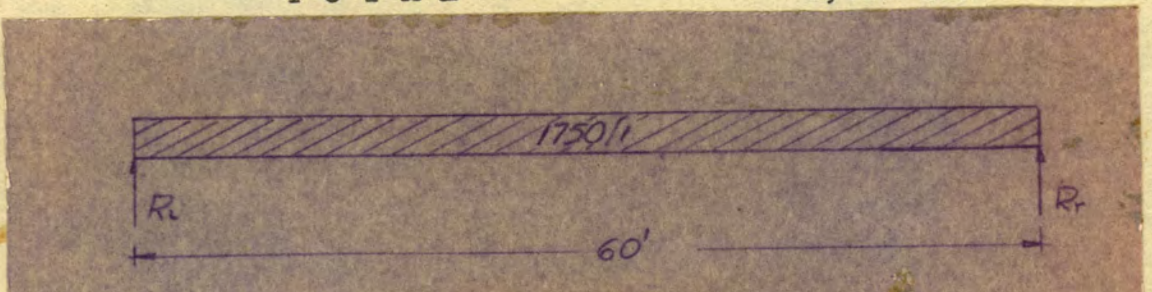

FIG. 14

End shear R_1 = 1,750 x 60/2 = 52,500 lb.

B) Dead Load Moments.- The maximum dead load moment is,

$$M_d = 1/8 \times 1{,}750 \times (60)^2 \times 12 = 9{,}450{,}000 \text{ in-lb.}$$

At 10' from support,

$$M_{10} = 5{,}250{,}000 \text{ in-lb.}$$

And at 20',

$$M_{20} = 8{,}420{,}000 \text{ in-lb.}$$

C) Live Load Moments.- A portion of each wheel load which rests on the exterior slab panel is supported by the exterior beam, namely beam "B". The proportioning of the load is determined by considering this panel, see Fig. 15, as simply supported and taking moments about the first interior support, we have,

$$\frac{3.25}{6.1} = 0.53 \text{ of the wheel load has to}$$

be supported on beam "B". The longitudinal proportion of the loading which will produce the masimum bending moment is the same as for beams "A" and hence the moments are directly proportional.

$$M_{max} = 5,660,000 \times \frac{0.53}{1.23} = 2,440,000 \text{ in-lb.}$$

Similarly,

$$M_{10} = 1,530,000 \text{ in-lb.}$$

and $$M_{20} = 2,220,000 \text{ in-lb.}$$

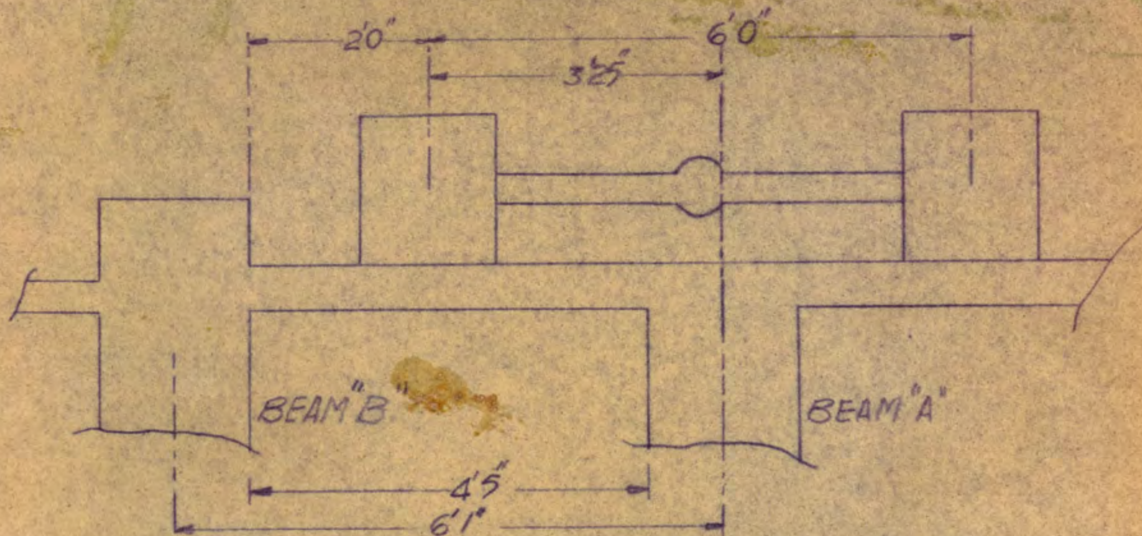

Fig. 15

D) Total Moments.- The total dead and live moment at intermediate points along the beam, are,

$$M_{max} = 2,440,000 + 9,450,000 = 11,890,000 \text{ inlb}$$

$$M_{10} = 1,530,000 + 5,250,000 = 6,780,000 \text{ in-lb.}$$

$$\text{and } M_{20} = 2,220,000 + 8,420,000 = 10,640,000 \text{ in-lb.}$$

$$d = \left(\frac{11,890,000}{20 \times 196}\right)^{\frac{1}{2}} = 55 \text{ in. which is more}$$

than the actual. Therefore compression reinforcement is required.

Allowing for three rows of steel, d = 50 - 6 = 44 in.

$$M_1 = Kbd^2 = 196 \times 20 \times 44^2 = 7,650,000 \text{ in-lb.}$$

$$M_2 = 11,890,000 - 7,650,000 = 4,340,000 \text{ in-lb.}$$

$$A_{s1} = \frac{7,650,000}{20,000 \times 44 \times 0.866} = 10.0 \text{ sq. in.}$$

$$A_{s2} = \frac{4,340,000}{20,000(44 - 2\ 1/2)} = 5.2 \text{ sq. in.}$$

$$A_s = 10.0 + 5.2 = 15.2 \text{ sq. in.}$$

$$A'_s = 5.2/2 \times \frac{1 - 0.403}{0.403 - \frac{2.5}{44}} \quad \text{(Urquhart page 109)}$$

$$= 4.5 \text{ sq. in.}$$

Therefore, use 5.2 sq. in. so that the stress in the compressive steel will not exceed that of the tensile steel. Sixteen No. 9 bars are selected for tension reinforcement arranged in three rows, five bars in each of the upper two rows, and six No. 9 bars are selected for compression reinforcement, furnishing 16.0 sq. in. and 6.0 sq. in. respectively.

Fig 16. c. shows the total moment diagram and the points where bars can be bent up.

E) Bond.- The perimeter of the steel bars required to satisfy the bond, is,

$$E_o = V_{max}/ u\ j\ d$$

$$= \frac{69,500}{0.045 \times 2,500 \times 44 \times 7/8} = 6.0 \text{ in.}$$

Two No. 9 bars are needed to satisfy this perimeter. Therefore, the lower row of six bars, shall be continued straight into the support. The remaining ten bars shall be bent up to assist resisting the diagonal tension developed in the bracket and the ordinary shearing stresses in the beam.

F) Shears.- The maximum end shear as found before is

V_{max} = 52,500 lbs.

At 10 ft. from the support,

$$V_{10} = 52,500 - 1750 \times 10 = 35,000 \text{ lbs.}$$

and at 20',

$$V_{20} = 17,500 \text{ lbs.}$$

The maximum net live load shear is proportional to the maximum net live load shear in beams "A" and is

$$V_{max} = 39,500 \times 0.53/1.23 = 17,000 \text{ lbs}$$

Similarly, at 10 ft. from support,

$$V_{10} = 12,700 \text{ lbs.}$$

and $V_{20} = 9,500$ lbs.

$V_{30} = 6,350$ lbs.

G) Total Shears.- The total shears at intermediate points along the beam, are,

$$V_{max} = 52,500 + 17,000 = 69,500 \text{ lbs.}$$

Similarly,

$V_{10} = 47,700$ lbs.

and $V_{20} = 27,000$ lbs.

$V_{30} = 6,350$ lbs,

H) Unit Shears.- At the support, the unit shear is,

$$\frac{69,500}{20 \times 7/8 \times 44} = 90 \text{ psi},$$

and similarly, at 10 ft. from support, 61 psi, and at 20 ft. from support, 35 psi.

Fig. 16.b. shows a graphical representation of the unit shear along the beam.

G) Web Reinforcement.- As shown in Fig. 16.b. the portion between the edge of the support and the section 9' 2" towards the center, the shearing stresses were taken care by bent up bars. At that section, the unit shear is 61 psi and the required spacing using No. 3 U stirrups is,

$$s = \frac{20,000 \times 2 \times 0.11}{(61-50)20} = 20 \text{ in.}$$

The maximum allowable spacing is 1/2 x 44 = 22 in.

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Therefore, use No. 3 U stirrups 20 in. on centers all along the beam with the first stirrup placed 24 in. from the edge of the support. Theoratically, the beam is over reinforced with stirrups and 2/3 of its length does not require it, however, they are meant to hold compression bars with tension bars more tightly and to reinforce the beam against cracking due to other causes.

Fig. 16.a. shows a cross section of the beam with the arrangement of stirrups, bent bars, compression reinforcement, and main reinforcement.

6) Design Of Bracket At Hinges.- The moment about the end of the bracket, assuming that all the load is concentrated at the farther end, is

$$M = 69{,}500 \times 2 \times 12 = 1{,}650{,}000 \text{ in-lb.}$$

and the steel area required, is,

$$A_s = \frac{1{,}650{,}000}{10{,}000 \times 0.866 \times 16} = 12.0 \text{ sq.in.}$$

This is furnished by 10 No. 9 bars placed in two rows. The area of this steel is 10.0 sq.in., i.e. 2.0 sq. in. less than the required. However, this amount is considered safe due to the very high factor of safety taken and due to the assumption of the worst location of loading which can never be true.

A) Web Reinforcement.- For web reinforcement, use the same stirrups with the same spacing as that of brackets of "Beams A" . (page 31)

Fig. 17. shows an enlarged section of the bracket of the simply supported middle span, and section B-B shows a cross section through that bracket. For the cantilever beam, the same steel and arrangement of stirrups and reinforcement shall be used, but in the opposite direction.

7) <u>Design Of Railings</u>.- The forces acting on the railings are,

Lateral = 300 lb. per foot

Wind = 50 lb. per sq. ft. of area

= 150 lb. per foot

Vertical = 100 lb. per foot

Own weight = $6/12 \times 3 \times 150 + 4 \times 6 \times \frac{150}{144}$

= 250 lb. per ft.

The points of application of the above loads are shown in Fig. 18 below.

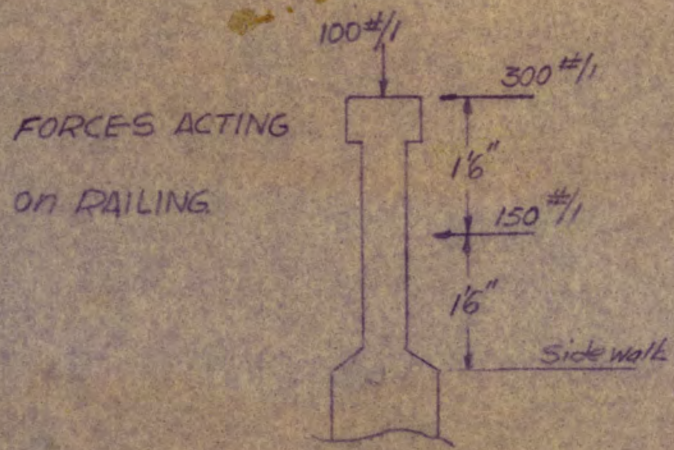


Fig. 18

Taking moments about the bottom of railings, we have,

$$M = 12(300 \times 3 + 150 \times 1.5) = 13,500 \text{ in-lb.}$$

$$d = \left(\frac{13,500}{12 \times 196}\right)^{\frac{1}{2}} = 3''$$, and therefore the 6" web is very safe, however, it shall be kept and d = 4.5".

$$A_s = \frac{13,500}{20,000 \times 0.866 \times 4.5} = 0.15 \text{ sq. in. per ft.}$$

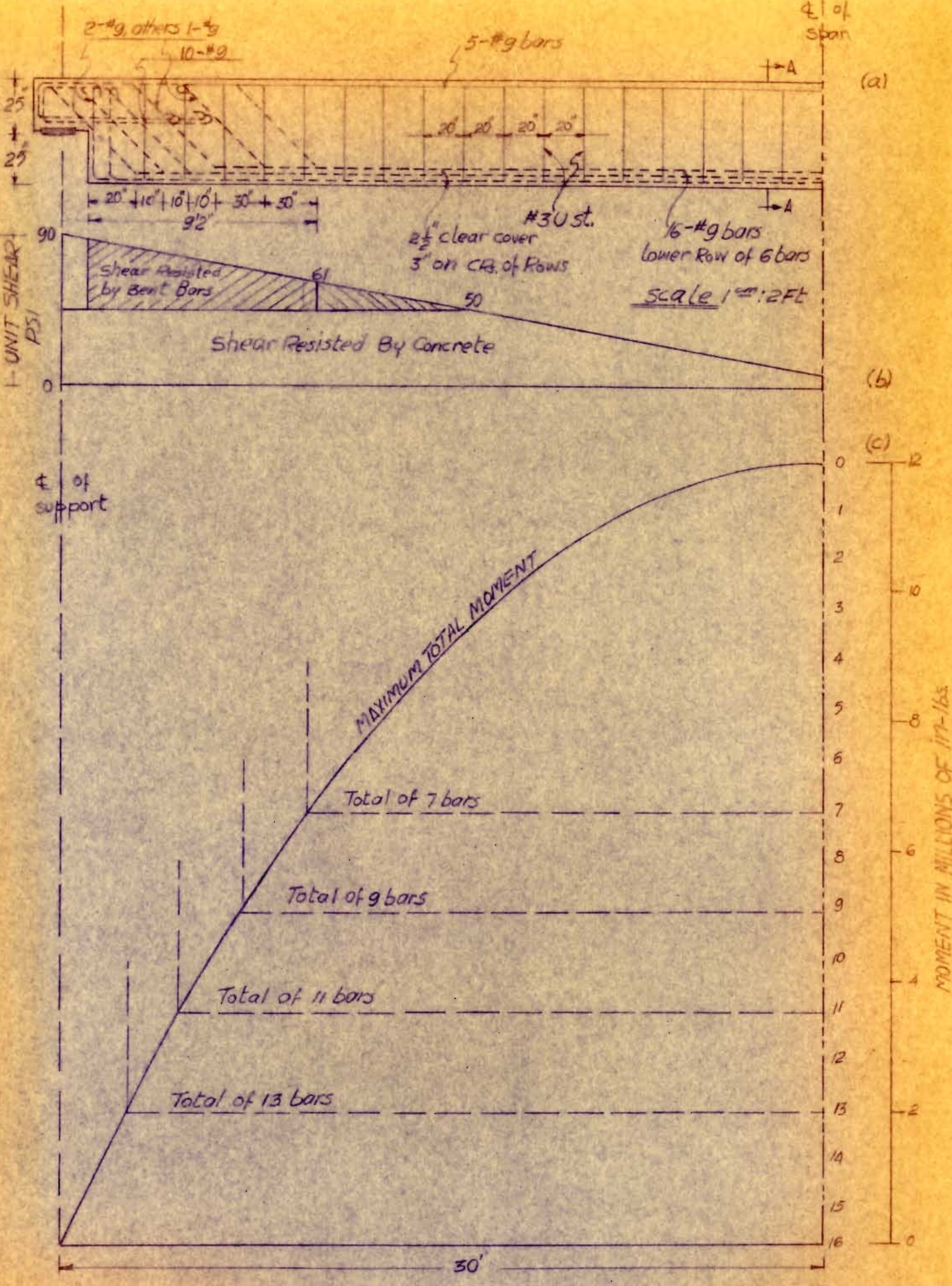
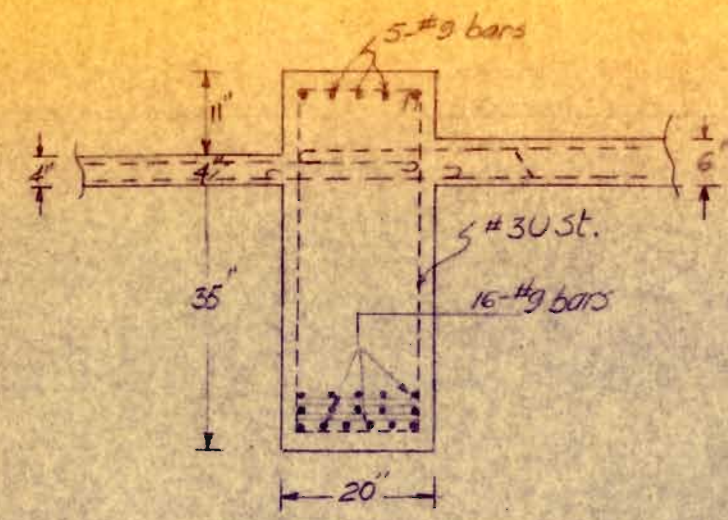


Fig. 16



SECTION A-A

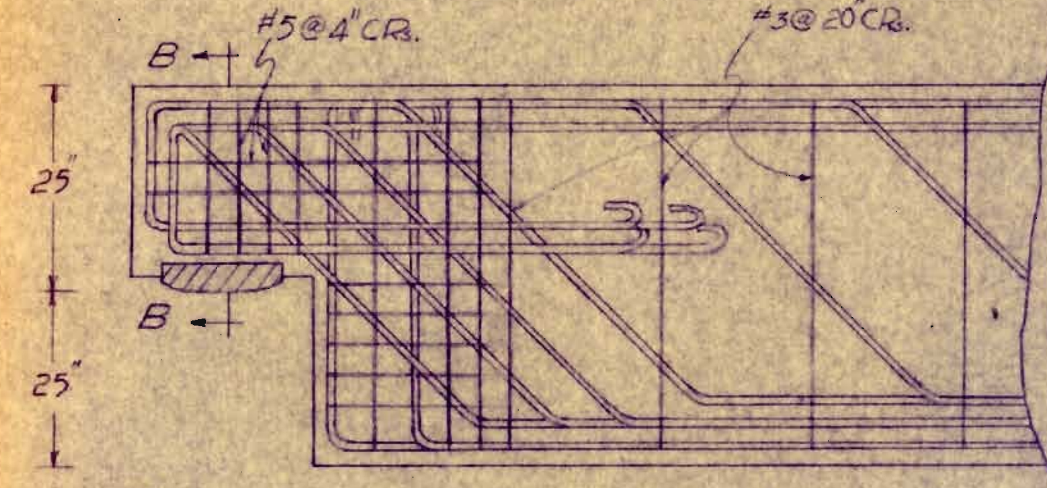
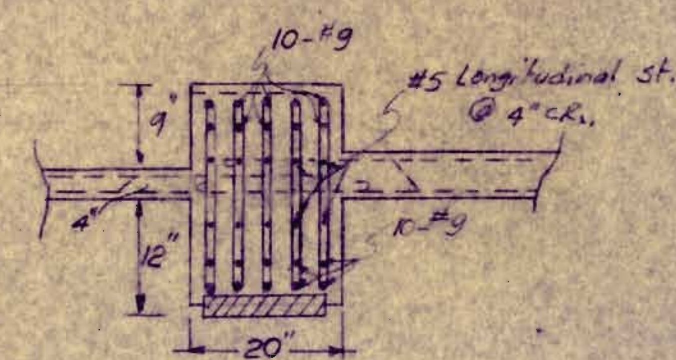


Fig. 17 Enlarged Section at Bracket.

Scale 1cm: 10 in.



SECTION B-B

This is furnished by No. 4 bars placed at 12" on centers.

On the outside of the railings, only 150 lbs. per foot is acting at the center of the railings. i.e., the wind forces only. The moment therefore, is less than that calculated above, and consequently, the steel area is less. The designer, however, prefers to use the same number of steel with the same spacing, to resist the force of the water in case any higher flood occurs.

For longitudinal reinforcement, no design has been carried out due to the fact that they are not used for structural purpose. Some bars however, have been supplied to take care of temperature stresses and to give the railing more rigidity.

Fig. 19. shows the reinforcement details of the railings.

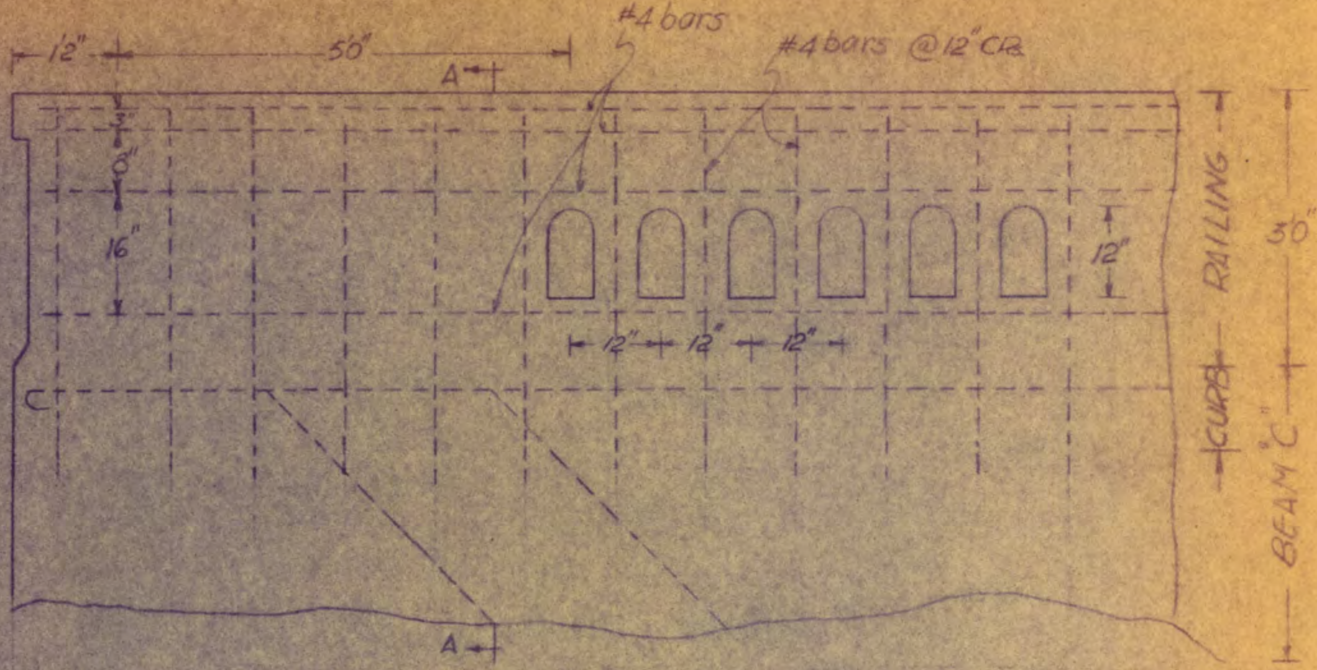
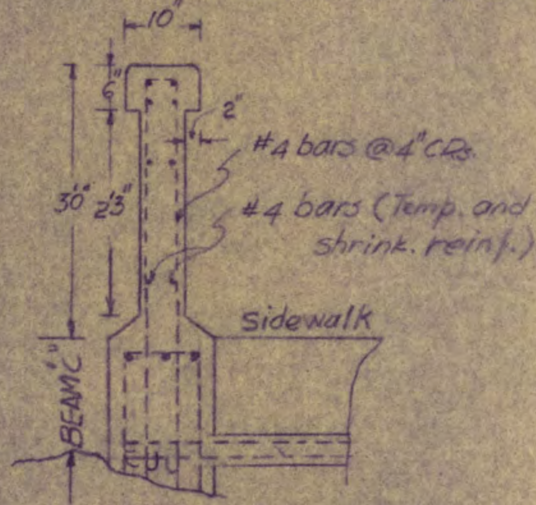


Fig. 19 LONGITUDINAL SECTION THROUGH RAILING. Scale 1 cm: 10 in.



SECTION A-A

8) <u>Design Of Beams "C"</u>.- (see Fig. 2.) Beams "C" shall be designed as rectangular beams with a 14" x 52" cross section.

The loads acting on the beam per linear foot of length, are,

From railing	=	250 + 100	=	350 lbs.
From side walk	=			475 lbs.
From own weight	=	$14 \times 52 \times \frac{150}{144}$	=	760 lbs.
T O T A L			=	1,585 lbs.

A) <u>Shears</u>.-

$$V_{max} = 1,585 \times 60/2 = 47,550 \text{ lbs.}$$
$$V_{10} = 31,700 \text{ lbs.}$$
$$V_{20} = 15,850 \text{ lbs.}$$

B) <u>Moments</u>.-

$$M_{max} = 1/8 \times 1,585 \times 60^2 \times 12 = 8,600,000 \text{ in-lb.}$$
$$M_{10} = 12(47,550 \times 10 - 1,585 \times 10 \times 5)$$
$$= 4,800,000 \text{ in-lb}$$
$$M_{20} = 7,550,000 \text{ in-lb}$$

From the maximum moment, we found that d = 57 in. which is more than the available d = 52 - 6 = 46 in. Therefore compression reinforcement is required.

$$M_1 = Kbd^2$$
$$= 196 \times 14 \times 46^2 = 5,850,000 \text{ in-lb.}$$

$$M_2 = 8,600,000 - 5,850,000 = 2,550,000 \text{ in-lb}$$

and the steel area required in each case, is

$$A_{s1} = \frac{6,500,000}{20,000 \times 0.866 \times 46} = 7.9 \text{ sq. in.}$$

$$A_{s2} = \frac{2,550,000}{20,000 \times (46 - 2.5)} = 2.3 \text{ sq. in.}$$

and $A_s = 7.9 + 2.9 = 10.2$ sq. in.

The compression reinforcement = 2.3 sq. in. (see page 36)

Using 11 No. 9 bars for tension reinforcement and 3 No. 9 bars for compression reinforcement, the steel furnished is 11.0 sq. in. and 3.0 sq. in. respectively.

C) <u>Bond</u>.- The perimeter of steel required is

$$E_o = \frac{47,550}{0.045 \times 2,500 \times 46 \times 7/8} = 10.5 \text{ in.}$$ This is

furnished by 6 No. 9 bars, and therefore the remaining five bars shall be bent up to resist the diagonal tension in the bracket and beam.

D) <u>Web Reinforcement</u>.- At the support, the unit shear is,

$$\frac{47,550}{14 \times 7/8 \times 46} = 85 \text{ psi, at the 5' section } =$$

69 psi, and the 10' section, it is = 53 psi. Similar procedure for stirrups' design in the beam and in the bracket, was followed as in beams "A" & "B". Fig. 20 shows the details of the reinforcement in the beam and bracket.

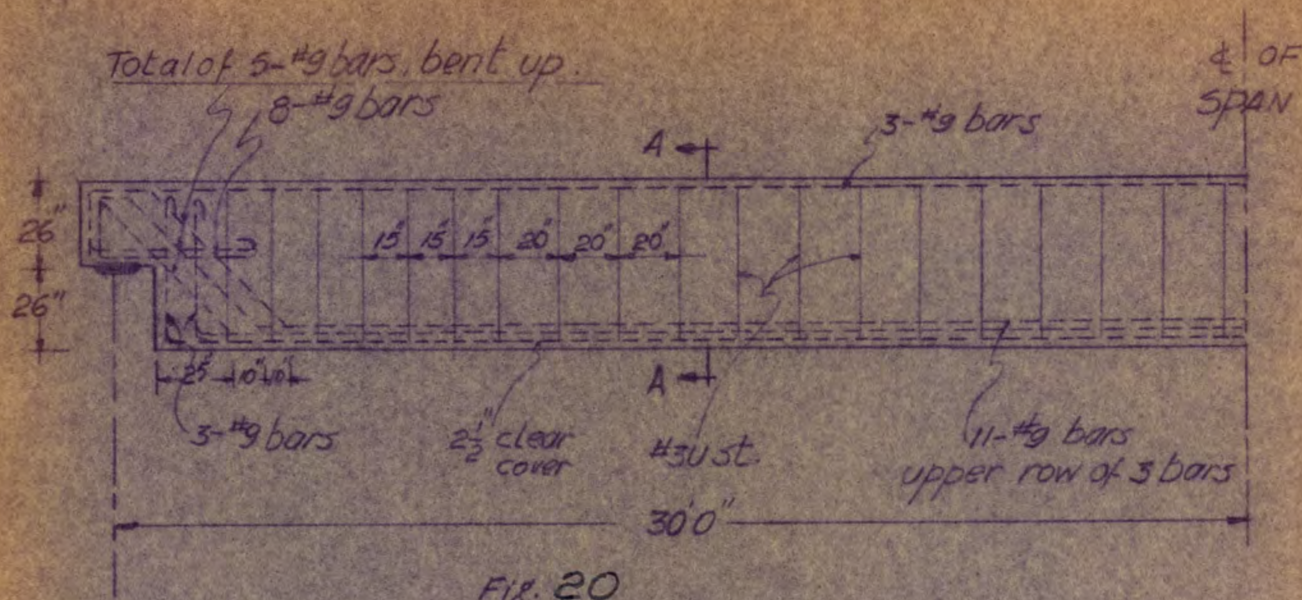
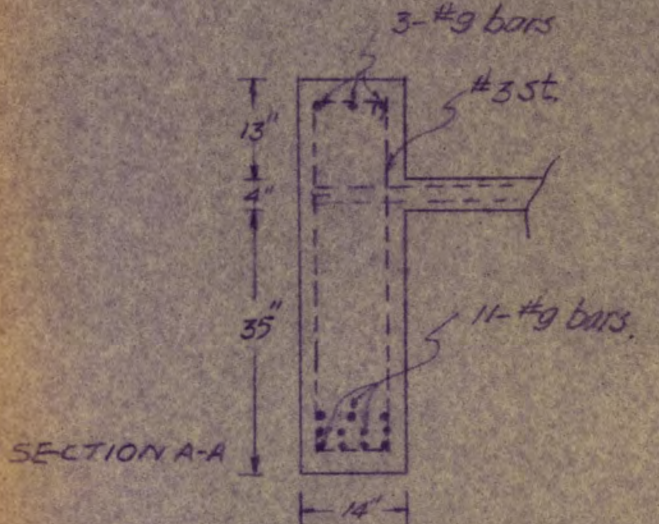
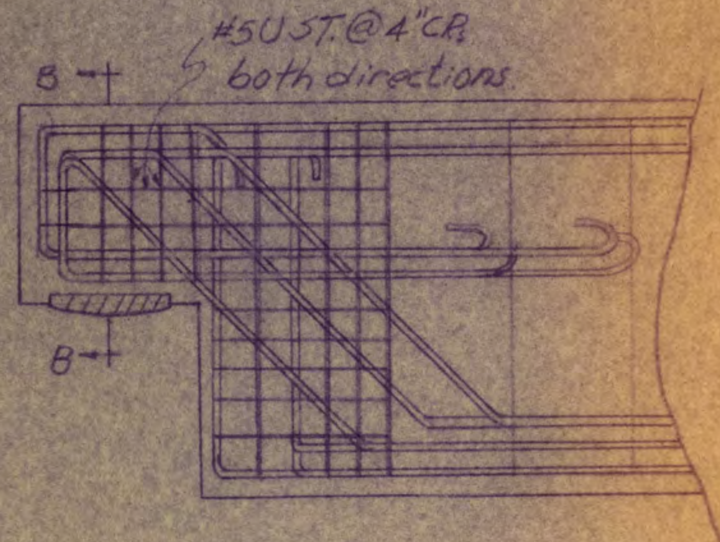


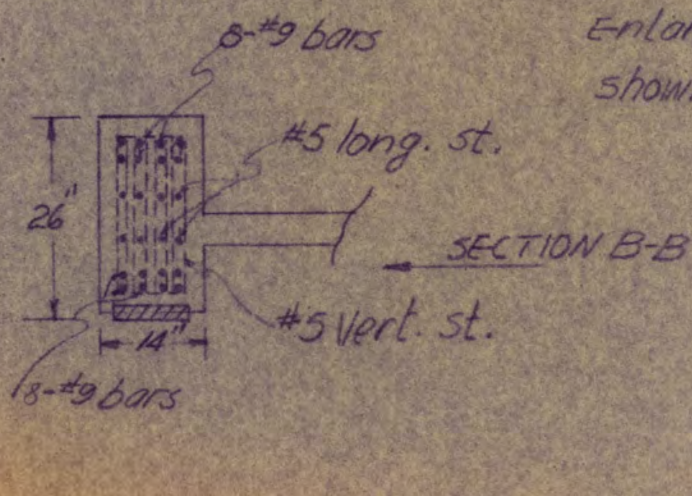
Fig. 20

SCALE 1cm : 2 Ft.





Enlarged section at Bracket showing St. Arrangements.



CHAPTER IV

DESIGN OF CANTILEVER AND END SPAN

1) Cantilever Of Beams "A".- (Refer to Fig. 2.)

A) Calculations For The Maximum Moment.- Assume a stem at the end of the cantilever of 20" x 65".

The dead load from the simply supported middle span acting at the free end of the cantilever, is, 41,250 lbs. (see page 20).

Therefore, the moment due to this load at the fixed end of the cantilever, is

$$M = 41{,}250 \times 15 \times 12 = 7{,}440{,}000 \text{ in-lb.}$$

and the moment due to the weight of the cantiliver itself, is,

$$M = 1{,}375 \times 15 \times 15/2 \times 12 + \frac{30}{12} \times \frac{15}{2} \times 150 \times \frac{15}{3} \times 12$$

$$= 2{,}134{,}000 \text{ in-lb.}$$

Therefore, the total dead moment is

$$M_t = 7{,}440{,}000 + 2{,}134{,}000 = \underline{9{,}574{,}000} \text{ in-lb.}$$

1. Live Moment.- The critical position of the truck-train to give the maximum live load moment at the end of the cantilever is as shown in Fig. 21.

The net live concentrated load at the free end of the cantilever of such loading is 39,500 lbs. (page 21)

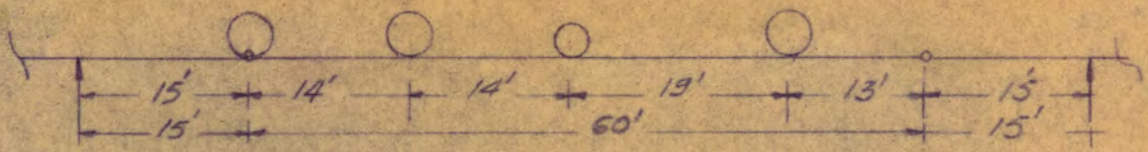


Fig. 21

and the moment due to this live load, is

$$M_1 = 39,500 \times 15 \times 12 = 7,100,000 \text{ in-lb.}$$

The maximum dead and live load moment is,

$$M_t = 9,574,000 + 7,100,000 = 16,674,000 \text{ in-lb.}$$

$$\text{and} \quad d = \left(\frac{16,674,000}{20 \times 196}\right)^{\frac{1}{2}} = 65 \text{ in.}$$

Allowing for three rows of steel with 3 in. on centers of rows and 2 1/2 in. clear insulation for the lower row, the effective depth would be 65 + 6 - 6 = 65 in. as assumed. Therefore, the design is safe with respect to moment and d = 65 in.

The steel area required at the fixed end of the canliver is,

$$A_s = \frac{16,674,000}{20,000 \times 0.866 \times 65} = 14.8 \text{ sq. in.}$$ This

is furnished by ten No. 11 bars, placed in two rows.

B) <u>Calculations For Moments At Intermediate Points.-</u>

At 5 ft. from the cantilever free end, the moment due to the smiply supported middle span, is,

$$M_{10} = 41,250 \times 5 \times 12 = 2,480,000 \text{ in-lb.}$$

and due to its own weight,

$$M_5 = 217,400 \text{ in-lb.}$$

and therefore, the total dead moment at the 5 ft. section is

$$M = 2,480,000 + 217,000 = 2,697,000 \text{ in-lb.}$$

The live load moment at the same section is,

$$M_l = 39,500 \times 5 \times 12 = 2,370,000 \text{ in-lb.}$$

And the total live and dead load moment is,

$$M_t = 2,970,000 + 2,370,000 = 5,067,000 \text{ in-lb.}$$

Similarly at the 10 ft. section from the free end, the moment due to simply supported middle span, is

$$M_{10} = 4,960,000 \text{ in-lb.}$$

and due to its own weight,

$$M_{10} = 908,500 \text{ inlb.}$$

Total dead moment is therefore,

$$M_t = 5,868,500 \text{ in-lb.}$$

The moment due to the live load at the same section, is,

$$M_l = 4,740,000 \text{ in-lb.}$$

And the total dead and live load moment at the 10 ft. section from the free end of the cantilever, is

$$M_t = 10,600,000 \text{ inlb.}$$

C) <u>Calculations For Shear</u>.-

1. <u>Dead Load Shear At Intermediate Points</u>.-

$$V_{max} = 41,250 + 1,375 \times 15 + \frac{30}{12} \times \frac{20}{12} \times \frac{15}{2} \times 150$$

$$= 65,550 \text{ lbs.}$$

$$V_{10} = 41,250 + 1,375 \times 10 + \frac{20}{12} \times \frac{20}{12} \times \frac{10}{2} \times 150$$

$$= 57,050 \text{ lbs.}$$

Similarly,

$$V_5 = 48,650 \text{ lbs.}$$

and $$V_0 = 41,250 \text{ lbs.}$$

2. Live Load Shear At Intermediate Points.- The truck-train positions to give maximum shears at intermediate points along the cantilever are shown in a,b,c & d of Fig 22. on page 49.

$$V_{max} = 39,200 + 4,900 \times \frac{47}{60} + 19,600 \times \frac{28+14}{60}$$

$$= 56,600 \text{ lbs.}$$

Similarly,

$$V_{10} = 48,800 \text{ lbs.}$$

and $$V_5 = 45,200 \text{ lbs,}$$

3. Impact Shear.- The impact coefficient for all three cases is ,

$$I = \frac{50}{75 + 125} = 25 \%$$

The maximum impact shear is

$$V_{max} = 56,600 \times 25 \% = 14,150 \text{ lbs.}$$

$$V_{10} = 12,200 \text{ lbs.}$$

and $$V_5 = 11,200 \text{ lbs.}$$

4. Net Live Load Shear.- The net live load shears at intermediate points along the cantilever, are,

$$V_{max} = (56,600 + 14,150) = 53,000 \text{ lbs.}$$

Similarly,

$$V_{10} = 46,000 \text{ lbs.}$$

$$V_{5} = 42,400 \text{ lbs.}$$

and $$V_{0} = 39,500 \text{ lbs.}$$

Table II on page 50, shows the total and unit shears, and the effective depths, calculated from the bending moments, along the cantilever length.

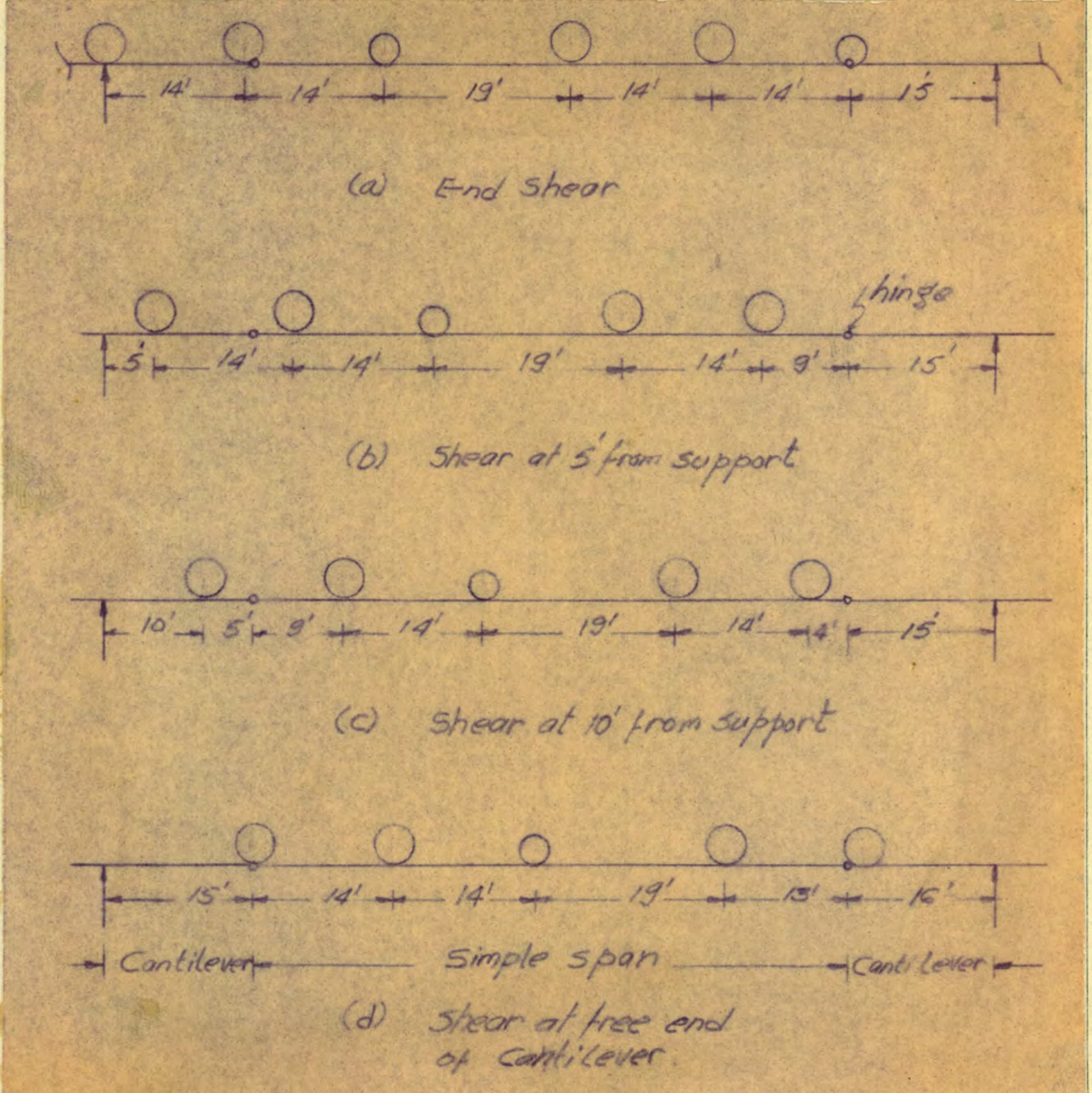


Fig. 22

TABLE II

SHEARS AND EFFECTIVE DEPTHS

Location	Total Shears lbs.	Unit Shears psi	Eff. Depth in.
Support	118,550	104	65
5' from support	103,000	113	52
10' from support	91,000	144	37
Free end of cant.	80,750	134	35

5. Web Reinforcement.- The maximum unit shear occurs at a section 10 ft. from the fixed cantilever end. The diagonal reinforcement would be kept throughout all the length, eventhough, at other sections, less stirrups are needed. All diagonal tension shall be resisted by bent bars, either from supplements, or from steel actually present but not needed for other purposes. To keep the same dimensions of steel, No. 11 bars supplements will be used.

The concrete at the section of maximum shear, resists a total of 0.02 x 2,500 x 20 x 0.866 x 37 = 31,200 lbs. Therefore, the steel has to resist 91,000 - 31,200 or 59,800 lbs. and the spacing between every two inclined bars is, (see page 88 of Urquhart)

$$s = \frac{3.14 \times 20{,}000 \times 0.866 \times 37}{0.7 \times 59{,}800} = 47 \text{ in.}$$

A spacing of 30 in. between every two bent bars shall be used throughout.

2) End - Span Of Beams "A".- Assume a span of 60 ft.

A) Dead Load Moments.- The dead load moments of the end span given below, are the same as the dead load moments of the simply supported middle span designed on page 20, chapter III. This is due to the fact that both spans are equal in length and both have the same section.

M_{max} = 7,450,000 in-lb (page 22)

M_{20} = 6,600,000 in-lb (page 24)

M_{10} = 4,120,000 in-lb (page 24)

It has to be noticed, that the moment due to the enlarged section next to the cantilever fixed end is very small and therefore has been neglected.

B) Net Live Load Moments.- Similarly, the net live load moments has been taken from the simply supported middle span due to the same reason.

M_{max} = 5,660,000 in-lb (page 23)

M_{20} = 5,150,000 in-lb (page 25)

M_{10} = 3,550,000 in-lb (page 25)

Fig. 23, shows the dead load, live load, and combined moment diagram for both the cantilever and end span. The combined diagram has been obtained by addition of ordinates. In adding up ordinates of unequal signs, a factor of safety of 2 has been counted.

CANTILEVER & END SPAN MOMENT DIAGRAMS FOR BEAMS "A"

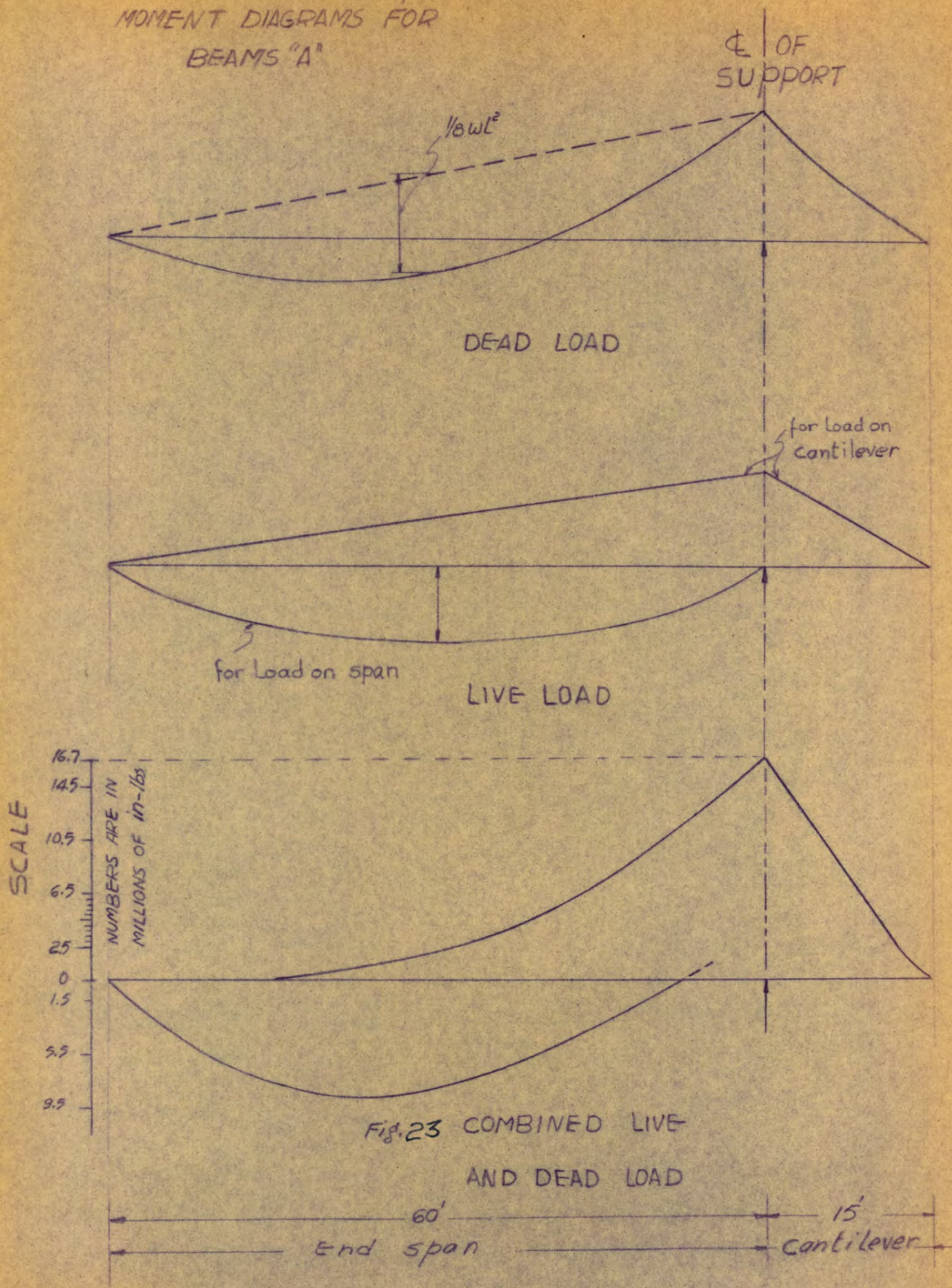


Fig. 23 COMBINED LIVE AND DEAD LOAD

C) <u>Dead Load Shears</u>.- Due to the cantilever effect, there is an uplift produced at the free end of the end span. It is specified (example, page 140 of Taylor) that the dead load shear at the free end of the end span shall be at least twice the uplift produced by the cantilever live loads.

		Free end	End nest to cantilever
Static end shear	1,375x30 =	41,250	41,250
Due to cantilever bending moment	$\frac{9,574,000}{12 \times 60}$	13,300 (-)	13,300 (+)
Due to increase of section next to cantilever		450	4,000
	T O T A L=	28,400	58,550

The uplift produced by the live loads when only the cantilever is loaded is,

$$\frac{7,100,000}{12 \times 60} = -9,850 \text{ lbs.}$$ which

is less than half the shear at the free end. Therefore, the design of the end span is safe with regard to uplift, and the assumed span length of 60 ft. shall be kept. The designer, however, tried a span length of 55 ft. and found out that the uplift produced was just over half the shear at the free end, and consequently, he considered it as unsafe.

D) Net Live Load Shear.- For the worst conditions of loadings, the cantilever shall be assumed to be loaded, while the end span shall be considered as unloaded.

	Free end	End next to cantilever
Due to truck loading	39,500	39,500
Due to cantilever bending moment		9,850
T O T A L	39,500	49,400

E) Total Shears.-

Due to all dead load causes	28,400	58,550
Due to all live load causes	39,500	49,400
∴ Total End Shears =	67,900 lbs.	107,950 lbs.

Fig. 24, shows the dead load, live load, and the combined dead and live load shear diagrams of both, the cantilever and the end span.

F) Reinforcement.- The maximum positive moment in the end span is 9,000,000 in-lb. determined from Fig. 23., and is located 25 ft. from the free end of the free end support. The steel required is therefore,

$$A_s = \frac{9{,}000{,}000}{20{,}000 \times (35 - 6/2)} = 14.0 \text{ sq.in,}$$ This

is furnished by 9 No. 11 bars placed in two rows.

CANTILEVER & END SPAN
SHEAR DIAGRAMS FOR
BEAMS "A"

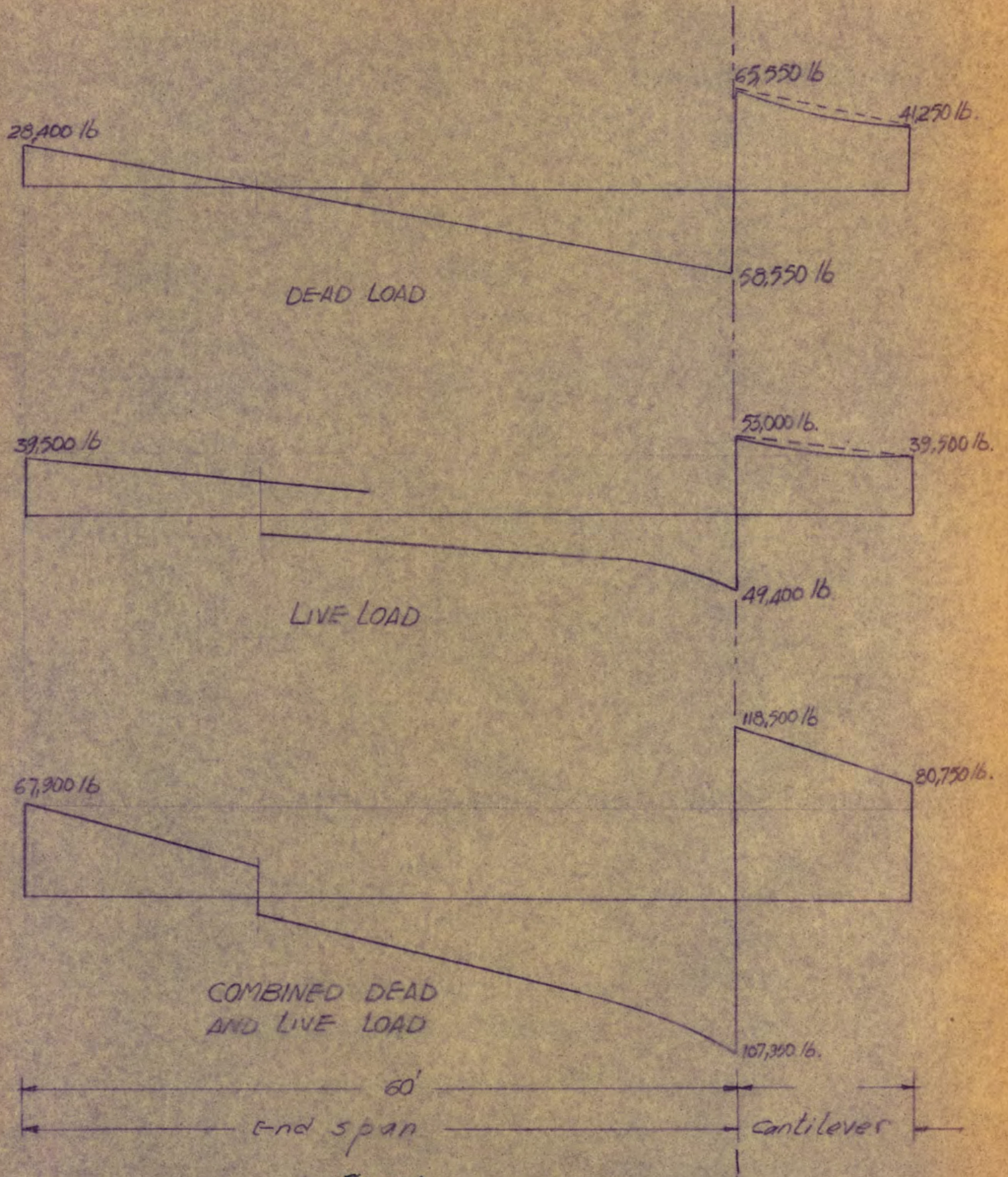


FIG 24

At 15 ft. from the support next to the cantilever end, the positive moment is 3,500,000 in-lb., and the steel area required is 14.0 x 3.5/9.0 = 5.5 sq. in. This amount is furnished by 4 No. 11 bars, and therefore, the lower of five bars shall be continued to the support in both ends. The upper row of four bars will be bent up 15 ft. from the fixed support and 5 ft. from the free end support to help resisting diagonal tension and negative bending moments.

Following, is a table giving negative moments in the end span at different sections and the respective steel areas required and the number of bars furnished.

TABLE III

NEGATIVE REINFORCEMENT IN END SPAN

Location*	(-) Moment 10^6 in-lb	A_s sq. in.	Bars supplied	Area furnished sq. in.
5	13.6	15.1	10 No. 11	15.62
10	10.0	11.1	8 No. 11	12.10
15	7.5	11.3	8 No. 11	12.10
20	5.6	9.2	6 No. 11	9.37
25	3.4	5.6	4 No. 11	6.25
30**	2.0	3.3	4 NO. 11	6.25

* Distances are from end next to cantilever.

** Beyond 30 ft., three No. 5 bars are used in top of beam until its free end. They are stirrup holders.

G) Web Reinforcement In End Span.- All the diagonal tension from the end of the cantilever, up to a distance of 25 ft. towards the free end of the span, has been practically supplied by bent bars either supplemented or bent from steel not needed over several sections. The total shear at the 20 ft. section is 50,000 lbs. (refer to Fig. 24) from which the concrete can resist a total of 0.02 x 2,500 x 20 x 0.866 x 35 = 30,000 lbs. The remaining 20,000 lbs. has to be resisted by the No. 11 bent bars. to check, for the worst condition, i.e., for 1 bent bar.

$$s = \frac{1.56 \times 20,000 \times 0.866 \times 35}{20,000} = 47 \text{ in.}$$

Therefore, a 30 in. spacing shall be used all through.

At the free end of the span, the 4 No. 11 bars resist the diagonal tension up to a distance of 5 ft. towards the other end. At that section, the total shear is, 57,000 lbs. Using two No. 3 U stirrups, the spacing would be s = 10". Referring to the shear diagram, Fig. 24. we notice that slope decreases with increase of distance. At 12 ft. towards the center, the shear is 40,000 lbs. and similarly s = 25 in.

H) Arrangement Of Stirrups.- Use two No. 3 U stirrups at 10" on centers up to a distance of 12' towards the cantilever end, with the first stirrup placed above edge of support. Also use two No. 3 U stirrups at 20 in. on centers up to a distance of 23 ft. from the fixed end, and use one No. 3 U stirrups for the rest of the end span and cantilever. Eventhough, this is not required, the stirrups are used to tie to with bottom bars and to hold in place the inclined bars in both portions.

Fig. 25 shows transverse and longitudinal sections through the end span and cantilever with details of main reinforcement and arrangement of incline bars and vertical stirrups. All are drawn to scale.

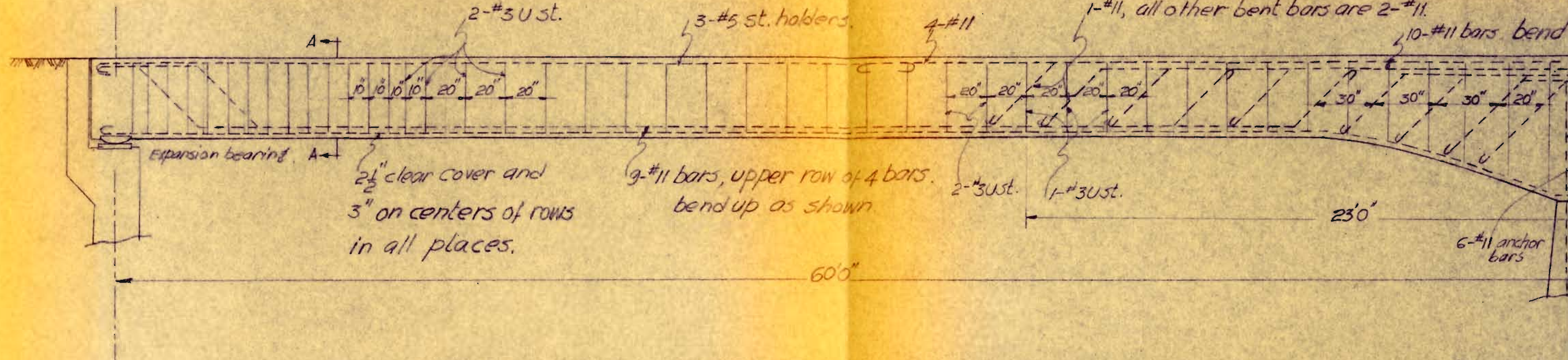
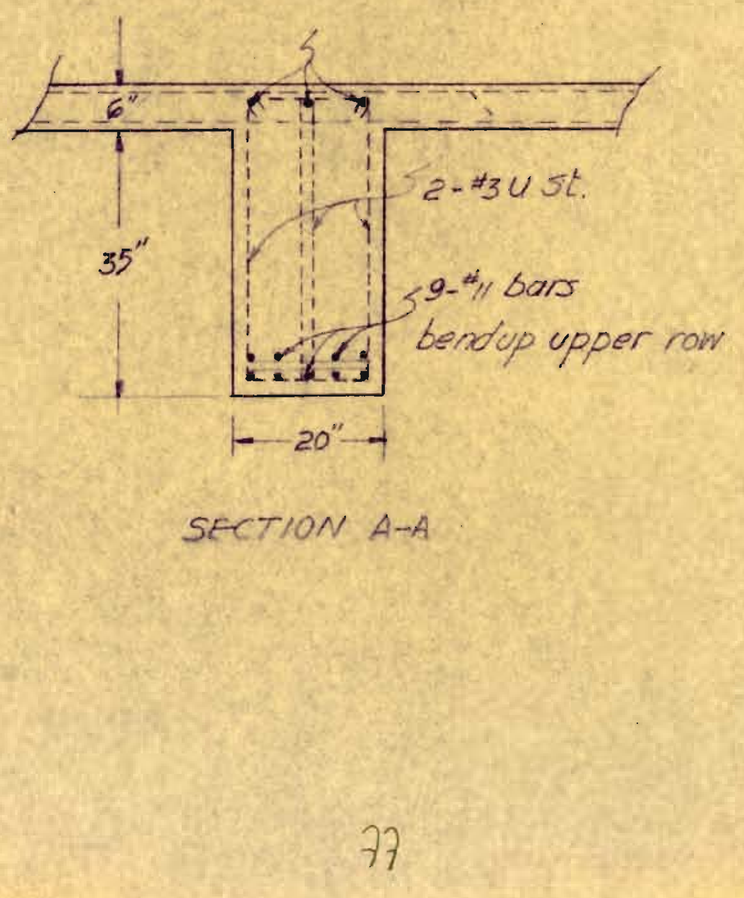
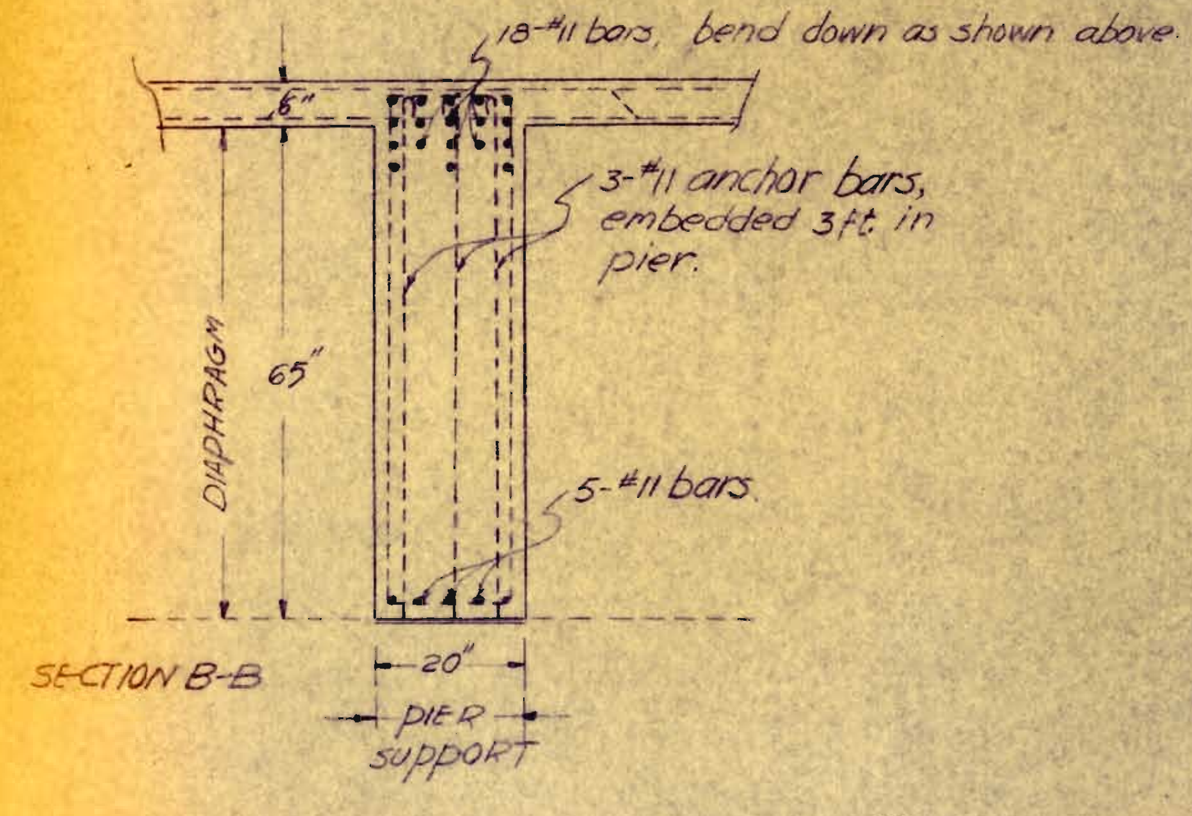


FIG.25 LONGITUDINAL SECTION THROUGH END-SPAN AND CANTILEVER SHOWING RE

scale: 1 cm : 20"



SECTION A-A



SECTION B-B

77 77

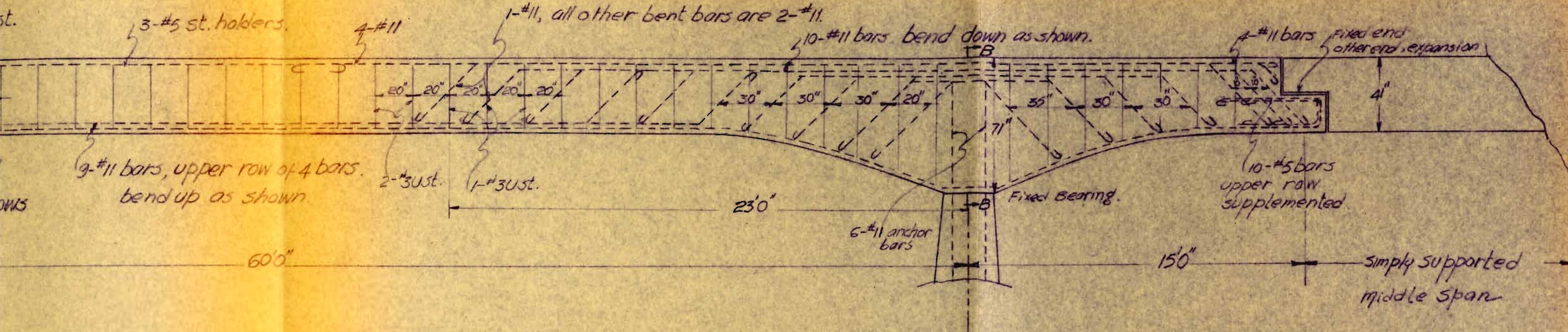
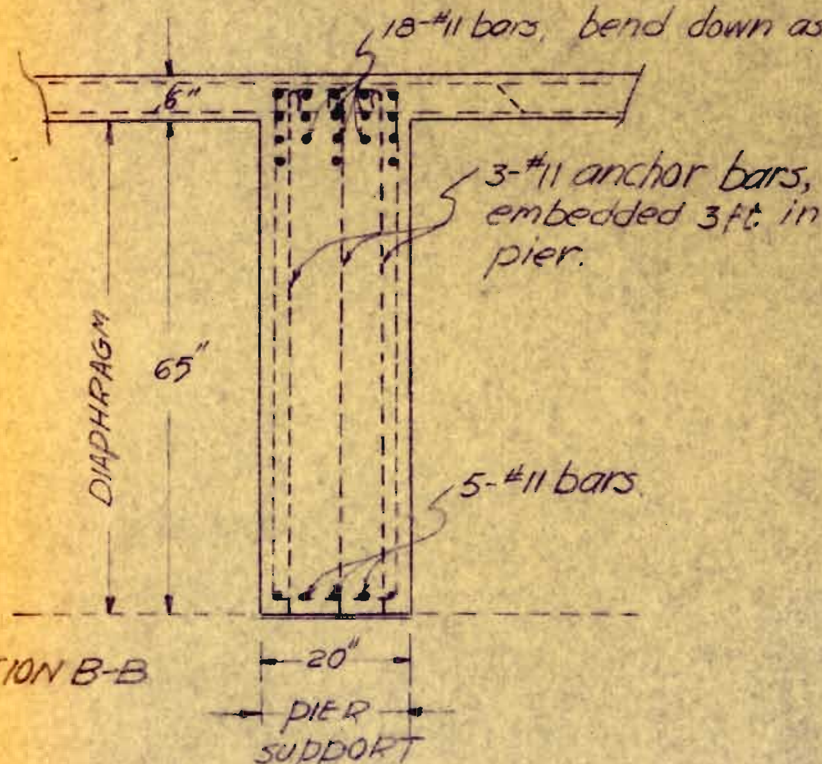


FIG.25 LONGITUDINAL SECTION THROUGH END-SPAN AND CANTILEVER SHOWING REINFORCEMENT

scale 1"=20"



SECTION B-B

IMPORTANT :-

1) For vertical and horizontal stirrups of cantilever bracket, refer to fig. 12

2) All bent bars are 45° bends.

3) For fixed end of simply supported middle span, use 3-#11 vertical bars/beam embedded to the end of both brackets.

4) For overlap of bars, provide 40 bar diameter Laps in tension members.

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3) Cantilever Of Beams "B".-

A) Calculations For The Maximum Moment.-

1. Dead Load Moment.- The section at the end of the cantilever is 20" x 80".

The dead load from the simply supported span acting at the free end of the cantilever is 52,500 lbs. (see page 34)

Therefore, the moment due to this load at the end of the cantilever is,

$$M = 52{,}500 \times 15 \times 12 = 9{,}450{,}000 \text{ in-lb.}$$

and the moment due to the dead weight of the cantilever, is,

$$M = 1{,}750 \times 15 \times \frac{15}{2} \times 12 + \frac{30}{12} \times \frac{15}{2} \times \frac{20}{12} \times 150 \times \frac{15}{3} \times 12$$

$$= 2{,}640{,}000 \text{ in-lb.}$$

The total dead moment is therefore,

$$M = 12{,}090{,}000 \text{ in-lb.}$$

2. Live Moment.- To get the maximum live moment at the end of the cantilever, the truck-trian position should be as shown in Fig. 21. and the maximum concentrated live load is 17,000 lbs. (page 37)

The maximum moment due to this live load is,

$$M = 17{,}000 \times 15 \times 12 = 3{,}060{,}000 \text{ in-lb.}$$

and the total maximum live and dead moment is = 15,150,000 in-lb.

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and $d = \left(\frac{15,150,000}{20 \times 196}\right)^{\frac{1}{2}} = 62''$

Allowing for two rows of steel, the effective depth would be 75", which is 13 in. too safe. The section need not be changed because this part is only a small fraction of the whole structure, and the extra cost of concrete would be offbalanced by the savings in steel.

The steel area required is

$$As = \frac{15,150,000}{20,000 \times 0.866 \times 75} = 12.0 \text{ sq. in. which}$$

is furnished by twelve No. 9 bars arranged in two rows.

B) Calculations For Moments At Intermediate Points.- The moment at the 10 ft. section from the free end need not be calculated, as all the steel shall be carried towards the free end to be bent down for diagonal resistance of stresses.

At 5 ft. from the cantilever free end, and due to middle span load, the moment is

$$M = 52,500 \times 5 \times 12 = 3,150,000 \text{ in-lb.}$$

and due to its own weight,

$$M = 273,000 \text{ in-lb.}$$

The total dead moment is

$$M_t = \underline{3,423,000} \text{ in-lb.}$$

Due to live load, the moment is,

$$M_l = 17,000 \times 5 \times 12 = 1,020,000 \text{ in-lb.}$$

The total dead and live load moment is

$$M_t = 4,443,000 \text{ in-lb.}$$

and the area of steel required is therefore 3.5 sq. in. Four No. 9 bars furnishing an area of 4.0 sq. in. will be contimued until the end. The other bars shall be bent down to resist the diagonal tension developed in the bracket due to sudden contraction of the section.

C) Shears.-

1. Dead Load Shear At Intermediate Points.-

$$V_{max} = 52,500 + 1,750 \times 15 + \frac{30}{12} \times \frac{20}{12} \times \frac{15}{2} \times 150$$

$$= 83,400 \text{ lbs.}$$

Similarly, at 10 ft. from the cantilever free end,

$V_{10} = 72,100$ lbs.

and $V_5 = 61,800$ lbs.

$V_0 = 52,500$ lbs.

2. Live Load Shear.- The truck-train positions to give the maximum live load shears at intermediate points is as shown in Fig. 22. and the net total live load shear is obtained by multiplying the net shears of beams "A" by the ratio, 0.53/ 1.23 = 0.43 (page 35)

$\therefore$ $V_{max} = 53,000 \times 0.43 = 22,800$ lbs. (page 49)

Similarly,

$V_{10} = 19,800$ lbs. (page 49)

and $V_5 = 18,300$ lbs. "

$V_0 = 17,000$ lbs. "

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4) End Span Of Beams "B".-

A.) Dead Load Moments.- The following dead load moments are the same as the dead load moments of the simpply supported middle span designed in chapter III .

$$M_{max.} = 9,450,000 \text{ in-lb.}$$

$$M_{20} = 8,420,000 \text{ in-lb.}$$

$$M_{10} = 5,250,000 \text{ in-lb.}$$

B) Net Live Load Moments.- Similarly, the net live load moments has been taken from the simply supported middle span due to the same reason.

$$M_{max} = 2,440,000 \text{ in-lb.}$$

$$M_{20} = 2,220,000 \text{ in-lb.}$$

$$M_{10} = 1,530,000 \text{ in-lb.}$$

Fig. 26. shows the dead load, live load, and the combined moment diagram for both, the cantilever and end span. When ordinates of unequal signs were added, a factor of safety of 2 was counted for.

C) Dead Load Shear.-

		free end	end next to cantilever
Static end shear		52,500	52,500
Due to cantilever bending moment	$\frac{12,090,000}{12 \times 60}$	16,800 (-)	16,800
Due to increase of section next to cant.		450	+4,000

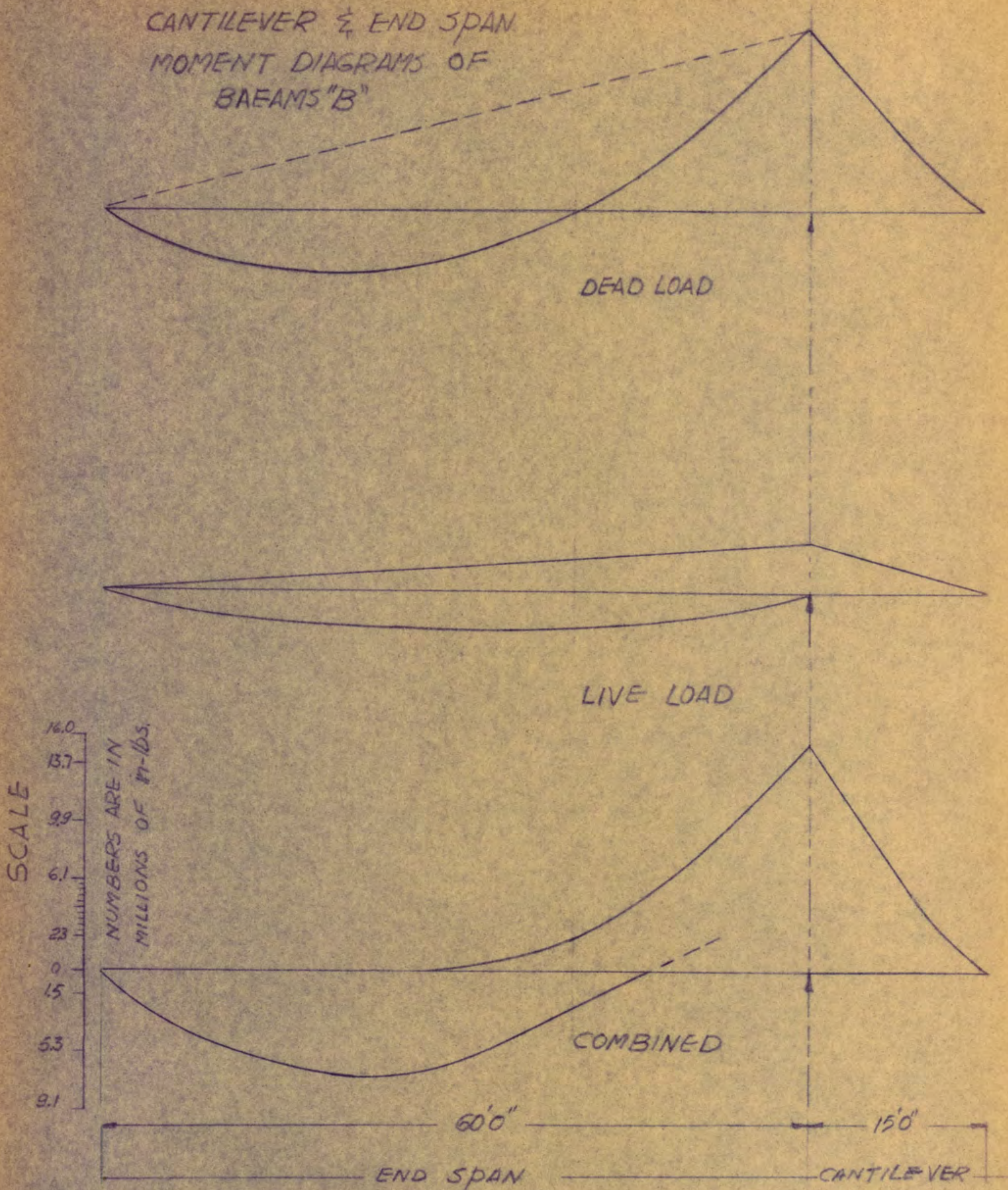
CANTILEVER & END SPAN
MOMENT DIAGRAMS OF
BAEAMS "B"
DEAD LOAD
LIVE LOAD
COMBINED
SCALE
NUMBERS ARE IN
MILLIONS OF in-lbs.
16.0
13.7
9.9
6.1
2.3
0
1.5
5.3
9.1
60'0"
15'0"
END SPAN
CANTILEVER

Fig. 26

The total free end shear is 36,350 lbs. and that next to the cantilever end is 73,300 lbs.

The uplift produced at the free end of the span when only the cantilever is loaded, is

$$\frac{3,060,000}{12 \times 60} = -\ 4,250 \text{ lbs.}$$

This is less than half the shear at the free end and therefore the design is safe.

D) Net Live Load Shear.-

	Free end	End next to cantilever
Due to truck loading	17,000	17,000
Due to cantilever bending moment		4,250
T O T A L	17,000	21,250

E) Total Shears.-

Due to all dead load causes	36,350	73,300
Due to all live load causes	17,000	21,250
T O T A L =	53,350 lbs.	94,550 lbs.

Fig. 27 shows the dead load, live load, and the combined shear diagrams of both, the cantilever and end-span.

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CANTILEVER & END SPAN SHEAR DIAGRAMS OF BEAMS "B"

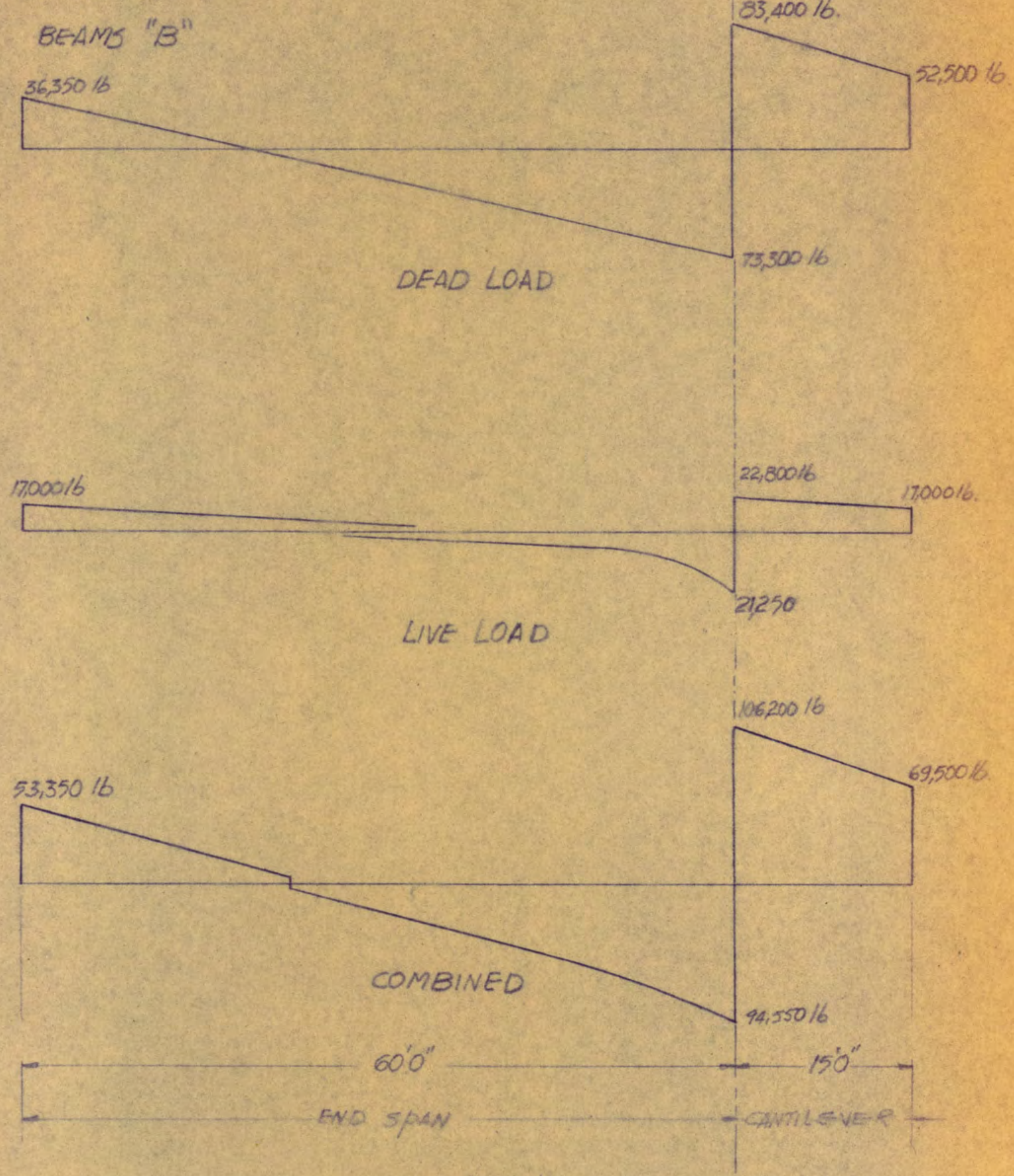


Fig. 27

F) Reinforcement.- The maximum positive moment in the end span is 7,300,000 in-lb. (refer to Fig. 26) and is located 25 ft. from the free end support. The area required is

$$A_s = \frac{7,300,000}{20,000 \times 0.866 \times 44} = 10.0 \text{ sq. in. which}$$

is furnished by ten No. 9 bars placed in two rows, five bars in each.

At 7 ft. and 38 ft. from the free end support, only five No. 9 bars are required and therefore the uppor row of five bars may be bent up at that section to resist the diagonal tension. The lower row, also of five bars, shall be continued straight into both supports.

For the negative moment, the steel required and that supplied at different sections with the corresponding moments are best shown by the following table on page 63.

G) Web Reinforcement.- All the diagonal tension from the end of the cantilever to a distance of 20 ft. towards the free end of the span, shall be taken care by No. 9 bent bars. At the 20 ft. section, the shear is 40,000 lbs., of which the concrete can resist a total of 38,400 lbs. leaving only 2,000 lbs. to be resisted by vertical stirrups. At the free end, the maximum shear has also been resisted by bent bars and almost no shear has been left to be resisted by vertical stirrups. However, some have to be used.

Therefore, use number 3 U stirrups 20" on centers throughout including the cantilever portion.

TABLE IV

NEGATIVE REINFORCEMENT IN END SPAN

Location*	(-) Moment 10 in-lb	A_s sq. in.	Bars supplied	Area furnished sq. in.
5	11.2	10.5	12 No. 9	12.0
10	7.6	9.3	12 No. 9	12.0
15	5.0	6.6	8 No. 9	8.0
20	2.2	2.9	4 No. 9	4.0
25**	0.5	0.8	4 No. 9	4.0

* Numbers under column headed location represent distances from the fixed end of the cantilever.

** Beyond the 25 ft. section, 2 No. 9 bars are continued until the free end of the end span.

Fig. 28 shows a detailed drawing with longitudianal and transverse sections through the cantilever and end span. Main top and bottom reinforcement, bent bars, and vertical stirrups are all shown to scale.

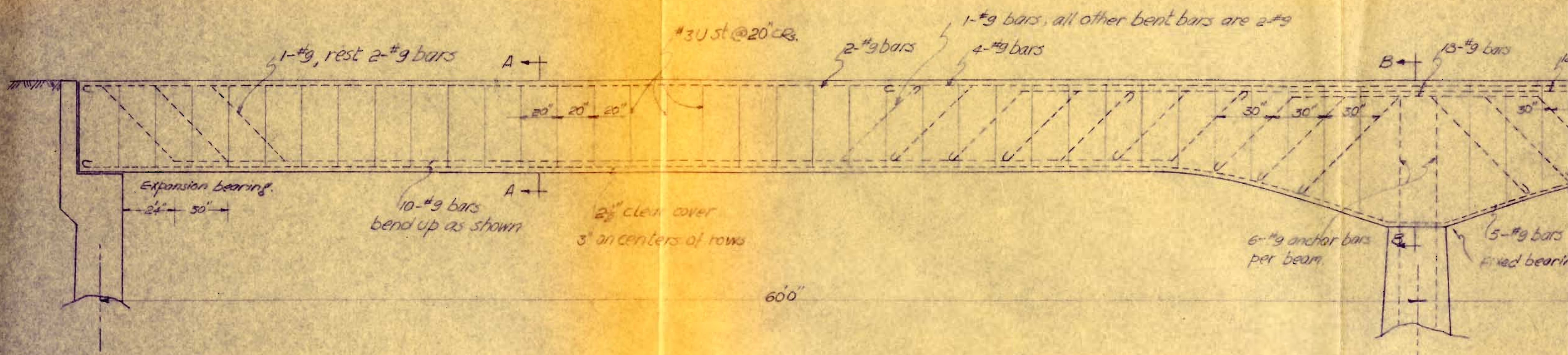
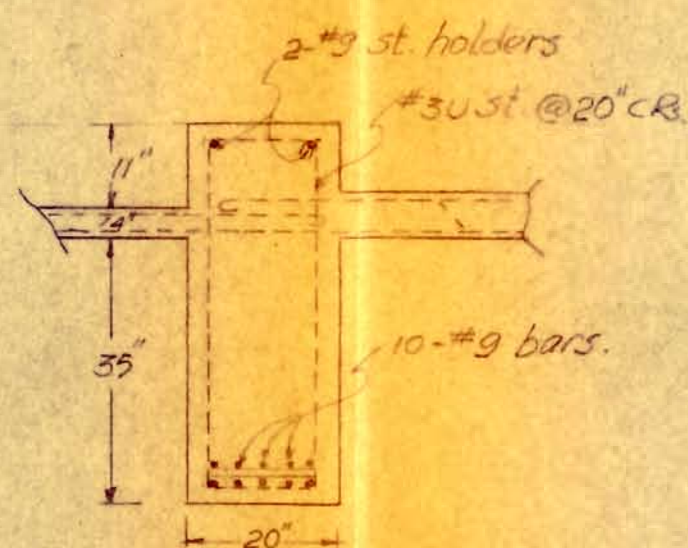
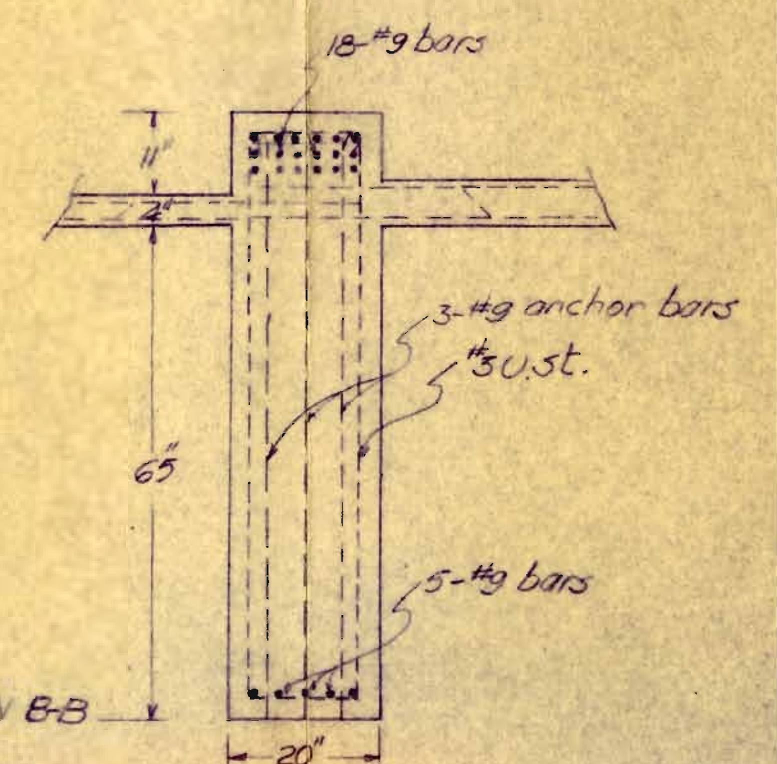


FIG 28 LONGITUDINAL SECTION THROUGH CANTILEVER AND END SPAN

Scale 1cm:20"



SECTION A-A



SECTION B-B

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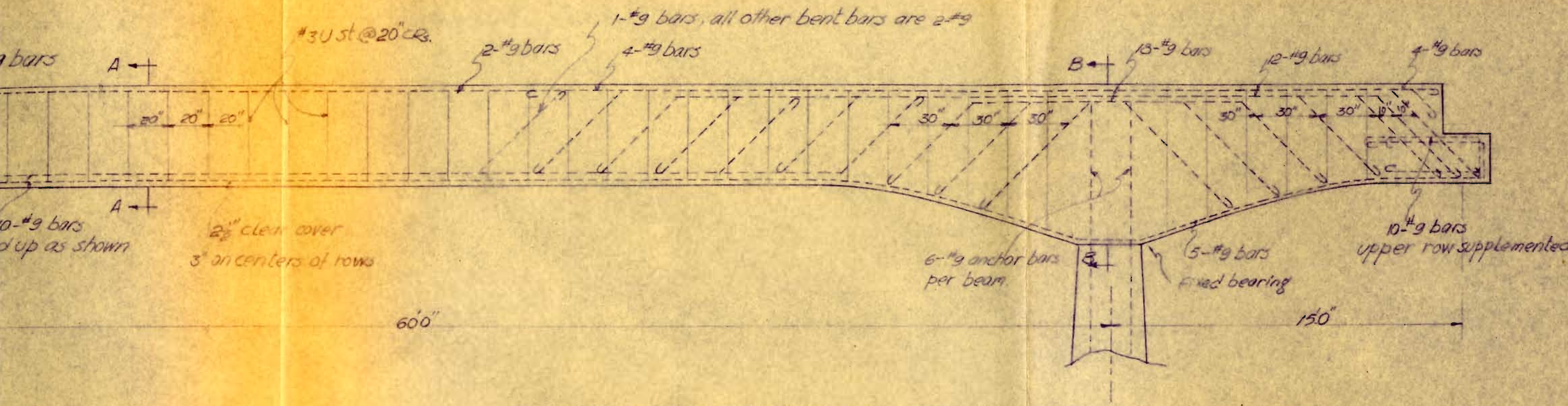
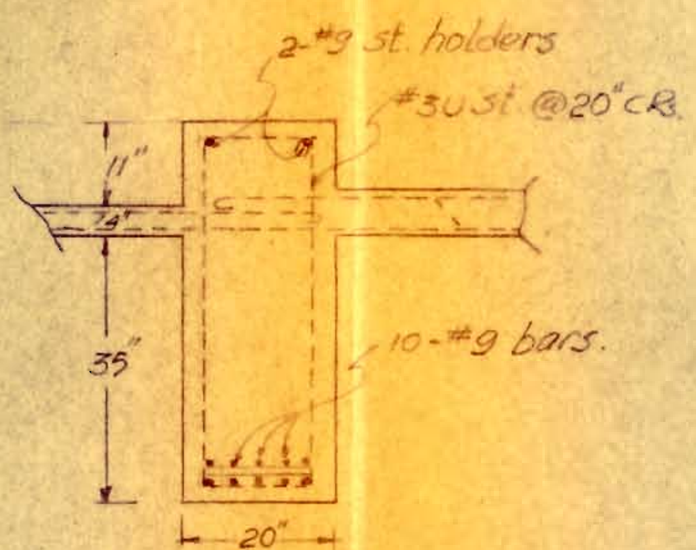
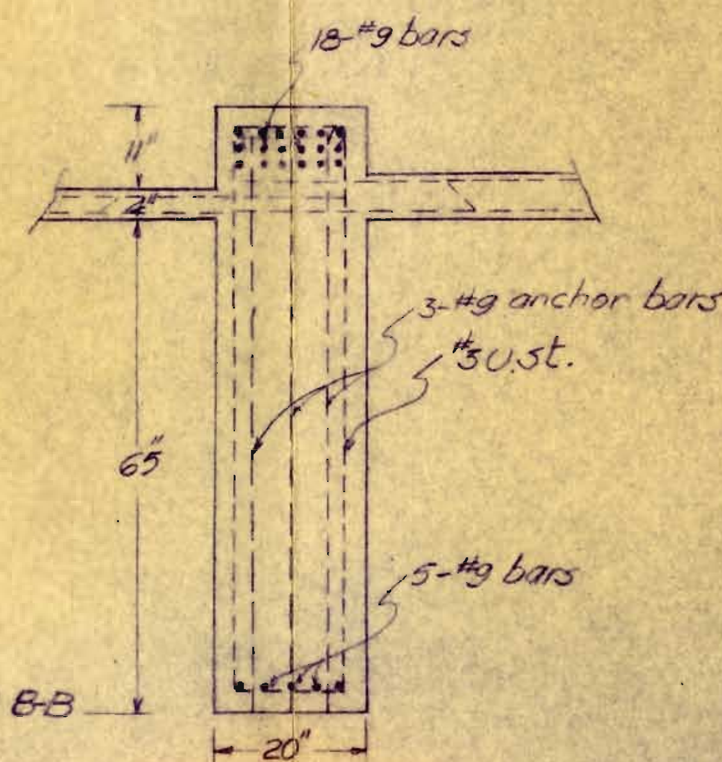


FIG 28 LONGITUDINAL SECTION THROUGH CANTILEVER AND END SPAN

Scale 1cm:20"



SECTION A-A



SECTION B-B

5) Cantilever And End Span Design Of Beams "C".-

A) Cantilever.- The section at the end of the cantilever is 14" x82".

The dead load from the simply supported span acting at the free end of the cantilever is 47,550 lbs. and the moment at the end of the cantilever due to it, is

$$M = 47,550 \times 15 \times 12 = 8,600,000 \text{ in-lb.}$$

and due to the dead weight of the cantilever, the moment is

$$M = 2,610,000 \text{ in-lb.}$$

The total dead moment is,

$$M_t = 11,210,000 \text{ in-lb.}$$

And hence, the section is very safe.

The steel area required is,

$$A_s = \frac{11,210,000}{20,000 \times 0.866 \times 76} = 9.0 \text{ sq. in. which}$$

is furnished by nine No. 9 bars arranged in three rows, 3 bars in each.

1. Shears.-

At the fixed end of the cantilever	= 74,850 lbs.
At 5' towards the free end	= 66,000 lbs
At 10' towards the free end	= 57,000 lbs.
At the free end	= 47,550 lbs.

B) End Span.- The maximum moment occuring at the center of the span is 8,600,000 in-lb. This moment includes the live load moment due to pedestrians' traffic as well as the dead loads' moment.

1. Shears in End Span.-

End shear	=	47,550 lbs.
At 10' from support	=	31,700 lbs.
And at 20'	=	15,850 lbs.

C) Reinforcement.- The maximum net positive moment in the end span is 6,000,000 in-lb and is located 25 ft. from the free end. (Refer to Fig 29.) The steel area required is

$$A_s = \frac{6,000,000}{20,000 \times 0.866 \times 56} = 6.3 \text{ sq. in.}$$

This is furnished by seve No. 9 bars arranged in two rows, the lower row being of four bars to be continued straight into the supports. The remaining three No. 9 bars shall be bent up.

D) Web reinforcement.- The maximum shear in the end span is just next to the end of the cantilever and is 50,000 lbs. (Refer to Fig 29) At that section, the concrete can resist a total of 46,500 lbs. and the remaining 3,500 lbs to be resisted by bent bars. In the middle portion, no web reinforcement is needed. This is evident from the shear diagram of Fig. 29. However, some have to be used. Therefore, use No. 3 U stirrups at 20 in. on centers

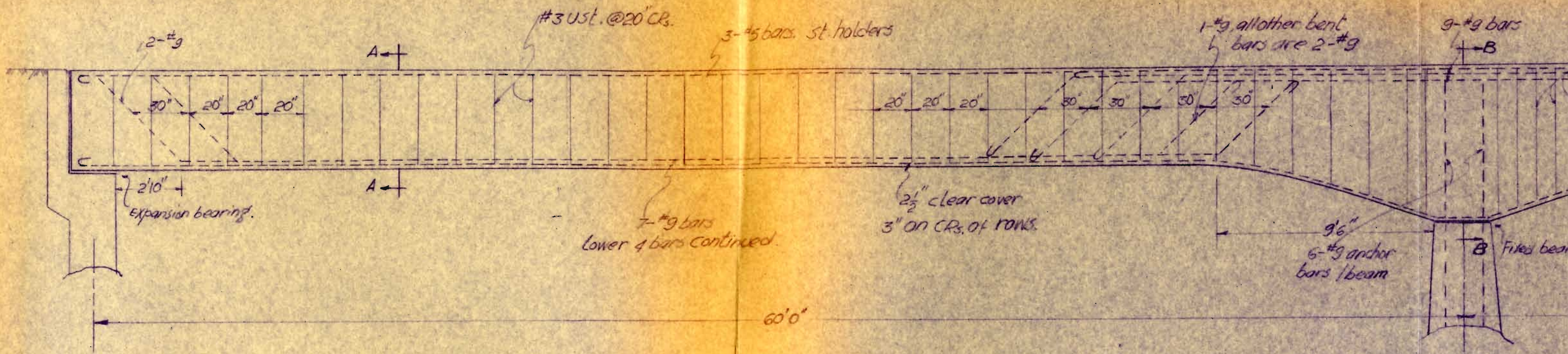
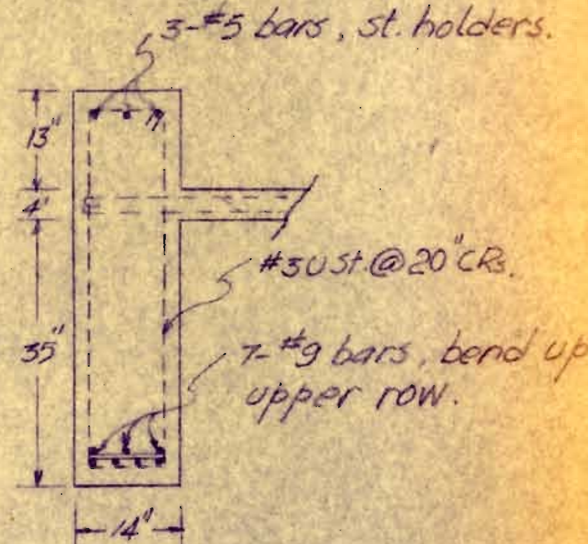
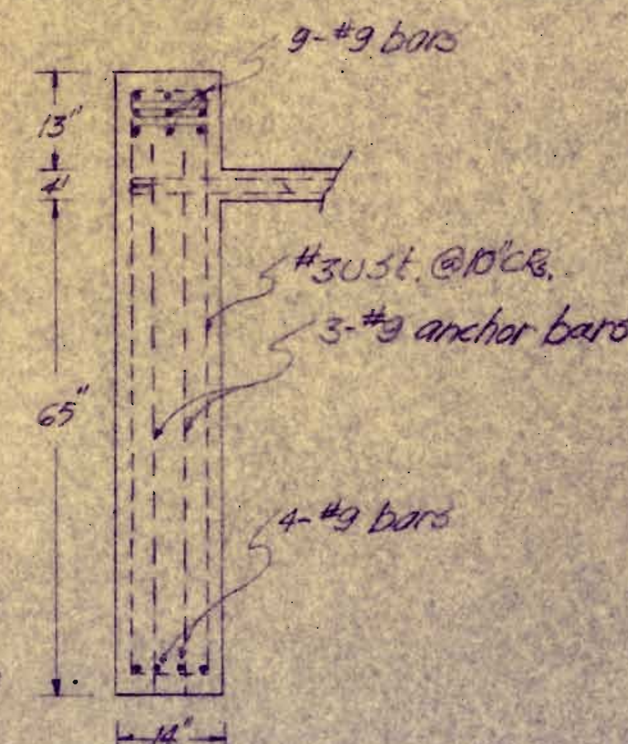


FIG. 30 LONGITUDINAL SECTION THROUGH CANTILEVER AND END SPAN

Scale: 1cm : 20"



SECTION A-A



SECTION B-B

90

90

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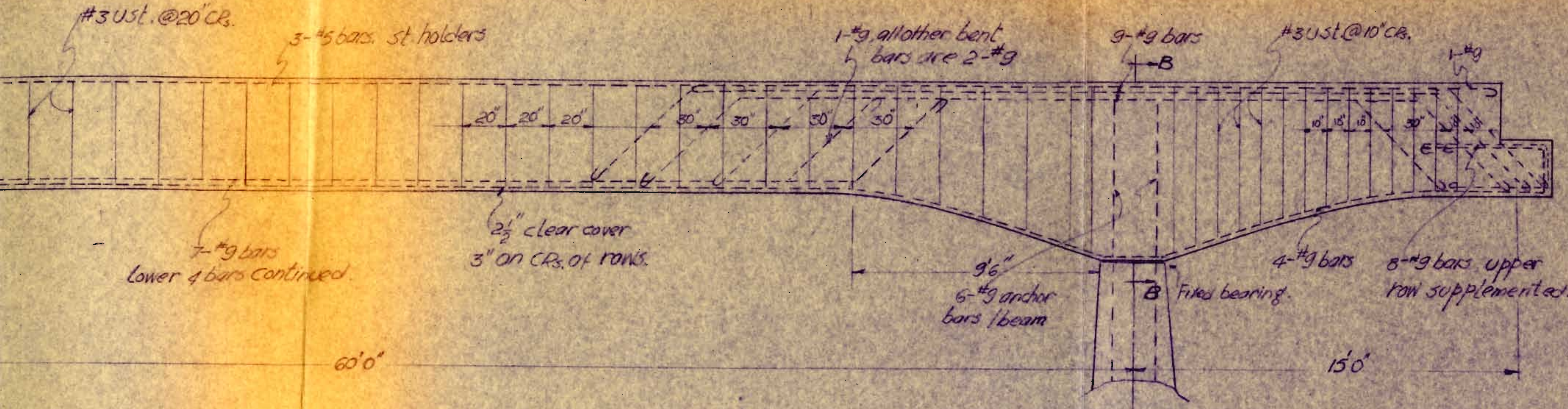
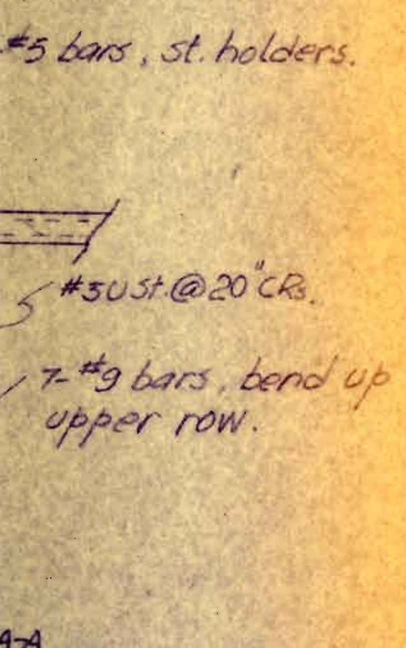
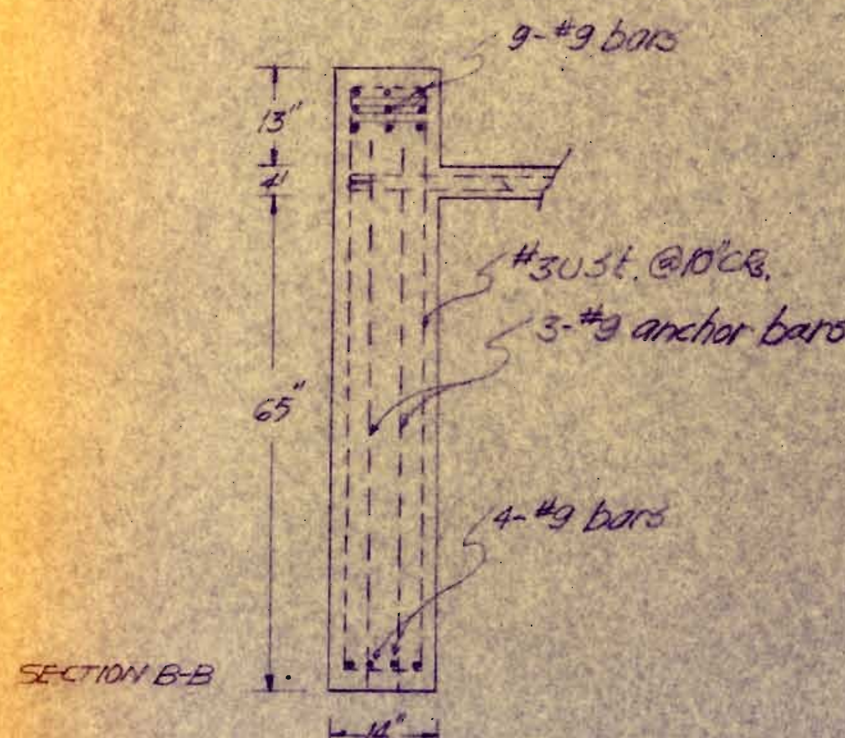


FIG. 30 LONGITUDINAL SECTION THROUGH CANTILEVER AND END SPAN

Scale: 1cm: 20"





throughout the length of the end span.

In the cantilever portion, the maximum shear is 74,850 lbs. The steel has therefore to resist a total of 74,850 - 46,000 = 28,850 lbs., and at 10' from the cantilever end, the shear is 66,000 lbs. from which, the concrete can resist 26,000 lbs. It is evident therefore, that the maximum unit shear occurs at the fixed end of the cantilever and the spacing between vertical U stirrups, using No. 3 bars all through the cantilever is s = 10".

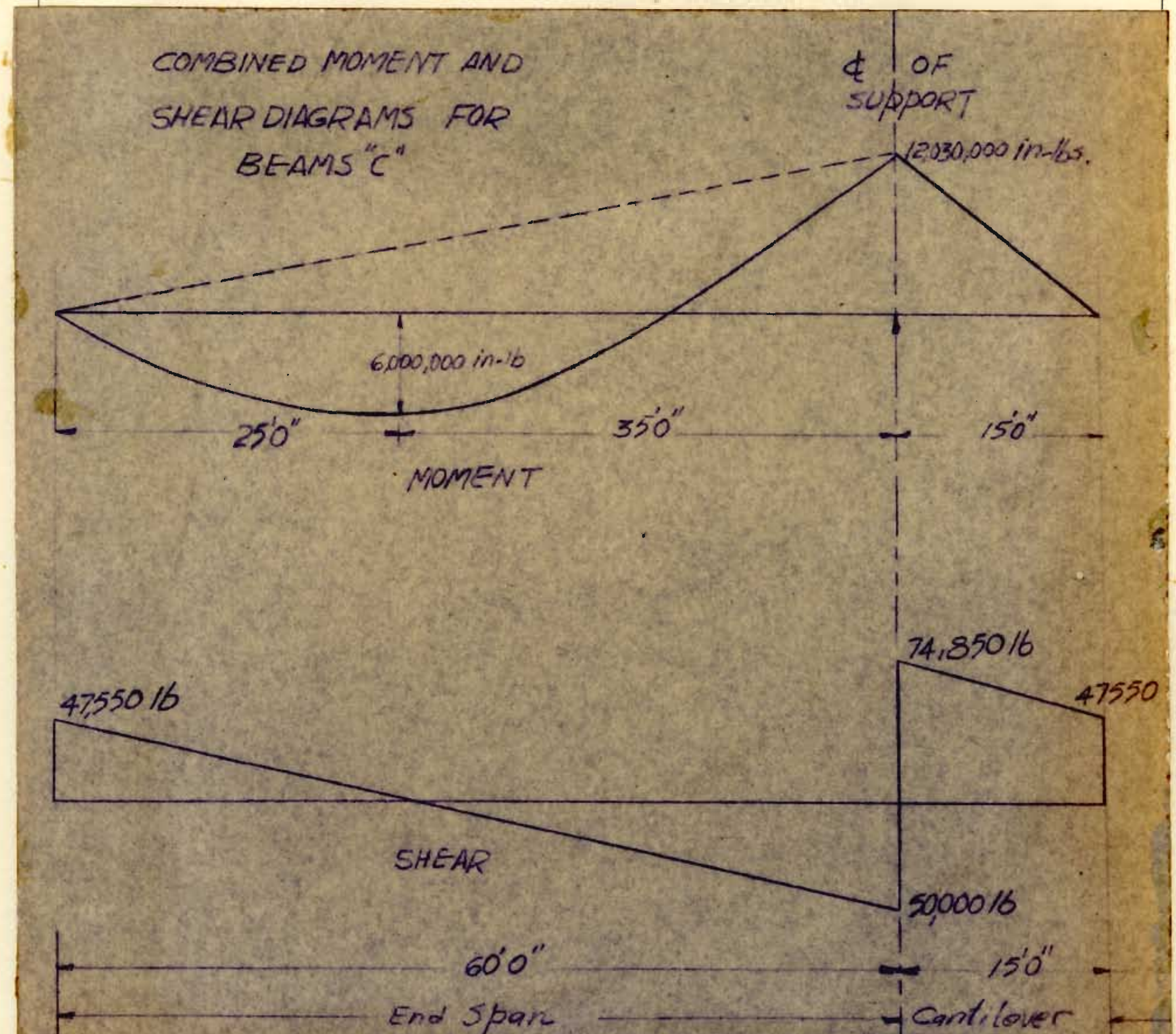


Fig. 29

6) Miscellaneous General Details.-

A) Cross Beams Or Diaphragms.- The main functions of cross beams are, to stiffen the girders laterally, and to equalize in a partly loaded bridge the deflection of the girders carrying heavy loading with those of the girders carrying partial loading.

There is no defenite rule for the spacing of cross beams; in German practice, they require a spacing of not more than one quarter the span length, nor more than 2.5 times the lateral spacing, whichever is less, of the main beam center lines. In the united states, many bridges were built without any cross beams, however, the Taylor Reinforced Concrete Bridges recommend a spacing of not more than 20 ft. on centers.

Therefore, a 12" x 41" cross beams spaced at 15 ft. on centers shall be used in this design. At the pier supports, beams of 24" x 71" shall be used. The respective steel areas being 0.003 x 12 x 41 = 1.5 sq.in. and 0.003 x 71 x 24 = 5.0 sq. in. Actually, no steel is required in beams above pier supports, but they are furnished to ensure proper uniform distribution of the loads from the main girders along the piers.

At the hinges, the cross beams shall be arranged in such a way to fix the sections of the middle span and cantilever. Fig. 31, shows a cross beam in the simple span.

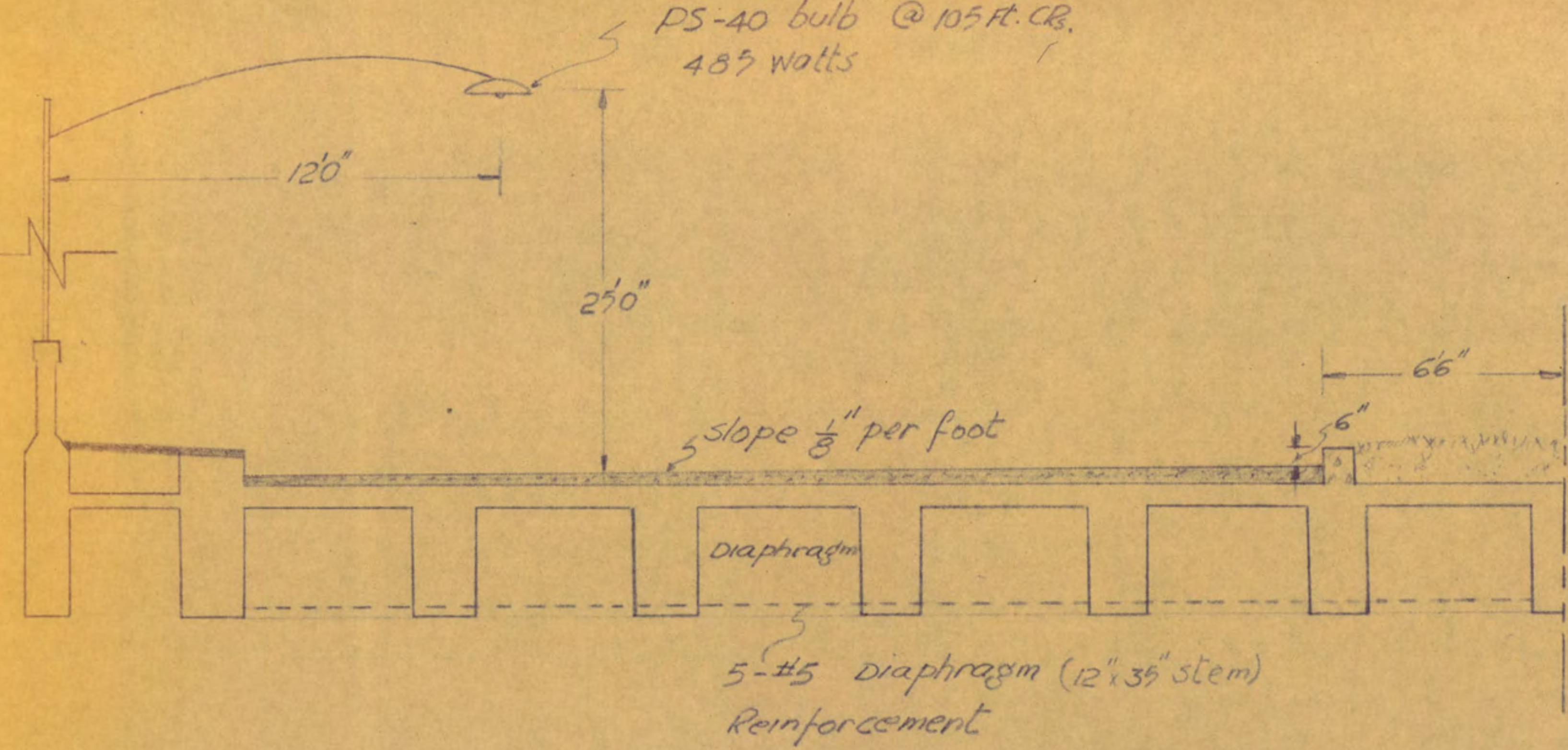


FIG 31. Details of Median strip, Drainage, Diaphragm, and illumination.

B) Fixed Bearings.- At one end of the simply supported middle span, and on the two piers, fixed bearings are necessary for the bridge.

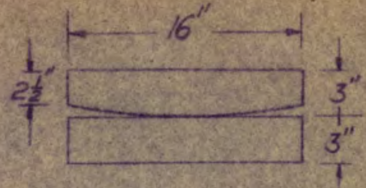
At the middle span of beams "A", three No. 11 bars per beam, placed 1 ft. from either end of the bracket, shall be used to connect the two brackets together. This steel does not resist any appreciable amount of bending moment and therefore, they were not designed for.

Similarly, at the pier supports, six No. 11 bars shall be used, arranged as shown in the respective sections. Notice that, for beams "B" & "C" , the same number of the steel is used but of the No. 9 size.

C) Expansion Bearings.- At all supports where fixed bearings are not used, expansion bearings shall be provided.

The expansion bearings of this bridge are adopted from Urquhart, page 401. They consist of pairs of steel plates placed under each girder. The lower plate shall be anchored to the concrete in the abutment, and the upper plate shall be fastened to the under side of the girder. A thin layer of zink is to be placed between the plates to reduce the friction.

Fig. 32,[a] shows a detailed section of the expansion bearings. The upper plate has a curved surface to allow



16"x 16" Expansion Bearing steel plates.
to be used all through except in beams "C"
instead, a 10"x10" bearing plates are used.

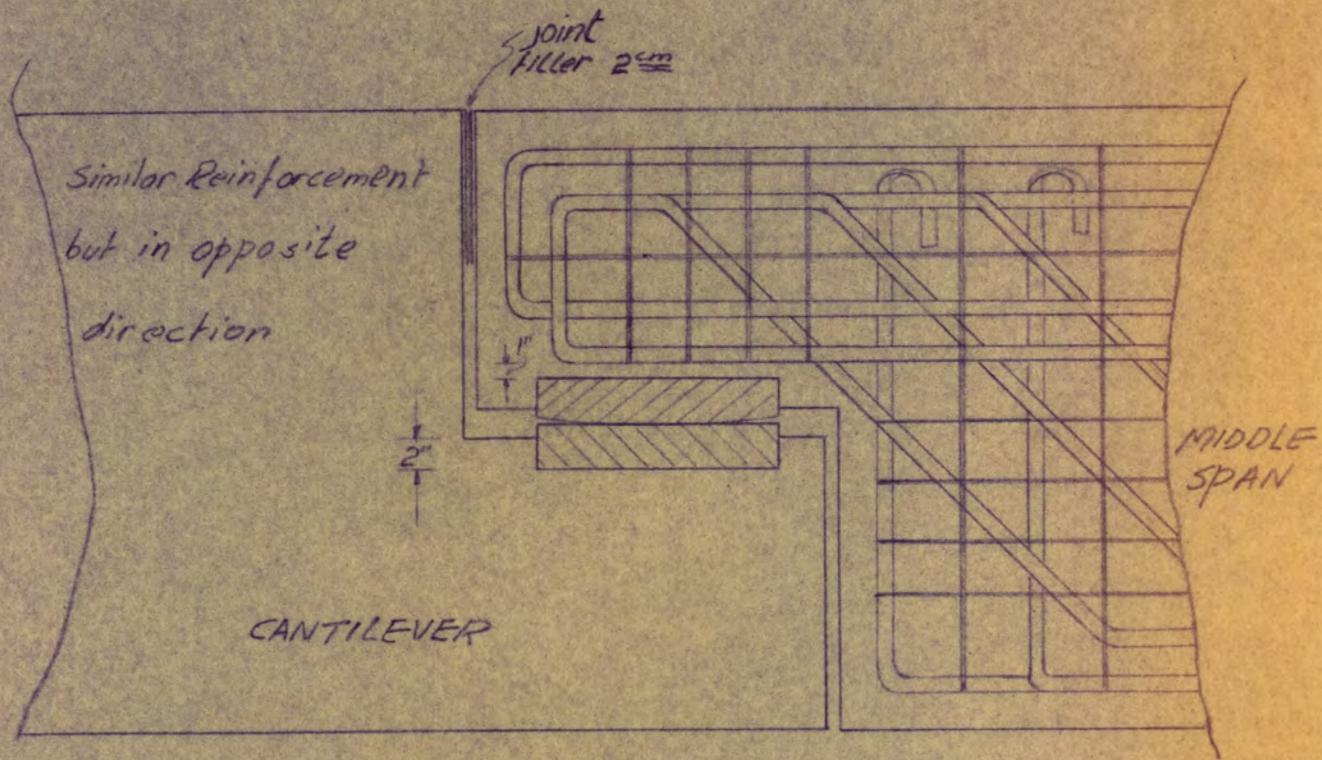


FIG 32(a) SECTION THROUGH BRACKETS OF CANT. AND MIDDLE SPAN SHOWING EXPANSION BEARING AND TYPICAL REINF. TO BE APPLIED IN ALL SIMILAR CASES.

for rotational movement, as well as horizontal, without causeing serious stresses to the members.

D) Expansion Joints.- At all points where expansion bearings are used, a small clearance shall be provided to allow for the expansion of the members due to temperature changes. Its depth along the longitudianl direction depends on :-

a. Local range of temperature change, assumed to be 30 degrees C. = 86 degrees F.

b. Length of the member to expand, Maximum length to be taken care by one expansion bearing is 90 ft.

c. Coefficient of expansion in concrete, = 0.000,005,5

Therefore, the total change in length is,

0.000,005,5 x 86 x 90 x 12 = 0.5" , and hence allow a clearance of 2 cm. to be filled with mastic.

E) Wearing Surface.- The wearing surface along the bridge shall be the same as that of the roadway, but not to exceed 2 1/2 in, nor to be less than 2 in.

F) Drainage.- To dispose of the precipitable water, crowning of the structure in both ways shall be used, since it has been decided that the Boulevard shall be level. The amount of crowning being 1/8 in. per foot. Therefore, at the center of the bridge, the rise shall be 106 x 1/8 or 13 in. decreasing uniformly to both ends of the bridge.

G) Median Strip.- The height of the median strip along the bridge shall be 6 in. higher than the surface of the road. Curbs shall be made of concrete, and casted monolothically with the slab. For details, see Fig. 31.

H) Illumination.- The bridge shall be illuminated for heavy pedestrian and medium traffic. Referring to Boast, Illumination Engineering, the following can be found out :-

Illumination required	=	1.0 ft. c.
Lamp flux	=	10,000 lumens
Mounting height	=	25 ft.
Spacing on centers	=	105 ft.
Bulb required	=	PS-40
Wattage	=	485 watts.
List price	=	1.85 dollars

In addition, lamps shall be 12 ft. from the center of the parapet towards the center of the roadway. The poles shall be 4 in. round empty pipes.

Fig. 31, shows the details of this illumination.

CHAPTER V

DESIGN OF PIERS AND ABUTMENTS

1) Design Consideration.- Due to the unavailability of subsurface borings from a convenient depth in the locality, the determination of the bearing capacity of the soil was impossible. However, to complete the design of the structure, the soil is assumed to be a mixture of sand and gravel. The characteristics for this type of soil are, (after Urquhart)

Unit weight	=	120 pcf.
Internal angle of friction	=	30 degrees
Coefficient of friction	=	0.5

The bearing capacity is assumed to be 6,000 psf., and the surcharge before the abutment to be 2 ft. of earth. The second assumption is based on the AASHO Specifications.

The design of piers and abutments is a matter of successive approximations. Reasonable dimensions are assumed and the various conditions of stability are checked for these dimensions. After the results of the first trial, the designer would then be able to tell whether dimensions are too safe or not safe, and accordingly, dimensions should be reassumed, and again rechecked, until proper design is obtained.

2) Design Of Piers.-

A) General.- The piers shall be designed of Plum concrete faced with masonary due to the following reasons:-

a. It is more economical,

b. For architectural purpose.

The proportions of the mixture shall be 60 % concrete and 40 % stones, giving an allowable stress of 8 kg. per sq. cm. which is equivalent to 110 psi.

The forces acting on the pier are,

a. The vertical reactions of the superstructure, i.e. the live, impact, and dead load transferred from the bridge to the pier.

b. The weight of the pier itself.

c. The horizontal traction force usually taken as 0.2 of the live load. (This item is neglected in most highway bridges design, and neglected here, since it has been considered before.)

d. The force of the flowing water in the direction of flow, and

e. The wind forces.

The above forces shall be combined to give the most unfavorable results. The location of the resultant should be found, and to prevent overturning, this resultant should lie within the middle third of the base. Stresses should be computed and to prevent failure, these stresses should not exceed the allowable unit stresses of which the pier is built. In addition, the structure should be checked against sliding, and in no case shall the factor of safety be less than 2.

Three feet from the top of the pier, a mesh of steel shall be furnished to provide better distribution of the superstructure loadings along the pier. Dowels, acting to make a fixed support at the pier, extending from the diaphragms and eventually from the longitudinal beams of the bridge, shall be continued down to this mesh. Usual good concrete therefore, shall be used in the upper three feet of thepier.

The footing of the pier shall be designed of reinforced concrete with additional vertical dowels spaced at reasonable intervals all along the length to ensure good bond. The depth of the footing shall be governed either bymoment or shear which ever requires a bigger depth.

B) <u>Design Of The Vertical Shaft</u>.- The maximum vertical force acting on the pier from the superstructure, is, (see Fig. 24) 107,950 + 118,550 or 226,500 lbs. per 6.17 ft. length of pier.
Assuming uniform distribution, the maximum vertical force per foot would be 37,000 lbs.

1. Weight Of Pier.- Referring to the assumed dimensions of the pier in Fig. 32, we find that the weight at the bottom per unit foot of length is,

$$\frac{2.5 + 5}{2} \times 35 \times 150 = 19,600 \text{ lbs.}$$

.·. The total vertical force at the bottom of the pier, is,

$$37,000 + 19,600 = 40,000 \text{ lbs.}$$

2. Check For Strength Of Pier.- At the top, the pier can resist 2.5 x 144 x 110 = 40,000 lbs. per foot length which is more than the actual 37,000 lbs. Therefore the section is O.K.

At the bottom, the pier can resist a total of 80,000 lbs. per foot , and this is more than the actual 56,600 lbs., and hence, the assumed dimensions of the pier are safe with respect to the unit stress of plum concrete.

3. Wind Forces. Due to the absence of a map showing the location of the bridge with respect to the north, and eventually, to the prevailing wind in our country, and due to a reconnaissance in the district by the designer, it can be fairly assumed that the center line of the bridge runs parallel to the north.

Referring to Fig. 32, we notice that the wind force hits the pier at an angle of 45 degrees, thus breaking the 50 lbs. per sq. ft. force into two 35 lbs/sq.ft.

componants, one being normal to the pier and the other, parallel to it. Hence the total force acting normal to the pier per foot of width, is,

$F = 35 \times 1.5 \times 21.4 = 1{,}123$ lbs. acting at 24.3 ft. above the top of the footing.

4) Check Against Overturning.- Neglecting the horizontal thrust due to the earth behind the pier and taking moments about point A of all the forces acting on the pier, we have,

$56{,}600 \times 2.5 - 1{,}123 \times 24.3 = x \times 56{,}600$, in which x is the distance in feet from point A towards the center of the pier.

and $x = 2$ ft. which locates the resultant within the middle third of the base and therefore, the structure is safe against overturning. (Actually, in checking against overturning, moments should be taken about the edge of the footing as overturning occurs at that point.)

5. Check Against Sliding.- The friction force is

$F = 0.5 \times 56{,}600 = 28{,}300$ lbs per foot.

and the factor of safety against sliding is,

$F.S. = \frac{28{,}300}{1{,}125} = 25$ which is much higher than the specified value. (The check against sliding should have been calculated after the design of footing is finished, as the coefficient of friction is concerned with soil and concrete. The procedure followed,however, is less safe.)

Notice that no check for the stability of the pier in the transverse direction of the roadway has been attempted. This is due to the fact that it has been found very safe in the longitudinal direction, which is the critical.

6. <u>Design Of Footing</u>.- Assuning the weight of the footing to be 2,400 lbs. per foot, the required width is

$$\frac{56,600 + 2,400}{6,000} = 10 \text{ ft.}$$

The net upward pressure caused by the pier load alone is

$$\frac{56,600}{10} = 5,660 \text{ lbs./ft.}$$

Page 215 of Urquhart, Design Of Concrete Structures, specifies that for footings supporting masonary walls, the maximum moment is computed midway between the middle and the face of the pier. Therefore,

$$M = 1/8 \times 5,660(10 - 5/2)^2 \times 12 = 430,000 \text{ in-lb.}$$

and $$d = \left(\frac{430,000}{196 \times 12}\right)^{\frac{1}{2}} = 15.0 \text{ in.}$$

For determining shear stresses, the vertical shear force is computed on a section "d" away from the face of the pier (same reference) thus, the shear force is,

$$V_s = 5{,}660(2.5 - 13.5/12) = 7{,}400 \text{ lbs.}$$

and the effective depth required for shear is,

$$d = \frac{7{,}440}{12 \times 7/8 \times 50} = 14 \text{ in.}$$

It is seen that the effective depths are approximately equal in both cases and a depth of 16 in. is selected. Allowing for 4 in. insulation, the overall depth would be 20 in.

The weight of footing is then $\frac{20}{12} \times 150 \times 10 = 2{,}500$ lbs. per foot, approximately as assumed.

The required area of reinforcement is ,

$$A_s = \frac{430{,}000}{20{,}000 \times 0.866 \times 16} = 1.6 \text{ sq. in./ft.}$$

The shear force to be used in bond computation is,

$V_s = 1/2 \times 5{,}660 \times 5 = 14{,}000$ lbs./ft. and

the required perimeter is

$$E_o = \frac{14{,}000}{113 \times 7/8 \times 16} = 8.8 \text{ in/ft.}$$

No. 7 bars, 4 in. on centers furnish an area of 1.8 sq. in. per ft. and a perimeter of 8.2 in./ft.

The required shrinkage reinforcement is 0.002 x 12 x 16 or 0.26 sq. in./ft. width of footing. No. 5 bars spaced at 12 in. on centers provide an area of 0.31 sq. in./ft.

Fig. 32 shows details of reinforcement.

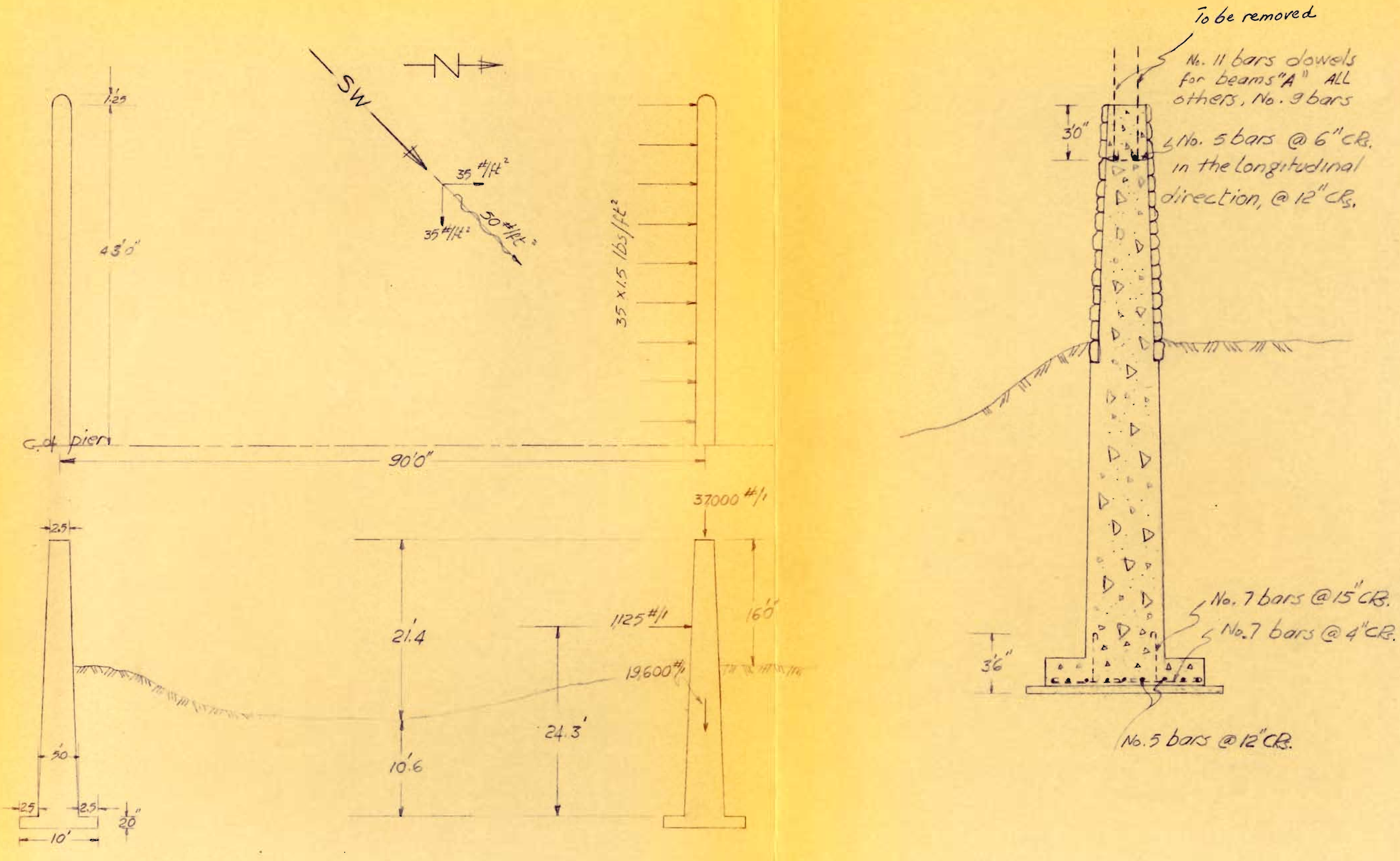


FIG. 32 PIER'S DETAILS

3) Design Of Abutments.-

A) General.- The purpose of the abutments is twofold :-

a. To transmit to the fondations the reactions of the superstructure,

b. To finish up the bridge and to retain the embankments.

Abutments for reinforced concrete structures may be built of reinforced concrete, plain concrete or masonary. The selection of the material to be used depends in most cases upon the cost, which in turn, depends upon local conditions. In the following design, reinforced concrete shall be adopted due to the excessive height of the abutment and due to the assumption that it is more ecomomical.

The same properties of steel and concrete shall be used with an assumption of 2 ft. surcharge due to traffic loading.

B) Design.- Fig. 33.A. shows the final dimension of the abutment and footing. A backwall was necessary to prevent the splitting of soil to the bearings of the bridge. With respect to the bottom of the arm, the force due to the earth thrust is, (page 332 of Urquhart)

$P = 1/2\ C_{ah}\ w\ h(h + 2h')$ in which,

$C_{ah} = 0.333$ (eq. page 330 of Urquhart)

$w = 120$ psf and $h' = 2$ ft.

$h = 33.7$ ft.

$$\text{and } P = 1/2 \times 0.333 \times 120 \times 33.7(33.7+4.0)$$
$$= 25,500 \text{ lbs.}$$

The vertical distance from the bottom of the arm to the point of application of this force is

$$Y = \frac{h + 3hh'}{3(h + 2h')} \quad \text{(same reference)}$$

$$= \frac{33.7 + 3 \times 33.7 \times 2}{3(33.7 + 4.0)} = 11.9 \text{ ft.}$$

Similarly, the force due the earth thrust under future roadway with no surcharge, for the worst condition, is

$$P = 3,920 \text{ lbs. and}$$
$$Y = 4.7 \text{ ft.}$$

The corresponding moments are, respectively, 304,000 ft-lb. and 18,400 ft-lb. opposite each other. The net moment therefore is, 304,000 - 18,400 = 3,410,000 in-lb.

$$\text{and } d = \left(\frac{3,410,000}{196 \times 12}\right)^{\frac{1}{2}} = 38 \text{ in. and for shear,}$$

$$d = \frac{25,500 - 3,920}{0.02 \times 2,500 \times 0.866 \times 12} = 42''.$$

A protective covering of 4" is required for reinforcement against ground. Hence, the minimum required thickness of the arm at its base is 46". This will be increased to 60" since the cost in such structures is usually more than balanced by the simaltaneous saving of steel, and hence d = 56".

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Following is a table showing the moments at various sections along the vertical height of the abutment due to the earth thrust with the corresponding effective depths.

TABLE V

EARTH PRESSURE MOMENTS IN VERTICAL ARM

Sect.	Dist. ft.	P lbs	Y ft.	Moment ft-lb.	Net Moment in-lb.	d in.
1	6.7	1,430	2.96	4,230	51,000	20
2	12.7	4,240	4.75	20,000	240,000	29
3	19.7	9,350	6.40	60,000	720,000	38
4	23.7	13,200	8.35	110,000	1,320,000	44
		- 320	1.33	427		
5	28.7	18,800	10.40	196,000	2,300,000	50
		1,620	3.00	4,870		
bot.	33.7	25,500	11.90	304,000	4,410,000	56
		3,920	4.70	18,400		

The moment diagram for the above moments is shown in Fig. 33.B.

The required steel area at the bottom is,

$$A_s = \frac{3,410,000}{20,000 \times 7/8 \times 56} = 3.5 \text{ sq. in per ft.}$$

which is furnished by No. 11 bars at 5 in. on centers.

Referring to Fig. 33.B., it is noticed that the bending moment decreases rapidly with increasing distance from the bottom. Therefore, some of the reinforcement is needed at higher elevations and some are not. They will be discontinued where they are no more needed.

The resisting moment provided by No. 11 bars at 10" on centers at the bottom is,

$$M = 7/8 \times 56 \times 1.87 \times 20{,}000 = 1{,}840{,}000 \text{ in-lb.}$$

and at point F at the top of the arm,

$$M = 7/8 \times 20 \times 1.87 \times 20{,}000 = 660{,}000 \text{ in-lb.}$$

Again, the resisting moment provided by No,11 bars at 20" on centers at the bottom is,

$$M = 920{,}000 \text{ in-lb.}$$

and at point F,

$$M = 330{,}000 \text{ in-lb.}$$

Hence, the two straight lines drawn in Fig. 33,B. indicate the resisting moment provided at any elevation by one half and one fourth the number of the main bars respectively. The intersection of these lines with the moment diagram at a distance of 9' 3" and 14' 9" from the bottom represents the points above which alternate bars can be discontinued. Allow 12 diameters beyond the theoratical cut off points, alternate bars therefore, shall be stopped at 10' 9" and 16' 3" respectively above the bottom of the base.

Table VI on page 83 shows all the weights acting downwards and the corresponding moments about the toe of the footing.

The soil pressure on plane "cd" is,

$$P = 1/2 \times 0.333 \times 120 \times 37.7(37.7 + 4.0)$$
$$= 31,000 \text{ lbs.}$$

and the vertical distance from the bottom of the footing to the point of application is,

$$Y = \frac{(37.7) + 3 \times 37.7 \times 2}{3(37.7 + 4.0)} = 13.2 \text{ ft.}$$

The overturning moment about "e" is,

$$M = 31,000 \times 13.2 = 410,000 \text{ ft-lb.}$$

The soil pressure in plane "ef" is similarly,

$$P = 6,400 \text{ lbs.} \quad \text{(no surcharge assumed)}$$

and $Y = 6.0$ ft.

The net overturning moment about "e" is

$$M = 410,000 - 6,400 \times 6.0 = 371,000 \text{ ft-lb.}$$

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TABLE VI

VERTICAL WEIGHTS AND BALANCING MOMENTS

	W - lbs	x*- ft	M=Wx ft-lb
W_o :	4,750	11.00	52,400
W_1 : 2 x 30 x 150	9,000	11.00	99,000
W_2 : 1 x 7 x 150	1,050	12.50	13,100
W_3 : $\frac{3}{2}$ x 27 x 150	6,100	13.00	79,400
W_4 : 2 x 7 x 120	1,680	14.00	23,500
W_5 : $\frac{3}{2}$ x 27 x 120	4,860	14.00	68,000
W_6 : 9 x 33.7 x 120	48,600	19.50	950,000
W_7 : 10 x 14 x 120	16,800	5.00	84,000
W_8 : 4 x 24 x 150	13,600	12.00	164,000
	106,440		1,533,400

* x is distance in feet from point "e". See Fig. 33.A.

C) Check Against Overturning.- The distance of the resultant from point "e" is

$$a = \frac{1,533,000 - 371,000}{106,400} = 10.70 \text{ ft. which}$$

locates the resultant within the middle third.

The factor of safety against overturning is,

$$F.S. = \frac{1,533,000}{371,000} = 4.10 \text{ which is ample.}$$

the corresponding maximum soil pressure at the toe is,

$$p_1 = (4 \times 24 - 6 \times 10.70)\frac{106,400}{(24)^2} \quad \text{(page 337 of Urquhart)}$$

$$= 5,900 \text{ lbs}$$

and at the heel,

$$p_2 = (6 \times 10.70 - 2 \times 24)\frac{106,400}{(24)^2}$$

$$= 3,000 \text{ lbs.}$$

The linear change of the soil pressure is shown in the stress distribution diagram of Fig. 33.

D) Check Against Sliding.- The resisting forces to sliding are,

1. friction between concrete and soil along the base
2. passive soil pressure, the value of which being,

$P = 1/2\ C_{ph}\ w\ h^2$ where,

C_{ph}= coefficient for passive earth pressure,

= 3.0 for a 30 degrees internal angle of friction, (see page 330 of Urquhart)

w = the unit weight of soil = 120 psf in this case.

h = the height of the effective soil above the base of the footing.

Friction along the base : $\frac{5,900 + 3,000}{2} \times 24 \times 0.5 = 53,500$

Passive earth pressure : 1/2 x 18 x 120 x 3.0 = 58,000

The total resisting forces against sliding is,

$$53,500 + 58,000 = 111,500 \text{ lbs.}$$

and the factor of safety against overturning is,

$$F.S. = \frac{111,500}{31,000 - 6,400} = 4.5 \text{ which is ample.}$$

These computations hold for the case that,

a. the surcharge extends from the right to point"c"

b. no surcharge on future roadway,

c. only dead load on bridge considered.

The other case of load distribution, when the surcharge is extended to both sides of the abutment and the live load considered as being transferred from the bridge, does not change the earth pressure on plane "cd". It does however, add to the sum of the vertical forces and will consequently increase both, the restoring moment and the friction along the base, and hence, more safety of the structure against overturning and sliding. The bearing pressure, on the other hand, increases, and hence,has to be calculated.

The total weight when all the three cases mentioned above,apply, is

$$W = 106,400 + 200 \times 9 + 200 \times 10 + 6,400$$
$$= 116,600 \text{ lbs.}$$

The resisting moment,

$$M_r = 1,533,000 + 1,800 \times 19.5 + 2,000 \times 5 + 6,400 \times 11$$
$$= 1,586,000 \text{ ft-lb.}$$

Then the resultant would be,

$$a = \frac{1,585,000 - 371,000}{116,600} = 11.0 \text{ ft. away}$$

from the toe and hence is located inside the middle third.

$\therefore$ The soil pressure,

$$p_1 = (4 \times 24.0 - 6 \times 11) \frac{116,000}{(24)^2}$$

$$= 6,000 \text{ psf}$$

and $$p_2 = (6 \times 66 - 2 \times 24.0) \frac{116,000}{(24)^2}$$

$$= 3,640 \text{ psf}$$

which are within the bearing capacity of the soil. The corresponding pressure distribution is shown in Fig. 33.

E) Design Of Footing.- Toe and heel are designed for the net results of moments and shears caused by the bearing pressure acting upward, and by the weight of fill, surcharge, and footing, all acting downward. Since the bearing pressure and the counteracting weights are different for the two cases of surcharge distribution, both conditions shall be checked.

CASE I.- No Surcharge Over Heel And Toe.

On the heel, the combined weights are,

$$W = 37.7 \times 120 + 4 \times 150 = 5,150 \text{ lbs/ft.}$$

Therefore, the net moment about "n" is

$$M_n = 12(5,150 \times 9^2/2 - 3,000 \times 9^2/2 - \frac{4800 - 3,000}{2} \times 9 \times \frac{18}{3}$$

$$= 650,000 \text{ in-lb.}$$

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and the shear is

$$V_n = 5,150 \times 9 - \left(\frac{3,000 + 4,200}{2}\right)9 = 14,000 \text{ lbs.}$$

On the toe,

$$W = 14 \times 120 + 4 \times 150 = 2,280 \text{ lbs/ft.}$$

Similarly,

$$M_m = -2,000,000 \text{ in-lb.}$$

and $$V_m = -27,700 \text{ lbs.}$$

CASE II.- Following the same procedure as in CASE I, the net results are,

$$M_n = 483,000 \text{ in-lb}$$

$$M_m = -2,510,000 \text{ in-lb.}$$

and $$V_n = 11,000 \text{ lbs}$$

$$V_m = -30,000 \text{ lbs.}$$

It is evident therefore, that CASE I governs the design of the heel and CASE II governs that of the toe. Checking the section against the maximum moment, we have, d = 33 in. and for shear, d = 38 in. Allowing for one row of steel and for 4 in. insulation, the overall depth would become 42 in. Therefore, the assumed section of the footing is 6 in. too safe. It however, need not be changed, as the additional cost of concrete will be offbalanced by the savings in steel.

F) Reinforcement of Footing.- On the heel,

$$A_s = \frac{650,000}{20,000 \times 0.866 \times 44} = 0.85 \text{ sq.in. which}$$

is furnished by No. 5 bars at 4.5 in. on centers.

On the toe,

$$A_s = \frac{2,510,000}{20,000 \times 0.866 \times 44} = 3.3 \text{ sq. in per ft.}$$

This is furnished by No, 11 bars at 5.5 in. on centers.

G) Drainage.- To prevent water from accumelating at the back of the abutment and consequently, to avoid pressure not calculated for, a longitudinal 6 in. in diameter, drain, shall be put at the bottom of the arm.

Fig. 33.C. shows a section of the abutment with reinforcement of the arm and footing, masonary cover, and the tile drain arrangement.

4) Wingwalls.- The wingwalls are usually of the same type as the abutment but differ from it in that they are topped by a simple coping without provision of surcharge and of support of any superstructure.

Fig. 34 and Fig. 35 show the plan and the east elevation of the bridge with suggested future boulevards along both sides of the river.

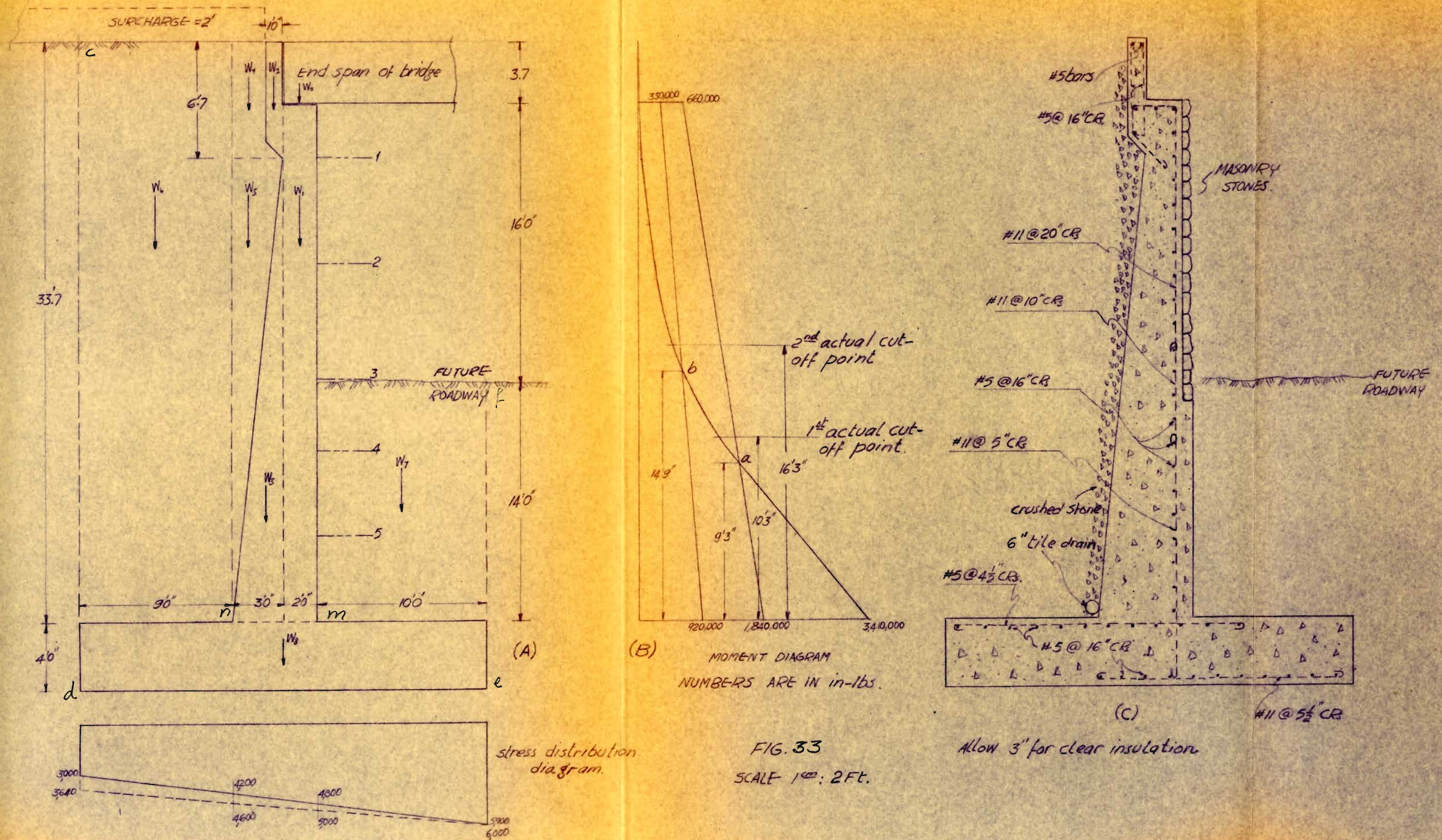

FIG. 33

SCALE 1cm : 2 Ft.

CHAPTER VI

BILL OF QUANTITIES AND COST ESTIMATES

1) Bill Of Quantities.- The following amount of steel and concrete has been obtained from the bar bending schedual of the structure shown at the end of this item.

A) Simply Supported Middle Span.-

Beams		Steel lbs.	Concrete cu.ft.
A	$89{,}520 \times \frac{11}{13}$	= 73,000	3,100
B		= 12,370	800
C		= 8,890	590

B) End Span And Cantilever.-

A	$\frac{11}{13} \times 115{,}800 \times 2$	= 195,000	10,970
B	$12{,}200 \times 2$	= 24,000	2,750
C	$7{,}940 \times 2$	= 15,880	2,060

C) Roadway Slab.-	= 52,800	7,700
D) Sidewalk Slab.-	= 3,200	460
E) Parapet.-	= 4,270	640
F) Cross Beams, av.-	= 6,000	2,200
G) Abutment.-	= 102,000	36,000

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		Steel lbs.		Concrete cu.ft.
H)	Piers.-	1,200		6,700
			Plum	20,000

TOTALS.- The following are the respective amounts of materials needed for the structure.

1.	Steel.-(plus 10 % wastage)	=	250 tons
2.	Concrete.-	=	2,070 cu.m.
3.	Plum Concrete.-	=	580 cu.m.
4.	Facing Stones.-	=	850 sq.m.
5.	Asphalt Surfacing.-	=	42 cu.m.
6.	Sidewalk Tiles.-	=	197 sq.m.
7.	Sidewalk Sand.-	=	42 cu.m.
8.	Expansion Bearings.-	=	45 two-plates
9.	Electric Bulbs.-	=	6 PS-40, 485 w.
10.	Electric poles.-	=	6
11.	Drain Pipes.-	=	2 30 m. each.

The amount of cement, sand, and gravel for the concrete required are,

a.	Cement	=	812 tons
b.	Sand	=	968 cu.m.
c.	Gravel	=	1,936 cu.m.

The following, are the weights per linear foot of the bars used in the design of the bridge, based on a specific weight of 490 lbs. per cubic foot. The wights in the following bar bending scheduals were calculated accordingly.

Bar No.	Weight Per Linear Foot lbs.
11	5.300
9	3.400
7	2.040
5	1.040
4	0.680
3	0.374

Bar Bending schedual and Bill of quantities of steel.

Description	Bar No.	No. of bars	SKETCH	Length per bar	Wt. per bar lbs	Total Weight lbs	REMARKS.
Beams "C" end span and Cant. No.=2/part section= 14"x52", 14"x15" Length, net=90', +30'	9	4		86'7"	294	2,350	Same as "A"
	"	2	6" 45'4" 5½'/7'	57'2"	194	776	
	"	1	3'0" 40'4"	54'8"	186	372	
	5	3	54'0"	55'2"	57	342	
	9	1	30'0"	31'2"	106	212	
	"	2	28'10" 5'0"	35'0"	119	476	
	"	2	28'8"	39'10"	133	532	
	"	4	23'8"	34'10"	119	952	
	"	4	8' 6' 1'6"	16'8"	57	456	Supplements of cant.
	3	33	5'10" 4½" 9"	10'2"	4	396	
	"	2	54"	11'4"	4	16	
	"	2	58"	12'0"	4	16	
	"	2	62"	12'8"	5	20	
	"	2	66"	13'4"	5	20	
	"	2	70"	14'0"	5	20	
	"	2	50"	10'8"	4	16	
	"	1	52"	11'0"	4	8	
	"	1	56"	11'8"	4	8	
	"	1	60"	12'4"	5	10	
	"	1	64"	13'0"	5	10	
	9	6	8'10"	10'0"	34	410	
	9	3	4'0"	5'0"	17	102	
						1418	
						7,940	

Description	Bar No.	No. of bars	SKETCH	Length per bar	Wt. per bar lbs.	Total Weight lbs.	REMARKS.
Roadway slab. t=6" L=212'0"	5	160x2	37'6"	38'0"	42	13,500	bot. bars
	"	160x2	2' 7" 3'4" 2'8"	41'7"	47	15,000	bent up bars
	"	160x2	37'6"	38'0"	40	12,800	Top bars
	3	72x2	212'	213'0"	80	11,500	dist. reinf.
Sidewalk slab No.=2 t=4" L=212'0"	3	160	2' 4" 2'	7'6"	2.8	900	bot. bars
	"	160x2	6'0"	6'10"	2.6	1,660	
	"	4	212'0"	213'0"	80	640	
parapet No.=2 section= 6"x36"	4	135	4" 4'6" 7"	10'6"	7.2	1,940	
	4	8	212'0"	212'0"	144	2,330	longitudinal reinf.
Cross beams No.=15	5	5	71'4"	72'4"	80	6,000	bottom bars
Abutment No.=2	5	50	88'6"	88'6"	100	10,000	
	"	237	17'0"	18'0"	19	9,000	Add 6" for hook.
	11	193	14'0"	15'0"	80	31,000	Foot. Reinforcement.
	"	105	14'0"	15'0"	80	16,800	Arm reinf. 1st cut off.
	"	52	33'6" 2'6" 1'0" 1'3"	39'3"	210	21,900	Continued.
	"	52	20'0"	21'0"	110	11,500	2d cut off.
Pier - No.=2	5	67	8" 5'6"	13'0"	13.5	1,800	
	7	133	9'0"	10'0"	20.4	5,450	
	7	136	3'	4'0"	8.2	2,200	
	5	11	88'0"	89'0"	100	2,200	
	5	177	2'6"	3'6"	4	1,420	
						169470	

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2) Cost Estimates.-

Item	Particulars	Unit	Rate LL	Cost LL
1	Steel- cost,bend and place	ton	520	130,000
2	Concrete- aggregate and workmanship	cu.m.	68	140,000
3	Plum Concrete- Do.	cu.m.	35	20,000
4	Facing Stones- cost and labor	sq.m.	15	13,000
5	Asphalt Surfacing,	cu.m.	25*	10,500
6	Sidewalk Tiles-cost and place	sq.m.	3	600
7	Sidewalk Sand- cost	cu,m.	10	400
8	Expansion Bearing,cost	unit	50*	2,000
9	Electric materials, total cost			200*
10	Drain pipes			200*
11	Excavation and refilling	cu.m.	3.50	11,000
				328,000

* Values are assumed.

Add 15 % for design and supervision, the total cost of the bridge would become,

$$328,000 + 328,000 \times 15\% = \underline{\underline{333,000}} \text{ L.L.}$$

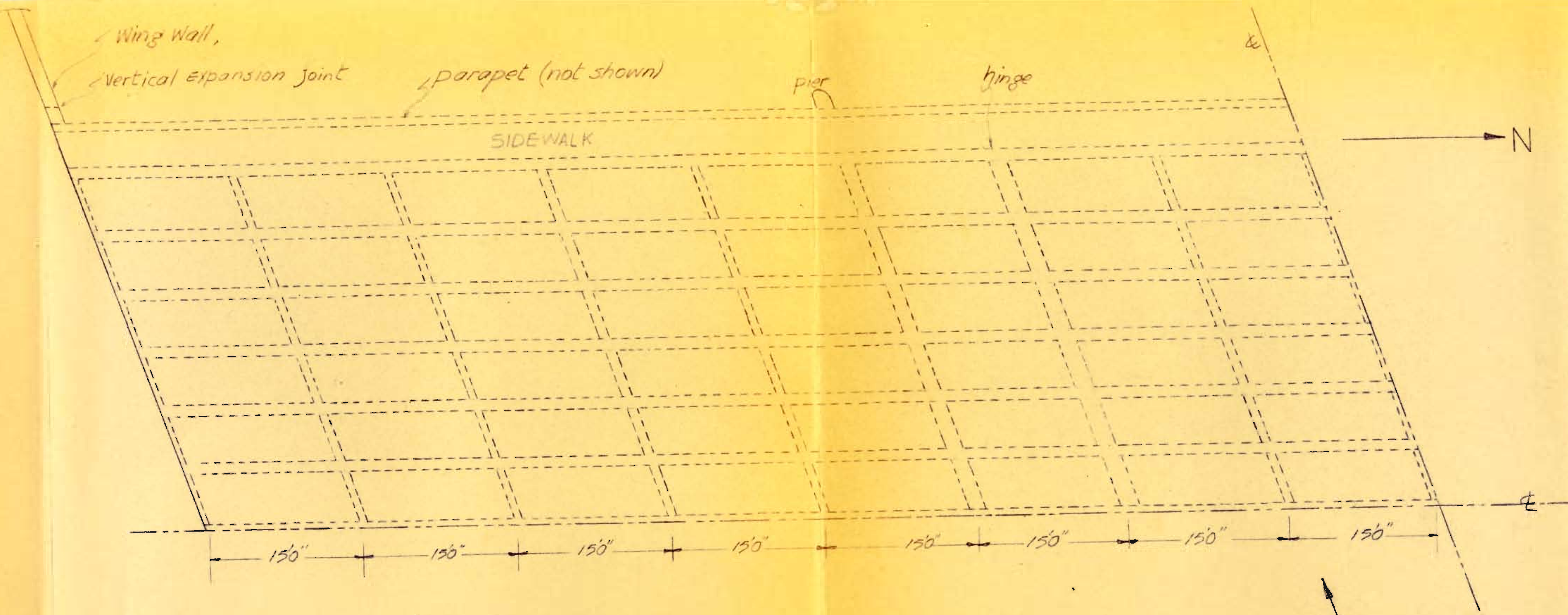


FIG. 34 $\frac{1}{4}$ PLAN, showing cross beams Arrangement.

Angle of skew 20°

Scale 1cm: 4 Ft.

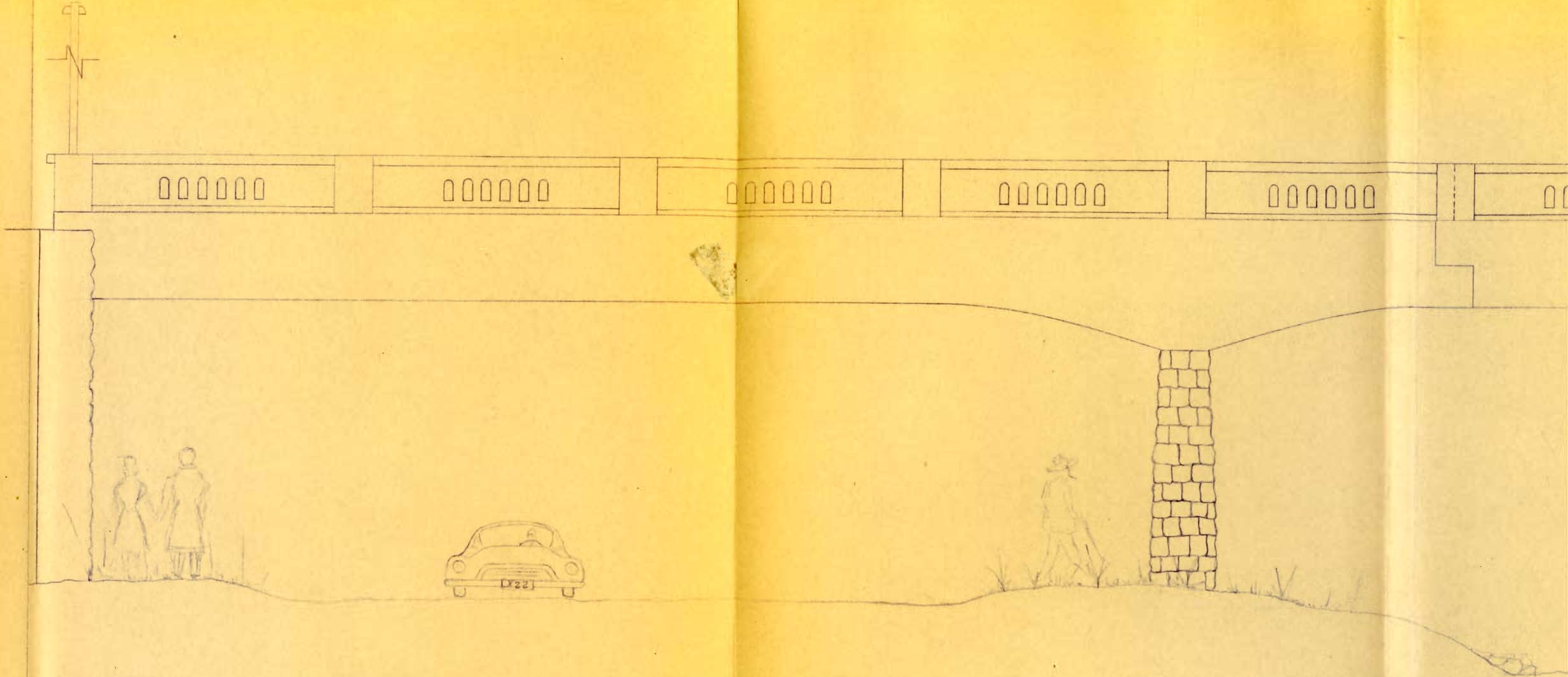


FIG. 35 EAST ELEVATION O

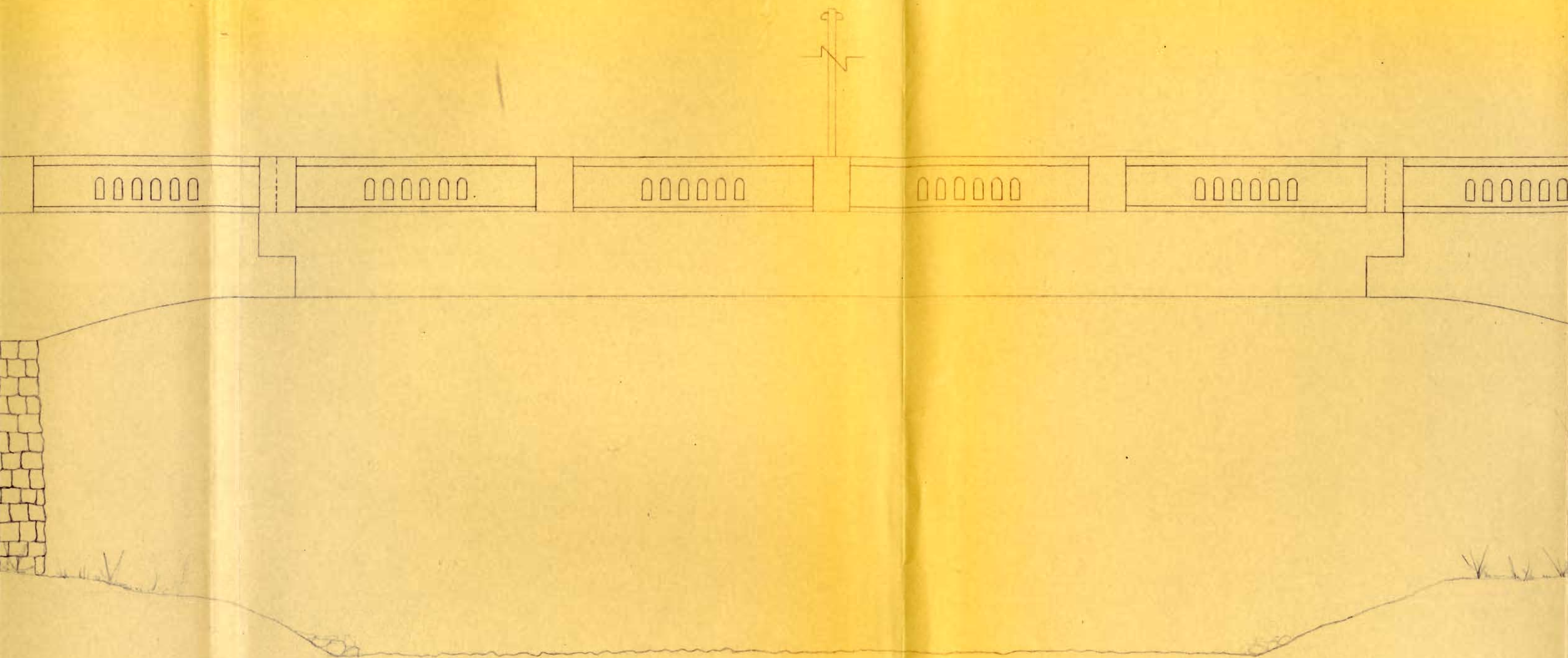
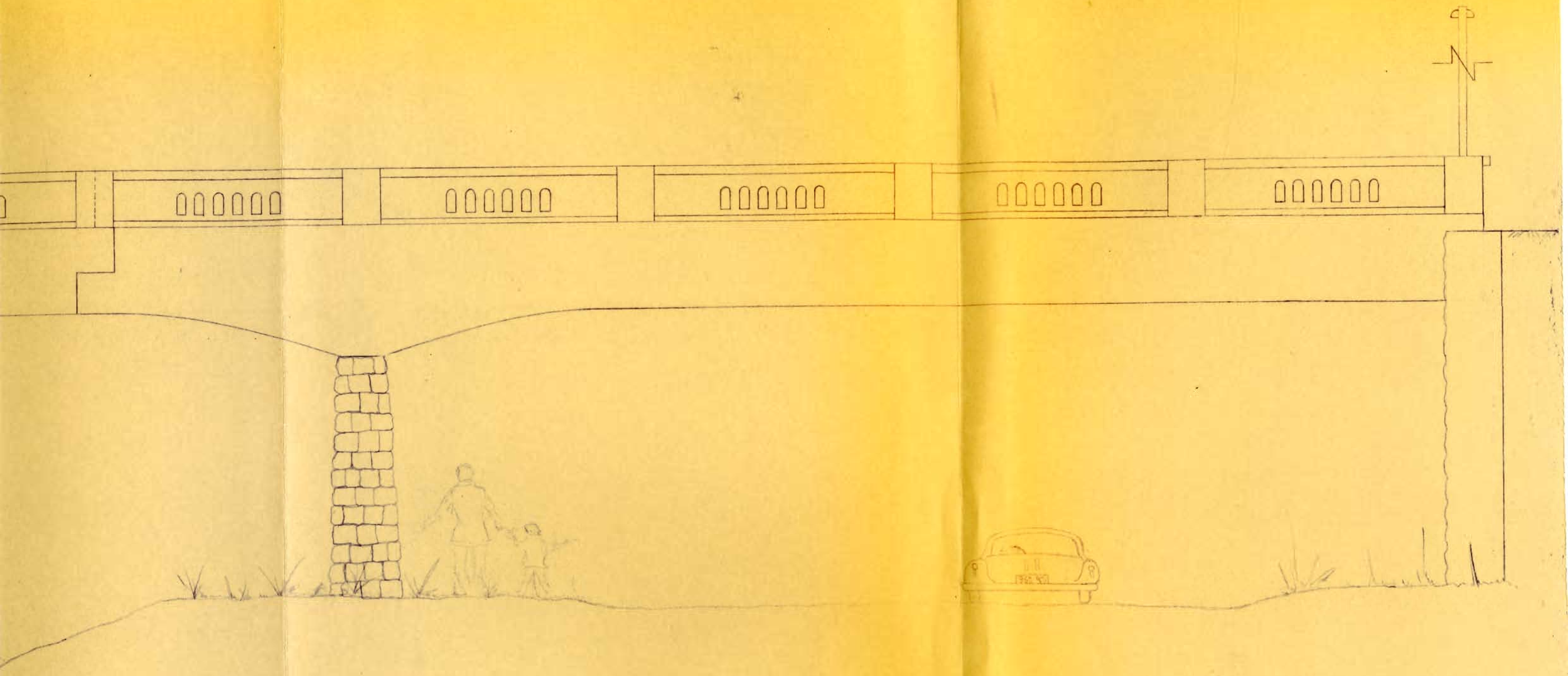


FIG. 35 EAST ELEVATION OF THE BRIDGE SHOWING FUTURE BOULEVARDS ON BOTH SIDES OF THE RIVER

Scale 1 cm: 2 Ft.



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B I B L I O G R A P H Y

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2. Reinforced Concrete Bridges by Taylor, Thompson, and Smulki.

3. Desing Of Reinforced Concrete Bridges by Legat Dunn and Fairhurst

4. Concrete Bridge Details, by Portland Cement Associations.

5. Design Of Concrete Structures by Urquhart and O'rourke

6. Journal Of The Institution Of Civil Engineers by The Instituion Of Civil Engineers, England.

7. Elementary Structures by wilbur and Norris

8. Illumination Engineering by Boast

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Bar Bending schedual and Bill of Quanties of Steel.

Describ-tion	Bar No.	No. of bars/-	SKETCH	Length per bar	wt. per bar lbs.	Total weight lbs.	Remarks
Beams "A" No. =13	11	5	56'6" 5'6"	65'6"	347	22,400	Add 1'ft. for hook
section of stem= 20"x35"	11	2	4'2" 55'6" 6' 11" 8'	85'0"	450	11,700	
Length, net = 58'	"	2	53'10" 1'4"	85'0"	450	11,700	
	"	1	52'10" 3'	85'8"	455	5,900	
	"	2	50'6" 3' 6'	85'0"	450	11,700	
	"	1	45'6" 5'6"	81'0"	430	5,500	
	"	1	40'6" 8'	81'0"	430	5500	
	5	3	52'0" 7"	53'2"	56	2,200	top reinforcement
	3	114	10" 3' 8"	8'2"	3	4,500	
	5	30	2" 3' 1'4"	7'6"	8	3,100	Vertical st. at bracket
	"	50	2" 1'4"	3'10"	3	1,950	
	"	10.	3'9"	8'8"	9	1,170	horizontal st. at bracket.
	"	40	20"	4'6"	5	2,600	
						89,720	

Describ-tion	Bar No.	No. of bars/-	SKETCH	Length per bar	wt. per bar lbs.	Total weight lbs.	REMARKS
Beams "B" No.=2	9	6	56'8" 5'6"	65'8"	224	2,700	Add 1'ft. for hook + bend.
section of stem = 20"x50"	"	2	5'2" 54'8" 4' 1'4" 8'	85'10"	290	1,160	
Length, net=58'.	"	2	53' 1'4"	86'2"	293	1,170	
	"	1	51'4" 2'8"	86'2"	293	590	
	"	2	49'8" 3' 6'	82'10"	282	1,128	
	"	2	44'8" 5'6"	82'2"	280	1,120	
	"	1	39'8" 8'	82'2"	280	560	
	"	5	52'0" 7" 7"	53'2"	180	1,800	Top reinf.
	3	35	10" 3'10" 1'2"	10'10"	4	280	
	5	30	2" 3'9" 1'4"	9'0"	10	600	Vertical st. at bracket.
	"	40	1'6"	4'6"	5	400	
	"	20	3'9"	8'8"	9	360	horizontal st. at bracket.
	"	50	20"	4'6"	5	500	
						12,370	

Describ-tion	Bar No.	No. of bars
Beams "C" No. = 2	9	6
Section= 14"x52"	"	2
Length, net= 58'	"	2
	"	1
	"	3
	3	38
	5	32
	"	32
	"	16
	"	16
Beams "A" End-span and Cant. No.= 13/part	11	5
	"	2
section of stem= 20"x35", 20"x15" (av.)	"	2
Length, net = 90', +30'	5	3

RKS

r hook +

st. at

/ st.

et.

Description	Bar No.	No. of bars	Sketch	Length per bar	Wt. per bar lbs.	Total weight lbs.	Remarks
Beams "C" No. = 2	9	6	56'8" 3'6"	65'8"	224	2700	
Section= 14"x52"	"	2	5.5' 53'10" 8' 1'6"	84'10"	290	1,160	
Length, net=58'	"	2	52'2" 10' 1'8"	85'6"	292	1,168	
	"	1	50'6" 6' 1'8"	81'6'	278	556	
	"	3	61'7" 6' 1'8"	77'11"	265	1,590	
	3	38	3'11" 9"	10'6"	4	304	
	5	32	2" 3'11" 1'4"	9'6"	10	640	Special stirrups at bracket.
	"	32	20"	4'6"	5	320	"
	"	16	3'9"	8'8"	9	298	"
	"	16	20"	4'6"	5	160	"
						8,890	
Beams "A" End-span and Cant. No.=13/part	11	5	8'7" 6' 1'2" 8'4" 2'6" 6'8"	86'7"	490	32,000	Bot. bars.
section of stem= 20"x35", 20"x15" (av.)	"	2	2' 45'6" 1'5" 1'	56'4"	320	8,050	
	"	2	4'6" 38'	51'4"	290	7,550	
Length, net=90', +30'	5	3	33'4"	34'6"	36	1,400	Top bars.
						49000	

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Description	Bar No.	No. of bars	Sketch	Length per bar	Weight per bar lbs.	Total weight lbs.	Remarks
	11	4	43'4" 1'	45'4"	256	13,300	Top bars
	"	1	32'8" 3'6"	39'8"	225	2,930	
	"	1	30'2" 3'4"	38'10"	220	2,860	
	"	2	29'10"	38'6"	218	5,660	
	"	2	24'2"	32'0"	187	4,900	
	"	2	16'2"	24'0"	140	3,640	
	"	2	4' 11'2"	21'2"	120	3,120	bar supplemented over pier support.
	"	2	4'6" 6'2"	17'6"	100	2,600	
	"	2	1'8"	13'0"	74	1,970	
	"	5	8' 6' 1'2"	17'2"	97	6,300	supplemented at bracket of Cant.
	3	60	1'4" 9" 3'6"	8'8"	3	2,350	
	"	9	1'3" 3'6"	9'8"	4	470	U st. in varying sections in Cant. and end span.
	"	2	1'5" 40"	10'4"	4	104	
	"	2	44"	11'0"	4	104	
	"	2	48"	11'8"	4	104	
	"	2	52"	12'4"	4	104	
	"	2	56"	13'0"	5	130	
	"	2	60"	13'8"	5	130	
	"	6	8'	10'0"	53	5,200	vertical bars in piers and brackets
	"	3	3'	5'0"	27	1,090	
						8820	sp. st. in bracket "A"
						66,800	

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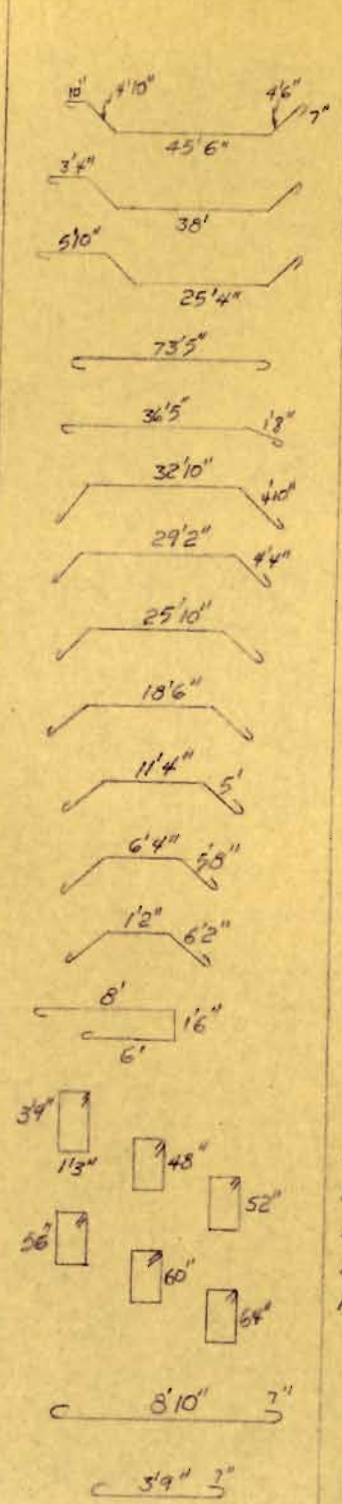
Description	Bar No.
Beams "B" End Span and Cant. No.=2 per one part.	9
	"
	"
Section= 20"x50", 20"x15".	"
	"
Length, net 90', +30'	"
	"
	"
	"
	"
	"
	3
	"
	"
	"
	9
	9

Length per bar	Weight per bar lbs.	Total weight lbs.	REMARKS.
45'4"	256	13,300	Top bars
39'8"	225	2,930	
38'10"	220	2,860	
38'6"	218	5,660	
32'0"	187	4,900	
24'0"	140	3,640	
21'2"	120	3,120	bar supplemented over pier support.
17'6"	100	2,600	
13'0"	74	1,970	
17'2"	97	6,300	supplemented at bracket of Cant.
8'8"	3	2,350	
9'8"	4	470	U st. in varying sections in cant. and end span.
10'4"	4	104	
11'0"	4	104	
11'8"	4	104	
12'4"	4	104	
13'0"	5	130	
13'8"	5	130	
10'0"	53	5,200	Vertical bars in piers and brackets
5'0"	27	1,090	
		8820	Sp. st. in bracket "A"
		66,800	

Description	Bar No.	No. of bars	SKETCH	Length per bar	Weight per bar lbs.	Total Wt. lbs.	REMARKS.
Beams "B"	9	5		86'7"	295	2,950	Same as "A"
End span and Cant.	"	2		56'10"	191	764	
No. = 2 per one part.	"	2		52'10"	180	720	
Section =	"	1		41'8"	142	284	
20"x50", 20"x15".	"	2		74'7"	254	1,016	
	"	2		39'3"	133	532	
Length, net 90', + 30'	"	2		43'8"	149	596	
	"	2		39'0"	133	532	
	"	2		35'0"	119	476	
	"	2		28'4"	96	384	
	"	2		22'6"	77	308	
	"	2		18'10"	64	256	
	"	2		14'8"	50	200	
	"	5		16'8"	57	570	supplements in bracket.
	3	32		11'2"	3	192	St. in varying section of Cant. and end span.
	"	2		11'8"	3	12	
	"	2		12'4"	3	12	
	"	2		13'0"	4	16	
	"	2		13'8"	4	16	
	"	2		14'4"	4	16	
	9	6		10'0"	34	410	
	9	3		5'0"	17	102	
						1,860	Sp. st. in bracket "B".
						12,200	



IMPORTANTANT NOTE.– Under Beams "A", The Weight Of Steel Calculated Was For A Total Of 13 Beams. Actually, They Are 11 Beams. Correction Is Made Under Bill Of Quanties Calculations. (see sheet of bar bending schedual.)