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'The Design and Construction of Composite Slab and Girder Bridges, with Particular Reference to the Leven Bridge at Ulverstone', 1935,

Thesis M.E. 1935

Folder Contains:

- **Leven Bridge Plans**
- **Thesis**
- **Published Article**
  "The design and construction of composite slab and girder bridges"
- **Appendix iii**
- **Album of photographs**

"THE DESIGN AND CONSTRUCTION OF COMPOSITE SLAB AND GIRDER BRIDGES WITH PARTICULAR REFERENCE TO THE LEVEN BRIDGE AT ULVERSTONE".

1. INTRODUCTION.

The theory on which this particular type of structure is based is given in the January issue of the "Journal of the Institute of Engineers" in a paper entitled "The design and construction of composite slab and girder bridges" and reference will be made to the subject matter of this paper. The paper can be regarded as part of this thesis.

Experiments on test pieces designed to investigate the possibility of reinforcing a structure consisting of a concrete deck slab supported by steel joists for shear between the top of the steel section and the concrete, were first conducted and indicated that the theoretical expectations were realised in practice. A 34ft span two beam bridge with a 10ft roadway was designed and constructed for testing the type of structure on a larger scale (Commonwealth Engineer, April 1933). Later a 1/6 scale model of one span of the proposed Leven bridge was also constructed for testing purposes, but rather from the point of view of load distribution between the beams than from strength considerations.

Two, three, four and five beam highway bridges of this type have now been constructed and it is of interest to note that the structural and economic advantage to be gained is likely to lead to a very extensive use in the future of this class of construction.

The Subject matter of the thesis, outlines the principles of the design and construction of composite beam bridges in general, and describes the Leven Bridge, which is a four beam bridge of this type, having 7 spans each 61 feet long. The deck of the Leven bridge carries a roadway 20ft between the kerbs, with a 4ft footway on each side. Reference is made, in describing the work, to the Journal paper and to the working drawings for the bridge.

The Leven River rises in the vicinity of Mt. Pearce about 35 miles from the coast, and flows into Bass Strait at Ulverstone on the North West Coast of Tasmania; the Coast Highway passes through the town less than a mile from the sea. The existing bridge is a timber structure 49 years old, and is in such a state of decay that it was considered to have reached the end of its useful life. Consideration was therefore given to the renewal of the structure which is a vital link in the communication system in this part of the State. A survey, of the locality, was therefore made and consideration given to five sites on which the bridge could be renewed; these are shown on Drawing 65L-2. Eventually the site marked 1.B. was selected by the Parliamentary Committee investigating the proposals and a decision made to renew the bridge with a permanent structure at an estimated cost of £13,700, exclusive of land resumption. Bore holes 66 feet apart were put down on the centre line of this site and indicated that a suitable foundation could not be obtained less than 35 feet below the river bed. A 10ft rise and fall of tide had also to be reckoned with. A number of designs were examined for the superstructure, including a three span high through truss bridge, several arrangements of welded pony trusses, and several arrangements of welded plate girders. Of these proposals a 7 span cantilever welded plate girder bridge was recommended as most suitable; the above designs were all prepared for a 18ft roadway and one 4ft footway. Later, however, an examination of the composite slab and girder type showed a marked saving over the latter bridge, but as the necessary funds had already been voted by Parliament, it was decided to increase the roadway width to 20feet, and add another footway, as it was estimated that the additional work could be undertaken for the difference in costs of the two types.

2. DESIGN.

SUBSTRUCTURE.

An examination of 65L-9 will show the varigated nature of the material forming the river bed and shows the difficulty of obtaining a type of substructure suitable for the full width of the river. On the eastern side serpentine carrying hard kernels of the original basalt was out-cropping, and extended to a considerable depth. At the eastern abutment four to five feet of clay and mud covered the serpentine, but at No.1 pier the rock was bare. The serpentine disappeared on the western side of No.2 pier, and in midstream the formation was mud and sand followed by clay, more sand, and finally the micacious schist at a depth of about 36 feet. Towards the western bank the blue clay disappeared altogether, and the material overlying the rock was almost wholly sand. The sand however was mixed with river shingle, which in some cases reached the dimensions of boulders, the shingle was more pronounced in some parts than in others, and in places occured in layers several feet thick. Near the western abutment the sand and shingle was cemented with a rich yellow-coloured clay which, however, responded easily to the action of the water jet.

The eastern abutment was designed as an ordinary spill abutment, the concrete foundation being spread and founded on the serpentine.

The main columns, as shown in 65L-11, are octagonal in cross section, 22" between the flats supported by a slab 7ft X 33ft, and connected at the top by a cross beam 30" X 15". To avoid any chance of damage by settlement of the earth filling, which is 9 feet high at the abutment, the pylons are directly connected to this cap and carried on the main foundation. The base of the slab is 7 feet below ground level and 12 feet below high water level.

The first pier is also supported by a spread foundation placed directly on the serpentine. The concrete slab 26ft X 5ft X 2ft carries three octagonal columns of the same dimensions as those of the abutment, but the columns are connected by an 8" curtain wall and a cap 26ft X 30" X 15"; the details of construction are given in 65L-9.

The remainder of the piers and the western abutment are supported by 22" octagonal concrete piles. There are three piles in each pier, connected at low water level by a concrete waling and above this waling by an 8" concrete curtain wall capped in the same manner as No 1. pier.

The longest piles were 65 feet in length and weighed 13 tons. The manufacture, handling and driving of these piles constitued the major portion of the substructure work. Various other types of substructure were investigated, the form finally adopted being regarded as the most economical for the site conditions indicated by the survey borings.

DESIGN OF CONCRETE PILES.

Design Stresses.

A compressive stress in the concrete of 33% of the ultimate strength as indicated by test blocks was allowed. The handling and driving stresses are usually the most severe for piles, it is only with piles having a considerable unsupported length above ground, that the working stresses require investigation on account of long column action; in such cases the unsupported length can usually be reduced by suitable bracing. The concrete mix was designed to give an ultimate strength of 5,000 lbs. per square inch, at 28 days, which gives an allowable working stress of 1,500 lbs. per square inch. The average strength of test blocks for the piles was actually as follows.

Mix	Age	Average Strength	No. of blocks
$1:1\frac{1}{2}:3\frac{1}{2}$	21 days	5,100 lbs. sq. inch	16
"	28 "	5,247 " " "	16
"	66 "	5,808 " " "	16

66 days was the average age of the piles at driving.

Modular Ratio.

Since strain is not proportional to stress for concrete, the value of the modular ratio depends on the stress at which the slope of the stress strain curve is measured. The L.C.C. regulations (1915) give the formula -

$$n = \frac{9,000}{c} \quad \text{where } c = \text{allowable compressive stress.}$$

Dr. Faber (Proc. Inst. C.E.- vol. 225) suggests that Young's Modulas for concrete is approximately equal ultimate strength X 1,000. For 5,000 lb concrete and E_s - 30,000,000 n by the above method is 6 in both cases. This value was therefore assumed in calculating the modulas of the pile section.

Tension Stress.

There is a diversity of opinion as to the allowable tension stresses for design of reinforced concrete piles. Usually a steel stress up to 24,000 lbs. per sq. inch due to handling loads is allowed, but in other works a maximum tension stress of 100 lbs. per sq. inch in the concrete is specified which for n = 6 limits the steel stress to 600 lbs. per sq. inch, a marked variation. For this particular work the latter figure was adhered to for the reasons given later.

The stresses for which a pile must be designed are as follows.

1. Bending and shear stresses due to handling.
2. Compression, shear and possibly buckling stresses due to driving.
3. Compression stress under working load.

The relative importance of these items depends to a large extent on the nature of the material through which the pile is to be driven. If this material provides any serious resistance to the pile in the early stages of the driving it is safe to say that the stresses for item 2. will be the most severe, and, therefore, the ones which should be given most consideration in design. Unfortunately the values of these stresses are difficult to determine, and this is apparently the reason for the development of various empirical rules for pile design. The formula commonly quoted for determining the size of the pile is the column formula.

$$P = A_c.f_c. + A_s.n.f_c.$$

where P = safe load in lbs.
A_c = net core area of concrete in square inches.
A_s = area of main reinforcing steel in square inches.
f_c = allowable compressive stress of concrete.
n = modulus of elasticity $\frac{\text{steel}}{\text{concrete}}$.

Use was made of this formula together with the reduction formula

$$r = 1.75 - \frac{L}{20d.}$$

where L = unsupported length in feet.
d = least dimension of the core section in inches.
r = the coefficient by which f_c in the short column formula must be reduced to give the safe stress for long columns.

The value of L can only be fixed by making allowance for the supporting effect of the material into which the pile is driven and of the curtain wall and cap of the pier. It should be noted that P, the safe load referred to in the formula is the maximum load which the pile can reasonably be expected to carry without damage; as it is necessary to allow a factor of safety on this load, against settlement of the pile, of from 2 to 5 according to the nature of the ground, the value of P used in the formula should be 2 to 5 times the working load. Allowing 15% for impact on live loads, the total working load per pile is 65 tons. The test bores indicated that a rock foundation could be reached, it was therefore considered safe to adopt a factor of safety of 2, for which the concrete stress on the pile section adopted is approximately 730 lbs. per square inch. The Leven Bridge is only a short distance from the sea, it was therefore necessary to provide adequate concrete cover on the steel rods to protect the steel from the effect of sea water. A minimum of 2¼" on the main rods was adopted for this. For the above section the allowable unsupported length is 25 feet and as this is not exceeded, consideration of the pile as a long column is unnecessary.

Observations made while driving piles for the Leven Bridge, and on other piling works, suggests that the concrete cover of the main rods has an important bearing on the allowable tension steel stresses caused by lifting and pitching the piles. It is obvious that concrete must fail if the tension stress exceeds its ultimate value, which, for first class concrete, is about 500 lbs. per square inch, but the result of this failure may not be apparent at the tension face of the concrete if the cracks are very small, owing to their distribution by the reinforcing effect of steel rods in close proximity to the concrete surface. If, however, the rods are some distance from the surface their effect in distrubuting the cracks in the concrete at the surface is negligable, with the result that one large crack of perhaps a serious nature developes in the place of many small ones that are not of serious consequence. An appreciation of this point is necessary in fixing the steel stresses by which the handling system is regulated.

In this case a maximum tension stress of 100 lbs per sq. inch in the concrete, and consequently n times this in the steel, was selected and a method of handling devised to keep the stress within this limit. In computing the modulus of the pile section, the concrete was taken to be effective in tension giving a value of Z = 2554. A stiff back, which consisted of a 24 X 7½ X 90lb R.S.J. 61ft long, was strapped to the pile to give additional support, and in calculating stresses the bending moments were assumed to be distributed in proportion to the moments of inertia of the two sections. Various arrangements of lifting gear were investigated and finally that shewn in the sketch was adopted. For some of the shorted piles the stiff-back was dispensed with and the same gear used on the pile alone.

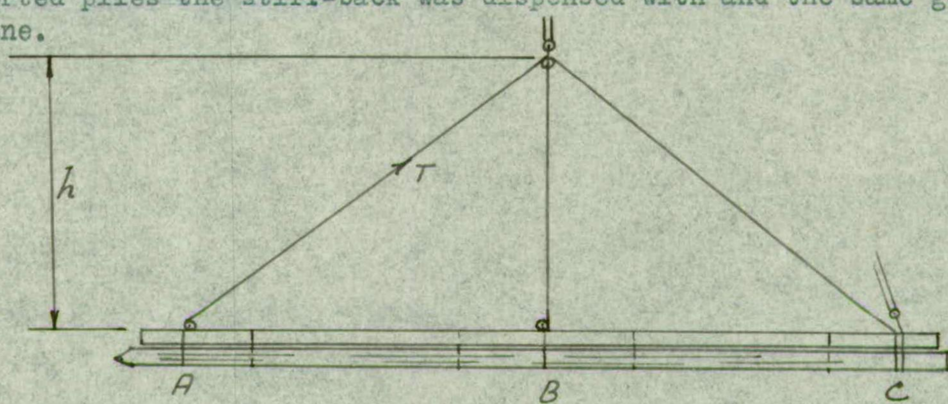


FIGURE 1.

For a given height h, which was actually 40 feet, the uniform tension T in the bridle can be resolved to find the vertical forces at A.B. and C. the sum of which equals the weight of the pile and stiff back. The distribution of bending moment can then be simply obtained and the position of A. and C. fixed by trial and error to give an equal positive and negative moment in the pile. Immediately the pile is swung to an inclined position it becomes unstable and tends to hang vertically, so that a guy is necessary to control this tendency once the pile is moved from the horizontal position by the head gear.

The longitudinal reinforcing must be designed to carry tension stresses in the pile caused by handling or driving. In this case the tension stresses due to handling were very low and the latter factor became of major importance. The quantity of steel for this purpose has been determined by experience with concrete piles of various design, and a minimum of 2% of the total area is recognised as a satisfactory amount. In this case it amounts to 2.45%, and consist of eight 1-1/8" diam. round rods spaced at the corners of the octagon and bent in to fit the form of the pile at the shoe and butt welded to the cast steel shoe, as shown on sheet 65L-14.

Generally speaking reinforcing is not required to resist shear stresses in the pile due to bending under its own weight, it is however necessary to resist impact stresses, to give resilience, to resist the bursting pressure of the concrete and to prevent buckling of the main rods; there is nothing gained by increasing the volume of the lateral reinforcement beyond that required to resist these forces. The lateral reinforcement is fixed by regulations in an empircal manner, but an application of Navier's theory* and the value of shear stress, which has been derived from it, enables the volume of the lateral reinforcing necessary to resist the bursting pressure of the concrete - its most important function - to be calculated in terms of the working compression stress.

Considering the shearing plane of a short concrete column to be inclined at 60° to the plane of loading, assume such a square column of side a and height 3a to be part of the core of a pile. If the maximum working stress is c.lbs per sq. inch then for $\theta = 60^\circ$ and $\phi = 30^\circ$ the working shear stress = .30 c approx.

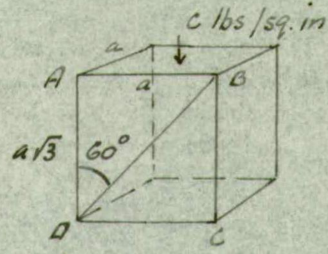


Figure II

Tangential stress on plane

$$BD = c \sin\theta \cos\theta = c\,\frac{\sqrt{3}}{4} = .433\ c.$$

There is therefore a shearing stress on plane BD of .133c greater than the material can sustain with a factor of safety of 3. This produces a tangential force = $.133\ c\ a^2\sqrt{3}$ lbs.

Resolving this force vertically and horizontally, the vertical component can be carried as a compression in the concrete; the horizontal component must be resisted by the hoop reinforcing if the factor of safety against shear within the core is not to be reduced below 3.

The horizontal component of the slipping force

$$= 0.133\ c\ a^2 \text{ lbs.}$$

Assume the links to form a square instead of an octogan, the side of the square being the same as the distance across the flats of the octogan. For a column hooped with these square links there are links in two planes to support the above force, therefore the required total sectional area of links on one plane

$$= \frac{.133\ c\ a^2}{2 \times 18{,}000} \text{ sq. inches}$$

But the total length of links = 4 a inches
therefore volume of links = $\dfrac{4\ c\ a^3 \times .133}{2 \times 18{,}000}$

The volume of the core ABCD = $\sqrt{3}\,a^3$ cu. inches.
therefore volume of links as a percentage of the core

$$= \frac{4\ c\ a^3 \times .133 \times 100}{2 \times 18{,}000 \times \sqrt{3}\,a^3}$$

$$= .00085\ c.$$

For c = 730 lbs per sq. inch the percentage is .62% and the spacing of $\frac{3}{8}$" diam. links to give this percentage for the pile is approximately 4.5". By this method an indication is given of the quantity of hoop reinforcing required, but the actual quantity used was fixed after an inspection of various designs for concrete piles which had proved satisfactory in practice. The hoop reinforcing is wrapped round the main rods in the form of a spiral, the spacing being decreased at the head and the shoe, to allow for the higher stresses at these points due to the effects of driving. At intervals of 4 feet bridles are located; these

<u>Note</u>.* Navier's theory proves that the tangential stress on the plane of rupture of a brittle compression specimen is compounded of the shearing stress plus a friction stress, the latter depending on the angle of internal friction of the material ϕ, and that the angle of the plane of rupture with the vertical axis is $\theta = 45^\circ + \phi/2$

are required to facilitate fabrication and also serve the same function as the hoop reinforcing. The details of the hoop reinforcing and other features of the pile design are shown on drawing 65L - 14.

The greater part of the material through which the piles were to be driven was of a sandy nature and this suggested that the water jet might be used to advantage. The water jet is eminently suitable for sinking piles in clean sand, a material which affords considerable resistance to penetration by piles under the hammer alone, its efficiency, however, is very much reduced if the sand contains river shingle or layers of clay which will block the jet. The difficulties due to the presence of shingle can be overcome by increasing the volume and pressure of the flow of water through the jet, in one instance piles penetrated through a rock filling twenty feet deep by this means. Under normal circumstances a volume of 10,000 gallons per hour at 150 lbs. per sq. inch pressure is sufficient to facilitate the penetration of the pile provided the material is in any way suitable for the method, and this quantity was adopted in this case. Sometimes the water is carried through the centre of the pile and through a nozzle formed in the shoe, or as an alternative two jets can be used externally and operated one each side of the pile. The chief factor claimed in favour of this arrangement is that a tendency for the pile to run at the shoe can be corrected by breaking up the material at the shoe of the pile with the hand jet. While this is to some extent true, any movement of the pile due to this process will generally jamb the hand jet between the pile and the material obstructing its path, and necessitate the use of a winch to withdraw it. In mud or sand, free from shingle, either arrangement is satisfactory. In designing the piles for the Leven Bridge it was recognised that the driving would be difficult and provision was therefore made for the central jet pipe, which could be used in conjunction with external jets if necessary. It is 2½" diameter with a right angle bend 5 feet from the head of the pile and connected to a nozzle of ¾" diam. formed in the cast steel shoe. Although this arrangement has been used to advantage on other works it did not give good results at the Leven, and after some experimenting the return outlets were blocked up and the central outlet increased to 1½" diam. This decreased the pressure to some extent but where the material was really suitable for jetting, the piles would sink steadily under their own weight.

P I E R S.

Excepting No. 1 pier which is supported by a slab foundation, all the piers are supported by piles. As the effect of the sea water on steel reinforcing is particularly severe, in the Leven River, the design of the piers was arranged to avoid breaking into the piles below high water level for the purpose of connecting the piles by a curtain wall. A concrete waling 27 feet x 3 feet x 9" thick was precast with holes to fit the three piles and slipped down over the piles to a point below low water level. It was supported at this level by timber clamps fixed at the correct level on the piles by a diver. The 8" curtain wall was then cast upwards from the waling, monolithic with the cross beam. The cross beam, details of which are given on 65L - 15, is designed to distribute the loads from the four main members of the superstructure to the three piles; provision is made for recesses to carry bearing plates, the levels of which are conveniently adjusted by means of the lower nuts carried by the holding down bolts - see 65L - 12.

ABUTMENTS.

The design of the abutment calls for little comment. As stated previously the eastern abutment - see 65L-11 - is of the spill type and is supported by a spread foundation. Account was taken of the end reaction from the first span and of the pressure of the filling on the curtain wall in examining the stability of the structure. A feature of the design is the method of supporting the pylons from the main abutment foundation in order to avoid any difficulties due to settlement of the filling. The western abutment is similar except for the fact that it is supported by three piles of the same dimensions as those used for the piers. The piles are connected by a concrete waling and curtain wall which reaches to high water level. Above this the design is the same as for the eastern abutment.

A timber retaining wall had previously been erected on the western bank of the river opposite the wharfs and it was decided to continue this wall round the face of the western bridge abutment and reclaim the area enclosed. A two-fold purpose was thus served, a foundation for the western approach was provided, and the scour in the river increased. Continuous dredging is necessary to remove sand deposited in the channel by tidal water and thus maintain the required depth of water for ships proceeding to the wharf immediately below the bridge, and this fact required consideration in arriving at the most suitable form of abutment. The pressure of the filling on the sides of the curtain wall of the abutment also provided longitudinal stability not only to the abutment but to the bridge as a whole.

SUPERSTRUCTURE.

The superstructure provides a roadway 20 feet wide and two footpaths each 4 feet wide, the overall width is 31 feet. It is on a slight grade, being 21" lower at the western end, which improves the approach grade to this end of the bridge. The bridge has a parabolic camber with a mid ordinate of 12", the deck is also cambered transversely to facilitate drainage. As stated previously various types of superstructure were examined and a decision made in favour of the composite beam type which showed a marked saving over other arrangements. The loading adopted for design purposes was the standard used by the Department for a bridge providing two traffic lanes and consists of a crusher train of a total weight of 34½ tons in one lane and a 10 ton motor truck in the other. The loads were not taken to be in the centre of the respective traffic lanes but were placed in the position to give the maximum reaction to any one of the supporting members and this reaction increased by 15% to allow for impact effect. The distribution of live loads to beams carrying a concrete deck slab of considerable stiffness presents a problem of which very little information is available and in the interests of accuracy and economy an effort was made to analyse this distributing effect of the slab. An account of this work will be found in the paper published in the Journal, and the calculation of the stresses in the members of the superstructure of the Leven Bridge is given as an example of the method of procedure for the design of composite slab and girder structures. In arriving at a suitable thickness of deck slab and size of steel member it will be found best to select what appears to be a suitable section, a simple matter after some experience with these designs and then take out the stresses due to the dead and live loads. The section can then be adjusted to give the required working stress in the steel and concrete. This method was adopted in arriving at the dimensions of themembers of the superstructure for the Leven Bridge and for which the design calculations are given in the paper referred to. As this section of the work is covered in the paper, to which reference should be made, it will not be repeated here. In conjunction with the work on distribution described in the paper for which a 1/6th scale model of one span of the bridge was constructed and tested it is interesting to refer to the results of a similar test to which the first span of the Leven Bridge was subjected the results of which are given in Appendix 11.

By assuming that the stresses in the steel are uniform throughout the section, a favourable condition for maximum loading, Prof. Burn has derived the propping moment necessary for this stress distribution and shown that the stresses in the concrete and steel can be expressed by simple formulae which are independent of the propping system. The method provides a convenient means for designing a suitable section quickly. The formulae are obtained as follows.

The properties of the section may be calculated by replacing the concrete by its equivalent steel area. In figure 111.

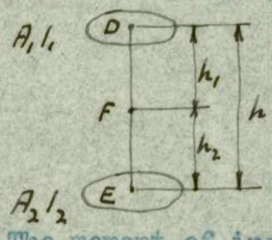


Figure III

D and E are the centroids of areas A_1 and A_2 with individual moments of inertia I_1 and I_2. If F is the centroid of the whole area then

$$h_1 = \frac{A_2 h}{A_1 + A_2} \qquad h_2 = \frac{A_1 h}{A_1 + A_2}$$

The moment of inertia of the whole is

$$I = I_1 + I_2 + A_1 h_1^2 + A_2 h_2^2$$

$$= I_1 + I_2 + \frac{A_1 A_2 h^2}{A_1 + A_2}$$

Suppose the moments at some point in the span (generally the centre) are:-

M_D due to dead loads)
M_F due to formwork) All taken as positive.
M_L due to live loads)
M_P due to propping system)

Then the steel alone has to carry a bending moment M_D $M_F - M_P$ which will generally be negative as the propping moment exceeds the others.

If V_1 and V_2 are the fibre distances to the top and bottom of the steel, the former negative and the latter positive and I_s the moment of inertia of the joist the tensile stresses are:-

At top of steel $(M_D+M_F-M_P)\dfrac{V_1}{I_s}$ At bottom of steel $(M_D+M_F-M_P)\dfrac{V_2}{I_s}$

Generally the former is tension (positive) and the latter compression (negative). When the concrete is set the props and formwork are removed, which is in effect equivalent to applying a positive moment M_P-M_F to the composite section. If the live load is then applied, the total bending moment acting on the section is $M_P - M_F+M_L$.

If the steel fibre distances are V_3 and V_4 to the top and bottom and I_c is the moment of inertia of the composite section, the tensils stresses are

At top of steel $(M_P - M_F + M_C)\dfrac{V_3}{I_c}$

At bottom of steel $(M_P-M_F+M_C)\dfrac{V_4}{I_c}$

Adding the stresses before and after propping the total steel stresses are

At top of steel $(M_D+M_F-M_P)\dfrac{V_1}{I_s}\ (M_P - M_F+M_L)\dfrac{V_3}{I_c}$

At bottom of steel $(M_D+M_F-M_P)\dfrac{V_2}{I_s}\ (M_P-M_F+M_L)\dfrac{V_4}{I_c}$

By equating the above expressions the propping moment to give equal stresses throughout the steel can be found

$$(M_P-M_F)\left(\frac{V_2-V_1}{I_s}-\frac{V_4-V_3}{I_c}\right)\quad M_D\left(\frac{V_2-V_1}{I_s}\right)+M_L\left(\frac{V_4-V_3}{I_c}\right)$$

But $V_2 - V_1 = V_4-V_3 =$ d the depth of the steel

$$\therefore\ (M_P-M_F)\left(\frac{1}{I_s}-\frac{1}{I_c}\right)=\frac{M_D}{I_s}+\frac{M_C}{I_c}$$

Giving $M_P = M_F + \dfrac{M_D I_c + M_L I_s}{L_c - I_s}$

Substituting $M_P - M_F$ from this expression in either of the expressions above gives the final steel stresses when equalised.

$$= \frac{M_D V_2}{I_s}+\frac{M_L V_4}{I_c}+\frac{M_D I_c+M_c I_s}{I_c - I_s}\qquad\left(\frac{V_4}{I_c}-\frac{V_2}{I_s}\right)$$

On reduction of this expression

$$fs = (M_D+M_c)\ \frac{V_4 - V_2}{I_c - I_s}$$

The final concrete stresses are those resulting form the moment M_P_M_P M_L

applied to the composite section. If V_5 and V_6 (generally both negative) are the fibre distances to the top and bottom of the slab, the concrete stresses are

$$\text{top of slab} \quad (M_P - M_F + M_L)\,\frac{V_5}{n\,I_c}$$

$$\text{bottom "} \quad (M_P - M_F + M_L)\,\frac{V_6}{n\,I_c}$$

substituting in these for the value of $M_P - M_F$ required to equalise the steel stresses

$$f_c\ (\text{top}) = \frac{(M_D + M_L)}{n\,(I_c - I_s)}\,V_5$$

$$f_s\ (\text{bottom}) = \frac{(M_D + M_L)}{n(I_c - I_s)}\,V_6$$

Although the actual shear force between the steel and concrete is given in the Journal also the bending moment in the deck slab, the actual nature of the reinforcing is not commented on. The shear reinforcing is shown on 65L-17 and c nsists of square reinforcing rods bent into the shape of hooks as illustrated and electrically welded to the top flange of the steel joist. The distribution of shear force S along the longitudinal member is shown diagramatically in Fig. 20C, and it is shown that the horizontal shear force at the top of the steel section is for this section .0317S. The force for which the stirrups must be designed is found in this way and the usual practice of bending the stirrups up at an angle of 45° in two directions and designing them to carry this horizontal force as a tension stress on the cross sectional area was followed. ½" and [illegible]" square stirrups were used and the spacing varied to provide the necessary reinforcement.

The 24 x 7½" R.S.Joists required splicing to make the length of 61ft. This splice was designed as a butt weld the webs and flanges being bevelled before welding and cover plates applied in the web to give an excess strength of 25% at the splice over the strength of the joist itself. These plates were spaced to distribute as far as possible the stresses in the web due to the attachment of the plate to the webs. More recent information indicates t e importance of shaping cover plates to reduce fatigue stresses due to alternating loads, experiments having shown that the strength of a member under this class of loading may even be reduced by attaching cover plates of improper design. The cover plate on the lower flange is attached by a continuous weld designed to prevent the effect of weathering and which gives ample strength; in this connection it is of interest to note that intermittent welding is subject to far higher fatigue stresses under alternating loading than the continuous weld run and for this reason should be avoided. The length of this cover plate was determined by examining the stresses on the section without the cover plate at points some distance from the centre line. Provision is made for expansion and contraction due to temperature changes in each span. On each pier there are four rocker bearings carrying the ends of the joists of the next. The design of the bearings is shown on 65L-12 and 65L-13 The reaction at each bearing is 26¼ tons and is transferred to a reinforcing mat of ½" diam. reinforcing rods cast in the concrete of the pier; a 10" length of a standard 100 lb. rail with welded stiffeners was used for the fixed bearing, a one inch diameter pin serving to fix the joist to the bearing. The size of the rocker of the movable bearing was obtained by using the formula P =600 D where P = load in lbs per lineal inch of rocker, D = diameter of roller required. The rocker and also the shoe and bearing plate are of cast steel, the height of the bearing plates could be conveniently and accurately fixed by adjusting the lower set of nuts on the holding down bolts before grouting in the plates and screwing down the top set of nuts. A ½" stiffener is placed between the flanges of the joist directly over each bearing.

The elasticity of the supporting beams was taken into account in designing the reinforcing for the deck slab and the live and dead load bending moments for different parts of the slab calculated by the method outlined in Section 4 of the Journal paper. The variation in stiffness of the beams for different parts of the span accounts for the variation in live load bending moment which are expressed in terms of Wl - and summarised in the graph of Figure 11. Mention is made of the method of distributing this moment over the effective width of the slab. This was determined as follows.

From test results an effective width, e, of slabs on rigid supports we find that at the support e = .71. In the following table the positive bending moment in the centre span of the deck slab due to a concentrated load is given in terms of Wl - the value of the moment is obtained by takingmoments of the external forces which are the reactions, as obtained from the corrected reaction diagram, due to the applied load W. The ratio of these moments to the moment at the support is calculated and it is assumed that the effective width of the slab increases in proportion to this ratio.

	M	Ratio $\frac{M}{M \text{ at support}}$	Effective Width e	
Support	.175Wl	1	.7x 93	= 65"
1/16 span	.213 "	1.22	.7x113	= 79"
1/8 "	.276 "	1.59	.7x148	= 104"
1/4 "	.362 "	2.06	.7x192	= 134"
1/2 "	.404 "	2.31	.7x215	= 151"

e for different parts of the deck slab was obtained by this means and the quantity of steel reinforcing proportioned in terms of this effective width and the dead and live load moments. An inspection of the quantities obtained enabled the reinforcing system detailed on drawing 65L-18 to be evolved. At the centre of the deck slab the maximum concrete stress in thetop of the slab is 680 lbs. per sq. inch dueto longitudinal bending giving a principle stress in the vicinity of 1090 lbs. per sq. inch. The concrete mix was designed for an ultimate strength of 3600 lbs.per sq. inch at 28 days and thefollowing results actually obtained.

Mix	Age	Average Strength	Number of Blocks
1:2:4	14	3267 lbs. per sq.inch	24
	21	4219 "	14
	28	4294 "	14

An average of 5.93 cub.ft. of cement was used per cub.yd. of concrete.

The deck slab is cast directly into a 7" x $3\frac{1}{2}$" channel at the end of each span. Some form of cross member is necessary at this point to carry wheel loads at the edge of the slab the assumption being made that the member takes half and the slab half - a 7" x $3\frac{1}{2}$" channel is found to serve this purpose and forms a convenient finish to the slab. The channel is bent to conform to the transverse camber of the deck and supported on stools made up from $\frac{1}{2}$" plate and welded to the top flange of the joist. Since the steel girders in this composite type of bridge are rigidly connected to the deck slab there is no need to provide bracing to carry wind loads and the practice has been followed with this bridge. The concrete deck of thebridge carries a bitumastic wearing surfact $\frac{1}{2}$" thick, the footways are built up above the road level and surfaced with precast concrete slabs 2" thick. This type of footway allows of a saving in dead load to be made. A light fence as protection from cattle crossing the bridge is provided between the foot and roadway on the upstream side of the bridge only. The main fence consists of reinforced precast concrete posts carrying a concrete coping on the top of two $1\frac{1}{4}$" diameter pipe rails. The method of attaching the coping * in a simple manner and at the same time to provide a means of adjusting minor errors in alignment of the posts. The general principle of precasting fence units etc. has much to commend it as it provides a simple method of disposing of small quantities of concrete left in the mixer from time to time and eliminates delay in constructing the fence when the rest of the bridge is completed. Lights are provided on each side of thebridge at the abutments and the second and fifth piers; the standards are of concrete and are precast.

* is designed to allow for expansion in the coping

APPROACHES.

The site for the new Leven Bridge has been the subject of keen discussion and although the site eventually adopted has advantages from the point of view of local traffic between the parts of the town on the two banks of the river, it has serious disadvantages in that existing roads and railways made it impossible to fix the position of the bridge to provide a good approach on the eastern side at a reasonable cost. Local interests pressed for the bridge to be made a continuation of Reiby Street. This proposal however was considered inadvisable owing to the proximity of shipping and the danger of boats striking the bridge in the strong tide. Failing this site the nearest practicable one to it was suggested as the most acceptable. At any point further upstream a level crossing in the wharf railway line was necessary in place of the overhead crossing at the Reiby St. site. With this condition for a level crossing, and that the maximum approach grade and minimum curvature on the approach road should be 1 in 20 and 1½ chains respectively, the position of the bridge became a matter of location. A subsidiary approach in a southerly direction at the eastern end was also provided, but this was of minor importance. On the western side the approach is straight with easy grades and involved the removal of a number of buildings and the construction of a new street to connect with the Main Coast Road approximately ¼ mile from the western abutment.

Details of the arrangements of the approach on the Western side as originally planned are shown on 65L-4,5 & 6. After construction work was commenced some alterations were made to provide easier grades at the entrance to Reiby Street, and these are shown on 65L-8 & 22. The stone filling in the banks of the approach roads was carried up 2 feet above high water mark to obviate any chance of erosion of the filling by the tidal waters.

3. C O N S T R U C T I O N.

1. GENERAL.

Owing to difficulties connected with the practice of carrying out bridge foundation work by contract, it has become the established practice in the Department to do this work by day labour, in fact contracts are only let for work which can be specified defintely and for which a reasonable price is tendered. The Leven Bridge however received special consideration as far as the contract versus day labour question was concerned owing to the fact that particularly accurate and careful work of an unfamiliar nature was required in the erection of the superstructure and also that the falsework required in connection with the substructure work could be made to serve also for the superstructure. These and other considerations influenced the Department in a decision to carry out the whole of the construction work by day labour, contracts being let only for the supply of materials. The work was therefore organised to allow construction of the substructure to commence from the eastern side followed by the construction of the superstructure from the same end as the piers were made ready for the beams. by this means the construction period was considerably reduced occupying only 10½ months from its commencement on January 9th 1934 to the official opening on November 26th 1934.

Two methods of driving the concrete piles of the substructure suggested themselves, one to drive the piles from a floating plant and the other from a fixed falsework. The former idea was rejected owing to the difficulty due to a 10 feet rise and fall of tide and the strong current; a timber falsework was therefore erected to carry the main pile driving frame and equipment, the piles in the falsework being driven in a convenient position to carry the screw jacks which supplied the propping forces necessary for the erection of the composite beams of the superstructure. The falsework was designed to carry the load from the jacks and support the pile frame as it was moved from pier to pier. At each pier the falsework was strengthened considerably to allow for the extra weight of a pile, the heaviest weighing 13 tons, and also to resist the effects of driving.

Plant.

The method of handling and driving the piles was influenced to some extent by the fact that although the piles were heavy and of considerable length necessitating heavy equipment the total number to be driven was only 18; it was obvious therefore, that the cost of the actual driving would be of minor importance compared with the cost of equipment and the cost of erecting and moving it so that speed in handling and driving could well be sacrificed if the cost of plant was thereby reduced. The piles were cast on the western bank of the river, rolled on to a punt and the punt floated across to the falsework. A steel pile frame 65 feet high carrying a 4 ton drop hammer and a double drum friction winch, operated by a 60 H.P. electric motor, was erected on the falsework and gear rigged to lift the pile off the punt from the frame itself using one drum of the winch to lift the pile and the other for the gear to pitch it. As the punt was available free of charge the cost of the pile driving plant was thus reduced to a minimum.

Electric power was supplied from a 6,600 volt line and transformed to 415 volts by a transformer at the welding bay on the western side of the river. An insulated 3 phase line was run across the river parallel to the bridge from this point and tapped at various points to supply power to the electric motor driving the winch on the pile frame and to a welding machine used at a later stage on the bridge deck. A second welding machine of sufficient capacity to provide for two welders was located at the welding bay itself.

Other machinery used on the work was part of the normal plant carried by the Department and arranged as self-contained units.

Material.

Contracts were let for the supply of all materials delivered to Ulverstone. Sand, cement andtimber were delivered by rail and the steel by boat to the wharf. This steel was picked up from the wharf by the punt, shipped across the river and transported on a light rail track to the welding bay, where it was stacked ready for fabrication. The steam crane on the punt was of sufficient capacity to load the steel beams from the wharf and directly to trucks after transportation to the western side of the river. A timber derrick of 4 ton capacity, hand operated, was provided in the welding bay for unloading this steel from the trucks and also for moving the beams about in the process of fabrication. The same derrick served to load the beams on to trucks on a line running across the falsework used for placing the beam in position on the bridge.

Falsework.

The falsework, as stated previously, served the dual purpose of carrying the pile driving gear for the concrete piles and the screw jacks which provided the propping forces at the centre and the quarter points of each span. The weight of the driving equipment was approximately 30 tons, exclusive of the pile which had a maximum weight of 13 tons, the prop load to each beam was 13.30 tons at the centre and 8.57 tons at the quarter points of each span; the general arrangement of the falsework to carry these loads is shown on plan 65L-16, but some additional piles were driven to facilitate the removal of the frame from pier to pier. All the piles in the falsework were driven from the punt by a gang of four men at an average rate of 7 per day, some difficulty was experienced in keeping them accurately in position owing to the strong current and this accounted for some delay. The piles were all driven about 10 feet with a 30 cwt. drop hammer. They were not shod but simply pointed with an axe. One set of piles in the sixth bay was omitted to allow for river traffic to use this opening and was not driven until the falsework had been removed from the second bay to allow boats to pass under the bridge at this point.

A second gang worked across the river bracing the piles of the falsework and sawing off the piles to the correct level. The falsework between the piers was not all braced at-once to allow the frame to be shifted, two bays were constructed to allow for this operation and after the frame had been moved the timber was transferred to the next unbraced bay and so on. Eventually when the falsework had to be removed, the braces were unbolted and taken ashore and a small charge placed in a hole bored in the pile at ground level by a diver. The charge was exploded by a submarine detonator andthe pile removed. This was found to be a simpler method than drawing the pile or sawing it off. A few of the piles were with-drawn, however, and used a second time.

Boring.

When the original examination of the bridge site was made test bores at one chain intervals were put down along the centre line of the bridge. A hand boring plant consisting of a hollow drill through which water was pumped at a pressure of approximately 100 lbs per sq. inch, was used for this purpose. Where the material was of a sandy nature or contained fine shingle a steel casing was used to prevent the drill from jaming. Different bits were used on the drill according to the nature of the material, but the calyx bit was generally found to be most suitable. This preliminary boring was done from a punt; it served to indicate in a general way the nature of the material in the river bed and from the information obtained the best type of substructure could be selected with confidence. It might also be mentioned that the results obtained indicated the recognised importance of using suitable equipment in testing the foundation material for bridges. A previous attempt to bore the river on a site close to the one adopted apparently showed solid rock within a few feet of the surface - an erroneous conclusion, for which unsuitable equipment was responsible.

From the evidence obtained as a result of this work the concrete pile substructure was adopted but there was so much variation in the material of the river bed that additional boring to determine the actual lengths of piles required was undertaken after the position of the piers had been fixed. At least two test bores were put down at each pier, one in the position of the up-stream pile, and the other in the position of the down-stream pile. If any marked difference was obtained another bore was put down between these two i.e. on the centre line of the bridge. All these holes reached a solid rock bottom, accurate records being kept of the various strata and plotted for each bore. Although this information was accurate it was not easy to know just how far a 22" pile could be expected to penetrate before reaching the required set. It was decided to make use of the water jet to facilitate driving in sandy materials and taking this fact into account the lengths of the piles were fixed on the assumption that they would reach their set when the head was level with the falsework and so allow the frame to be moved to the next pier. However if there was any doubt as to the length of pile required they were made longer rather than find when they came to be driven that they were too short. Generally speaking the lengths adopted were satisfactory. At No2. pier the piles could not be driven through the serpentine and about 15 feet had to be cut from each of the three piles. On all the other piers they drove approximately to the expected depth, in one or two instances the guides ofthe frame were extended and the pile driven a foot or so below the falsework before the set was obtained.

The extent of the boring that can be undertaken to advantage in and concrete pile work varies with the nature of the material through which the piles are to be driven and the uniformity in levels of any layer to which the poin is driven. The nature of the ground warranted a thorough investigation in this case the information being of value both in determining the lengths of the piles and during the driving. The same hand plant was used as for the preliminary boring but in this case the work was done from the falsework and a reciprocating jetting pump of 3,500 gallons per hour capacity at 150 lbs per sq. inch pressure was used. The cost including all items amounted to 4/7 per foot of bore hole.

SUBSTRUCTURE.

1. Eastern Abutment.

At high tide the water was 5 feet deep at the eastern abutment and 7 feet of soft clay and mud overlay the level of the foundation. A cofferdam 33 x 7' was constructed by driving 9" x 3" hardwood timbers round the outside of the foundation with a 10 cwt drop hammer from a timber frame. The cofferdam was suitably braced and the material excavated by hand from the inside. A 3" centrifugal pump unit was used to keep the cofferdam dry while the tide was high but towards the end ofoperations it become almost watertight due to swelling of the timber and the effect of clay being forced into the cracks from the outside. The excavation was carried about a foot into the serpentine and the concrete slab foundation then cast. Steel reinforcing dowels were cast into the slab to fix the columns and curtain wall and these members built up from the slab in the usual way.

No 1 Pier.

The serpentine was outcropping at this pier as shown on 65L-9 and at low tide was about 2 feet out of the water. It was only necessary to excavate deep enough to prevent any possibility of damage due to scour and this work was done between tides. As many men as possible were put on the excavation in order to reduce to a minimum the number of times the hole had to be dewatered. The foundation slab was cast and the columns and curtain walls constructed without any trouble. The reinforcing grids were electrical welded in the steel yard and placed in position as complete units. Provided the reinforcing is not too heavy the steel yard is the best place to do this work, as the time of fabricatio can be reduced by that means particularly if a number of similar units have to be made. There is very little to choose between tying and welding for holding the reinforcing in position. If a grid is to be moved about it will be a simpler job to use the welding - it should be noted too that additional reinforcing is often necessary to enable the grid to be moved without damage - but fabrication with the tie wire is as cheap and is to be preferred in that the section of the reinforcing rod is not reduced as is generally the case with the welded connection. A combination of tying and welding was used on the Leven Bridge according to the nature and situation ofthe work.

Piles - Manufacture.

The main rods for the pile reinforcing grids were bought in 30 feet lengths and had therefore at least one join in the length of the pile. A butt weld was used for this join and in addition some rods that had been cut to waste were put in at the splice. The main rods were spaced on jigs and the hoop reinforcing and stirrups, which had previously been bent to shape were slipped on and tack welded at the correct spacing along the length of the pile. The ends of the main rods were butt welded to the cast steel shoe care being taken to fix the shoe symmetrically on the end of the rods. The whole of the fabrication of the reinforcing grids was done by electric welding special electrodes suitable for striking an arc quickly being used. This is important because with some classes of electrods practically as much time is taken up trying to strike an arc as in actual welding. Care is necessary to see that the grids are not damaged when they are moved into the forms before concreting, either they should be designed to resist the stresses involved when they are lifted at one or two points only or adequate supports along the whole length of the grid should be provided during every stage of their transfer from the welding bay to the forms.

The casting bay for the piles was located on the reclaimed land on the western bank of the river; the reclaimed material was a mixture of sand and river shingle and therefore provided an excellent foundation. 9" x 3" timbers previously used for the cofferdam round the eastern abutment were founded on the sand at 2 feet centres and levelled with a surveyors level. 6" x 4" timbers at 2 feet centres were then placed at right angles on these and the forms for the piles built up on this foundation. The boxing for each pile was made as a separate unit although the opposite sides of the vertical posts between the piles carried the boxing for adjacent piles.

The concrete aggregates used for the piles and all other parts of the work consisted of crushed beach shingle and sand obtained from the Blyth River 10 miles away. The course aggregate was crushed from the particularly hard round stone which abounds on the N.W.Coast beaches, only pieces which would be retained on a 6" screen being used. It was crushed to 2" to $\frac{3}{4}$" "crusher run" for everything except the piles for which the specification called for crusher run metal of $1\frac{1}{4}$" to $\frac{1}{4}$". Although the reinforcing in the piles was closely spaced it was found possible to use the large aggregate and as higher strength was possible with this, at least a proportion was used and the surplus $1\frac{1}{4}$" aggregate placed elsewhere. The sand is recognised as of first class quality and calls for little comment. The mixture used was approximately $1:1\frac{3}{4}:3\frac{1}{2}$ and considering that no mechanical aids for tamping were available the high strengths indicated by the test blocks are very satisfactory. A number of cylindrical blocks either 10" x 5" or 12" x 6" were cast whenever concrete was placed and details of the mix, age, compressive strength etc. recorded. A synopsis of the results of test blocks taken from the piles is given on P. 2.

The slump test was not used in any of the work as the mixing, placing etc. was in the hands of experienced men who were conversent not only with the methods in use but also with the aggregates. The quantity of water was cut to a minimum and particular attention given to tamping and ramming the concrete into the forms. The latter feature no doubt accounts to some extent for the high strengths obtained but extensive testing on a large number of other bridges with various aggregates and procedure in mixing and placing has indicated that the crusher run course aggregates and Blythe River sand used on the Leven Bridge are the best obtainable locally and if these are used under close supervision, 28 day strengths of 4,000 lbs per sq. inch with a nominal 1:2:4 mix can be guaranteed. The piles were covered with jute bags after the concrete was placed and hosed three times a day for a week after casting.

As only a limited number of piles were required there was no necessity to shift them until they were to be driven. Owing to their length and weight particular care was necessary to avoid the development of tension cracks in the concrete due to handling stresses, the punt previously used for driving the timber piles of the falsework was fitted with supporting bearers and skids placed from the edge of the pile casting bay to the bearers on the punt. Two hand winches were rigged on the side of the punt farthest from the pile and when the tide brought the top of the bearers to the same level as the bearers on the casting bay the pile was rolled on to the punt by winding in wire ropes run from the winches to the pile, a turn being taken round the latter. To prevent the pile from bumping as it was rolled towards the punt two additional guys were taken from the pile to be moved to the adjacent pile on the casting bay. The guys were passed round this pile and payed out as the other was rolled forward by the winch ropes. The general arrangement is clearly indicated in one of the photos of Appendix 111. As the lengths of the piles had previously been determined from the borings and the piles cast on the casting bay in the order they would be required for driving no difficulty was experienced in obtaining the pile required without the services of a lifting crane.

A pile having been transferred to the punt the latter was warped across the river and moored against the falsework at the pier over which the pile frame was situated. As stated in the section on design, a 24 x 7½" steel joist was used as a stiff back for lifting the piles; it was kept on the punt and lifted on the uppermost face of the octagonal pile by a rope from the pile frame where it was secured by four wire ropes long enough to pass completely round the pile and stiff back twice and spaced at intervals along the length of the pile. Each rope was clamped back on itself and tightened by driving timber wedges under the rope at the top flange of the stiff back. Pieces of timber two inches thick were placed between the pile and stiff back to allow the head gear which was attacked to the pile alone to be fixed without fouling the stiff back. These details can be seen in photo No 26.

The steel pile frame, 65 feet in height and 16ft x 20ft at the base was erected on two 24 x 7½" steel joists 50 feet in length placed symetrically on either side of the centre line of No 2 Pier. Rails were welded to the top flanges of these joists and steel shoes put on the rails under the main members of the base of the frame. The winch and gear operating it was mounted on the back of the frame itself and by rigging the necessary gear the whole frame could be traversed on the rails by winding in a rope running from the friction winch on the frame to the joists supporting it. Originally this frame was equiped with wheels which were attached to the base of the frame and served to traverse it on rails but experience has shown that it is more satisfactory when driving piles to have the frame solidly supported and depend on sliding it when it has to be moved. The frame then was traversed to the downstream side of the falsework and after lifting and fixing the monkey and dolley at the top of the frame one gear was attached to the bridge for lifting the pile, and the second directly to the head of the pile for pitching it. Three heavy strops were passed completely round the pile and stiff back and shackled to one end of the bridle and to the pulleys through which the bridle passed at the points of support. The weight of the pile was then taken on the bridle gear, the punt floated clear and the pile swung into the vertical position by winding in the head rope. Additional guys were necessary on the head and point of the pile to steady its motion while it was being pitched. On reaching the vertical position the weight of the pile was taken on the head rope alone and the bridle and stiff back lowered again to the punt. The pile was then swung into the leaders where it was fixed by clamps, the frame carrying the pile traversed across the falsework to the position for driving and the dolley fitted in position on the head of the pile. After the three piles had been driven at No2 pier the frame and supporting beams were moved bodily by sliding them on rails running along the two sides of the falsework. Three hand winches supplied the power for this purpose, the frame being moved in stages by this means to the western abutment where it was dismantled and taken ashore.

Piles - Driving.

It has been pointed out elsewhere concerning the choice of hammer for driving concrete piles that the drop hammer is to be preferred to the steam hammer for most circumstances on the grounds of economy. In many cases the actual driving time is only a small percentage of the total time required for the work so that even a saving of half the driving time does not represent a large proportional saving in total time, this fact combined with the higher capital cost of the steam hammer, the large amount of steam required and the heavy tackle and equipment required to handle almost double the weight of hammer made the decision in favour of the drop hammer for the Leven Bridge piles an obvious one. Although there are some thirty or fourty pile formulae in existence and at least half a dozen in common use, the subject having received a great deal of consideration, there is still much information to be desired on the subject. It is not proposed to treat the subject in anyway exhustively here but the points which received consideration in deciding the set required to ensure that the piles would carry a 65 ton load will be indicated.

In the first place the weight of the drop hammer necessary to drive the heaviest piles of 13 tons is required. It is important to notice the effect on the efficiency of the driving of hammers of different weight. There are two main stages in pile driving, in the first the pile is hit by a hammer and set in motion, and in the second the resistance of the earth stops the pile. The effectiveness of the impact is a function not so much of the energy as of the momentum per blow.

If W = weight of hammer
P = " " pile
V = velocity of hammer at moment of impact
v = resulting velocity of pile and hammer together

Then momentum of the hammer before striking = WV., after the impact the total mass in motion is $(W+P)$ and the momentum is $(W+P)v$. Equating the momenta gives

$WV = (W+P)v$ or $\frac{v}{V} = \frac{W}{W+P}$, that is the ratio of the velocities before and after impact is the inverse of the masses in motion. The kinetic energy before impact is $\frac{WV^2}{2g}$, and the energy of the total moving mass after impact is $\frac{(W+P)v^2}{2g}$. Therefore the kinetic energy is reduced by the impact in the ratio $\frac{W}{W+P}$, and the resulting kinetic energy of the blow is

$\frac{WV^2}{2g} \cdot \frac{W}{W+P}$. In actual practice the available energy will be even less than this. The following analysis relates to the 13 ton pile being driven by drop hammers of various weights from one to ten tons, the fall being that necessary to give a blow of 20 ft tons in each case.

The figures show that the driving effect is much greater with a heavy hammer than with a light one for the same energy of blow, and the risk of cracking the pile is considerably greater with a light hammer and a high velocity than with

Hammer weight tons	Fall feet	W+P tons	V ft/sec.	v ft/sec.	K.E. after impact ft tons	K.E. after impact %	K.E. lost by impact. %
1	20	14	35.8	2.6	1.43	7.1	92.9
2	10	15	25.3	3.4	2.66	13.3	86.7
4	5	17	17.9	4.2	4.70	23.5	76.5
6	3.33	19	14.6	4.6	6.31	31.5	68.5
8	2.50	21	12.6	4.8	7.62	38.1	61.9

a heavy hammer and a small fall. The figures also show that any pile formula based directly on the product of hammer weight and fall is misleading. Fig. 4 shows a graph of driving energy plotted against the weight of the hammer expressed as a percentage of the weight of the pile. In practice the driving energy is decreased by the energy required to overcome friction and the elastic compression in the pile itself so that the resulting effective driving energy at the pile point is indicated by some such curve as the dotted one, which shows that over a certain range of weight ratio of hammer and pile there can be no driving, and that increase in the weight of the hammer has a very great effect.

The Brix pile formula is a driving resistance formula which taken account of the weight of the pile but no allowance is made for work done in compressing the dolley and packing or in compressing the pile itself but to offset this the available energy of the drop hammer after impact is neglected. This formula gives good results for sets greater than $\frac{1}{4}$". Hiley has gone a step further and makes allowance for these items, also for loss of energy due to errors in centring the pile and due to longitudinal vibrations.

The formulae is

$$R = \frac{12\ Wh}{s + \frac{c}{2}} \times \frac{W + P\varepsilon^2}{W + P}$$

where
R - ultimate driving resistance (tons)
h - equivalent height of free fall of hammer.
s - set per blow in driving (inches)
c - equivalent compression in pile and dolley.
ε - coefficient of resitution of dolley.

A 4 ton hammer was used at the Leven and the piles driven to a set of 1 inch for the last 7 blows with a 4 foot drop; inserting in the formula the other constants applicable for the conditions of driving

$$R = \frac{12 \times 4 \times 36}{.14 + .18} \times .24.$$
$$= \underline{108 \text{ tons.}}$$

It is recognised that this formulae gives a reliable estimate of the driving resistance for all values of the setwhich more recent work has shown to tend, if anything, to the conservative side for small values. As the piles were driven to a rock bottom with the assistance of the water jet, and the set measured before the effects of the water had entirely disappeared it was considered that there was no justification in driving the piles to a smaller set than the average "adopted of .14 " for each of the last seven blows. The piles were all driven to the same set although the shorter piles would indicate a somewhat higher driving resistance from the formula.

Particular care was necessary to see that the dolley retained a symmetrical position on the head of the pile. The dolley was of cast steel with a blue gum block in the top section and packing between the central diaphram and the head of the pile. The most effective arrangement of the packing was found to consist of two layers of pine separated by one of hardwood with woodwool

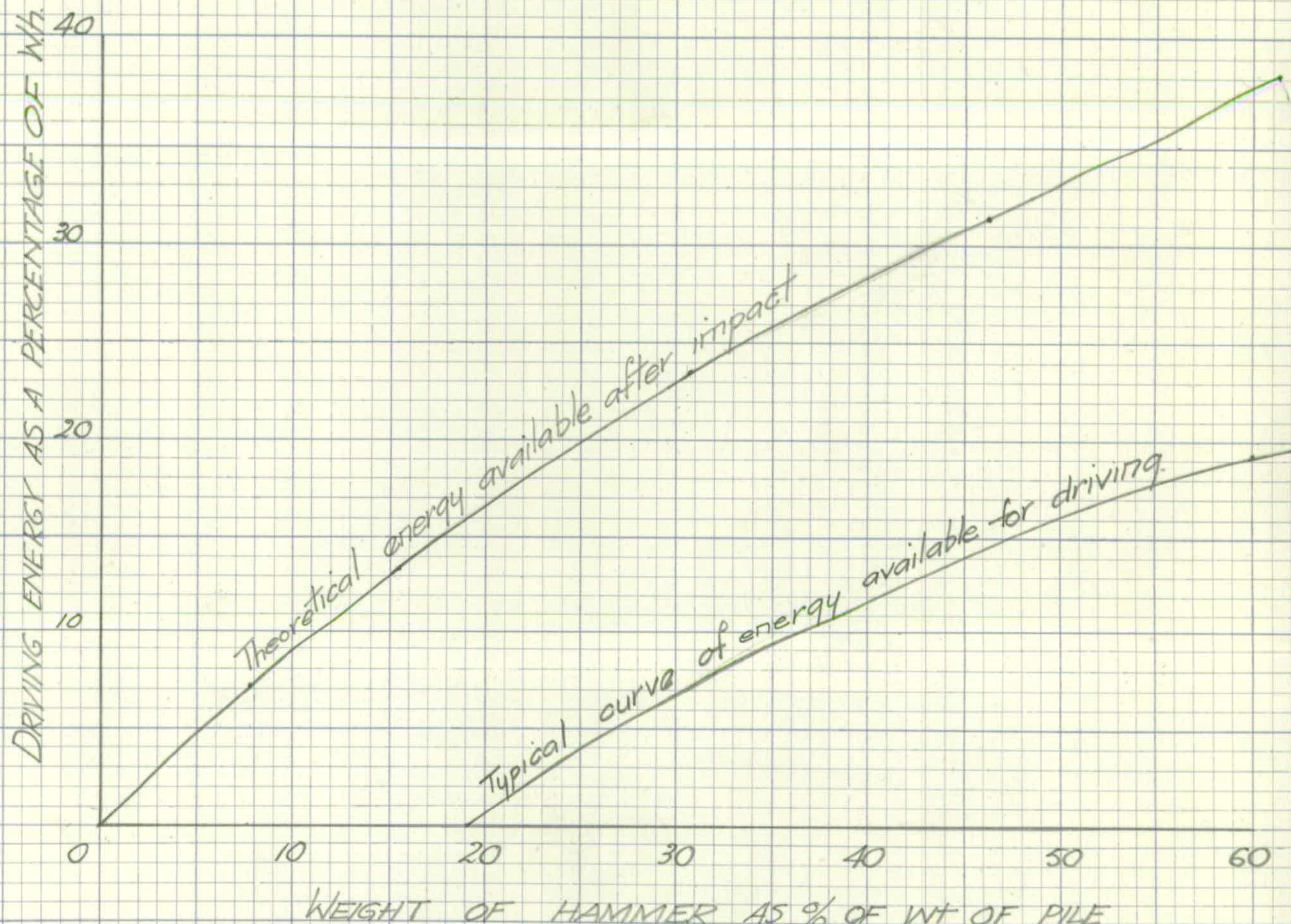

DRIVING ENERGY AS A PERCENTAGE OF Wh.
40
30
20
10
0
10
20
30
40
50
60
Theoretical energy available after impact
Typical curve of energy available for driving.
WEIGHT OF HAMMER AS % OF Wt OF PILE

FIG. 4.

between the pine and the head of the pile. This woodwool was only used to drive one or two piles and was then replaced by another lot. Two lugs cast on the sides of the dolley served to hold it in position, the lugs fitting round the leaders of the frame.

The nozzle shown in the plans was used in driving the first few piles but some alterations were made to the remainder. A reciprocating pump is generally used to supply the water to the jet, a safety valve being used to relieve the excess pressure should the jet become blocked. As it was intended to use the pump on various other works as a general purpose pump a centrifigal pump was used in this case with the idea of reducing weight. It was specially designed for the job with four stages, head 350 feet and capacity 10,000 gallons per hour and proved entirbly satisfactory. Care however is necessary to ensure that the delivery through the pile and nozzle is not unduly restricted otherwise the quantity of water is reduced below that required for effective jetting. The unit was belt driven by a Stutz petrol engine the whole outfit being mounted on the punt, connection being made to the pile through a fire hose and coupling. The nozzle as originally designed was not satisfactory although it may have been if a greater supply of water had been used, the four return outlets were therefore blocked up but without any appreciable improvement. The size of the central nozzle was then experimented with, a diameter of $1\frac{1}{2}$" finally being adopted as the best. The pump runner was of stainless steel to resist the corrosion of the salt water which was pumped from the river through the jet.

The work of driving the piles proved a particularly difficult proposition owing to the nature of the material to be negotiated. At the second pie with the water jet in operation the piles only needed light blows from the hammer to reach the level of the serpentine where after a tendency to run at the toe they soon reached the specified set. This tendency for the piles to run was difficult to prevent without damaging the piles. Heavy timber frames were constructed at low water level, the piles of the falsework being used as supports and the pile driven down through this frame. If the obstruction in the path of the pile was a boulder of any appreciable size the bending moment induced in the pile, due to the restraining influence of the frame, would either cause the concrete to crack or else force the frame out of position. To relieve this situation frequent resort was made to the hand jet the vicinity of the obstructior being thoroughly explored with the jet before any further driving was attempted. The extra lengths of the piles were cut off at the second pier when it was obvious that the expected penetration would not be attained and the pile frame moved to the third pier. The surface of the river bed at this pier was covered by several feet of sand and heavy shingle with the result that the jet water forced the sand away and left a solid mass of stone for the pile to penetrate. This hard patch on the surface made it difficult to keep the pile in the correct position only a small error at this stage of the driving causing considerable difficulty later on. The fact that it was necessary to keep the piles within a few inches of the correct position was the chief cause of trouble as once the best size of the jet had been obtained no great difficulty was met in obtaining the penetration. This ranged between 26 and 30 feet at this pier. At the fourth pier the driving was good, the penetration ranging between 34 and 37 feet. At the fifth pier boulders caused a great deal of trouble, three days being taken in driving one pile. The pile was withdrawn several times to facilitate the work of removing the obstructions which were of such a nature that on the downstream side of the bridge the pile only reached a penetration of 20' - 2" whereas on the upstream side the penetration amounted to 34' - 11"for the same set. The material at the sixth and seventh pier and at the western abutment was of a more consistent nature, mainly sand carrying fine to course shingle and the driving did not give very much trouble.

As the timber retaining wall on the western sideof the river was erected before the piles for the western abutment were driven it was necessary to transfer the piles from the punt to the filling behind the wall again before pitching them. This was done by hauling the pile off skids placed on the punt on to rollers carried by a 24" X $7\frac{1}{2}$" joist on it's flat, the joist being supportec so that the rollers were just above the level of the timber sheathing. This was done on a falling tide and the piles moved at such a rate that the rise in the level of the punt due to the removal of the weight of the pile equalled the fall in the water level. By this means the pile was removed without difficulty, sufficient rollers being used to reduce the bending moment in the pile to the required limit, and brought opposite the pile frame by which it was lifted from its supports and pitched ready for driving.

An accurate driving record was kept for each pile showing the number of blows, height of drop, jet pressure at various stages of the driving and so on. Curves have been plotted from these results showing penetration of pile per foot lb of energy delivered for various sizes of jet but the material was of such a varigated nature that no useful conclusions can be derived from them. About 2,000 blows of 3 feet drop was an average value for the number of blows required before the specified set was obtained but very often half the blows were spent in negotiating thin layers of shingle or boulders. The water jet was kept in action until the pile had reached a penetration sufficient to bring it on to a sound foundation as indicated by the test bore, it was then turned off and the driving continued under the hammer alone. For some cases, where the point of the pile had not reached solid rock, the additional penetration after the water was turned off was only a few inches, which illustrates the effectiveness of the jet.

A number of the piles developed small cracks during the driving particularly in the early stages of the work. The hard driving required to force the pile through the surface shingle was responsible for some of these but the chief cause was from the restraint placed on the piles to prevent them from running. This can be overcome in most instances by the use of external jets but here they were only of limited value. Even if the pile is allowed to follow a direction other than the vertical without restriction damage can be expected near the head of the pile owing to the fact that the frame is fixed. Both the position and the direction of the guides would need to be adjustable to overcome this trouble. However the cracks were not serious and had it not been for the large cover allowed on the steel would probably not have been visable, regular inspection both above and below water level shows that most of these have now disappeared. In no case was it considered necessary to sleeve the pile.

Any length of pile projecting above the level of the falsework was removed by chipping the concrete away from the main reinforcing rods with gads and then cutting the rods with an oxy torch. The piece was then easily pulled off. Instead of casting the concrete walings on the shore as was originally intended these were cast directly over the three piles of the pier, a piece of the curtain wall about three feet in height being cast on the waling between the piles. Sufficient clearance was left round the piles to allow the waling to be lowered by two chain blocks on to three timber clamps placed at the required level, one on each pile, by a diver. The space between the waling and the piles was filled with concrete placed through the water at lowtide and the curtain wall extended and cast monolithic with the cross beam. When the concrete was set the clamps were removed from the piles and transferred to the next pier. Considerable advantage was gained by pre-casting some of the curtain wall on the slab as only the spring tides were low enough to allow work at this level to be done in the dry. The reinforcing grids for the cross beams were fabricated in the steel yard but for convenience in handling were made in two pieces. $1\frac{1}{8}$" diam. rods were welded to those of the same size in the piles to run into the crossbeams to ensure adequate bond between these members. The holding down bolts for the bearing plates were cast in position in the top of the pier the lower of the two nuts on each bolt providing a particularly convenient method of adjusting the levels of the plates. The level of the plate having been obtained it was removed and the space underneath filled with cement mortar, on replacing the plate and screwing down the top set of nuts the excess mortar was squeezed round the sides of the plate.

SUPERSTRUCTURE.

The whole of the structural and reinforcing steel required for the superstructure was stacked adjacent to the crane in the steel yard. A welding bay was constructed on a timber foundation and two lengths of the steel beams set up in position on the bay ready for splicing. The beams were cut to length and the ends prepared for welding by a mechanically operated oxy-acetylene torch; the cuts were hand finished before being welded. The cover plates, splice plates etc, were all cut to shape with the torch and welded in position while the beams were on the welding bay. The square stirrups were bent by hand with the use of a jig and set in position by a template, first just being tacked in position and finally welded afterwards. It was found advantageous to use extra fluxed electrodes for this work the E.M.F. electrodes of this type giving good satisfaction. For all other work excepting the reinforcing grids for which rods designed for ease in striking the arc were used, the New Era electrodes manufactured by the E.M.F. Co. were used. The stirrups were put on while the beam was still on the welding bay, no difficulty being encountered in handling the beams afterwards. On completion of this phase of the work the member was placed on two bogey trucks carried by a line running across the falsework and hauled into position on the various spans. By fabricating the steel work and placing the four beams of each span in position as each pier was completed the difficulties attached to moving the beams over the tops of a number of piers were avoided. Only one track was used for taking out the beams; greased timber placed on the top of the piers serving as a base to slide the beams from this track to their respective lateral positions in the span.

Having welded the fixed bearing in position on the bearing plate it was a simple matter to drop this end of the beam into position and thus set the other end on the rocker. The four beams were then connected by the 7 x 3" channel by welding the channel to the stools which were already in position on the webs of the joist. When the deck concrete had set the bolts were removed and theholes filled up with weld metal. In addition to these bolts reinforcing rods were welded to the flange of one beam near a support and to another at a point ten or fifteen feet from the pier, a few of these braces was sufficient to fix the direction of the beams until the concrete was set.

Heavy timber beams across the top of each set of four piles provided the propping forces calculated in the section on design. The four jacks at the centre of the span were placed in position first and the beams jacked up the required amount; the ends of the beams were held on the bearing plates by steel clamps attached to the concrete cap of the pier and all measurements taken from a datum level established by stretching 22 gauge piano wires between the piers a few inches directly beneath each beam. The jacks at the quarter points were brought into contact with the bottom flange of the beams and then thedistance between datum and the flange at each jack checked and tabulated for reference. These measurements were checked at intervals while the deck concrete was being placed and any alterations necessary were made by adjusting the jacks. It was found better to set the jacks a little higher in the first instance as the load was sufficient to cause a loss in height of about $\frac{1}{4}$" in the supports, in any case it was easier to lower the jack than to raise it. All the jacks were of the screw type of 15 tons load capacity.

The boxing for the under side of the deck was made up in the form of shutters from $\frac{7}{8}$" hardwood flooring supported from bearers placed on the lower flanges of the beams. By handling the shutters carefully and painting with oil each time they were used the one complete set lasted for the seven spans, as a result the cost of this work was kept at a minimum. The jacks were not removed until the test blocks indicated a strength of 3,000 lbs per sq. inch in the deck concrete, the precast fence posts were then set in position and cast into the kerb. In each panel of the fence a break was made in the kerb-- to relieve the compressive strength at the top of the kerb due to bending under live load. If the pipe rail is threaded through the holes in the posts before the kerb is cast it saves any difficulty in doing this afterwards due to slight errors in alignment. The posts in the footway fence were also set up in position before any of the kerb was cast.

48 cubic yards of concrete were placed in each span of the deck in one operation. Two petrol driven concrete mixers and a gang of twenty men placed this concrete in about seven hours, the whole of the work being done

from the eastern side to avoid lifting the concrete from the level of the falsework to the level of the deck which would have been necessary from the other end. The mixing plant was moved out on to the deck after two spans had been cast in order to reduce the distance over which the concrete had to be barrowed. Although the reinforcing system in the deck slab was fairly complicated no difficulty was experienced in placing concrete containing 2" metal; use was made of heavy tamping rods to assist in placing the concrete the continual ramming of these heavy rods giving a very satisfactory job.

As successive spans of the deck were cast the beams were cleaned and painted and the temporary falsework removed. A primer known commercially as "Fishoilene" was applied to the steel first and then followed by a coat of aluminium paint. This paint is exclusively used by the Department for either high or pony type through trusses owing to the excellent lighting effect given the trusses by reflection from car lights and as it has given satisfactory service as a paint it is also used for steelwork in other bridges.

APPROACHES.

The stone filling of the eastern approach was placed by contract, also some of the earth filling at both ends of the bridge. These contracts were schedule rate contracts the material being paid for at so much per cubic yard measured either in position or in trucks as specified. Other than some stone for the retaining wall on the eastern side and for the road foundation which was obtained by contract the whole of the remaining work was done by day labour. This proved a convenient method as it formed a stock job for a number of labourers which were only required on the bridge deck when concrete was being placed. The work of demolition of buildings, road construction etc. was all straight forward and calls for little comment.

C O S T S.

An accurate costing system was maintained throughout the whole of the period of construction as it was realised that the information obtained would be of particular value in estimating the cost of future work. This information however is not available for inclusion in this thesis, but it is of interest to note that the actual cost of the bridge was slightly less than the estimate.

A P P E N D I X 11.

As the design of the longitudinal members and the reinforcing of the deck slab was based on the test results obtained from the 1/6th scale model of the bridge it was of interest to check these results by making a similar test on the bridge after it has been constructed. The first span was selected for this test and the 4 ton hammer applied as a concentrated load at points on the centre line of the span the deflections of the longitudinal members for each position of the load being measured by gauges.

The gauges were set up, the hammer moved into the first position and the resulting deflections indicated by the gauges recorded. Owing to the weight of the hammer and the consequent difficulty in moving it about it was necessary to reduce this movement to a minimum, the hammer was therefore moved across the deck in steps, the gauges being read at each position, and finally removed from the span. It was found then that the zero error in the gauges was sufficient to render the previous readings valueless. This error was traced to the effect of atmospheric temperature changes above and below the deck, the deflections produced by the normal temperature changes in a few hours being greater than those due to the hammer itself. This difficulty was partly overcome by setting the gauges at zero, loading the hammer in one particular position and then removing the hammer from the span to check up any zero error. This process was slow but sufficient information was obtained by this means to indicate the form of the influence lines for reactions. The influence line diagram plotted from the results is substantially the same as that obtained from a similar test on the model the results of which are given in Table 11. in the Journal. The diagrams are superimposed in Figure 5.

Very little useful information can be obtained from a test which involves the use of a load at a number of points, the complicated distribution of these loads rendering the results unintelligable; this fact accounts for the use of the concentrated load in the above test. It might be noted that the deflections measured for this type of structure for a given load are invariably less than those indicated by deflection formula. The elasticity of the structure under load was a feature of the Leven Bridge test.

An attempt was made to obtain some check on the deflections caused by atmospheric temperature changes. Figure 6. shows a graph of deflections of the two inside beams plotted against time and shows how the deck slab rises as the surface concrete expands due to the increased temperature. It is worth noting that these deflections exceed those due to the 4 ton hammer used for the distribution test but whether they involve any stress in the material of the superstructure depends on the distribution of temperature through the concrete deck and steel beams. If the temperature distribution between the deck surface and the bottom of the steel beam is linear then there is no stress but if the temperature distribution from the top to the bottom of the slab is linear and the temperature in the steel constant - a more probable arrangement - then temperature stresses are involved and these can be calculated. Observations indicate that the difference in temperature between the top and bottom of the slab might be as much as 20° F.

LEVEN BRIDGE TEST.

Influence line for Reactions

½ span.

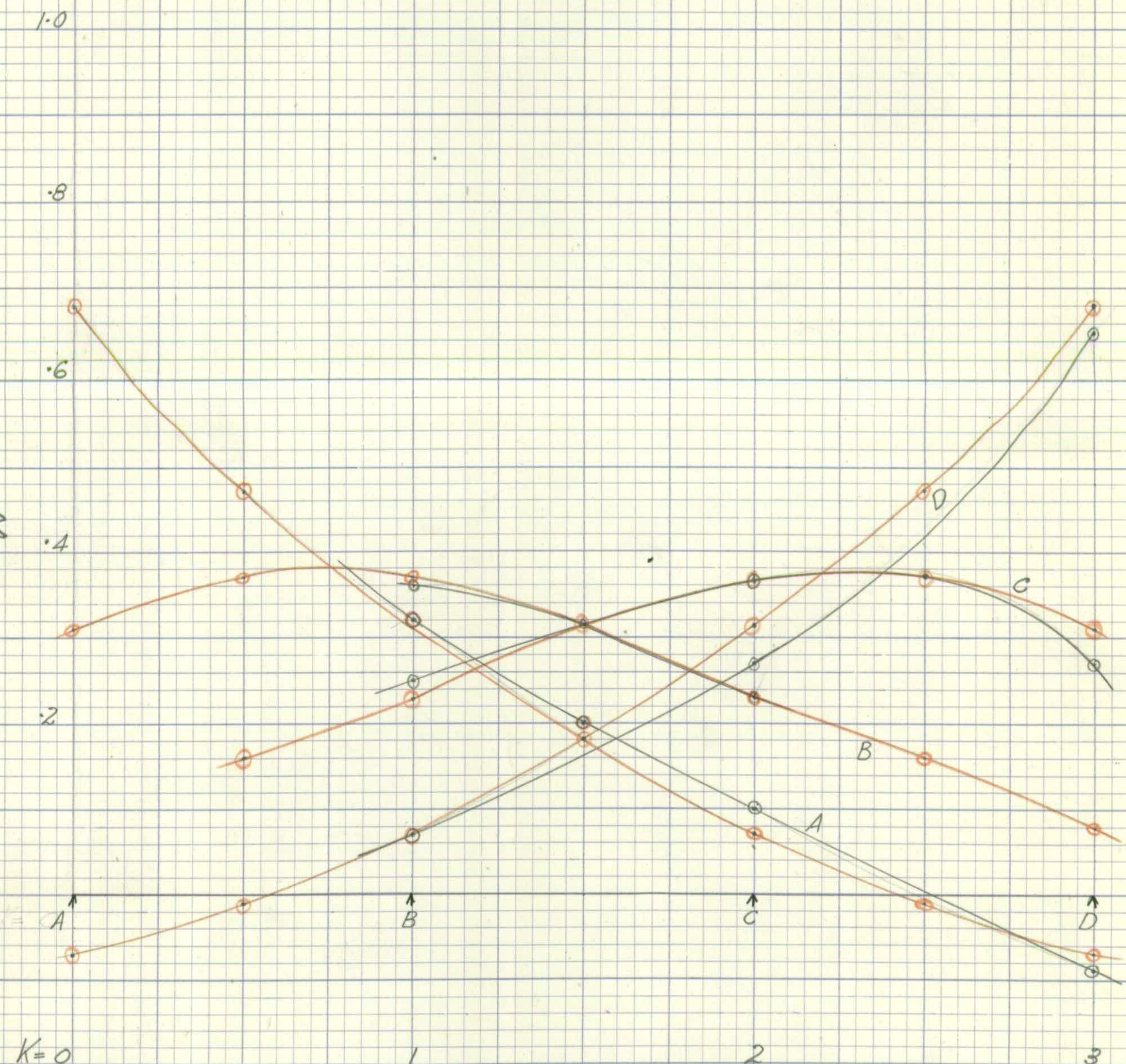


FIG. 5.

Model test thus

Leven Bridge test thus

UPWARD DEFLECTION
·05"
·04"
·03"
·02"
·01"
9
10
11
12
1
2
3
4 pm
TIME
Nº3 Beam
Nº2 Beam

FIG. 6.

The Design and Construction of Composite Slab and Girder Bridges.

By ALLAN WALTON KNIGHT, B.Sc. B.E.

*Junior.**

Summary.—The paper deals with the design of composite slab and girder bridges under the following headings :—

1. Introduction ; 2. Notation ; 3. Properties of the longitudinal member ; 4. Application to bridge design ; 5. Construction of composite beam bridges ; 6. Economics of the structure ; 7. Conclusion and 8. Acknowledgment.

An Appendix deals with the question of continuous beams on elastic supports.

1. INTRODUCTION.

In an attempt to develop economic methods for the design and construction of single and double track road bridges, close attention has been paid to this phase of the Public Works Department's activities in connection with the replacement of the wooden bridges throughout Tasmania by permanent structures.

The introduction of concrete slab decks for bridges has made a thorough investigation into the methods of design developed for timber decks and later applied to concrete decks essential. Despite the apparent simplicity of the problem, it becomes, in reality, extremely complicated owing to the complex nature of the distribution of a slab deck to elastic beams acting as supports.

In a paper† by G. D. Balsille, A.M.I.E.Aust., a description was given of the existing data and methods available for the design of simple bridges with concrete decks. In a second paper dealing with the distributing effect of the slab, read before the Tasmania Division of The Institution, an experimental method of obtaining the reactions to longitudinal supporting members for various ratios of stiffness of slab to beams was described.

Though closely allied to the distribution problems of the above papers, this paper is mainly a discussion of matters concerning the design and construction of a new type of bridge which consists of steel beams, in the direction of the span, carrying a concrete deck, which, with the steel, forms composite beams, the horizontal shearing forces in the section being provided for by the introduction of suitable steel reinforcement. The main features of the type of bridge to be described, as compared with other forms of road bridges, are :—

(1) The increased economy due to :
(*a*) the more effective use of the concrete of the deck slab ;

*This paper No. 470, originated in the Tasmania Division of The Institution and will be presented before the Engineering Conference in Hobart in February, 1934.

†See THE JOURNAL, Vol. 5, No. 2, February, 1933, p. 60.

In trusses of moderate span where the height is insufficient to allow of top sway bracing being used, the top chords must have sufficient transverse stiffness to prevent failure by buckling. This stiffness is provided partly by the member itself and partly by the web members, which provide an elastic resistance at each panel point.

First taking the case where the top chord is considered as hinged at the panel points, and suppose that the web system offers equal elastic resistance at each panel point, given by $W = Cy$, where y is the displacement.

If P be the thrust in the top chord, l the panel length, and y_1, y_2 and y_3 displacements at three successive panel points, then

$$W_2 = \frac{P}{l}(2\,y_2 - y_1 - y_3) = C\,y_2$$

so that

$$C = \frac{P}{l}\left(2 - \frac{y_1 + y_3}{y_2}\right)$$

By making successive deflections equal in magnitude and of opposite sign the maximum stiffness required becomes $C = 4\,P/l$. In practice, this would be multiplied by a factor to provide a suitable margin of safety.

When the top chord is continuous, the stiffness required at the supports is obviously less than when the joints are hinged. It does not necessarily follow, however, that the lowest buckling load occurs when the panel point deflections are alternately equal and opposite, since this involves the greatest bending of the top chord.

An exact analysis is difficult, but an approximate estimate of the buckling load may be made by assuming the elastic resistance of the web members uniformly distributed along the chord instead of being concentrated at the panel points.

If the elastic resistance per unit length is $w = c\,y$, it can be shown that the member bends in a sine curve. If the length of a half wave of this curve be L, the buckling load is found to be :—

$$P = \frac{\pi^2 EI}{L^2} + \frac{cL^2}{\pi^2}$$

It may be reasonably assumed that the length of the half wave will be such that the buckling load is a minimum. This length

bers may be different at different panel points, the above results may be applied in the design of open type bridges with some further margin of safety if the maximum thrust is used and the highest vertical members taken in determining the stiffness.

In calculating stiffness, flexure of cross members as well as of posts should be taken into account, and it is advisable to neglect the stiffness due to diagonal web members, which will generally be small.

It must not be forgotten that the results given concern the buckling or critical loads, and that adequate factors of safety must always be applied.

APPENDIX II.

By G. D. Balsille.

The experience gained by the Public Works Department in the design and construction of welded bridges has pointed to the following tentative conclusions :—

High Trusses Versus Low Trusses.

Owing to the greater homogeneity of welded work and its facility for transmitting vibration owing to the lack of "damping out" effect due to slip of joints, it would appear that stiffness of trusses should be given more consideration than in riveted work. This points to the adoption of the high truss in welded work when the field is now taken by the low truss in riveted work. The reduction of dead weight of some 20% to 30% of steel weight also lessens the inertia of the structure as a whole, and for similar live loading calls for a greater moment of inertia.

It is possible that the field of high trusses will extend, certainly to the 100 feet span with 20 feet panel, and probably down to the 90 feet span with 18 feet panel, and possibly down to the 80 feet span with 16 feet panel.

Economical Panel Length.

Economy points to the adoption of the larger panel.

Truss work is costly, but the floor system is relatively cheap, particularly where the floorbeams can be carried on top of the bottom

(*b*) the uniform tensile stress throughout the steel; and
(*c*) the suitability of rolled, as against more costly fabricated sections, for relatively long spans.

(2) The absence of unsightly deflections.

2. Notation.

I	moment of inertia (inches [4])
Z	modulus of resistance (inches [3])
E_s	Young's modulus for steel
E_c	Young's modulus for concrete
n	ratio $\frac{E_s}{E_c}$
f_{shear}	shear stress (lb. per sq. in.)
f_s	steel stress (lb. per sq. in.)
f_c	concrete stress (lb. per sq. in.)
M	bending moment (inch tons)
P	propping force (tons)
S	vertical shear force (tons)
s	horizontal shear force (tons)
y	distance from neutral axis to any layer (inches)
w_1	weight of beam, cover plate and stirrups (tons per ft. run)
w_2	weight of formwork and deck slab (tons per ft. run)
w_3	weight of formwork alone (tons per ft. run)
δ	deflection (inches)

3. Properties of the Longitudinal Member.

In the type of bridge which consists of a concrete slab supported by steel joists, a type which is very economical owing to the simplicity of the formwork, the concrete slab itself acts as a beam in the direction of the span with the piers or abutments as supports. Experiments have shown that the adhesion between the concrete and the upper face of the flanges of the joists is practically negligible, thus it can be assumed that the joists and slab bend independently of one another. The modulus of the section as a whole is therefore the sum of the moduli of the steel and concrete sections. The introduction of suitable shear reinforcing at the surface of contact of the steel and concrete serves to transform the section composed of two independent parts as described above to a composite section of greatly increased strength. This is the basic principle involved in the design of the particular type of bridge with which this paper is concerned.

Thus the longitudinal member consists of a steel beam connected to a slab of concrete, and its form must be such as to allow it to act as the main supporting member and at the same time as the deck slab. As far as the steel section is concerned, a rolled steel joist is suitable both on the grounds of its suitability for supporting the formwork and of economy. Steel stirrups electrically welded to the top flange provide the shear reinforcing.

The concrete section must be symmetrical, the thickness of the concrete being proportioned from considerations of deck slab design. Small variations in the thickness of the deck slab have only a small effect on the concrete stress due to longitudinal bending owing to the fact that this variation in thickness alters the value of y_c (max.) for the section. If it is necessary to reduce the maximum concrete stress the width of the section of the slab attached to each beam must be increased. The introduction of a concrete haunch between the joist flange and the slab greatly increases the modulus of the composite section and more than offsets the increased cost of formwork. The fact that a symmetrical section is required necessitating cantilever supports to the formwork for the outside members is to some extent an advantage in that the distribution of dead load to the longitudinal members is approximately the same for each. It therefore obviates the necessity of special methods of construction necessary to ensure that the distribution of the dead load of the liquid mass of concrete is the same as for the concrete in the form of a stiff slab—the form for which the most economical design of the longitudinal members can be obtained.

A typical section is shown in Fig. 1. It consists of a 24 in. x $7\frac{1}{2}$ in. x 90 lb. rolled steel joist carrying a 6 in. x $\frac{3}{4}$ in. cover plate welded to the bottom flange, a $7\frac{1}{2}$ in. concrete haunch with side slopes of 45° and a 7 in. slab with a width of 93 in. per joist. The moment of inertia of the composite section is calculated by replacing the concrete by the equivalent area of steel (taking n, the ratio of Young's modulus of steel to concrete as 12).

For the composite section

Height of neutral axis	= 27.05 in.
Moment of inertia	= 15,470 in.[4]
Section modulus for steel stress	= 572 in.[3]

For steel alone

Height of neutral axis	= 10.95 in.
Moment of inertia	= 3032 in.[4]
Z for top fibre	= 219.7
Z for bottom fibre	= 276.9

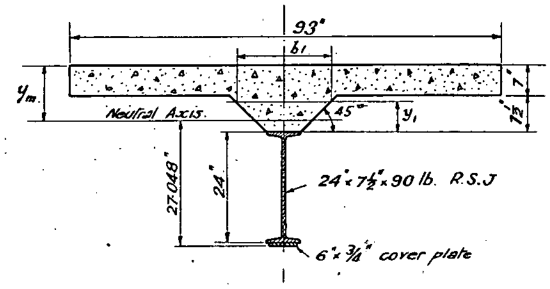


Fig. 1.—*Typical Section.*

The variation in the value of n for different classes of concrete has a considerable effect on the strength of the section and for any particular design it is desirable to make a close estimate of its expected value. For lower values of n there is a relative increase in the strength of the concrete to the strength of the steel, the neutral axis of the section moves away from the slab, tending to increase the maximum concrete stress. Since, however, the Z value of the section is increased and the stress is carried by a higher strength concrete the effect is not detrimental. Later in the paper a graph is given showing the variation in stresses for different values of n for a particular system of loading applied to a given section.

TABLE I.

The value of n is taken as 12.

Beam	Slab	Y_s	I	Z c.s.	Z r.s.j.	$\frac{Z_{c.s.}}{Z_{r.s.j.}}$
24in. x $7\frac{1}{2}$in. x 90lb. 2-9in. x 1in. C.P.	90in x 7in	27.28	21176	775	405	1.91
22in. x $7\frac{1}{2}$in. x 75lb.	,,	26.13	9076	348	153	2.27
20in. x $6\frac{1}{2}$in. x 65lb.	,,	24.55	7054	287	123	2.34
18in. x 6in. x 55lb.	,,	23.00	5291	230	94	2.45
16in. x 6in. x 50lb.	,,	21.53	4214	196	77	2.55
15in. x 6in. x 45lb.	,,	20.82	3779	182	66	2.76
14in. x $5\frac{1}{2}$in. x 40lb.	,,	19.87	3056	154	54	2.85

It is interesting to note the marked increase in strength of the composite section over the steel section alone as illustrated by Table I. Unless special steel reinforcing is provided to carry the shear stresses no tee-beam action is developed so that in these cases the effect of the concrete is neglected. The longitudinal members consist of rolled steel joists carrying 7 in. x 90 in. concrete slabs on haunches.

Shear stresses in the composite section can be calculated in the usual manner by the equation,

$$f_{\text{shear}} = \frac{S}{I\,b_1}\int_{Y_1}^{Y_m} b y\,\delta y \qquad \ldots\ldots\ldots\ldots(1)$$

In view of what follows, it should be noted that when the section is propped, the shear must be calculated in two parts.

(1) due to reversed prop loads; and
(2) due to live loads.

By evaluating equation (1) for any particular section, the horizontal shear stress on any plane is obtained in terms of the vertical shear.

The maximum shear stress in the concrete occurs at the junction of the concrete and steel, owing to the minimum breadth occurring there and to the neutral axis being close to the top of the section.

The amount of steel reinforcing required in the concrete portion of the section is therefore determined by the shear on this plane and although the shear for planes above this is progressively less, it is not practicable to reduce this amount owing to the necessity of obtaining sufficient bond for the steel.

4. The Application to Bridge Design:

It will be obvious that, if the formwork for the concrete is carried from the steel joists, when the concrete is placed dead loads will be carried by the steel alone. In order to make the composite section effective for dead as well as for live loads it is necessary to support the steel joists in some way until the concrete has set. A further advantage can be gained by the use of temporary supports in that a negative bending moment can be applied to the steel by propping it in a suitable manner to give an initial camber.

After the concrete has set and the props are removed the resulting stresses in the steel are tension throughout, and by careful adjustment of the amount of initial upward deflection it is possible to make the steel stresses under maximum load conditions practically uniform throughout, thus utilizing the steel to the best possible advantage.

Additional shear stresses are thereby introduced, but the saving in steel in the joist more than offsets the additional shear steel required.

The first step in the design of a longitudinal member for a bridge is the determination of the bending moments and shear forces acting on it. This involves a knowledge of the dead and live loads acting on the structure; in this respect this type of bridge is no different to any other form of concrete slab and girder bridge, in that an accurate determination of the distribution presents a problem of very great difficulty.

Actually the structure consists of an elastic slab on elastic supports, the elasticity of the slab being approximately constant throughout the span length, and the elasticity of the supports varying from zero at the piers to a maximum at the centre of the span. Under such varying conditions the load distribution is very different for different parts of the span and although at first sight it might not appear necessary to go to such refinements in design as to take account of these differences, yet in so far as the design of the deck slab is concerned a close investigation of the problem shows it to be very necessary in that even a slight departure from the condition of rigid supports might cause a total reversal of bending moment in the slab.

One of the difficulties of the general problem of load distribution by slab decks supported by beams is the lack of consistent information regarding the stiffness of reinforced slabs. If accurate information of this nature were available, or if it could be determined by analytical methods, there would be little difficulty in arriving at a reasonably accurate design. Because of this lack of information, it is desirable to resort to models of a proposed structure, but unless such models are almost exact replicas of the actual structure the introduction of new variables complicates the results to such an extent that they can only be used as a general guide, and are unsatisfactory from the point of view of design. The reason why there is very little useful information on the subject is to be found in this wide divergence of test pieces from the conditions obtained in the actual structure.

The distribution of the loads to each of the main longitudinal members varies, of course, with their number. The number of beams required depends on the width and to a lesser extent on the span of the bridge. It is found that a spacing of between 6 and 9 feet is convenient so that for ordinary widths of road bridge 2, 3, 4 or 5 longitudinal members are required. Since it is advantageous both from the points of view of design and construction to have similar members, it is economical to arrange them so that the maximum bending moment to each is, as far as possible, the same.

On account of its close connections to the problem in hand, extensive reference will be found, in what follows, to the reactions of a continuous beam on elastic supports. The appendix gives an outline of the general method involved in obtaining the reactions and support bending moments for an elastic beam on any number of supports and also the solution for the 3, 4 and 5 beam types. The 2 beam type is statically determinate in this connection although this is not the case when the beam is replaced by a slab.

In order to illustrate the method of design, the four beam bridge shown in section in Fig. 2 will be examined. The span is 60 feet; the live load consisting of a crusher train of $34\frac{1}{2}$ tons total weight and a passing 10 ton truck. is also shown in the figure. A live load of 80 lb. per sq. ft. is allowed on the foot paths. The 7 in. concrete deck slab carries a 20 ft. roadway and two 4 ft. footpaths. The longitudinal members are spaced at 7 ft. 9 in. centres and

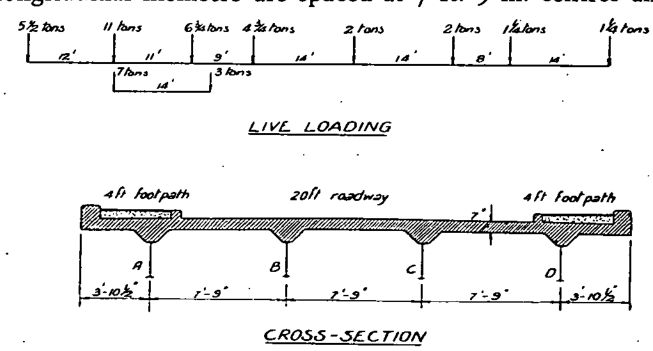


Fig. 2.—Cross Section of Four Beam Bridge.

are similar to the sections for which the properties are calculated earlier in the paper. The stresses in the outside members and the moments in the deck slab will be calculated.

As a preliminary step in the design of this bridge, influence lines for reactions are obtained analytically for a beam on four elastic supports for various values of the ratio $x = \dfrac{\text{stiffness of slab}}{\text{stiffness of supports}}$ the stiffness of the slab being defined as the central concentrated load required to give unit deflections over a span of $2l$, where l = spacing of the longitudinal members, and the stiffness of the support as the load required to give unit deflection of the longitudinal member over the span length, i.e., 60 feet. From various experimental information available, the values of x for the structure can be estimated. By using the corresponding reaction influence lines, obtainable from the equations of the appendix, the maximum reactions to the longitudinal members can be obtained for the dead and live load systems. It will be noticed that an approximation is involved here in that although the analytical solution for reaction influence lines is for a beam on elastic supports, these equations are actually applied to a slab on elastic supports. It is necessary to make this approximation since the analytical solution for the slab on elastic supports is not available. The results, however, are sufficiently accurate for a preliminary design of the cross section of the longitudinal member to carry the loads at satisfactory working stresses. In order to check the validity of the methods used in determining the load distribution to the beams, a $\frac{1}{6}$ scale model of this bridge has been built and extensive deflection experiments made on it. It consists of 4 in. electrically welded plate girders with $1\frac{1}{4}$ in. x $\frac{3}{16}$ in. flanges and $3\frac{5}{8}$ in. x $\frac{1}{8}$ in. webs. The lower flange carries a 1 in. x $\frac{1}{8}$ in. cover plate. A deck slab $1\frac{3}{8}$ in. thick with 10 and 12 gauge wire as deck reinforcing is cast on the beams. The concrete haunches are $1\frac{1}{4}$ in. high with 45° side slopes.

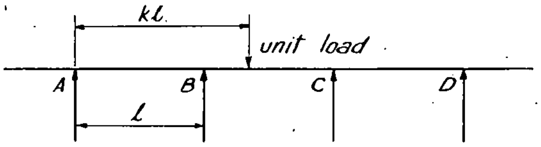


Fig. 3.

TABLE II.

Position of Load.	Reacts.ion			
k	A	B	C	D
— 0.25	0.752	0.291	0.048	— 0.095
0.0	0.683	0.309	0.078	— 0.070
0.5	0.477	0.369	0.161	— 0.007
1.0	0.326	0.370	0.232	0.072
1.5	0.180	0.320	0.320	0.180
2.0	0.072	0.232	0.370	0.326
2.5	—0.007	0.161	0.369	0.477
3.0	—0.070	0.078	0.309	0.683
3.25	—0.095	0.048	0.291	0.752

Note.—Position of load is expressed in terms of kl measured from support A. See Fig. 3.

Table II. gives the reactions for the centre span section of the model, obtained by measuring deflections under each of the four longitudinal members as a single concentrated load is moved in steps across the deck slab. Individual deflection readings are expressed as a proportion of the sum of the four readings, the result being taken as a measure of the reaction from the slab to the particular beam for which the deflection is measured. This is not necessarily correct as the load distribution probably varies for the different beams. The results obtained, however, appear to indicate that in this case the assumption is allowable.

The influence lines for reactions obtained in this way for the centre line section of the span are shown in Fig. 4. Plotted on the same diagram for a value of $x = 9$ are the reaction influence lines for a beam on elastic supports. The figure serves to illustrate the distributing effect of the slab as compared with the beam.

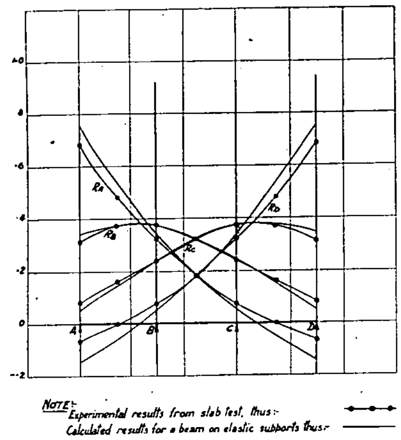


Fig. 4.—*Influence Lines for Reactions*, $\frac{1}{2}$ *Span*.

Although this experimental method of determining reactions would prove satisfactory for a beam on elastic supports it is not a true indication of the distribution of the load by a slab, as no account is taken of torsional moments which are negligible for the beam but considerable for the slab. The method of adjusting the results to allow for this torsion is illustrated by the following example. Test results from the model give the following reactions for a unit load over the left hand beam on the centre line section of the span.

$R_A = 0.683 \quad R_B = 0.309 \quad R_C = 0.078 \quad R_D = -0.070$

The forces acting on the system are shown in Fig. 5.

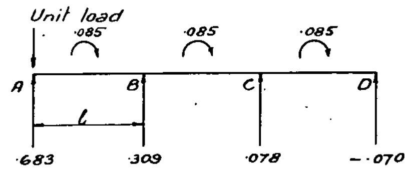


Fig. 5.

Any such system of forces must be in equilibrium. Resolving the forces in a vertical direction :—

$$0.683 + 0.309 + 0.078 - 0.070 = 0.$$

Taking moments about A

$$0.309 \times 1 + 0.078 \times 2 - 0.070 \times 3 = M_T$$
$$0.309 + 0.156 - 0.210 = M_T$$
$$0.225 = M_T \qquad \text{taking } l = \text{unity.}$$

M_T is the moment which must be introduced to the system to give equilibrium. Actually it is supplied by the torsional resistance of the vertical faces of any transverse strip of the slab and therefore not shown as a reaction by the deflection measurements. This torsional moment in a slab under load constitutes one of the fundamental differences between a slab and a beam. If the above reactions are adjusted in terms of this moment they will correspond to the reactions obtained for a beam under the same condition of loading.

Assuming M_T evenly distributed over the sides of a transverse strip of the slab, the moment for a length l of the strip $= \frac{0.255}{3} = 0.085$ acting in the direction shown in Fig. 5. Each of these component moments can be resolved into a pair of equal and opposite forces of 0.085 acting at the adjacent supports. Summing the forces at each support the adjusted reactions are obtained :—

$$\begin{aligned} R_A &= 0.683 + 0.085 = 0.768 \\ R_B &= 0.309 - 0.085 + 0.085 = 0.309 \\ R_C &= 0.078 - 0.085 + 0.085 = 0.078 \\ R_D &= -0.070 - 0.085 = -0.155 \end{aligned}$$

Adjusting the corresponding reactions, in this way, for each position of the load, a new set is obtained; this set is shown plotted in Fig. 6. Also plotted on the same diagram are the reactions as calculated from the analytical equations for the four beam bridge for a value of $x = 9$. (This value of x is obtained by substituting values of reactions taken from the adjusted test results in the appropriate equations and solving for x).

What discrepancy there is between the two sets of curves may be partly due to the assumption that the torsional moment is evenly distributed.

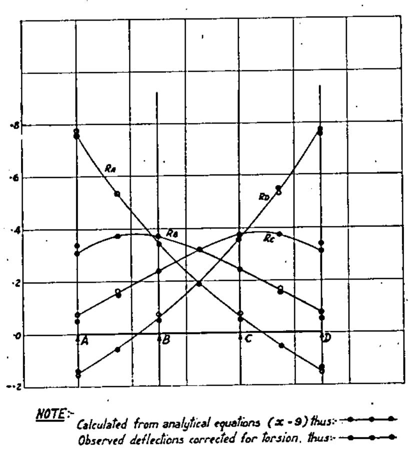


Fig. 6.—Reaction Influence Lines.

Reaction influence line curves obtained experimentally for the sections at $\frac{1}{8}$, $\frac{1}{4}$ and $\frac{3}{8}$ of the span length confirm the supposition that the stiffness of the slab is approximately constant throughout the whole span whereas the stiffness of the longitudinal member varies from zero at the abutments to a maximum at the centre of the span. It is a simple matter to calculate the stiffness of the longitudinal member at any particular point; the value of x at the centre is 9. It therefore follows that x varies approximately as the reciprocal of the stiffness of the longitudinal member, i.e., is 0, 0.494, 1.723, 5.062 and 9 at the abutment, $\frac{1}{16}$, $\frac{1}{8}$, $\frac{1}{4}$ and $\frac{1}{2}$ span points respectively.

Actually the values obtained experimentally were 0, 1.5, 3, 5.7 and 9 and reaction influence lines for the above values of x are plotted in Figs. 7 to 10. They serve to show the high distributing effect of the slab over the greater portion of the span, also the sudden change in distribution for a slight departure from the condition of rigid supports and the more gradual change in distribution when the value of x becomes appreciable. These changes in distribution are of extreme importance as far as the design of the deck slab itself is concerned.

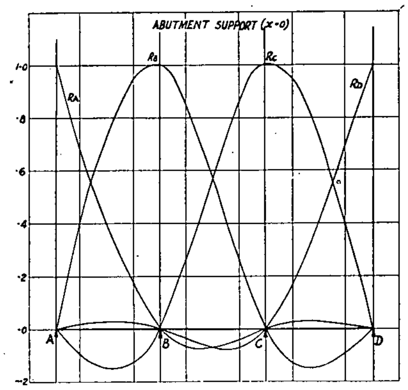


Fig. 7.—Reaction Influence Lines.

The live load bending moments in the slab can now be calculated. The maximum moment curves obtained by drawing envelopes over the series of bending moment diagrams obtained by moving two unit loads spaced at 6 feet centres from the outside beam to the centre of the deck in steps can be obtained for the sections of the slab for which the values of x are noted above. The bending moment for any point, in terms of Wl where W is a wheel load, is simply obtained by taking moments of the external forces as obtained from the corrected reaction diagram. (If the value of x is known, the reaction diagram can be determined analytically.)

Fig. 11 shows a diagram of the maximum live load moments for the inside and outside slab spans and the inside supports plotted against the span length. A positive moment is defined as one producing compression in the top of the slab. The coefficient of maximum moment in the inside spans increases from 0.149 at the abutment support to 0.566 at $\frac{1}{2}$ span and in the outside span from 0.166 to 0.476. The support moment coefficient varies from —0.174 at the support to —0.150 at $\frac{1}{2}$ span. An interesting feature of this diagram is the curve showing the ratio of maximum negative to maximum positive moment in the slab. This ratio is practically constant at 0.28 over $\frac{7}{8}$ of the span length; the ratio of steel area required in the top and bottom of the slab for live load moment is, of course, the same. The moments shown in the graph must be distributed over the effective width of the slab and added to the dead load moments (which are constant for any section of the span) to obtain the moments for design purposes.

The introduction of the idea of effective width is necessary in order to form a practical basis for the design of the reinforcement. Extensive tests (see Vol. 7 No. 1 *Public Roads*) indicate that the effective width of a slab can be expressed in terms of the slab width, which in this case is the distance between the longitudinal members. The results of these tests are applicable at the abutment where under a distributed load the slab has a point of contraflexure at each support but for other points in the

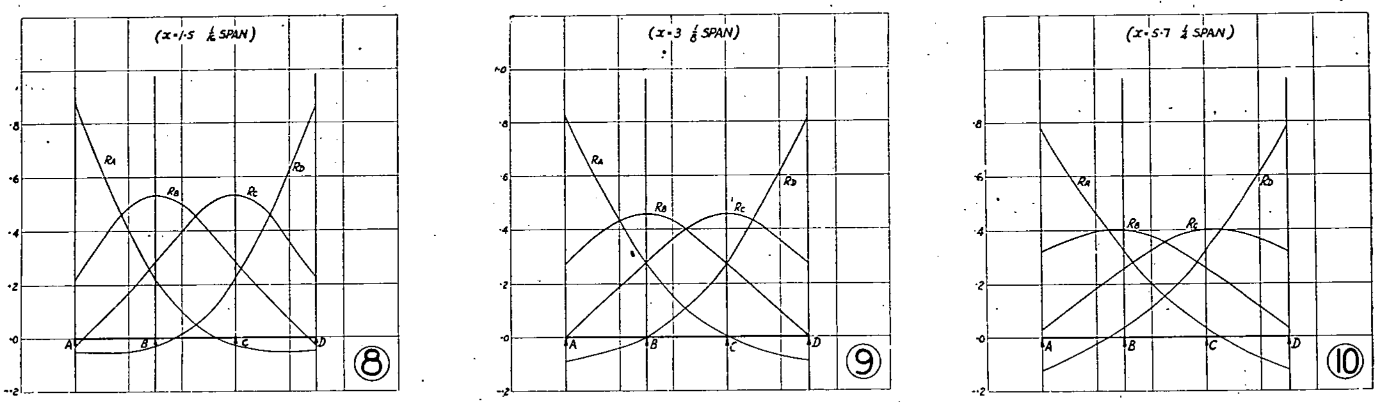


Fig. 8, *Fig* 9, *Fig.* 10.—*Reaction Influence Lines.*

span where, owing to the elasticity of the supporting members, the slab tends to bend in a smooth curve throughout its full width, the slab width mentioned above is increased with a consequent increase in the effective width. By comparison of the bending moment diagrams for a unit load on the slab at the abutment and at the centre of the span it is found that the effective width at the centre is approximately twice that at the abutments.

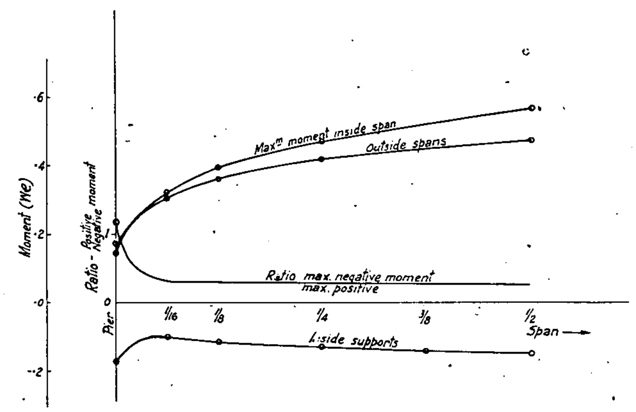


Fig. 11.—*Maximum Moments in Slab, Live Load Coefficients of Wl.*

The above analysis demonstrates the possible errors involved in assuming the longitudinal members of a bridge deck rigid. So much for the deck slab.

By using Fig. 4, the dead and live load reactions to the longitudinal members can be obtained by ordinary influence line methods. It is not necessary to consider the variation of the distribution near the abutment supports as over the greater portion of the slab (the portion where the loads produce the greatest moments) this variation is only slight. Having obtained the maximum wheel load reactions on any member by the above method the bending moment and shear forces acting on the member can be calculated and hence the stresses in the member due to these moments and shears.

To illustrate the method of calculation of the stresses, it is proposed to work out for the section already referred to, the dead load stresses due to the weight of the structure and the propping system and the live load stresses for the loading adopted as a standard for bridge design in the Public Works Department and shown in Fig. 2. The points which need consideration in deciding what form of propping system should be adopted are worth noting.

The propping forces required, even for long spans, are relatively small so that it is not a matter of dividing the load among a number of props. The important consideration is the stress distribution throughout the section in the direction of the span. It is possible to express analytically the stresses in terms of the live loads and the propping forces; the latter can then be evaluated for any particular distribution and applied by a suitable adjustment of the prop levels. In the example the system adopted consists of producing the initial stresses in the steel joist by the application of a single propping force at the centre of the span. Two additional props are then placed in contact with the cambered beam at the quarter points and maintained at this level while the formwork is fixed in position and the concrete placed. It will be seen that the stresses produced are not a maximum at the centre but at two points, one on either side of the centre; it is necessary to adjust the propping forces to suit the properties of the section so that a reasonably good distribution is obtained.

The value of the initial stresses to be placed in the steel joist are fixed arbitrarily but although any initial value can be obtained by adjustment of the prop levels, this value will be modified by the loads due to first the formwork and later the concrete of the deck. Suppose the upward deflection is required to produce a compressive stress of approximately 3 tons per sq. in. in the steel. If this stress is produced by a single prop at the centre of the span:

$$\text{then } \delta = \frac{Pl^3}{48E_s I}, \text{ where } P = \text{propping force} \quad \dots . (2)$$

$$\text{and } f = \frac{My}{I} = \frac{Ply}{4I}$$

$$\therefore \delta = \frac{fl^2}{12Ey} = \frac{3 \times 2240\,(60 \times 12)^2}{12 \times 30 \times 10^6 \times 10.95} = 0.884\text{in.}$$

The downward deflection due to the dead load of steel joist, stirrups and cover plate is given by

$$\delta = \frac{5}{384}\frac{wl^4}{EI} = \frac{5 \times 0.046 \times 60\,(60 \times 12)^3 \times 2240}{384 \times 30 \times 10^6 \times 3032}$$
$$= 0.329\text{in.}$$

and the corresponding stress is 0.893 tons per sq. in., tension. If an upward camber equal the sum of these deflections i.e. $0.884 + 0.329 = 1.213$ in. is introduced by the central prop an initial stress of $3 + 0.893 \times \frac{5}{4} - 0.893 = 3.224$ tons per sq. in. is obtained. The force on the central prop required to produce this deflection is given by

$$P = \frac{48E_sI\delta}{l^3} \quad \text{(by transposing equation (2))}$$

$$= \frac{48 \times 30 \times 10^6 \times 3032 \times 1.213}{(60 \times 12)^3 \times 2240} = 6.334 \text{ tons}$$

Two additional props are now introduced at the quarter points and placed in contact with the lower flange of the joist. The support bending moments and reactions for the system of loading shown in Fig. 12 are as follows:

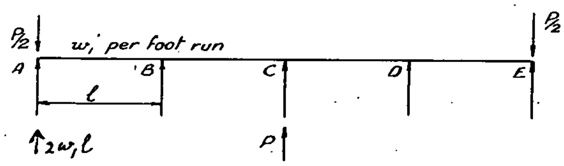


Fig. 12.

where w_1 = weight of beam, cover plate and stirrups per foot = 0.046 tons

$$M_A = M_E = 0$$

$$M_B = M_D = \frac{P}{2}.l - \frac{3w_1l^2}{2} = \frac{6.334}{2} \times 15 - \frac{3 \times 0.046 \times 15^2}{2} = \cancel{42.330}\ 39.75 \text{ ft. tons.}$$

$$M_C = \frac{P}{2}.2l - 2w_1l^2 = \frac{6.334}{2} \times 30 - 2 \times 0.046 \times 15^2 = 74.310 \text{ ft. tons.}$$

$$R_A = R_E = -\frac{P}{2} + \frac{4w_1l}{2} = -\frac{6.334}{2} + \frac{4 \times 0.046 \times 15}{2} = -1.787 \text{ tons.}$$

$$R_C = P = 6.334 \text{ tons.}$$

The shear and bending moment diagrams for this system are shown in Fig. 13.

The three props are maintained at their original level while the formwork and deck reinforcing is placed in position and the deck slab cast. For this condition the joist acts as a continuous beam of four equal spans, of length l feet, carrying a distributed load of w_2 tons per foot run.

The support bending moments and reactions are

$$M_A = M_E = 0$$

$$M_B = M_D = 3/28w_2l^2 = \frac{3}{28} \times 0.500 \times 15^2 = 12.054 \text{ ft. tons}$$

$$M_C = \frac{1}{14}w_2l^2 = \frac{1}{14} \times 0.500 \times 15^2 = 8.036 \text{ ft. tons}$$

$$R_A = R_E = \frac{11}{28}w_2l = \frac{11}{28} \times 0.500 \times 15 = 2.946 \text{ tons}$$

$$R_B = R_D = \frac{8}{7}w_2l = \frac{8}{7} \times 0.500 \times 15 = 8.571 \text{ tons}$$

$$R_C = \frac{13}{14}w_2l = \frac{13}{14} \times 0.500 \times 15 = 6.964 \text{ tons}$$

where $w_2 = 0.469 + 0.031 = 0.500$ tons per ft. run
$l = 15$ feet.

The bending moment and shear diagrams for this system are shown in Fig. 14.

Superimposing these bending moments and shearing forces on those of the previous system, the following total moments and reactions are obtained on the steel before removal of the props or formwork.

$$M_A = M_E = 0$$
$$M_B = M_D = \cancel{42.330}\ 39.75 + 12.054 = \cancel{54.384}\ 51.80 \text{ ft. tons}$$
$$M_C = 74.310 + 8.036 = 82.346 \text{ ft. tons}$$
$$R_A = R_E = -1.787 + 2.946 = 1.159 \text{ tons}$$
$$R_B = R_D = 8.571 \text{ tons}$$
$$R_C = 6.334 + 6.964 = 13.298 \text{ tons}$$

The total moment acting on the steel is shown in Fig. 15 (obtained by summing the diagrams of Figs. 13 and 14).

The maximum steel stresses at the centre of the span before removal of the props are therefore given by

$$f_{s1} = \frac{My}{I_s} = \frac{82.346 \times 13.80 \times 12}{3032} = 4.498 \text{ tons per sq. in.}$$

$$f_{s2} = \frac{82.346\,(-10.95) \times 12}{3032} = -3.569 \text{ tons per sq. in.}$$

The corresponding stress diagram is shown in Fig. 16*a*.

When the concrete has set, the formwork and the props at the points *B*, *C* and *D* can be removed. The stresses resulting from the removal of the above falsework will be carried by the composite section. Suppose the formwork weighs, w_3 tons per foot run and that it is removed before

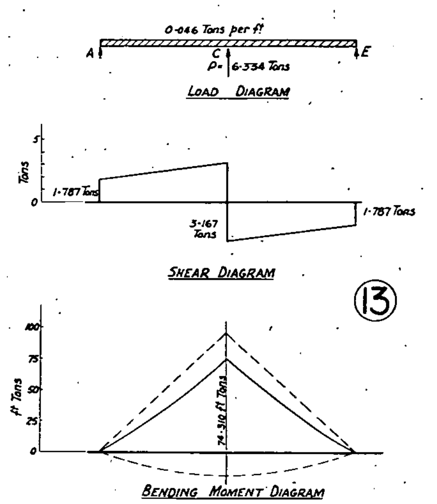


Fig. 13.

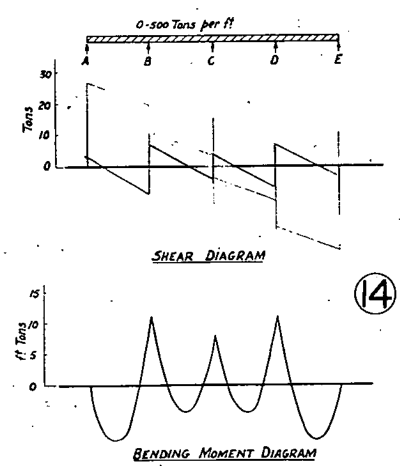


Fig. 14.

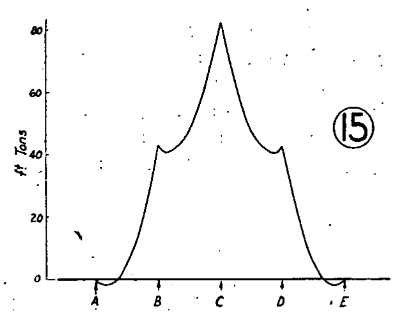


Fig. 15.—*Total Bending Moment Diagram. Props in Position and Concrete Placed.*

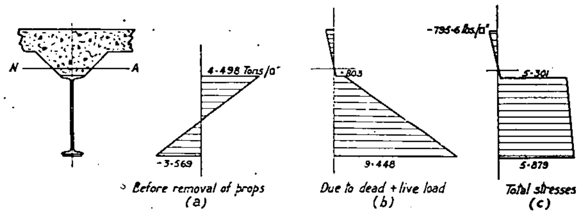


Fig. 16.

the props. Then moments and reactions at the supports due to its removal are:

$$M_A = M_E = 0$$

$$M_B = M_D = -\frac{3}{28}w_3 l^2 = \frac{-3}{28} \times 0.031 \times 15^2 = -0.747 \text{ ft. tons}$$

$$M_C = -\frac{1}{14}w_3 l^2 = -\frac{1}{14} \times 0.031 \times 15^2 = -0.498 \text{ ft. tons}$$

$$R_A = R_E = -\frac{11}{28}w_3 l = -\frac{11}{28} \times 0.031 \times 15 = -0.182 \text{ tons}$$

$$R_B = R_D = -\frac{8}{7}w_3 l = -\frac{8}{7} \times 0.031 \times 15 = -0.531 \text{ tons}$$

$$R_C = -\frac{13}{14}w_3 l = -\frac{13}{14} \times 0.031 \times 15 = -0.432 \text{ tons}$$

Thus the prop loads removed are

$R_B = R_D = 8.570 - 0.531 = 8.039$ tons
$R_C = 13.298 - 0.432 = 12.866$ tons

They are shown diagramatically in Fig. 17 together with the reactions at A and E caused by their removal.

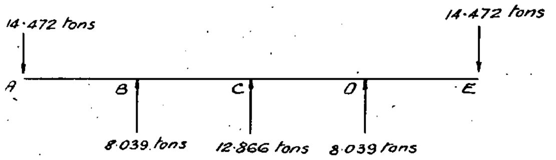


Fig. 17.

The reversed bending moments (obtained by taking moments of the forces of Fig. 17) are therefore

$M_A = M_E = 0$
$M_B = M_D = -14.472 \times 15 = -217.080$ ft. tons
$M_C = -14.472 \times 30 + 8.039 \times 15 = -313.575$ ft. tons

They are shown plotted in Fig. 18.

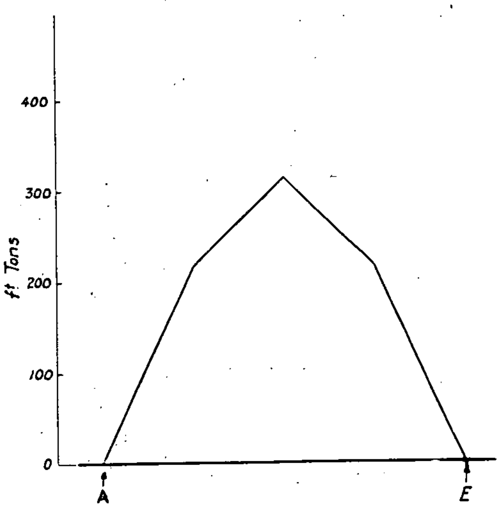


Fig. 18.—*Reversed Bending Moments Carried by Composite Beam when Props are Removed.*

It remains to calculate the live load bending moment before the total stresses in the section can be calculated. The live loads are placed in the centre of their respective traffic lanes and the wheel reaction ordinates read from the influence line diagram of Fig. 4. Multiplying the ordinates by the moments produced by the corresponding train of wheel loads on a 60 ft. span, i.e., 150 ft.-tons for the crusher train and 62.5 ft.-tons for the truck, the maximum moment to each longitudinal member is obtained. This moment is increased by 15% for impact but as no allowance is made for the distributing effect in the direction of the span of the slab on the concentrated loads, the impact allowance is actually in the vicinity of 25% to 30%. It is assumed that the live load on the foot path is negligible when the full vehicular live load is on the bridge.

Thus for the outside beams at the centre section of the span:

Live load moment =
$-1.15\ \{ 150\ (0.493 + 0.243) + 62.5\ (0.131 = 0.004) \}$
$= -1.15\ \{ 110.50 + 7.98 \}$
$= -136.252$ ft. tons.

Hence the total moment for this section is
$M_C = -313.575 - 0.498 - 136.252 = -450.325$ ft. tons.

and the stresses due to this moment acting on the composite section are:

$$f_{c(\text{max.})} = \frac{M_c y}{nI} = \frac{-450.325 \times 12 \times 12.20 \times 2240}{15471 \times 12} = -795.6$$
lb. per sq. in.

$$f_{c(\text{min.})} = \frac{-450.325 \times 12\ (-2.30) \times 2240}{15471 \times 12} = 149.8$$
lb. per sq. in.

$$f_{s(\text{min.})} = \frac{M_c y}{I} = \frac{-450.325 \times 12\ (-2.30)}{15471} = 0.803 \text{ tons per sq. in.}$$

$$f_{s(\text{max.})} = \frac{-450.325 \times 12\ (-27.048)}{15471} = 9.448 \text{ tons per sq. in.}$$

These stresses are shown plotted in Fig. 16*b*. These stresses can be superimposed on the steel stresses before removal of the props, giving final stresses at the centre of the span of

$f_{c(\text{max.})} = -795.6$ lb. per sq. in. (compression)
$f_{c(\text{min.})} = 149.8$ lb. per sq. in. (tension)
f_s (top of steel section) $= 4.498 + 0.803 = 5.301$ tons per sq. in. (tension)
f_s (bottom of steel) $= -3.569 + 9.448 = 5.879$ tons per sq. in. (tension)

The corresponding stress diagram is plotted in Fig. 16*c*.

The central deflection under dead and live load conditions is obtainable from the steel stresses at this point thus

$$\delta = \frac{f_s l^2}{12Ey} = \frac{(5.879 - 5.301)\frac{10.95}{24.75} \times (60 \times 12)^2 . 2240}{12 \times 30 \times 10^6 \times 10.95} = 0.075\text{in.}$$

(approximately for the actual conditions of loading)

The deflection is downwards because the stress is greater in the bottom flange than in the top. If an accurate determination of this deflection is required the deflection for each stage of the loading should be calculated and the results summed.

It is desirable to examine the steel stresses adjacent to the centre of the span under full load conditions. When propped and the concrete is placed $M_C = 82.346$ ft. tons and $M_B = M_D = 54.384$ ft. tons. There is a distributed load of 0.546 tons per foot throughout. Hence the bending moment at x feet from C may be represented by the general equation.

$$M = \frac{15-x}{15} \times 82.346 + \frac{x}{15} \times 54.384 - \frac{0.546}{2}\, x\,(15-x)$$
$$= 82.346 - 5.959x + 0.273x^2$$

Multiplying by 12 to reduce the units to inch tons and dividing by the section modulus of the steel alone, i.e., $\frac{3032}{13.80} = 219.71$ at the top flange, and $\frac{3032}{-10.95} = -276.89$ at the bottom:

$$f_s \text{ (top flange)} = \frac{M}{Z} = 4.498 - 0.3254x + 0.01491x^2$$
$$f_s \text{ (bottom flange)} = -3.569 + 0.2582x - 0.01183x^2$$

The moment at x feet from the span centre line when props and formwork are removed is

$$M = -\left\{313.575 + 0.498 - 6.4164x + \frac{0.031}{2}.\ x\,(15-x)\right\}$$
$$= -\{314.073 - 6.1844x - 0.0155x^2\}$$
$$f_s \text{ (top flange)} = \frac{My}{I} = -\{314.073 - 6.1844x - 0.0155x^2\} \times$$
$$\times \frac{12\ (-2.298)}{15471} = 0.5598 - 0.01102x - 0.0000276x^2$$
$$f_s \text{ (bottom flange)} = -\{314.073 - 6.1844x - 0.0155x^2\} \times$$
$$\times \frac{12\,(-27.048)}{15471} = 6.5892 - 0.1297x - 0.000325x^2$$

Combining these with the previous stress equations the total dead load steel stresses are given by the equations

$$f_s \text{ (top flange)} = 5.0581 - 0.3364x + 0.01494x^2$$
$$f_s \text{ (bottom flange)} = 3.0202 + 0.1284x - 0.01216x^2$$

Assuming the maximum moment curve for live loads is a parabola, the moment at any point x ft. from the span centre line is $136.252\left(1 - \frac{x^2}{30^2}\right)$

and the corresponding stress equations are fs (top flange)

$$= -\left\{136.252 - 0.15139x^2\right\} \frac{12\,(-2.298)}{15471}$$
$$= 0.2429 - 0.000270x^2$$
$$f_s \text{ (bottom flange)} = -\left\{136.252 - 0.15139x^2\right\} \frac{12\,(-27.048)}{15471}$$
$$= 2.8586 - 0.003176x^2.$$

Combining with the dead load stress equations we obtain the final stress equations.

$$f_s \text{ (top flange)} = 5.3010 - 0.3364x + 0.014669x^2$$
$$f_s \text{ (bottom flange)} = 5.8788 + 0.1284x - 0.01533x^2$$

Fig. 19 shows the live and dead load steel stresses for points adjacent to the centre of the span.

Design of Shear Reinforcing.

Since there is no shear in the composite section before removal of the formwork and props, the shear when they are removed is that due to these loads reversed, and is shown plotted from the load diagram in Fig. 20*a*. The shear due to the live load must be added to the above. Assuming each longitudinal member to carry the same proportion of shear force due to the live load as of bending moment, i.e., $\frac{127.2}{300} = 0.424$ for the crusher train and $\frac{9.32}{125}$ $= 0.074$ for the passing truck; then maximum live load shear at end of span:

$$= \left(11 + \frac{49}{60} \times 6.75 + \frac{40}{60} \times 4.75 + \frac{26}{60} \times 2 + \frac{12}{60} \times 2 + \frac{4}{60} \times 1.25\right)$$
$$0.424 + \left(7 + 3 \times \frac{46}{60}\right) 0.074$$
$$= (21.030 \times 0.424) + (9.3 \times 0.074) = 9.605 \text{ tons}$$

and with the heaviest load at the centre the shear at this point

$$= \left(5.5 \times \frac{18}{60} + 11 \times \frac{30}{60} + 6.75 \times \frac{19}{60} + 4.75 \times \frac{10}{60}\right) 0.424$$
$$+ \left(7 \times \frac{30}{60} + 3 \times \frac{16}{60}\right) 0.074$$
$$= (10.079 \times 0.424) + (4.300 \times 0.074) = 4.592 \text{ tons.}$$

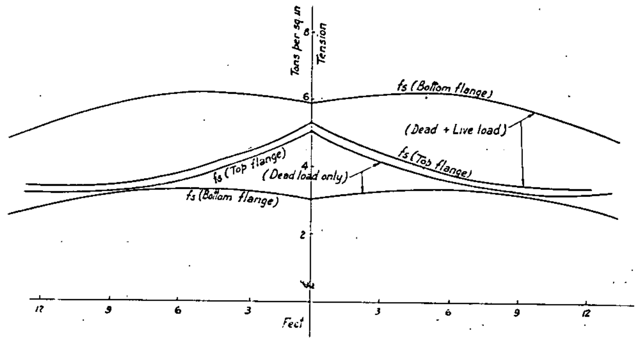


Fig. 19.—*Steel Stresses for Points Adjacent to Centre Line of Span.*

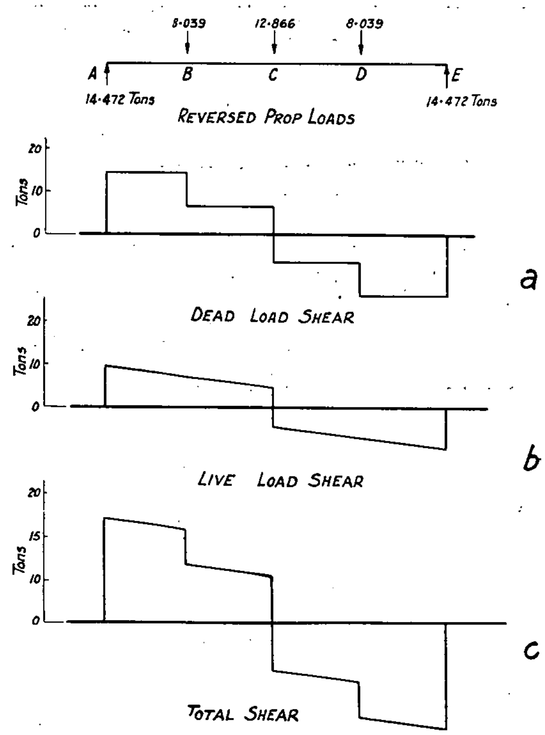


Fig. 20.

The live load shear can be assumed linear between these points. The corresponding shear diagram is shown in Fig. 20*b* and the total shear diagram in Fig. 20*c*.

The horizontal shear in the composite beam section is given in terms of the vertical shear of Fig. 20*a* by formulae (1) i.e.:

$$s = f_{\text{shear}_1} = \frac{S}{I}\int_{y_1}^{y_m} b y \delta y$$

It is necessary to reinforce the concrete in the section for the horizontal shear. The shear force s is a maximum at the plane of contact of the top flange and the concrete haunch; evaluating the equation for this plane

$$s = \frac{S}{15471}\left\{\frac{93}{12}\int_{5.20}^{12.20} y\delta y + \left(1.008 + 0.1667y\right)_{-2.30}^{5.20} \int_{-2.30}^{5.20} y\delta y\right\} = \frac{S}{15471}\left\{472.08 + 18.52\right\} =$$
$$= 0.0317S$$

Ordinary round steel reinforcing rods welded in the form of stirrups to the top flange of the joist, form a convenient type of reinforcement for this purpose, either the size or the spacing being varied to provide the required amount. The rods are bent up into the top of the slab and provided with hooks in order to develop the necessary bond. As to whether the rods should be designed for shear or tension stress depends on the angle at which they are inclined to the joist. The practice in the Public Works Department is to bend the rods up at an angle of 45° in two directions and design them for a tension stress on the cross sectional area of the rod.

Sufficient reinforcement must be provided to carry the shear in the concrete at the vertical sections between the flange and the stem of the tee beam formed by the composite section. It will be found that the deck reinforcing is sufficient for this purpose without the introduction of any extra material.

Since the process of welding the stirrups on one flange only tends to distort the joist it is convenient, in cases where the steel section requires strengthening by cover plates, to place the plate or plates on the lower flange only. Not only is this the most effective position to place the extra steel but it tends to correct the distortion effect noted above. There is some difficulty in welding round steel to a flat surface such as the flange of a joist; for this reason square reinforcing steel may be preferred, in which case this difficulty does not arise.

5. Construction of Composite Beam Bridges.

The general procedure to be followed in constructing the bridge superstructure has been outlined in the section dealing with the design. Having decided on the points at which the longitudinal members are to be propped and having calculated the propping forces required at these points, it remains actually to provide these forces. Since the propping force is fixed by an accurate adjustment of the level of a prop it is important that the prop should be of such a form that its original level once fixed is maintained during the progress of the work.

The method of supporting the props will vary with the natural conditions of the site but it can definitely be said that an arrangement whereby the prop level can be adjusted so as to maintain the correct propping force is essential. Screw jacks of suitable capacity supported either on piles or timber bents have proved satisfactory for this purpose, as any sinking of the prop due to the weight of the concrete deck slab can be adjusted as the work proceeds.

In order to measure the initial camber placed in the joist, and to ensure that this camber is maintained, a suitable means of measuring the level of the top flange of the joist must be adopted. It is desirable that such measurements should be made to $\frac{1}{100}$th of an inch; for this purpose a surveyor's level is of little use, particularly as the sight distance might be considerable. A simple method which has been used with success is to stretch a steel piano wire between adjacent piers an inch or so from the lower flange of each joist, clear of any falsework; if the wire is kept at the same tension by attaching a suitable weight to one end and passing it over a pulley attached to the pier, all deflections can be measured from this wire.

The necessity for wind and sway bracing is obviated when the composite beam type of construction is used, the concrete deck slab resisting these forces. Some form of bracing, however, is desirable during construction when the span length is greater than about 45 feet. The introduction of bracing which has a stiffening effect in the transverse direction leads to a complicated state of affairs as far as distribution is concerned, making it extremely difficult to compute the maximum moments and shearing forces carried by each longitudinal member. Disregarding the effect of the bracing on the main members there still remains the fact that the elasticity of these members under load is responsible for far higher stresses in the bracing than those for which it is usually designed.

The phenomena of plastic flow in concrete must be considered in designing this type of bridge. If this were appreciable it might be expected that the dead load compression in the concrete would be relieved at the expense of extra stress in the steel joist. Various experiments on plastic flow of concrete have given results which indicate that its effect is seriously to increase the stresses in reinforcing steel particularly in reinforced concrete columns, but as far as the 34 ft. span experimental bridge of this type is concerned the effects of plastic flow are small. Deflection measurements made over a period of 9 months gave the results shown in Fig. 21.

Any creep in the concrete would result in an increased deflection in the beams with a consequent increase in dead load steel stresses proportional to this deflection. Since the steel joist alone is comparatively slender an appreciable deflection is necessary to give any serious increase in stress in the steel; for the maximum sag measured in the above bridge the corresponding steel stress amounted to 1.046 tons per sq. in. The fact that at one stage the sag appeared to be decreasing is possibly due to a redistribution of stresses in the section due to the change in the modulus of elasticity of the concrete as its strength increases with age. On the other hand it may be due to temperature changes.

The use of pneumatic rammers for placing the concrete, enabling a reduction in the $\frac{\text{water}}{\text{cement}}$ ratio, should have the dual effect of increasing its ultimate strength and at the same time minimizing the plastic flow. Provided the stress in the steel joist due to plastic flow in the concrete can be estimated it is a simple matter to provide an equal and opposite stress in the steel by means of the props, thus neutralizing the effect. Results to date indicate that the effects of plastic flow in this class of structure are unimportant.

Plastic flow is probably closely connected with the value of Young's modulus for the material and in this con-

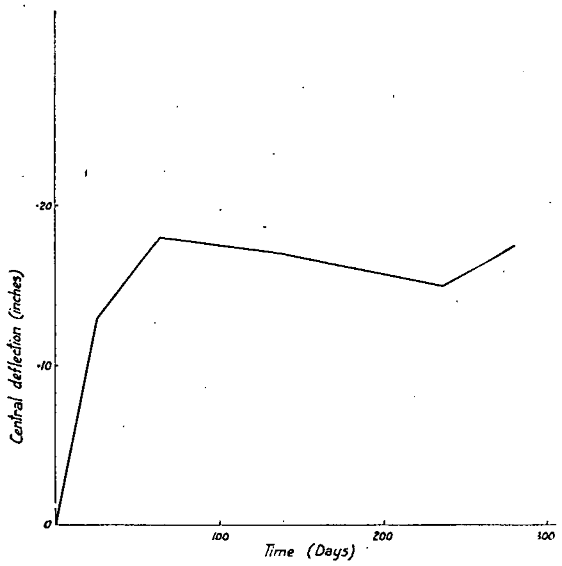


Fig. 21.

nection it is interesting to refer to Fig. 22 which shows the maximum steel and concrete working stresses (for the section treated throughout the paper) under full live and dead load, plotted against various values of n, the ratio of Young's modulus of steel/concrete. The factor of safety in the steel and concrete is also plotted against n. E for the concrete was taken to be proportional to its ultimate strength which in the light of at least some test data appears reasonable.

The steel stresses show a slight decrease for lower values of n, though this might not be the case with all sections examined, but there is an appreciable decrease in the concrete stresses and a marked increase in the factor of safety. A study of this graph shows the importance of a reasonably close determination of Young's modulus for the concrete used in the deck slab.

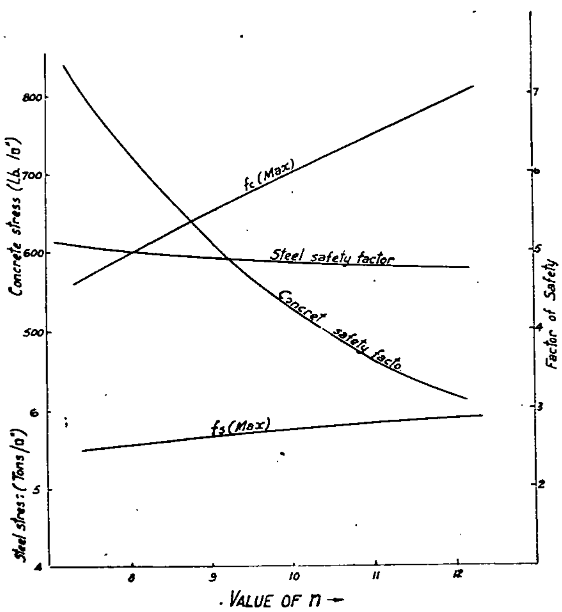


Fig. 22.—*Effect of n on Steel and Concrete Stresses.*

It might be noted that, in determining the type of wearing surface to be used on the deck, one formed by an increase in the thickness of the deck slab is a better proposition than a layer of some other material (other than a thin coating of a plastic material to reduce impact effect) since the extra concrete serves to increase the strength of the longitudinal member. The practice of placing a thick wearing surface on road bridge decks is to be condemned as it results in a saving of perhaps 1 inch in deck slab thickness at the expense of 3 inches of surface material with a consequent increase in dead load moment. This effect is of course more pronounced on long spans.

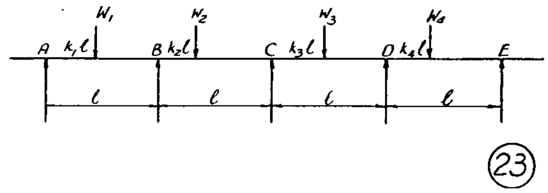


Fig. 23.

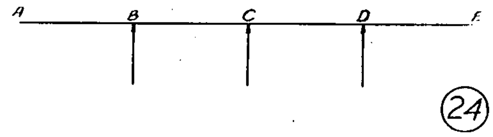


Fig. 24.

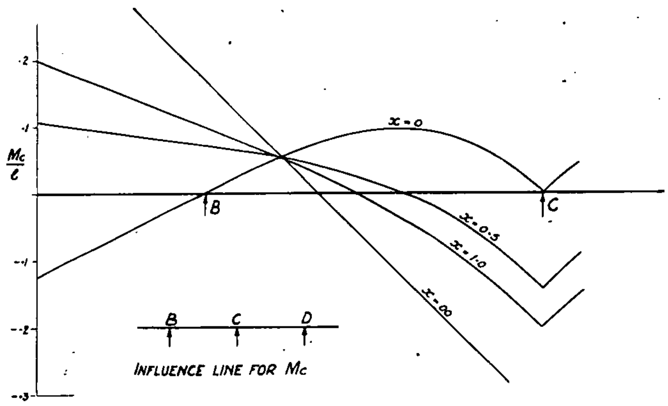


Fig. 25.

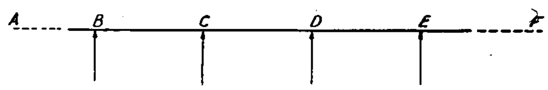


Fig. 26.

A feature of this type of bridge is its stiffness as measured by test compared with the calculated stiffness. The explanation is probably due to an underestimation of Young's modulus of the concrete and also to lack of knowledge regarding the distributing effect of the slab through which the test load is applied. Recent tests show that the error due to the latter reason would be quite sufficient to account for the discrepancy between the calculated and the actual values under test.

The rigid connection of the beams to the deck slab appears to increase the resistance of the bridge to impact effect, a noticeable feature of the structure being its elasticity under loads causing these effects.

6. Economics of the Structure.

A perusal of the section moduli of the composite sections of Table I. will indicate the economy in these sections as compared with the ordinary rolled sections. The only figures of cost available are those for the steel superstructure for the 34 feet single span experimental bridge which gave a saving of 29.5% over ordinary methods, and an estimated saving of 35% on the steelwork of the superstructure of a 427 ft. bridge of 7 spans.

7. Conclusion.

In conclusion, mention should be made of the question of slab bridge deck design which has been discussed in conjunction with the other matters of the paper. A review of the development of design methods for slabs for bridge decks shows a certain lack of appreciation of the problem in hand. Although it is an accepted principle never to place continuous structures on non-rigid supports it has frequently been done with bridge decks without any provision being made for the changes in bending moment which are bound to occur. A certain amount of research on the distributing effect of slabs supported by a number of beams has been undertaken, but little attention has been paid to the effect of this distribution on the slab itself with the result that the design of deck slabs along the lines accepted as standard practice involves at one and the same time a waste of reinforcing steel and a low factor of safety.

8. Acknowledgment.

The thanks of the author are due to G. D. Balsille, A.M.I.E.Aust., Director of Public Works, for his assistance and encouragement during the period in which the material for this paper was collected, and also to Professor A. Burn, A.M.I.E.Aust., of the University of Tasmania to whom the general solution for the beam on elastic supports given in the appendix is due, and without whose valued help this paper could not have been produced.

APPENDIX.

By Prof. A. Burn.

The Continuous Beam on Elastic Supports.

The usual equations for a continuous beam are derived on the assumption that the supports do not deflect. It may not be generally realized that when the supports are elastic, as for example when a beam is supported on a number of other beams, the reactions at the supports and the bending moments in the beam are very different to those occurring with rigid supports.

The following analysis gives a general method of treatment, and also the solutions for the special case of a uniform beam with a number of equal spans on 3, 4 and 5 supports.

The general method is based on the identity of the slope over a support for adjacent spans. The end slope in one span is expressed in terms of the load on the span, the end moments, and the end deflections, and equating values gives an equation involving the load on two spans, three moments and three deflections.

For elastic supports, the deflections are the reactions divided by the stiffness of the supports, the latter being defined as the load to produce unit deflection.

Each reaction in turn can be expressed in terms of the loads on two adjacent spans and the three moments at the end of those spans. On substitution, a five moment equation is obtained. The five moment equations for all interior supports together with the end moment conditions provide sufficient equations to enable the support moments to be determined.

Expression for End Slopes.—In the most general case where the beam is of variable cross section the use of the M/EI diagram gives the simplest expression for the slopes due to bending moments. Adopting the notation used by R. C. Robin in his paper on Statically Indeterminate Frames (The Journal, May, 1933) and writing the slopes for a span A B:—

$$i_A = -M_A A_1 \frac{l-x_1}{l} - M_B A_2 \frac{x_2}{l} + A_w \frac{l-x_w}{l} + \frac{y_B - y_A}{l}$$

$$i_B = M_A A_1 \frac{x_1}{l} + M_B A_2 \frac{l-x_2}{l} - A_w \frac{xw}{l} + \frac{y_B - y_A}{l}$$

The last terms are the additional slopes resulting from the deflections y_A and y_B of the supports.

Expression for Reactions.—If A, B, and C are three consecutive supports of spans l_1 and l_2, and r_B is the reaction which would occur at B if the spans were discontinuous, then

$$R_B = r_B + \frac{M_B - M_A}{l_1} + \frac{M_B - M_C}{l_2}$$

and if u_B is the stiffness of the support B

$$y_B = \frac{R_B}{u_B}$$

Case of Equal Spans and Uniform Beam.—Referring to Fig. 23, the load on each span is kl from the nearest support to the left, with the appropriate suffix.

For all spans, $x_1 = x_2 = l/3$

$$A_1 = A_2 = \frac{l}{2EI}$$

$$x_w = \frac{(1+k)\,l}{3} \qquad A_w = \frac{Wk(1-k)\,l^2}{2EI}$$

For the second span:—

$$i_C = +\frac{M_B l}{6EI} + \frac{M_C l}{3EI} - \frac{W_2 k_2 (1-k_2)(1+k_2)\,l^2}{6EI} + \frac{y_C - y_B}{l}$$

For the third span:—

$$i_C = -\frac{M_C l}{3EI} - \frac{M_D l}{6EI} + \frac{W_3 k_3 (1-k_3)(2-k_3)\,l^2}{6EI} + \frac{y_D - y_C}{l}$$

Reactions.—

$$R_B = k_1 W_1 + (1-k_2)\,W_2 + \frac{2\,M_B - M_A - M_C}{l}$$

$$R_C = k_2 W_2 + (1-k_3)\,W_3 + \frac{2\,M_C - M_B - M_D}{l}$$

$$R_D = k_3 W_3 + (1-k_4)\,W_4 + \frac{2\,M_D - M_C - M_E}{l}$$

Dividing these by u gives the deflections.

In order to simplify the final equation the symbols, $K_L = k(1-k)(2-k)$ and $K_R = k(1-k)(1+k)$ will be used. These quantities occur very frequently in problems on continuous beams and a table of values for intervals of 0.1 is therefore given.

k =	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
K_L =	0.171	0.288	0.357	0.384	0.375	0.336	0.273	0.192	0.099
K_R =	0.099	0.192	0.273	0.336	0.375	0.384	0.357	0.288	0.171

For distributed loads over a whole span the average values of k and K are used, namely $k = \frac{1}{2}$ and $K = \frac{3}{8}$ for interior spans.

The Stiffness of the Beam will be defined as $v = \frac{6EI}{l^3}$. This definition arose from consideration of the case of 3 supports and the load to give unit deflection at the centre with the centre support removed. It has proved the most convenient measure for other cases as well. Omitting intermediate steps, multiplying the slopes by $\frac{6EI}{l}$ equating and rearranging with $x = \frac{v}{u}$ leads to the equation:—

$$x\,M_A + (1-4x)\,M_B + (4+6x)\,M_C + (1-4x)\,M_D + x\,M_E = K_{R_2} W_2 l + K_{L_3} W_3 l + xl\,\{k_1 W_1 + (1-3k_2)\,W_2 + (3k_3-2)\,W_3 + (1-k_4)\,W_4\}$$

This is the five moment equation for the uniform beam on equal spans with equal stiffness of supports.

For plotting influence lines a unit load on one span only is taken, so that the right side of the equation is simplified by the omission of other terms.

End conditions are that the end moments are zero except when there is a load on an overhung end, in which case the actual value of this end moment can be inserted.

Overhung ends are invariably shorter than the main spans, but for construction of influence lines they may be taken of the same length, and only the required range of k used.

Some doubt may be felt as to the applicability of the five moment equation above when there are less than five supports, or for end spans. Since however the terms in M_D and M_E only appear as a result of substituting for the reactions R_B and R_D, correct results are obtained by putting $M_A = 0$ when B is the first support and $M_E = 0$ when D is the last support.

In the case of an overhung end which is considered as the first span, $M_A = 0$ and the term $k_1 w_1$ on the right must be retained when the cantilever carries a load. M_B then has the value $(1 - k_1) W_1 l$.

Solutions for Three Supports.—Referring to Fig. 24. Load on 1st span:—$M_B = (1 - k_1) W_1 l$, $M_A = 0$, $M_D = 0$, $M_E = 0$.

The elastic equation is:—

$$(1 - 4x)(1 - k_1) W_1 l + (4 + 6x) M_C = x k_1 W_1 l$$

giving $$M_C = \frac{k_1 - 1 + x(4 - 3k_1)}{4 + 6x} W_1 l$$

Load on 2nd span:— $M_A = M_B = M_D = M_E = 0$.

The elastic equation is:—

$$(4 + 6x) M_C = W_2 l \{ K_{R_2} + x(1 - 3k_2) \}$$

giving $$M_C = \frac{K_{R_2} + x(1 - 3k_2)}{4 + 6x} W_2 l$$

These results are plotted as influence lines for $x = 0$, $x = 0.5$ and $x = 1.0$ in Fig. 25.

In general the reactions are best deduced from the moments by means of the expression already given. In this case the values reduce to the following.

With the load on the overhung span AB:—

$$\frac{R_B}{W_1} = (2 - k_1) + \frac{(1 - k_1) - x(4 - 3k_1)}{4 + 6x}$$

$$\frac{R_C}{W_1} = \frac{x - 3(1 - k_1)}{2 + 3x}$$

$$\frac{R_D}{W_1} = \frac{(1 - k_1) - x(4 - 3k_1)}{4 + 6x}$$

With the load on the second span:—

$$\frac{R_B}{W_2} = (1 - k_2) - \frac{K_{R_2} + x(1 - 3k_2)}{4 + 6x}$$

$$\frac{R_C}{W_2} = \frac{K_{R_2} + 2k_2 + x}{2 + 3x}$$

$$\frac{R_D}{W_2} = -\frac{K_{R_2} + x(1 - 3k_2)}{4 + 6x}$$

Solution for Four Supports.—Referring to Fig. 26, two five moment equations are required in this case, together with the end conditions.

Loads W_1 W_2 W_3 only need be considered.

$$M_A = M_E = M_F = 0, \quad M_B = (1 - k_1) W_1 l.$$

The elastic equations are then:—

$$(1 - 4x)(1 - k_1) W_1 l + (4 + 6x) M_C + (1 - 4x) M_D = K_{R_2} W_2 l + K_{L_3} W_3 l + xl \{ k_1 W_1 + (1 - 3k_2) W_2 + (3k_3 - 2) W_3 \}$$

$$x(1 - k_1) W_1 l + (1 - 4x) M_C + (4 + 6x) M_D = K_{R_3} W_3 l + xl \{ k_2 W_2 + (1 - 3k_3) W_3 \}$$

The solutions for M_C and M_D are, for load on 1st span, the overhung end only:—

$$\frac{M_C}{W_1 l} = \frac{(5x + 4)(4x - 1) - k_1(14x^2 + 7x - 4)}{(2x + 5)(10x + 3)}$$

$$\frac{M_D}{W_1 l} = \frac{(10x^2 - 12x + 1) - k_1(6x^2 - 11x + 1)}{(2x + 5)(10x + 3)}$$

For load on second span BC:—

$$\frac{M_C}{W_2 l} = \frac{(K_{R_2} + x)(6x + 4) - k_2 x(14x + 13)}{(2x + 5)(10x + 3)}$$

$$\frac{M_D}{W_2 l} = \frac{(K_{R_2} + x)(4x - 1) - k_2 x(6x - 7)}{(2x + 5)(10x + 3)}$$

For load on third span CD:—

$$\frac{M_B}{W_3 l} = \frac{(K_{R_3} + x)(4x - 1) + (K_{L_3} - 2x)(4 + 6x) + 3k_2 x(2x + 5)}{(2x + 5)(10x + 3)}$$

$$\frac{M_C}{W_3 l} = \frac{(K_{R_3} + x)(6x + 4) + (K_{L_3} - 2x)(4x - 1) - 3k_3 x(2x + 5)}{(2x + 5)(10x + 3)}$$

General expressions for reactions will not be given in this case. They can be deduced as in the case of three supports.

Solution for Five Supports.—Proceeding in the same way three equations can be written down and solved for M_C, M_D and M_E. The results are as follows:—

$$\frac{M_C}{W_1 l} = \frac{(4x - 1)(15x^2 + 52x + 15) - k_1(40x^3 + 137x^2 - 7x - 15)}{2(5x + 4)(5x^2 + 34x + 7)}$$

$$\frac{M_D}{W_1 l} = \frac{(9x^2 - 12x + 1) - k_1(5x^2 - 11x + 1)}{2(5x^2 + 34x + 7)}$$

$$\frac{M_E}{W_1 l} = \frac{(4x - 1)(5x^2 - 16x + 1) - k_1(10x^3 - 57x^2 + 19x - 1)}{2(5x + 4)(5x^2 + 34x + 7)}$$

For load on second span BC:—

$$\frac{M_C}{W_2 l} = \frac{(K_{R_2} + x)(2x + 5)(10x + 3) - k_2 x(40x^2 + 157x + 49)}{2(5x + 4)(5x^2 + 34x + 7)}$$

$$\frac{M_D}{W_2 l} = \frac{(K_{R_2} + x)(4x - 1) - k_2 x(5x - 7)}{2(5x^2 + 34x + 7)}$$

$$\frac{M_E}{W_2 l} = \frac{(K_{R_2} + x)(10x^2 - 12x + 1) - k_2 x(10x^2 - 47x + 7)}{2(5x + 4)(5x^2 + 34x + 7)}$$

For load on third span CD:—

$$\frac{M_C}{W_3 l} = \frac{(K_{R_3} + x)(4x - 1)(5x + 4)}{2(5x + 4)(5x^2 + 34x + 7)} + \frac{(K_{L_3} - 2x)(2x + 5)(10x + 3) + k_3 x(10x^2 + 123x + 58)}{2(5x + 4)(5x^2 + 34x + 7)}$$

$$\frac{M_D}{W_3 l} = \frac{(K_{R_3} + x)(7x + 4)}{2(5x^2 + 34x + 7)} + \frac{(K_{L_3} - 2x)(4x - 1) - k_3 x(5x + 16)}{2(5x^2 + 34x + 7)}$$

$$\frac{M_E}{W_3 l} = \frac{(K_{R_3} + x)(4x - 1)(5x + 4)}{2(5x + 4)(5x^2 + 34x + 7)} + \frac{(K_{L_3} - 2x)(10x^2 - 12x + 1) - k_3 x(10x^2 + 13x - 30)}{2(5x + 4)(5x^2 + 34x + 7)}$$

For more than five supports, involving more than three simultaneous equations, solutions in terms of x, the stiffness ratio, become so clumsy that it is better not to set them out but to solve the equations directly with numerical coefficients for particular values of the stiffness ratio as required.

Solutions for Uniformly Distributed Dead Loads.—If the beam has cantilever ends, it is of course necessary to know their length. It will be assumed in the solutions given below that the length of the cantilevers is half the length of the interior spans.

Case of Three Supports.—Using the general equation, $M_A = M_E = 0$, $M_B = M_D = \frac{wl^2}{8}$, $W_1 = \frac{wl}{2} = W_4$, $W_2 = wl = W_3$, $k_1 = \frac{3}{4}$, $k_2 = k_3 = \frac{1}{2}$, $k_4 = \frac{1}{4}$, $K_{R_2} = K_{L_3} = \frac{1}{4}$.

Substituting these values:—

$$(1 - 4x)\frac{wl^2}{4} + (4 + 6x) M_C = \frac{wl^2}{2} + xwl^2 \{ \tfrac{3}{8} - \tfrac{1}{2} - \tfrac{1}{2} + \tfrac{3}{8} \}$$

giving

$$M_C = \frac{3x + 1}{3x + 2} \cdot \frac{wl^2}{8}$$

Case of Four Supports.—Similar treatment gives

$$M_B = M_E = \frac{wl^2}{8}$$

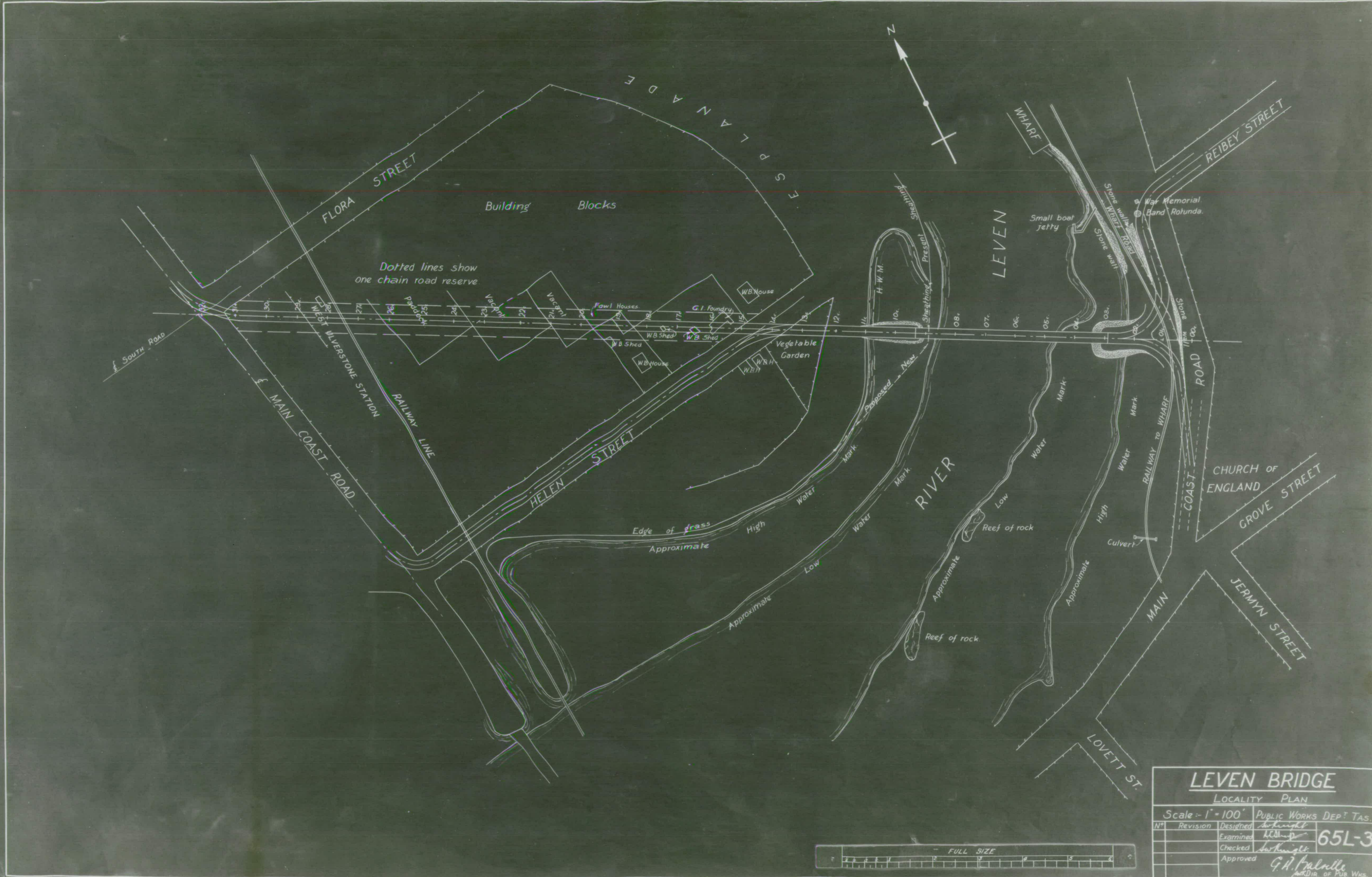
and $$M_C = M_D = \frac{2x + 3}{2x + 5} \cdot \frac{wl^2}{8}$$

Case of Five Supports.—$M_B = M_F = \frac{wl^2}{8}$

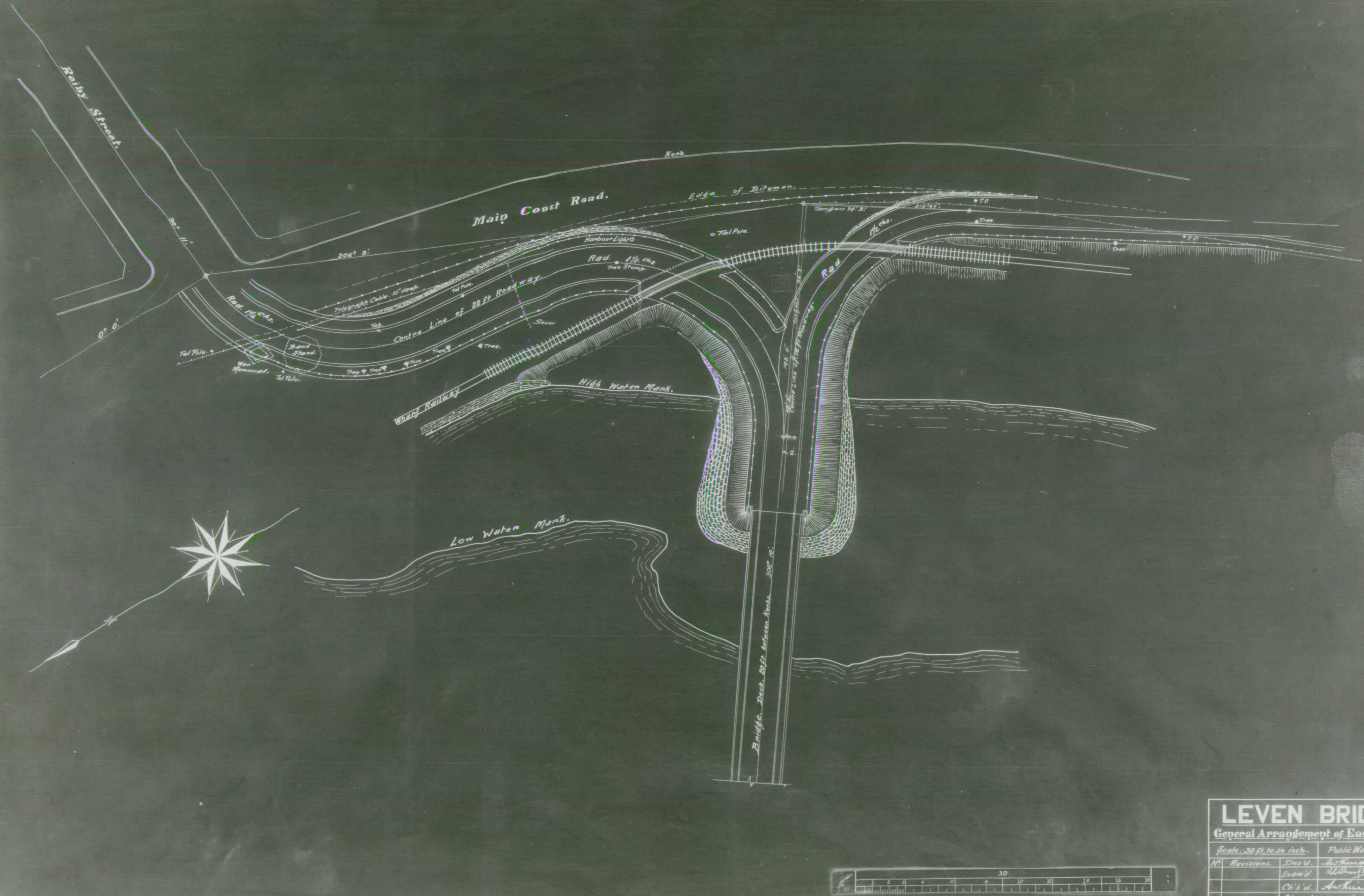
$$M_C = M_E = \frac{5x^2 + 24x + 4}{5x^2 + 34x + 7} \cdot \frac{wl^2}{8}$$

$$M_D = \frac{5x^2 + 19x + 5}{5x^2 + 34x + 7} \cdot \frac{wl^2}{8}$$

LOCALITY PLAN
LEVEN BRIDGE
AT
ULVERSTONE
17-3-33
65L-2
JERMYN ST
LOVETT ST
GROVE ST
Fence
MAIN ROAD
RAILWAY
H. W. MARK
Police Buildings
Creek
Rocks
Rocks
RIVER
LEVEN
No 3 SITE
No 2 SITE
No 1B SITE
No 1 SITE
No 4 SITE
No 5 SITE
EXISTING ROAD BRIDGE
RAILWAY BRIDGE
APPROX. L.W.M.
APPROX. L.W.M.
Edge of Grass
H.W. MARK
REIBEY ST
Band Rotunda
S.A. War Memorial
STONE RETAINING WALL
WHARF
SHEATHED PILING
W.B. Hut
W.B. House
OLD POWER HOUSE SITE
HELEN ST
TO BURNIE



ESPLANADE
FLORA STREET
Building Blocks
Dotted lines show
one chain road reserve
W.B. House
Fowl Houses
G.I. Foundry
W.B. Shed
W.B. Shed
W.B. Shed
W.B. House
Vegetable
Garden
Paddock
Vacant
Vacant
SOUTH ROAD
WEST ULVERSTONE STATION
RAILWAY LINE
MAIN COAST ROAD
HELEN STREET
Edge of grass
Approximate High Water Mark
Approximate Low Water Mark
Proposed New Sheathing
Present Sheathing
H.W.M.
LEVEN
RIVER
Reef of rock
Reef of rock
Approximate Low Water Mark
Approximate High Water Mark
Small boat
jetty
WHARF
Stone wall
Wharf Road
Stone wall
Stone
War Memorial.
Band Rotunda.
REIBEY STREET
RAILWAY TO WHARF
COAST ROAD
MAIN
CHURCH OF
ENGLAND
GROVE STREET
JERMYN STREET
LOVETT ST.
Culvert
N
FULL SIZE
LEVEN BRIDGE
LOCALITY PLAN
Scale :- 1" = 100'
PUBLIC WORKS DEPT. TAS.
No. Revision
Designed
Examined
Checked
Approved
G. H. Balcille
DIR. OF PUB. WKS.
65L-3

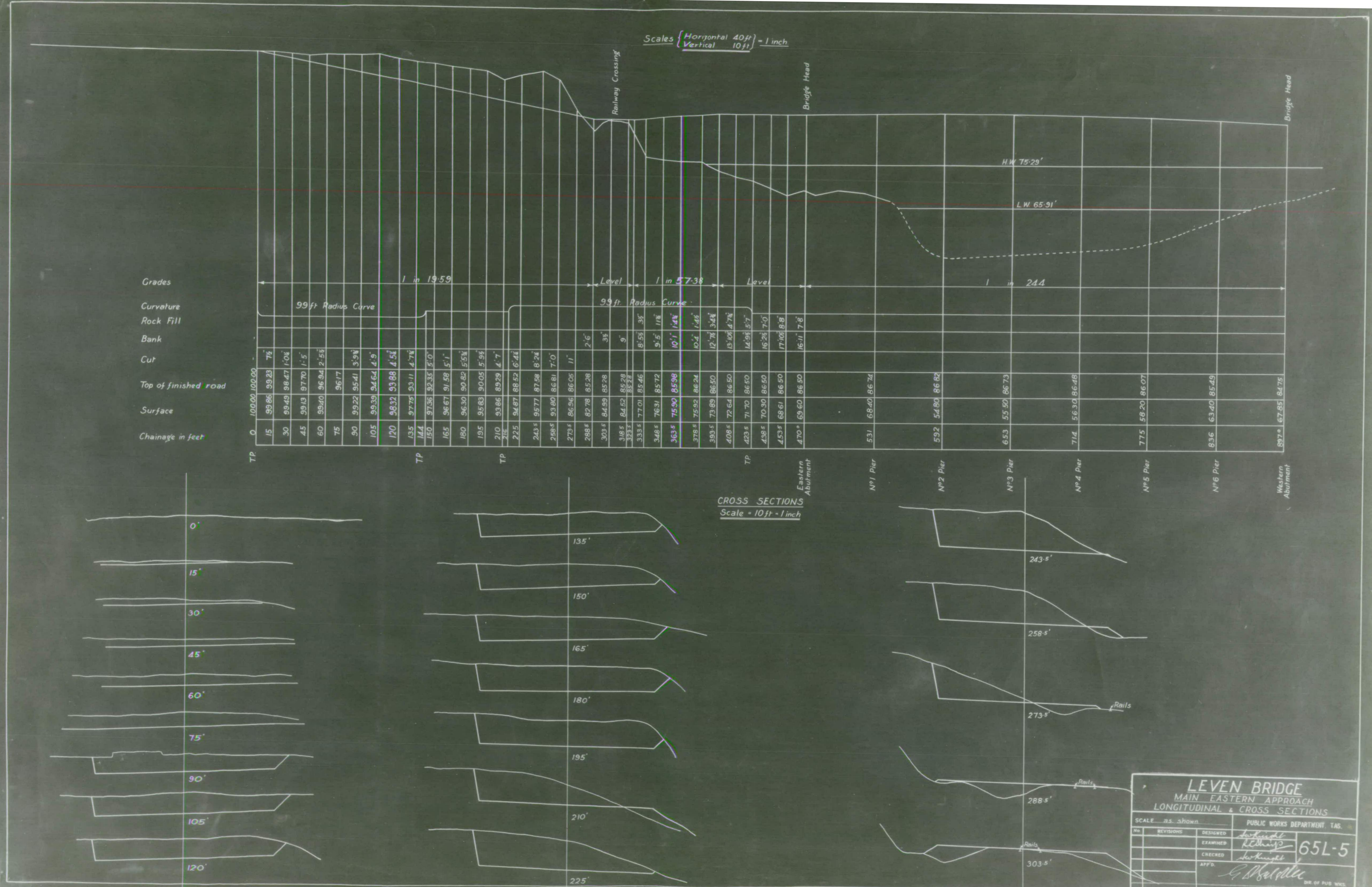


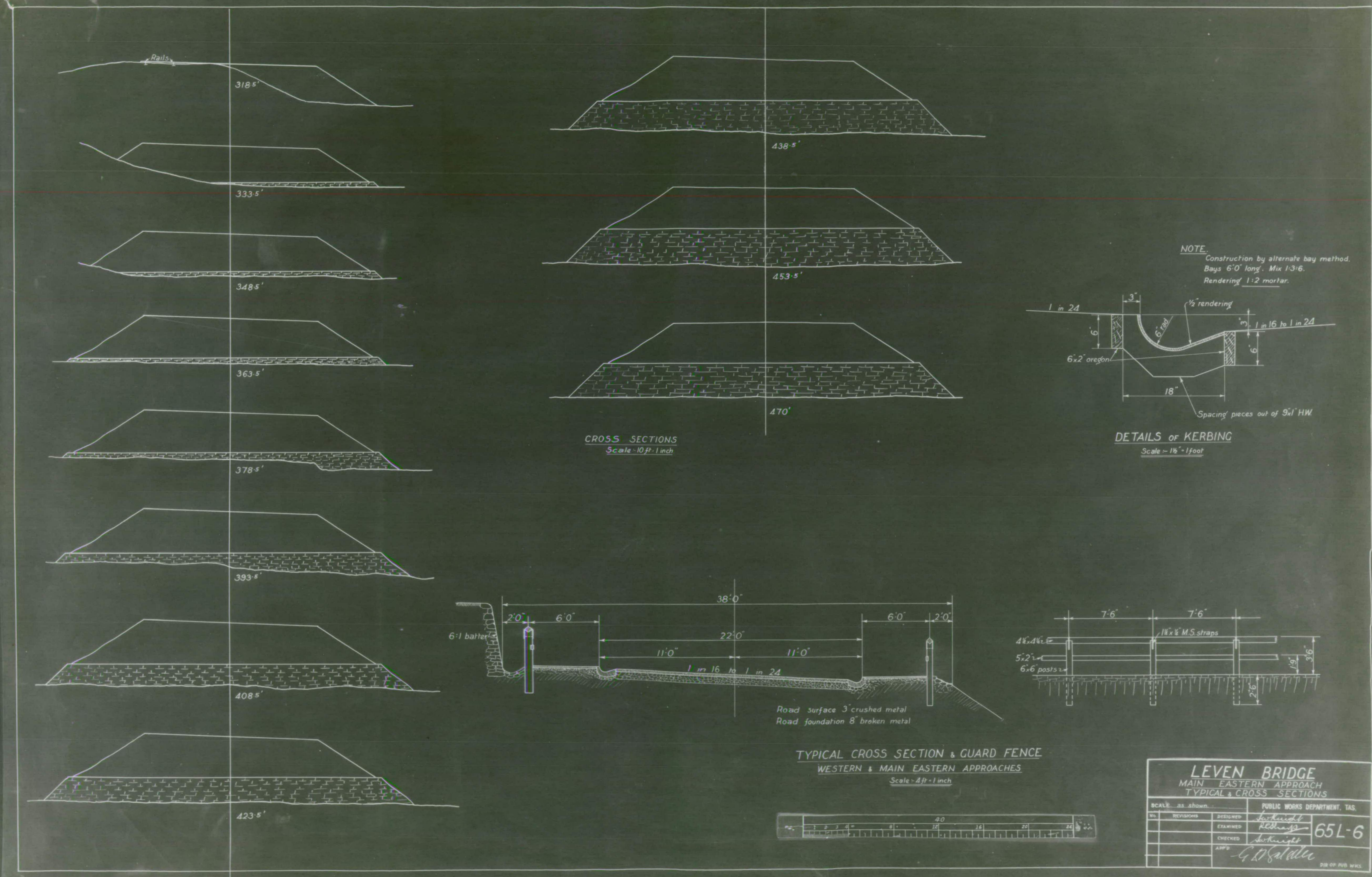
A.A.H. 24-11-53

LEVEN BRIDGE.

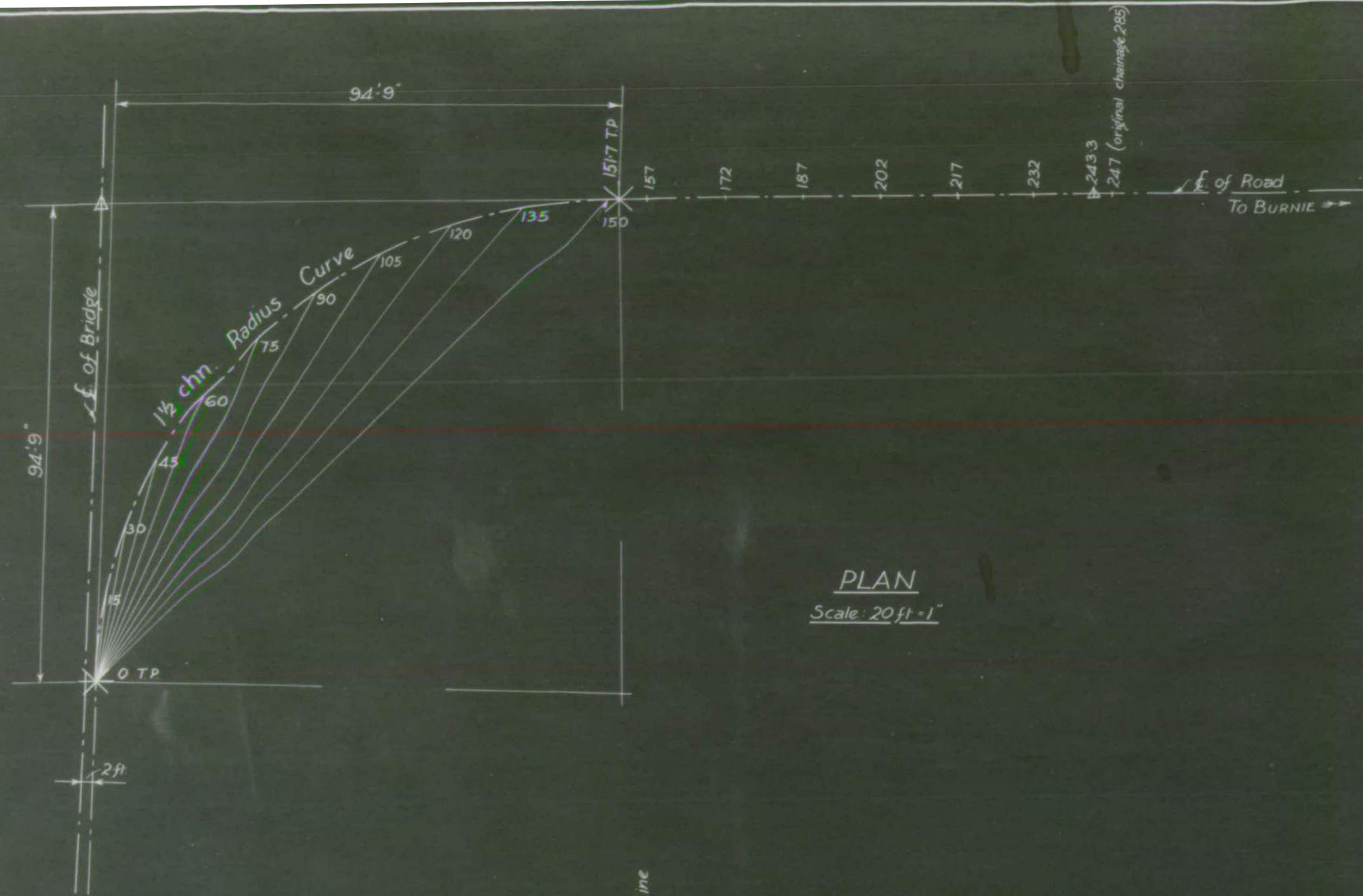
General Arrangement of Eastern Approach.

Scale. 30 ft. to an inch.			Public Works Department. Tas.	
No.	Revisions.	Des'd.	A W Knight	65L-4
		Exam'd.	[illegible]	
		Ch'k'd.	A W Knight	
		Approved.	G [illegible]	
			DR. OF PUB. WKS.	


Scales { Horizontal 40 ft / Vertical 10 ft } = 1 inch
Railway Crossing
Bridge Head
Bridge Head
H.W. 75·29'
L.W. 65·91'
Grades
1 in 19·59
Level
1 in 57·38
Level
1 in 244
Curvature
99 ft Radius Curve
99 ft. Radius Curve
Rock Fill
Bank
Cut
Top of finished road
Surface
Chainage in feet
0 15 30 45 60 75 90 105 120 135 144 150 165 180 195 210 216 225 243·5 258·5 273·5 288·5 303·5 318·5 323·5 333·5 348·5 363·5 378·5 393·5 408·5 423·5 438·5 453·5 470·0 531 592 653 714 775 836 897·0
T.P.
Eastern Abutment
Nº1 Pier
Nº2 Pier
Nº3 Pier
Nº4 Pier
Nº5 Pier
Nº6 Pier
Western Abutment
CROSS SECTIONS
Scale = 10 ft = 1 inch
0'
15'
30'
45'
60'
75'
90'
105'
120'
135'
150'
165'
180'
195'
210'
225'
243·5'
258·5'
273·5'
288·5'
303·5'
Rails
LEVEN BRIDGE
MAIN EASTERN APPROACH
LONGITUDINAL & CROSS SECTIONS
SCALE as shown
PUBLIC WORKS DEPARTMENT, TAS.
No. REVISIONS
DESIGNED
EXAMINED
CHECKED
APP'D.
65L-5
DIR. OF PUB. WKS.

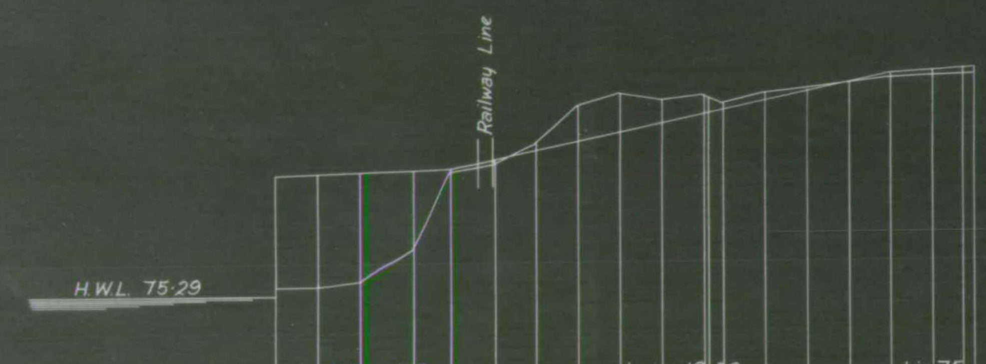


Rails
318·5'
333·5'
348·5'
363·5'
378·5'
393·5'
408·5'
423·5'
438·5'
453·5'
470'
CROSS SECTIONS
Scale - 10 ft - 1 inch
NOTE.
Construction by alternate bay method.
Bays 6'0" long. Mix 1:3:6.
Rendering 1:2 mortar.
1 in 24
3"
½" rendering
6" rad
1 in 16 to 1 in 24
6"
6"x2" oregon
18"
Spacing pieces out of 9"x1" H.W.
DETAILS OF KERBING
Scale - 1½" - 1 foot
38'0"
2'0"
6'0"
22'0"
11'0"
11'0"
6'0"
2'0"
6:1 batter
1 in 16 to 1 in 24
Road surface 3" crushed metal
Road foundation 8" broken metal
TYPICAL CROSS SECTION & GUARD FENCE
WESTERN & MAIN EASTERN APPROACHES
Scale - 4 ft - 1 inch
7'6"
7'6"
1½"x⅛" M.S. straps
4½"x4½"
5"x2"
6"x6" posts
1'9"
3'6"
2'6"
40
LEVEN BRIDGE
MAIN EASTERN APPROACH
TYPICAL & CROSS SECTIONS
SCALE as shown.
PUBLIC WORKS DEPARTMENT, TAS.
No.
REVISIONS
DESIGNED
EXAMINED
CHECKED
APP'D
65L-6
DIR OF PUB WKS.



PLAN

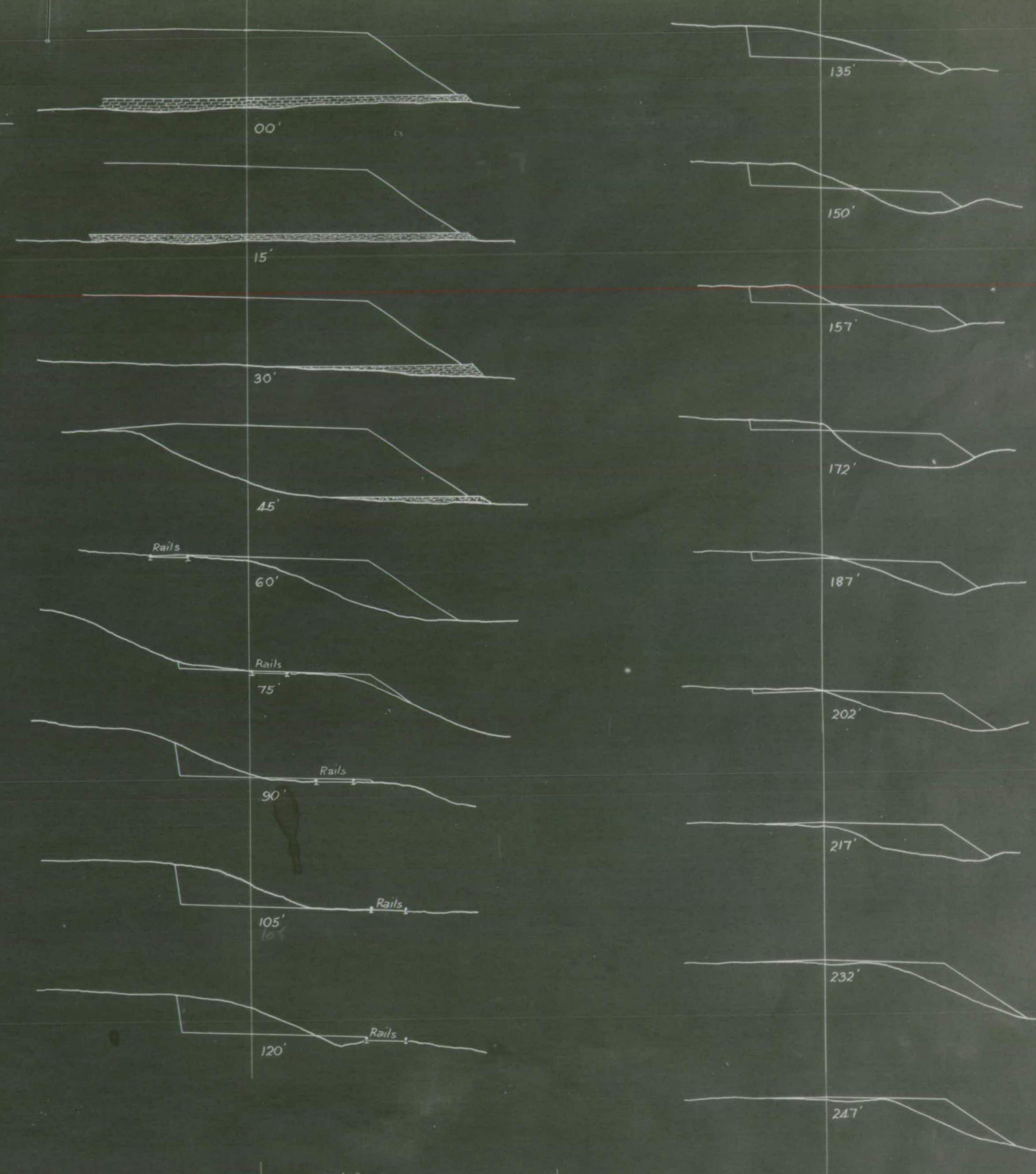
Scale 20 ft = 1"



Grades	1 in 115						1 in 18·30										1 in 75		
Curves	Radius 1½ chns											Straight							
Bank	10'3½"	10'4½"	10'0"	7'3¾"	0'3¾"	0'4¾"									—	0'5"	0'5¼"		0'6½"
Cutting							0'7¼"	3'4"	3'6½"	2'2¼"	1'10"	0'6½"	0'8½"	0'3¾"	—				
Top of finished road	86·50	86·63	86·76	86·89	87·02	87·84	88·66	89·48	90·30	91·12	91·94	92·32	93·14	93·96	94·78	95·60	95·80		96·00
Surface	76·21	76·27	76·76	79·57	86·71	87·44	89·27	92·81	93·85	93·31	93·76	92·87	93·84	94·28	94·79	95·19	95·36		95·47
Chainage (in feet)	00	15	30	45	60	75	90	105	120	135	150	157	172	187	202	217	232	243·3	247
	T.P.										T.P.							Reference Peg	

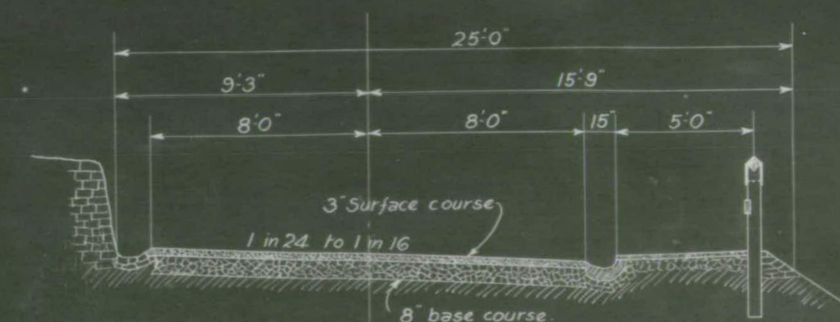
LONGITUDINAL SECTION

Scales {Horiz' - 40' / Vert' - 10'} = 1"



CROSS SECTIONS

Scale - 10' = 1"



TYPICAL CROSS SECTION

Scale - 4' = 1"

LEVEN BRIDGE

SOUTH EASTERN APPROACH

LONGITUDINAL & TYPICAL SECTION & CROSS SECTIONS

Scales :- as shown		PUBLIC WORKS DEPT TAS.	
Nº	Revisions	Designed	[signature]	65L-7
		Examined	[signature]	
		Checked	[signature]	
		Approved	[signature] DIR. OF PUB WKS.	

Scales { Horizontal, 100 ft / Vertical 10 ft } = 1 inch

Grades: 1 in 60 | 1 in 31·3 | 1 in 80·7 | Level | 1 in 48·22 | 1 in 209·3

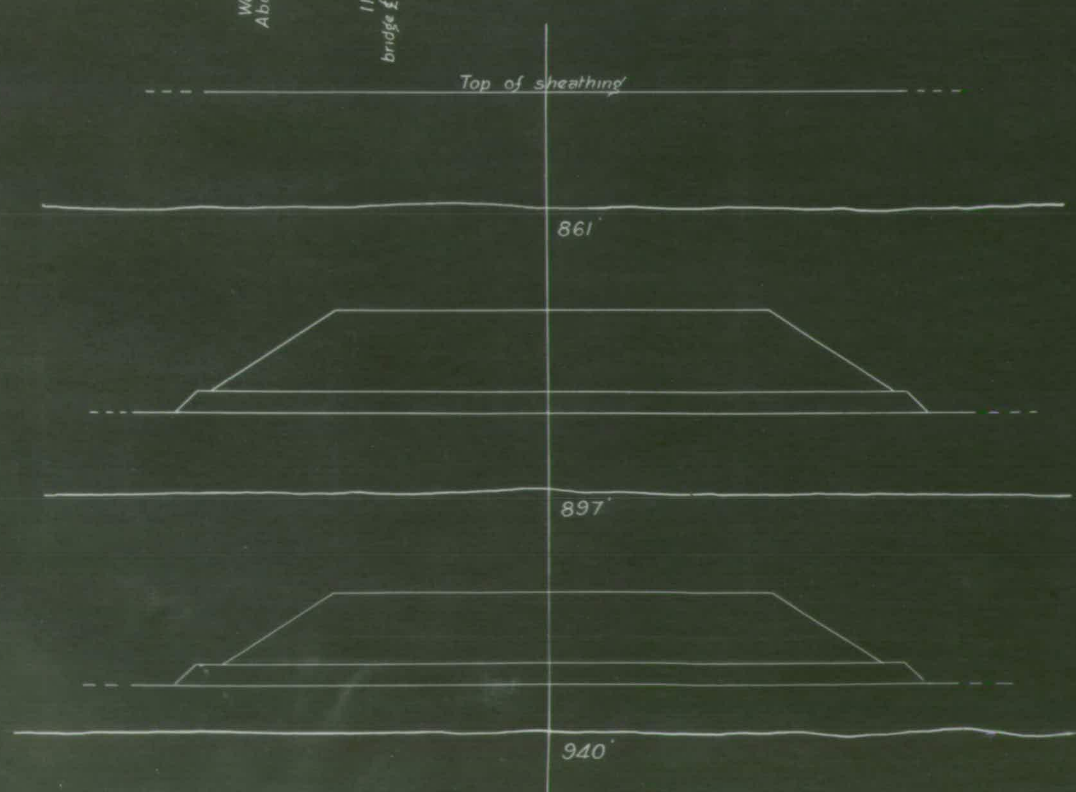
Straight

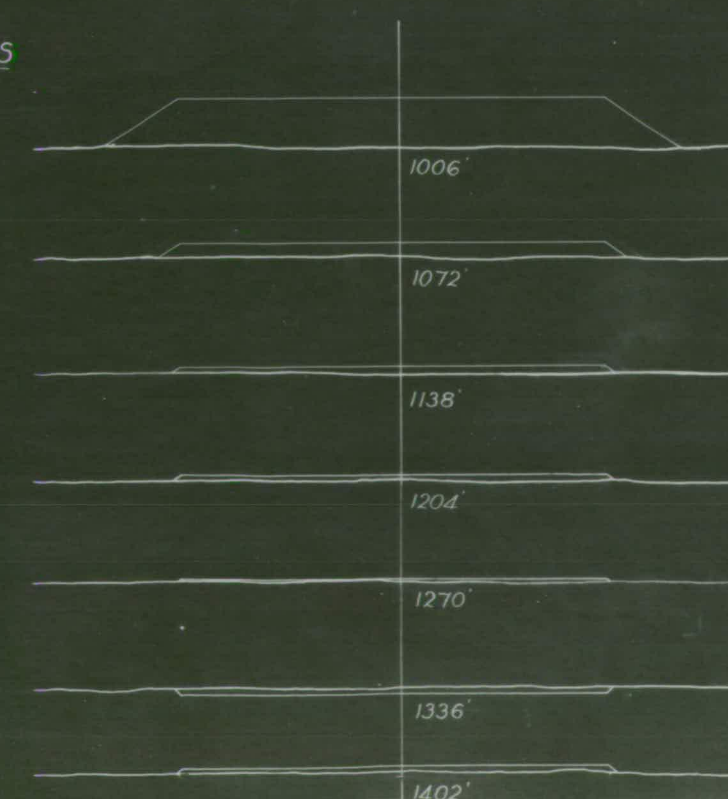
Chainage in feet	Sand Fill	Rock Fill	Bank	Cutting	Top of Finished Road	Surface
861	9'7"	0'0"	0'0"		75·29	65·70
897	7'7"	2'0"	7'5¾"		84·75	67·70
907	[illegible]	2'0"	7'3"		84·52	68·19
930					84·20	
940	4'4"	2'0"	6'8½"		83·88	70·85
981	0'0"	2'0"	5'3½"		82·57	75·29
982		0'7¾"	5'3"		82·54	76·66
1006		0'0"	4'2¾"		81·77	77·55
1072			1'3"		79·66	78·40
1086			1'0"		79·21	78·21
1138			8½"		78·57	77·85
1204			5¼"		77·75	77·31
1207 / 1227			0'0"	0'0"	77·71 / 77·71	77·71
1247					77·71	
1270			2¼"		77·71	77·52
1336				3½"	77·71	77·90
1402				2¼"	77·71	76·92
1468				0½"	77·71	77·76
1534			7"		77·71	77·14
1600			4½"		77·71	77·35
1666				1'1"	77·71	78·78
1732			0¾"		77·71	77·65
1798				3"	77·71	77·97
1864			1'4"		78·38	77·06
1930			2'1"		79·05	76·97
1996			1'6¼"		79·72	78·20
2062			6"		80·39	79·88
2074					80·52	
2095			0'0"	0'0"	80·42	80·42
2099 / 2121				9¼"	80·40 / 80·30	80·36 / 81·07
2138			1'1"		80·27	79·19
2194			7"		79·95	79·35
2260				0¾"	79·64	79·70
2326				2¾"	79·32	79·55
2392			0'0"	0'0"	79·01	79·01
2399			0'0"	0'0"	78·97	78·97

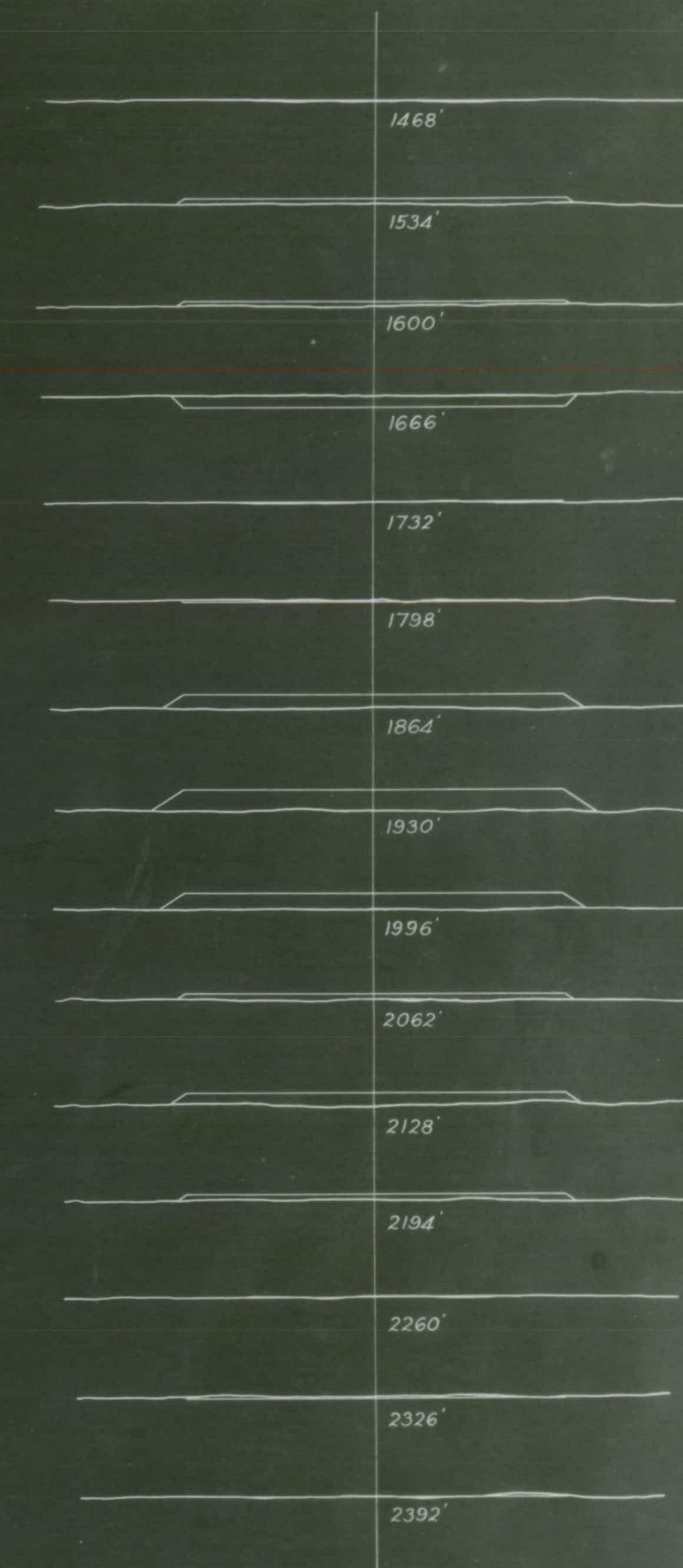
Western Abutment

℄ on bridge ℄ survey

CROSS SECTIONS
Scale - 10 ft = 1 inch

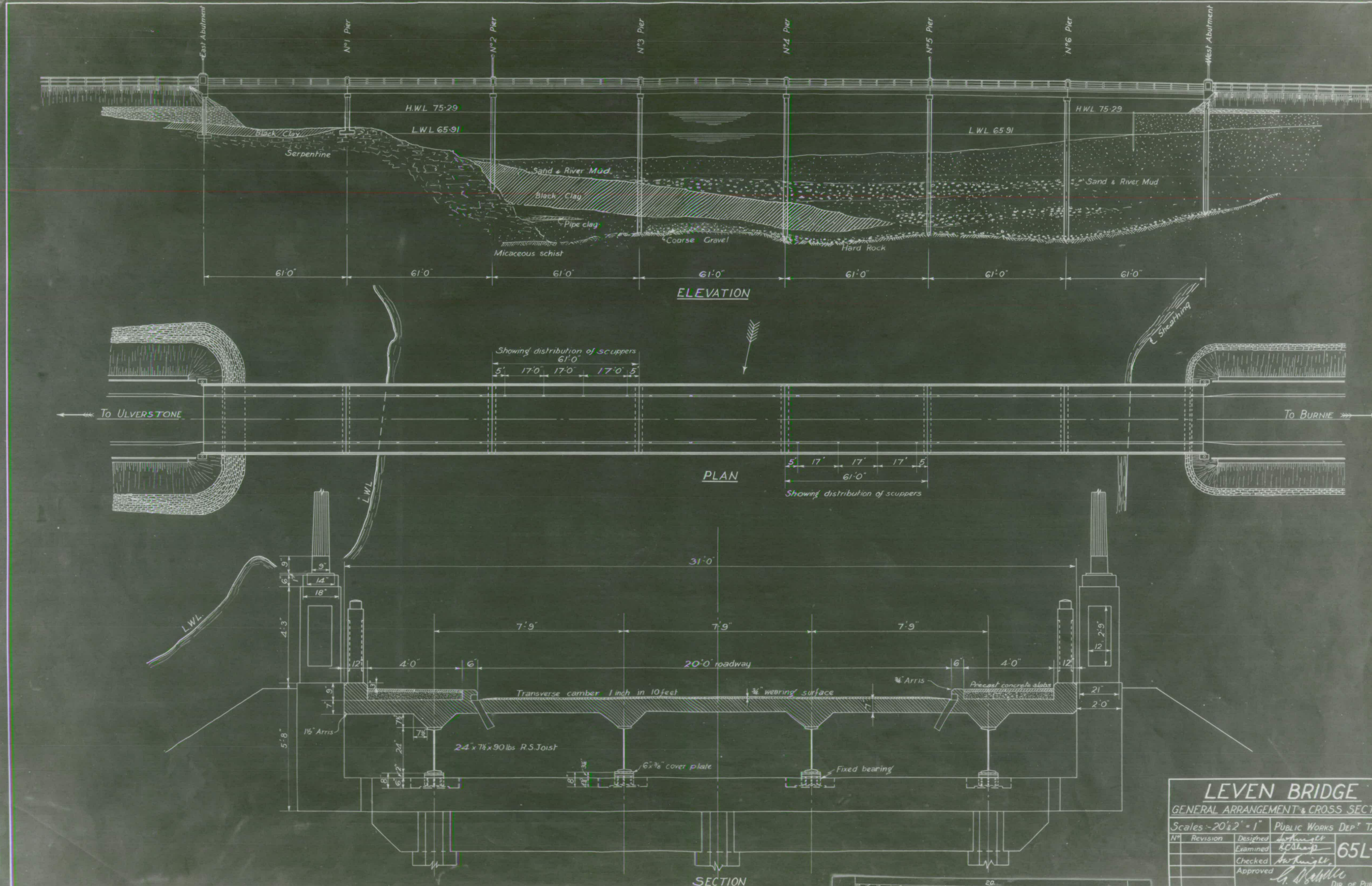






LEVEN BRIDGE
WESTERN APPROACH
LONGITUDINAL & CROSS SECTIONS

SCALE	as shown
	PUBLIC WORKS DEPARTMENT, TAS.
DESIGNED	[illegible] Knight
EXAMINED	[illegible]
CHECKED	[illegible] Knight
APP'D	[illegible]
	65L-8


East Abutment
Nº1 Pier
Nº2 Pier
Nº3 Pier
Nº4 Pier
Nº5 Pier
Nº6 Pier
West Abutment
H.W.L 75·29
L.W.L 65·91
Black Clay
Serpentine
Sand & River Mud
Black Clay
Pipe clay
Coarse Gravel
Micaceous schist
Hard Rock
61'0"
ELEVATION
Showing distribution of scuppers
61'0"
5' 17'0" 17'0" 17'0" 5'
To ULVERSTONE
To BURNIE
Sheathing
L.W.L
PLAN
5' 17' 17' 17' 5'
61'0"
Showing distribution of scuppers
31'0"
7'9"
20'0" roadway
4'0"
6"
12"
Transverse camber 1 inch in 10 feet
¾" wearing surface
¾" Arris
Precast concrete slabs
1½" Arris
24" x 7½" x 90 lbs R.S. Joist
6" x ⅜" cover plate
Fixed bearing
5'8"
4'3"
2'9"
21"
2'0"
18"
14"
9"
L.W.L
SECTION
LEVEN BRIDGE
GENERAL ARRANGEMENT & CROSS SECTION
Scales:- 20' & 2' = 1"
PUBLIC WORKS DEPᵀ TAS.
Nº Revision
Designed
Examined
Checked
Approved
65L-9
DIR. OF PUB. WKS

STEEL LIST

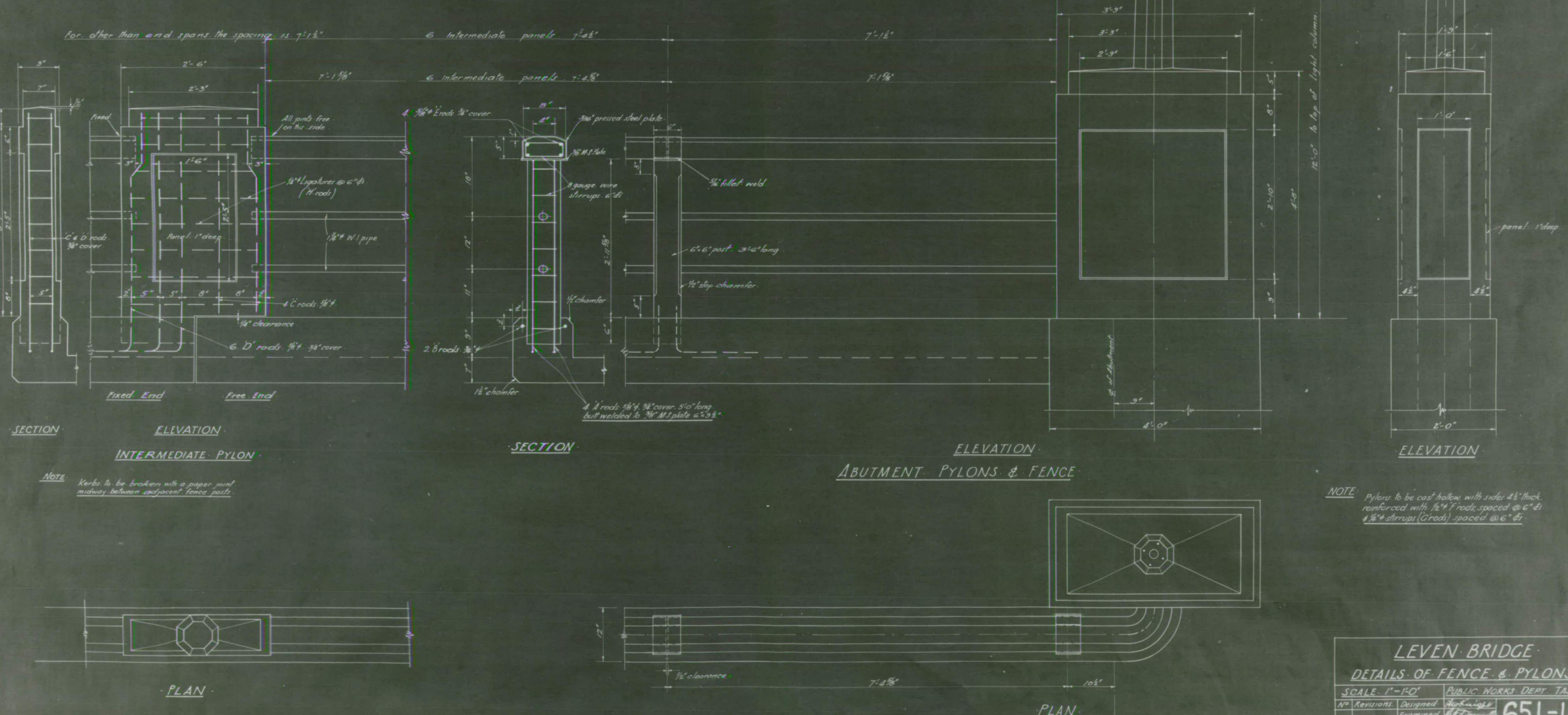
Mark	Size	Nº	Length	Wt	Total Wt	Shape	Remarks
A	5/8"ϕ	392	5'-0"	5.2 lbs	2040 lbs	Straight	Fence posts
B	3/4"ϕ	224	7'-0"	10.50 "	2353 "	"	kerbs
C	5/8"ϕ	48	3'-9"	3.90 "	187 "	As shewn	Pier posts
D	5/8"ϕ	72	6'-0"	6.24 "	449 "	"	"
E	5/8"ϕ	448	7'-3"	7.54 "	3380 "	"	coping
F	1/2"ϕ	90	4'-6"	3.02 "	272 "	"	Pylons
G	1/4"ϕ	32	11'-0"	1.84 "	59 "	"	Pylons ligatures
H	1/4"ϕ	96	7'-0"	1.19 "	114 "	"	Pier post ligatures
J	3/4"ϕ	32	8'-0"	12.0 "	384 "	"	Light standard

9244 " — 4.13 tons

MATERIAL LIST

Item	Size	Number
Fence Posts	6"×6"×3'-6"	98
Pier "	2'-6" × 9"	12
Coping rails	8"×5" ×7'-4"	112
M.S Plate	3/8" × 6" × 9½"	98
"	3/16" × 6" × 18"	98 (pressed to shape)
W.I. Pipe	1¼"ϕ 14'-8" long	112 pieces.
Light standards		8 off
Wire	8 gauge	4500 lin ft

Light standard (Located at Abutments Nº2 & Nº5 Piers)
6" octagonal at top
3" " " bottom
2"ϕ hole down centre
Standard reinforced with
4. 3/4"ϕ 'J' rods 3/4" cover.



SECTION

ELEVATION

INTERMEDIATE PYLON

NOTE Kerbs to be broken with a paper joint midway between adjacent fence posts.

SECTION

ELEVATION

ABUTMENT PYLONS & FENCE

ELEVATION

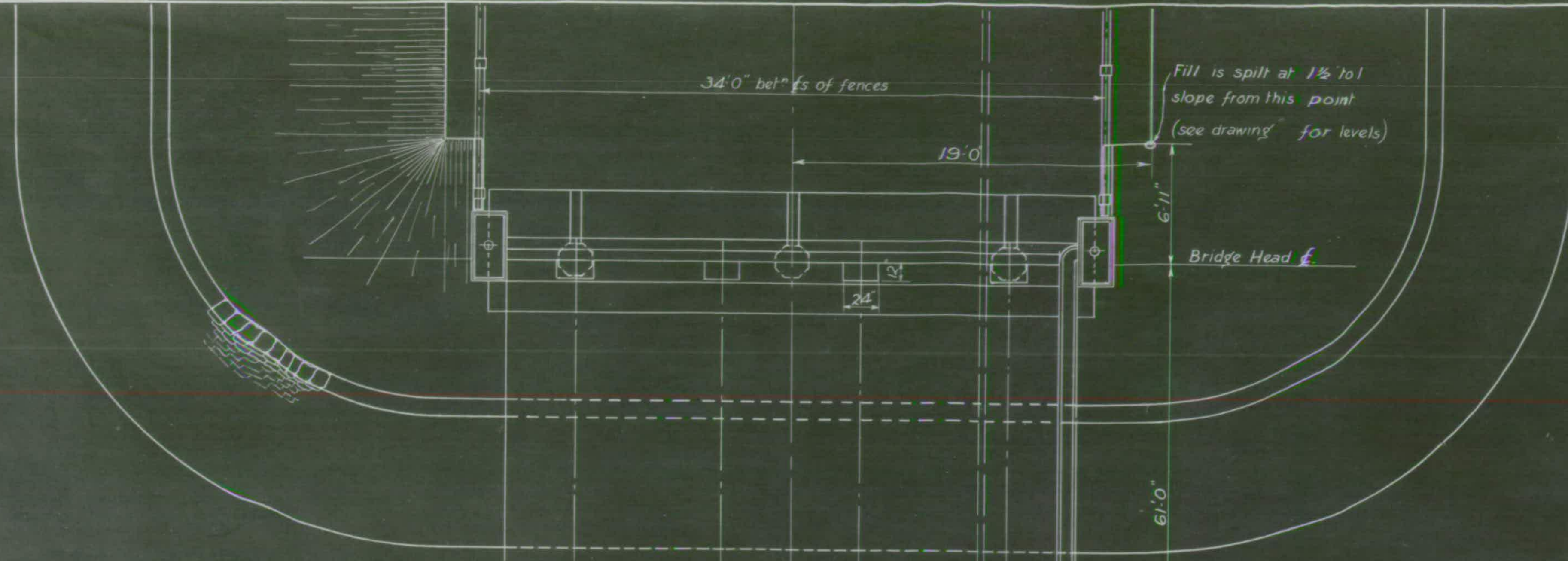
NOTE: Pylons to be cast hollow with sides 4½" thick, reinforced with ½"ϕ 'F' rods spaced @ 6" c/s & ¼"ϕ stirrups ('G' rods) spaced @ 6" c/s

ONE INCH = ONE FOOT

LEVEN BRIDGE

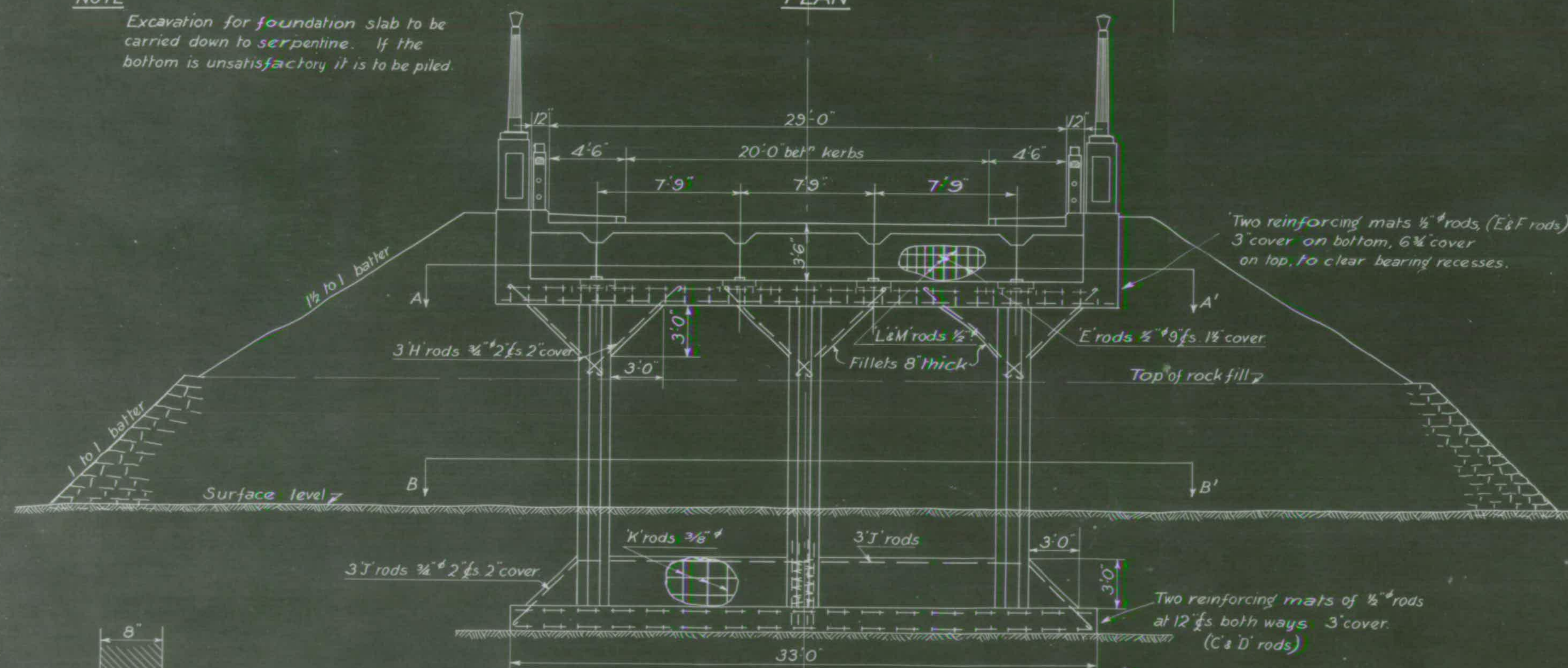
DETAILS OF FENCE & PYLONS

SCALE 1"=1'-0"		PUBLIC WORKS DEPT TAS	
Nº	Revisions	Designed	[signature]	65L-10
		Examined	[signature]	
		Checked	[signature]	
		Approved	[signature] DIR OF PUB WKS	

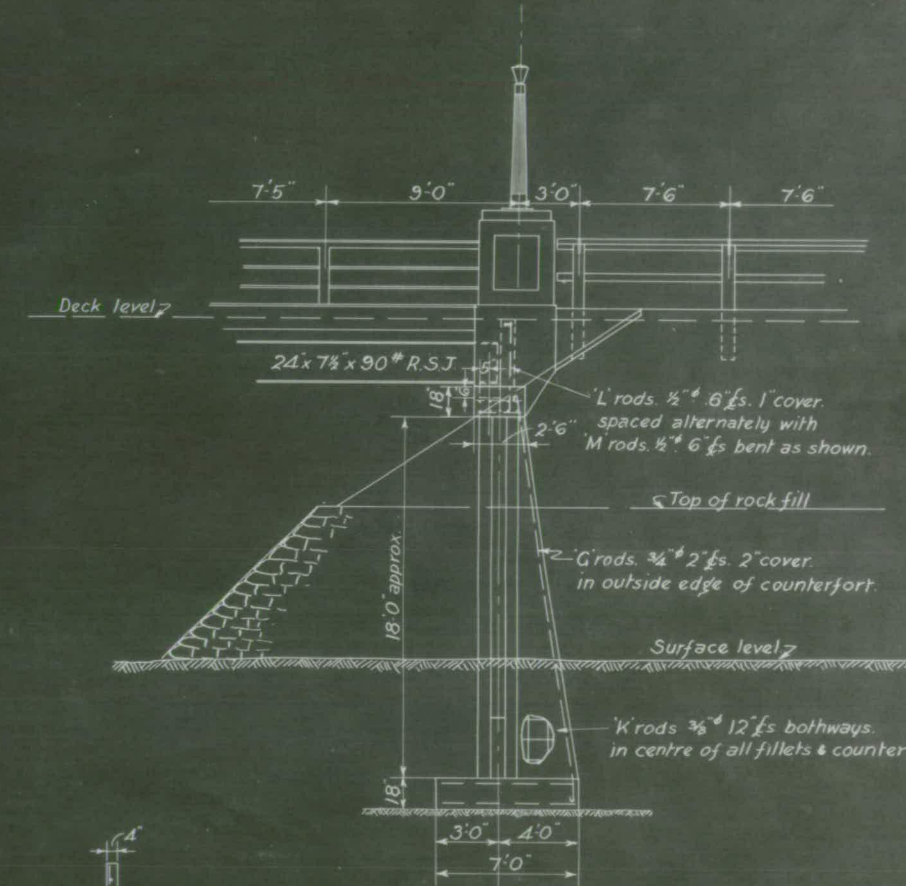


PLAN

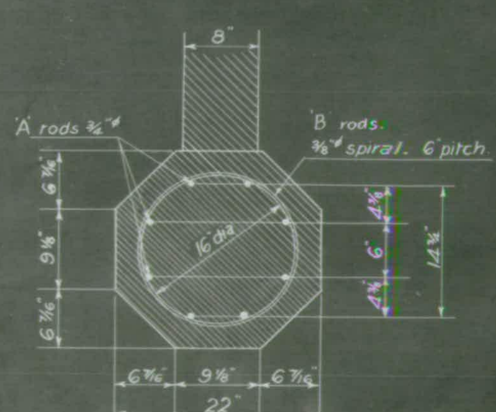
NOTE
Excavation for foundation slab to be carried down to serpentine. If the bottom is unsatisfactory it is to be piled.



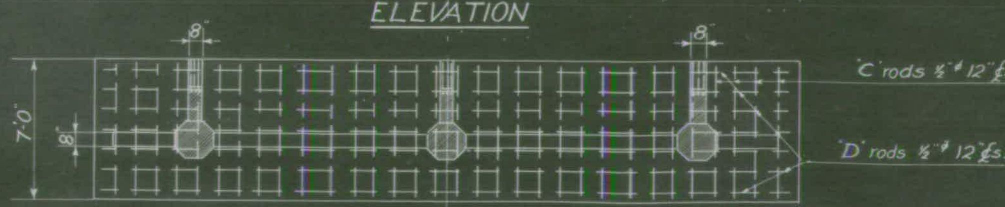
ELEVATION



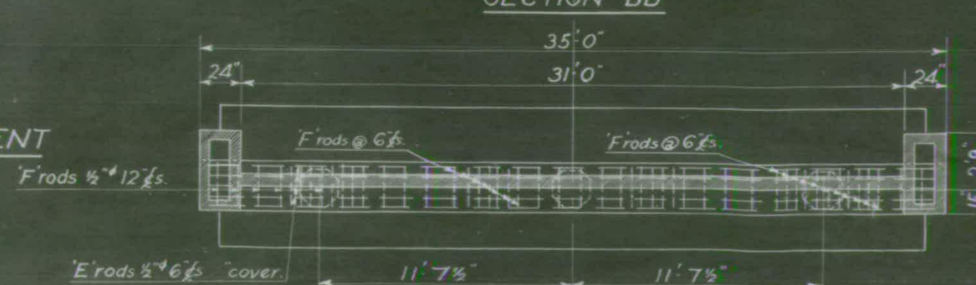
SIDE ELEVATION



DETAIL OF COLUMN REINFORCEMENT
1"-1' scale

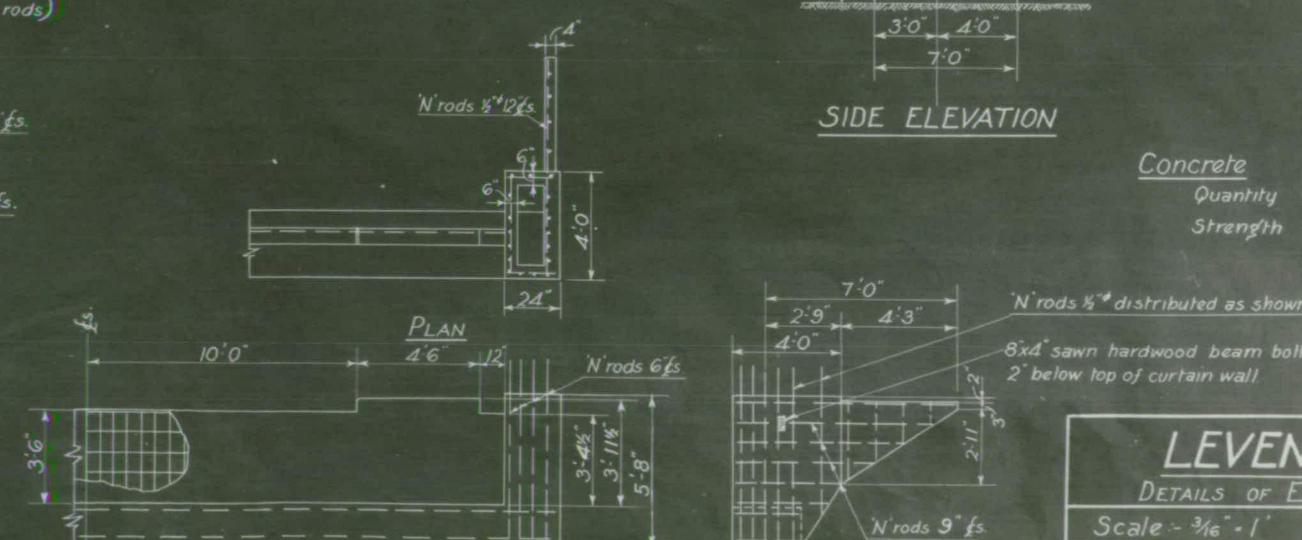


SECTION BB'



SECTION AA'

¼ OF AN INCH = ONE FOOT



PLAN

END ELEVATION

SIDE ELEVATION

DETAILS OF CURTAIN WALLS (PYLONS REMOVED)
½"-1' scale

STEEL LIST

Mark	Size	No of	Length	Wt of ea.	Total Wt	Shape	Remarks.
A	¾" dia.	24	20'0"	30 lbs	721 lbs.	Straight	Columns (verticals)
B	⅜" "	27	20'0"	7·6 "	203 "	Spiral	"
C	½" "	64	6'0"	4·0 "	257 "	Straight	Foundation slabs.
D	½"φ "	28	17'0"	11·4 "	319 "	"	" "
E	½" "	32	18'0"	12·0 "	386 "	"	Cap & curtain wall (transverse)
F	½" "	50	2'0"	1·34 "	67 "	"	"
G	¾" "	9	20'0"	30·0 "	270 "	Bent as shown	Counterforts.
H	¾" "	18	8'0"	12·0 "	216 "	" " "	Fillets (betn cap & column)
J	¾" "	9	12'0"	18·0 "	162 "	" " "	Fillet (betn slab & columns)
K	⅜" "	16	20'0"	7·6 "	122 "	" " "	Fillets & counterforts.
L	½" "	31	4'6"	3·0 "	94 "	Straight.	Curtain walls.
M	½" "	31	6'0"	4·0 "	125 "	4'6" × 18	" "
N	½" "	20	14'0"	9·4 "	188 "	Straight	To cut pylon reinforcing

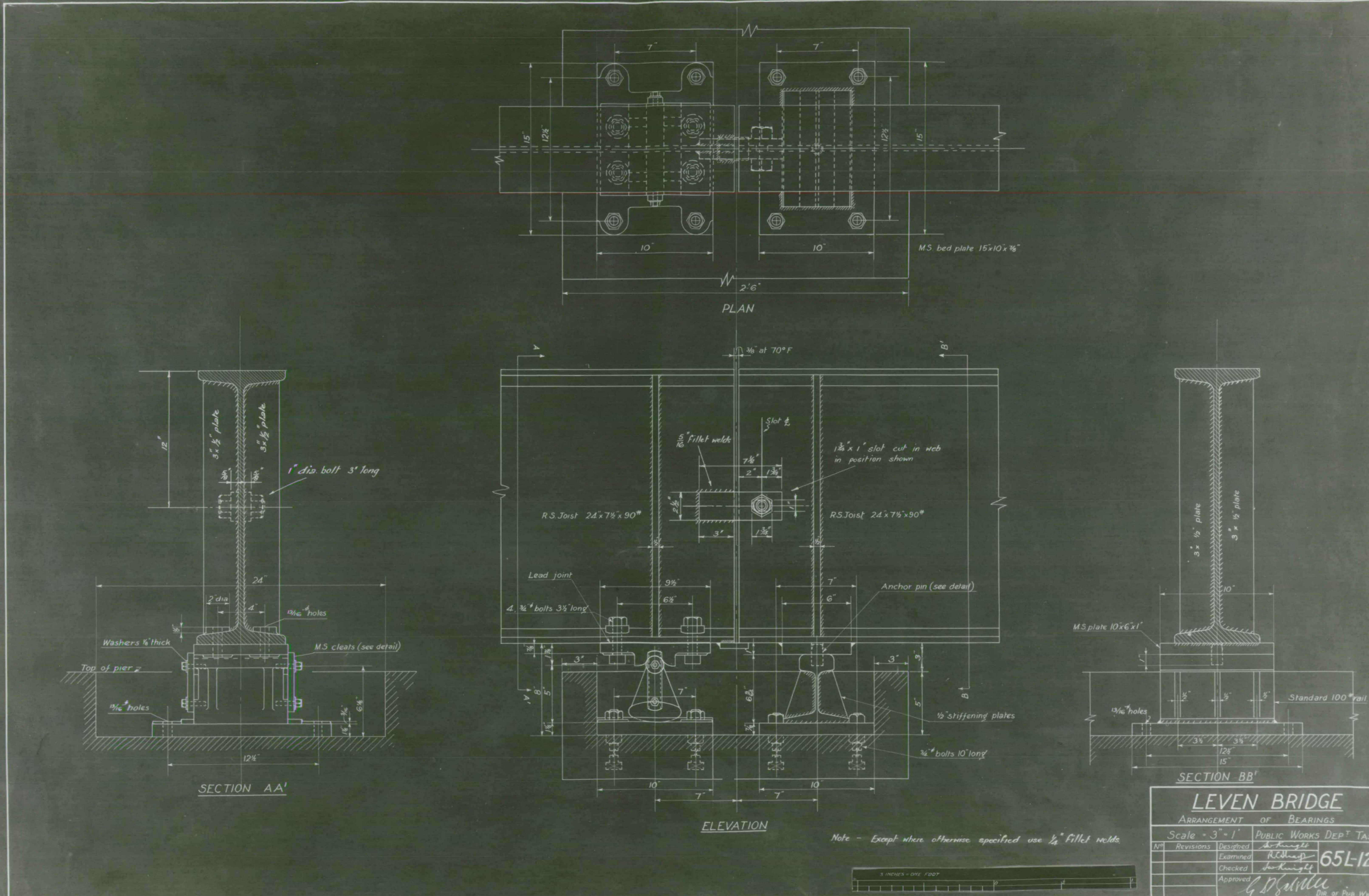
Total weight of steel = 1 ton 8 cwt.

Concrete
Quantity - 33 c yds.
Strength - base slab 1500#/□"
columns, fillets 2000#/□"
cap, curtain wall, etc. 2500#/□"

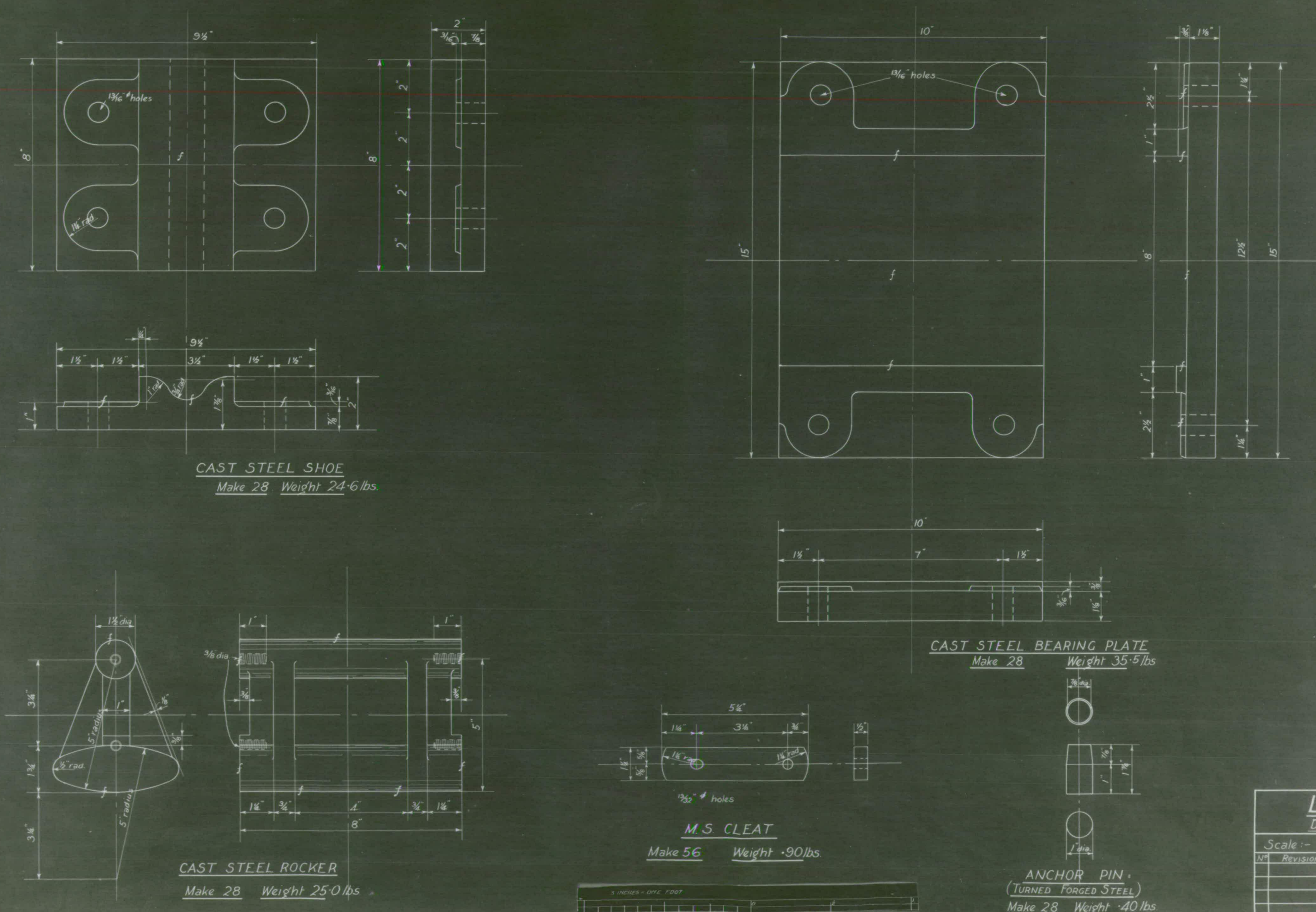
LEVEN BRIDGE
DETAILS OF EASTERN ABUTMENT

Scale :- 3/16"-1'		PUBLIC WORKS DEPT TAS.	
No	Revision	Designed	65L-11
		Examined	
		Checked	
		Approved	

DIR OF PUB WKS



PLAN
M.S. bed plate 15"x10"x 7/8"
2'-6"
ELEVATION
3/8" at 70° F
Slot ℄
5/16" Fillet welds
1 3/4" x 1" slot cut in web in position shown
R.S. Joist 24"x 7 1/2" x 90#
Lead joint
4. 3/4" bolts 3 1/2" long
Anchor pin (see detail)
1/2" stiffening plates
3/4" bolts 10" long
SECTION AA'
3"x 1/2" plate
1" dia. bolt 3" long
2" dia
13/16" holes
Washers 1/8" thick
M.S. cleats (see detail)
Top of pier
SECTION BB'
M.S. plate 10"x6"x1"
Standard 100# rail
Note - Except where otherwise specified use 1/4" Fillet welds
3 INCHES = ONE FOOT
LEVEN BRIDGE
ARRANGEMENT OF BEARINGS
Scale - 3" - 1'
PUBLIC WORKS DEPT TAS
No
Revisions
Designed
Examined
Checked
Approved
65L-12
DIR. OF PUB. WKS

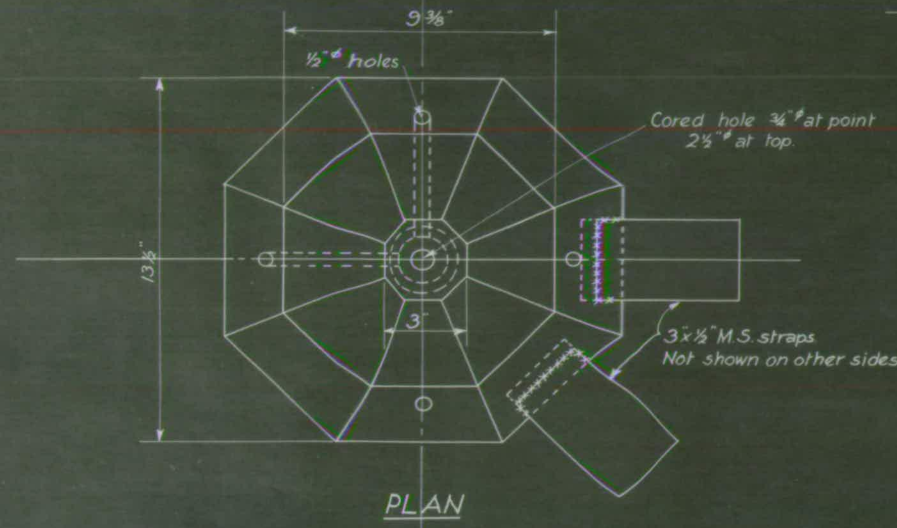


LEVEN BRIDGE

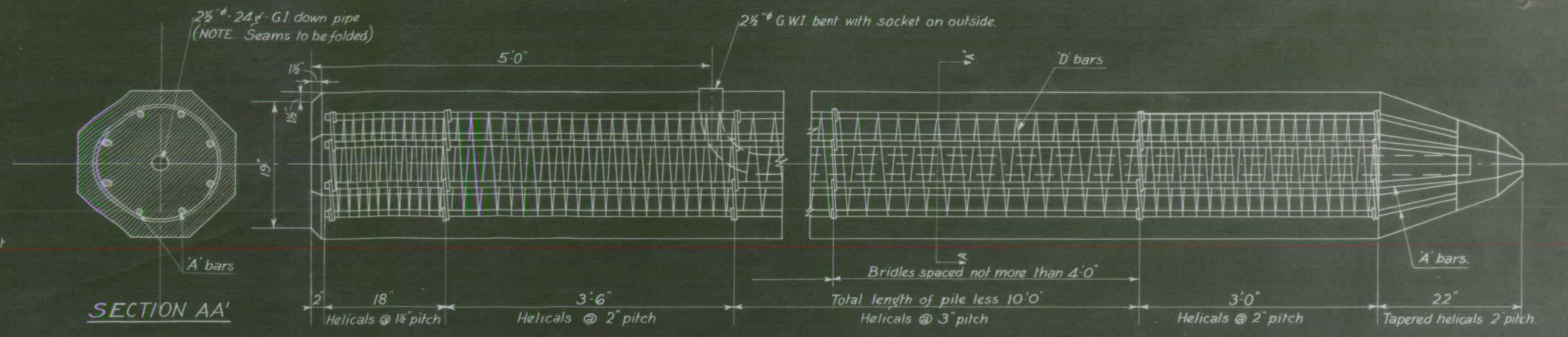
DETAILS OF BEARINGS

Scale :- 6" = 1'	PUBLIC WORKS DEPT TAS.	
Nº Revision	Designed	65L-13
	Examined	
	Checked	
	Approved	

DIR. OF PUB. WKS.



PLAN



SECTION AA'

ELEVATION

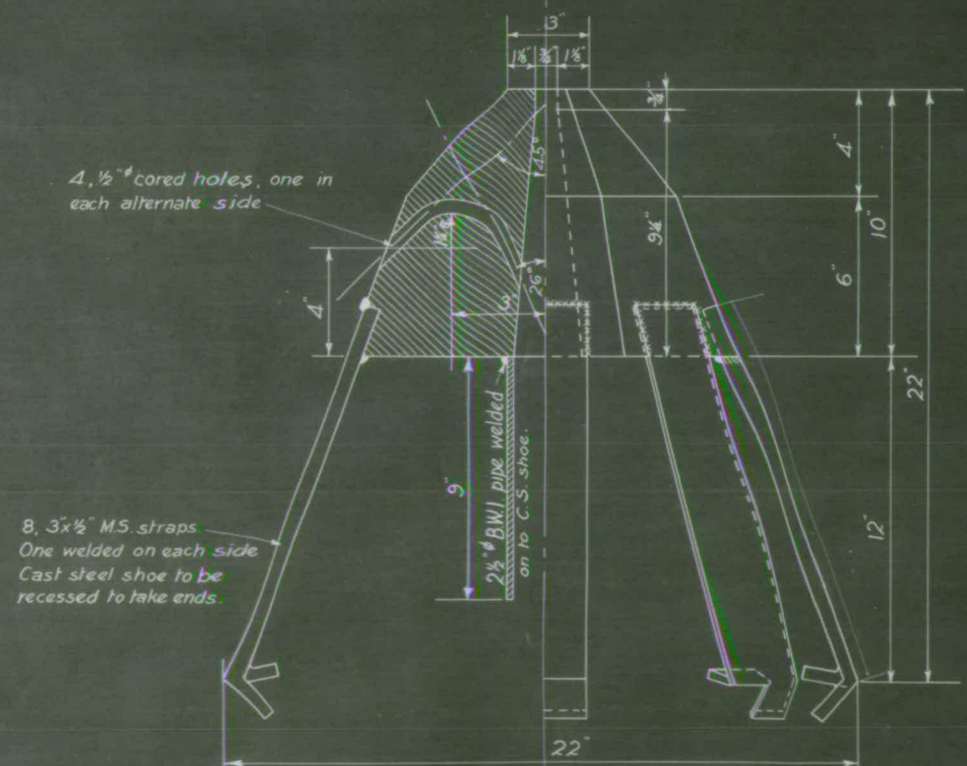
DETAIL OF REINFORCEMENT IN PILES

Scale:- 1"=1'

STEEL LIST

Mark	Size	Number of	Length	Wt of each	Total Wt	Shape	Remarks
A	1⅛" dia.	315	30'0"	101·4 lbs	14·20 tons	Straight	Vertical reinforcement
B	½" "	165	16'0"	10·69 "	0·79 "	Bent as shown	Bridles
C	¼" "	200	12'0"	2·004 "	0·18 "	" " "	M.S. supports
D	⅜" "	566	40'0"	15·04 "	3·80 "	Spiral as shown	Helical spirals

Total = 18·97 tons (Sufficient for 6 piles)



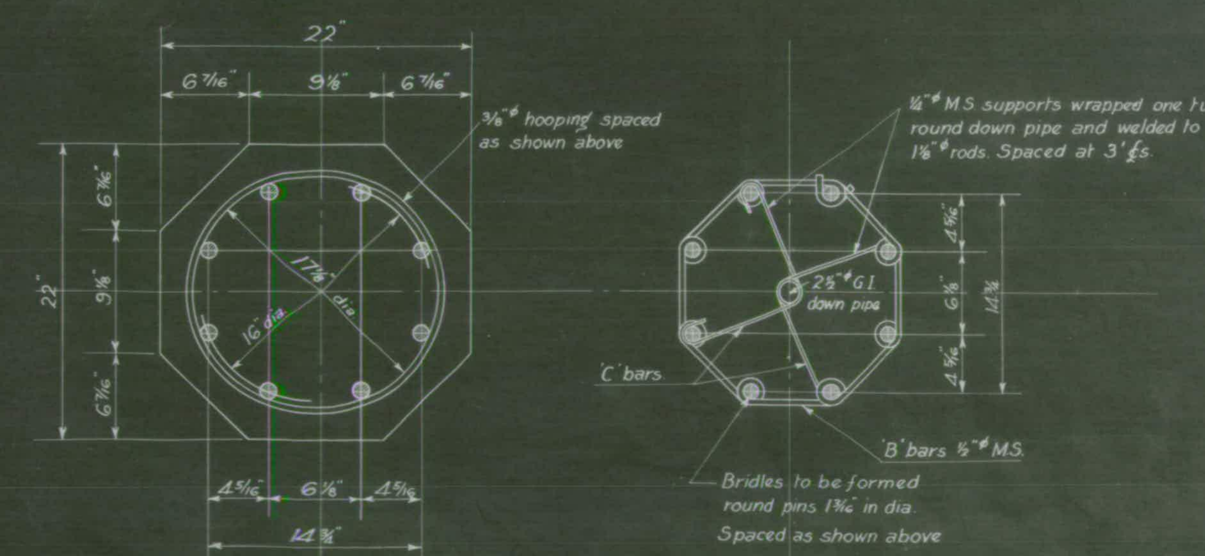
HALF SECTION HALF ELEVATION

CAST STEEL PILE SHOE

Scale - 3" = 1'

Make 18

Wt of straps per pile = 53 lbs.
Wt of cast shoe = 214 lbs.
Quantity of piping



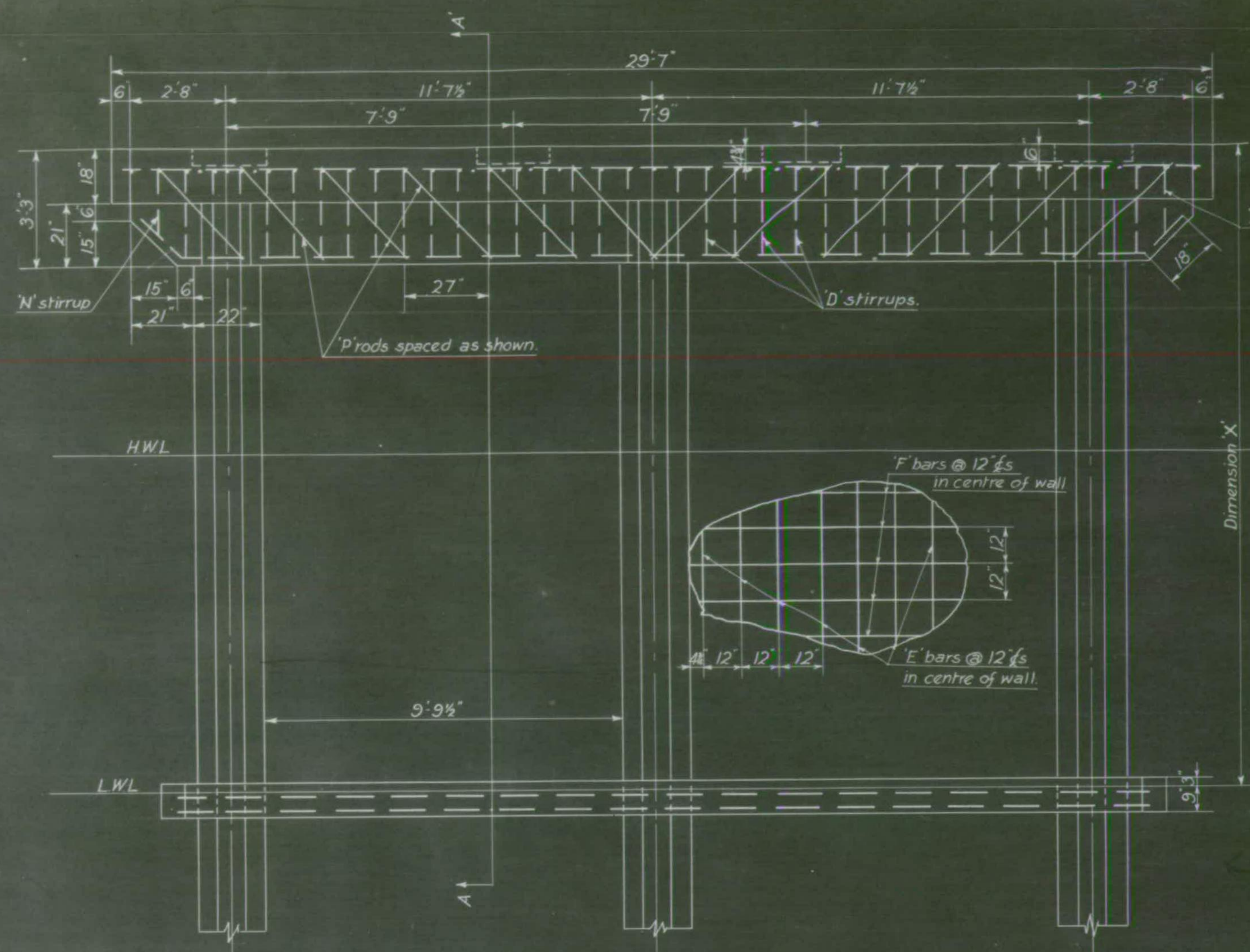
SECTIONS OF PILE

Scale - 1½" = 1'

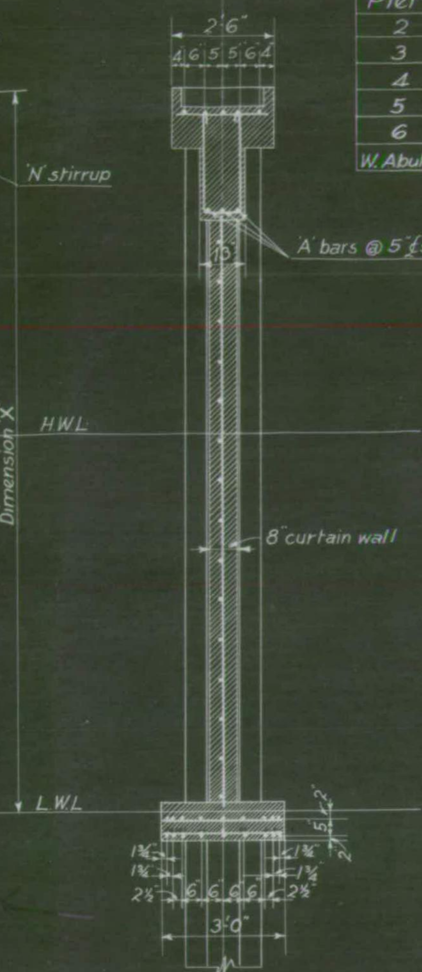
ONE INCH = ONE FOOT

3 INCHES = ONE FOOT

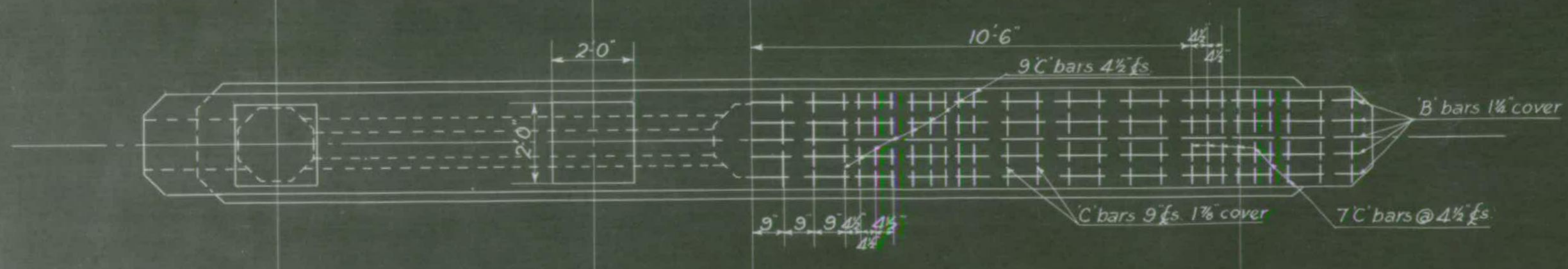
LEVEN BRIDGE			
DETAILS OF PILES			
Scales :- 1", 1½" & 3" = 1'		PUBLIC WORKS DEPT TAS	
No	Revisions	Designed	65L-14
		Examined	
		Checked	
		Approved. DIR. OF PUB. WKS.	



ELEVATION



SECTION AA'

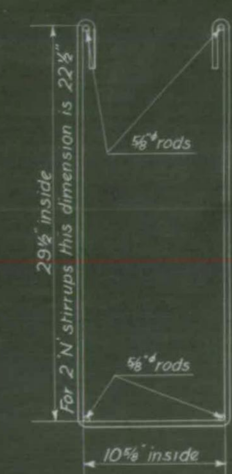


PLAN

Scale - ⅜" = 1'

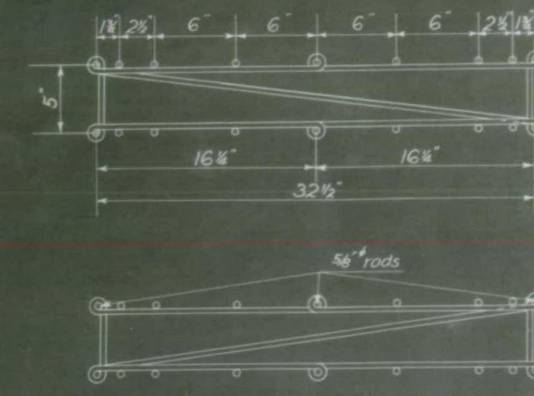
TABLE OF DIMENSION 'X'

Pier	Dimension X
2	17'5"
3	17'4"
4	17'1"
5	16'8"
6	16'1"
W. Abut^mt	15'4"



CAP STIRRUPS ('D' & 'N')

Make 35



'O' STIRRUPS

SPACED ALTERNATELY IN PRECAST FOUNDATION SLAB

Make 10

Scale - 1½" = 1'

STEEL LIST

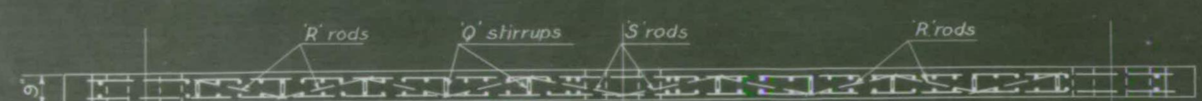
Mark	N° Off	Size	Length	W^t of ea.	Total W^t	Shape	Remarks
A	6	⅝" ϕ	16'0"	16·69 lbs	100 lbs	Bent as shown	Bottom of cap (longitudinal)
B	10	⅝" ϕ	16'0"	16·69 "	167 "	Straight	Top of cap (longitudinal)
C	53	⅝" ϕ	2'3"	2·35 "	124 "	"	" " " (transverse)
D	35	⅜" ϕ	6'6"	2·44 "	86 "	See detail	Cap stirrup
E	20	½" ϕ	15'0"	10·20 "	201 "	Straight	Curtain wall (verticals)
F	30	½" ϕ	9'6"	6·35 "	191 "	"	" " (horizontal)
G	16	½" ϕ	16'0"	10·69 "	171 "	"	Top & bottom of foundation slab (longitudinal)
H	8	½" ϕ	3'9"	2·51 "	20 "	"	" " " " " " (longitudinals by end piles)
J	4	½" ϕ	5'0"	3·34 "	13 "	"	" " " " " " (" " central piles)
K	20	½" ϕ	9'3"	6·18 "	124 "	"	" " " " " " (longitudinals)
L	88	½" ϕ	2'10"	1·89 "	166 "	"	" " " " " " (transverse)
M	24	½" ϕ	2'3"	1·51 "	36 "	"	" " " " " " (diagonals)
N	2	⅜" ϕ	5'3"	1·97 "	4 "	See detail	Cap stirrup
P	24	⅜" ϕ	3'6"	1·32 "	32 "	Straight	Bracing in cap
O	10	⅜" ϕ	10'0"	3·76 "	38 "	See detail	Foundation slab stirrups
R	16	⅜" ϕ	2'3"	0·85 "	14 "	Straight	" " braces
S	4	⅜" ϕ	2'9"	1·04 "	4 "	"	" " "

NOTE
5 piers and Western Abutment to be as detailed on this plan.

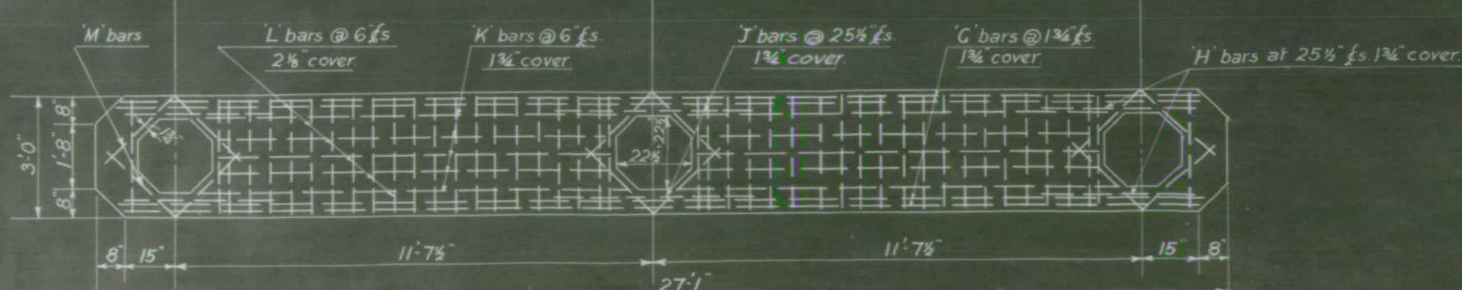
QUANTITIES

CONCRETE	Foundation slab	- 2·7 cub yds
	Curtain wall	- 7·0 " "
	Cap	- 6·0 " "
STRENGTH	3000 lb/□"	Total - 15·7 " "
STEEL	Total Weight - 13½ cwt	
	32 holding down bolts	

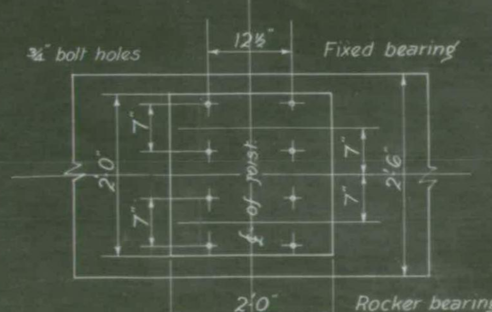
NOTE All quantities are per pier.



NOTE
Slab to be picked up at points 5'5" from ends
Weight - 3 tons 16 cwt.



DETAILS OF PRECAST FOUNDATION SLAB
(For Piers 2 to 6 & Western Abutment)



Fixed bearing bolts to project 1⅞" above concrete
Rocker " " " " 2⅛" " "

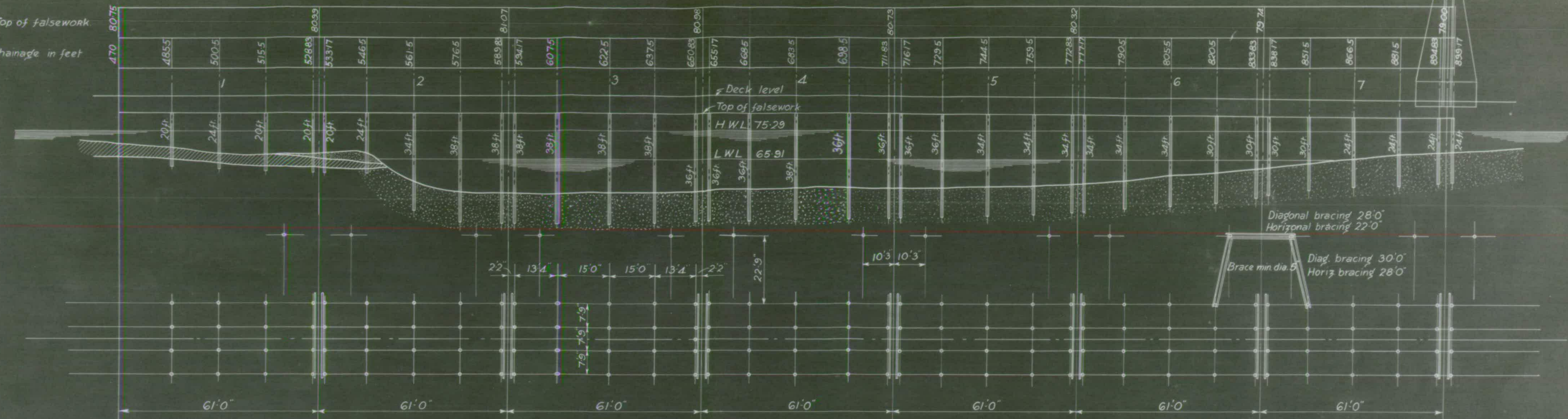
DETAIL OF BEARING RECESS

Scale - ¾" = 1'

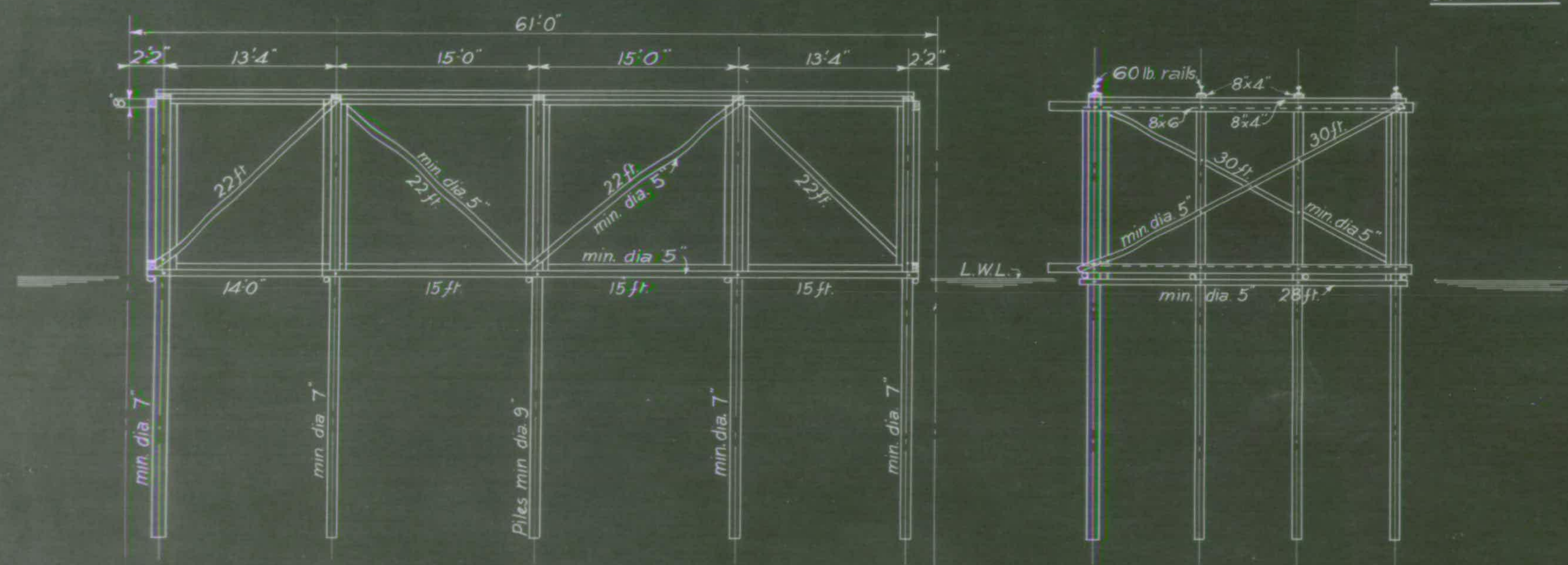
⅜ OF AN INCH = ONE FOOT

¾ OF AN INCH = ONE FOOT

LEVEN BRIDGE			
DETAIL OF PIERS			
Scales - ⅜", ¾" & 1½" = 1'	PUBLIC WORKS DEP^T TAS		
N°	Revision	Designed	
		Examined	65L-15
		Checked	
Approved	G. D. Balsille	Dir. of Pub. Works	

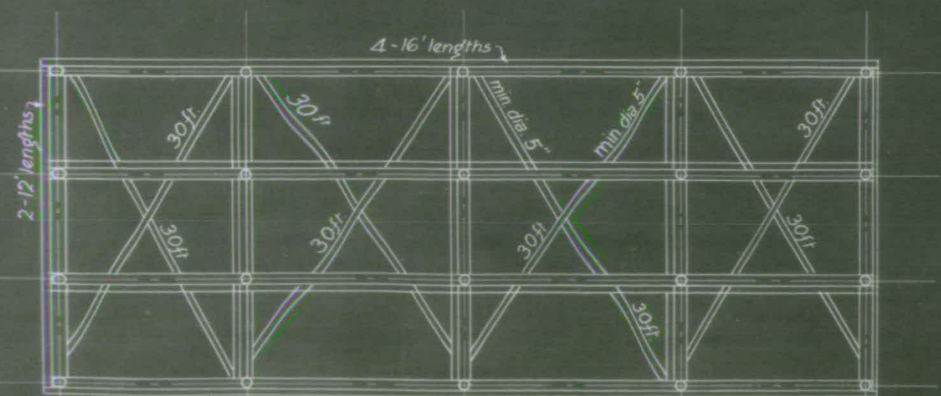


GENERAL ARRANGEMENT
Scale 20' = 1"



ELEVATION

END ELEVATION



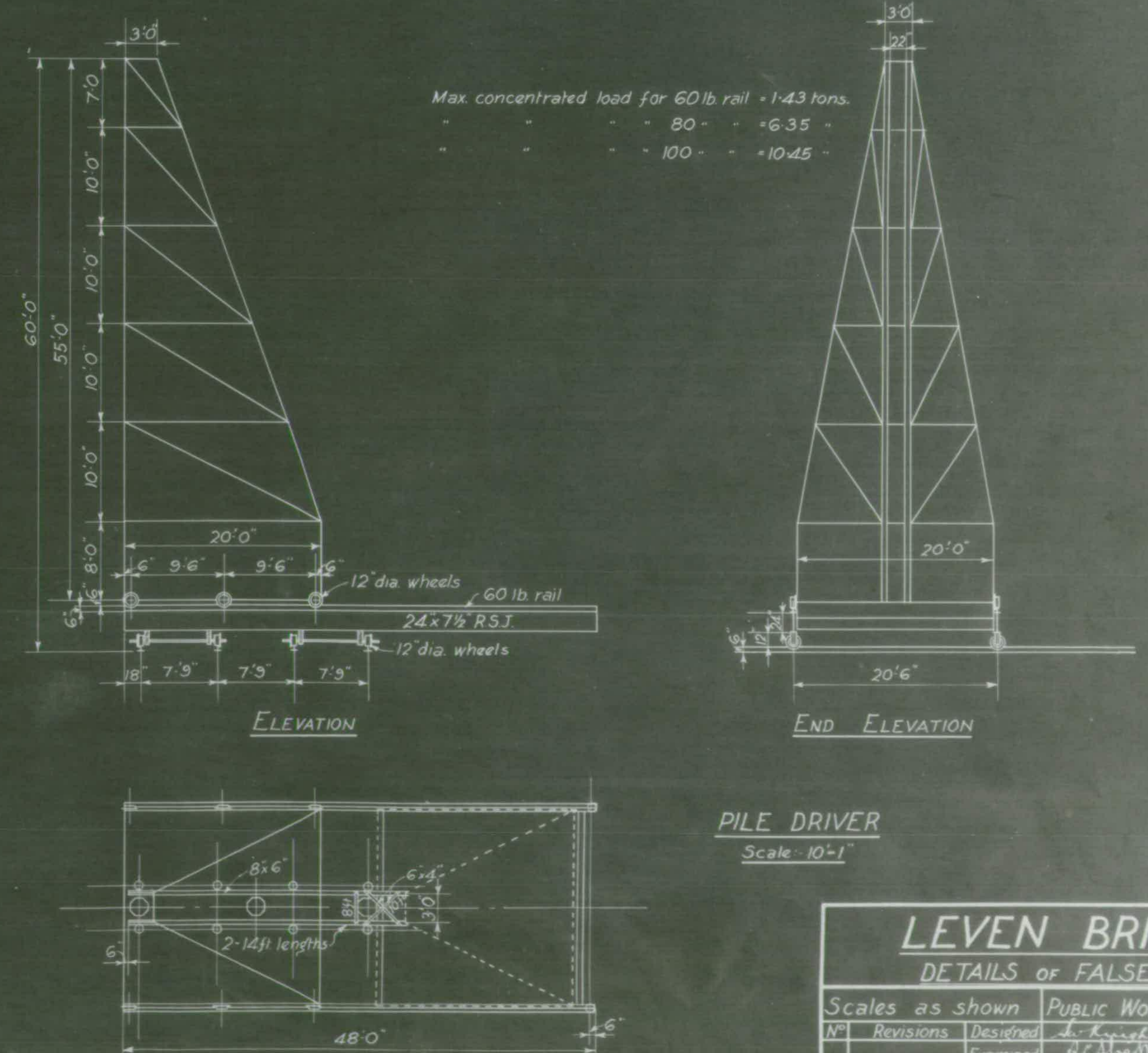
PLAN

SPAN 3
Scale 8' = 1"

MATERIAL LIST

Round Timber			Sawn Timber		
Diameter	Length	Number	Size	Length	Number
5"	22'0"	63	6"x4"	18'0"	8
5"	28'0"	76	8"x4"	17'0"	54
5"	30'0"	210	8"x4"	15'0"	56
7"	20'0"	17	8"x4"	14'0"	56
7"	24'0"	19	8"x4"	22'0"	7
7"	30'0"	18	8"x6"	14'0"	28
7"	34'0"	18	Bolt Steel		
7"	36'0"	32	Diameter	Length	Number
7"	38'0"	22	7/8"	20'0"	45
9"	24'0"	8	3/4"	20'0"	45
9"	34'0"	12			
9"	38'0"	8			

Max. concentrated load for 60 lb. rail = 1·43 tons.
" " " 80 " = 6·35 "
" " " 100 " = 10·45 "



ELEVATION

END ELEVATION

PLAN

PILE DRIVER
Scale 10' = 1"

LEVEN BRIDGE
DETAILS OF FALSEWORK

Scales as shown
PUBLIC WORKS DEPT TAS

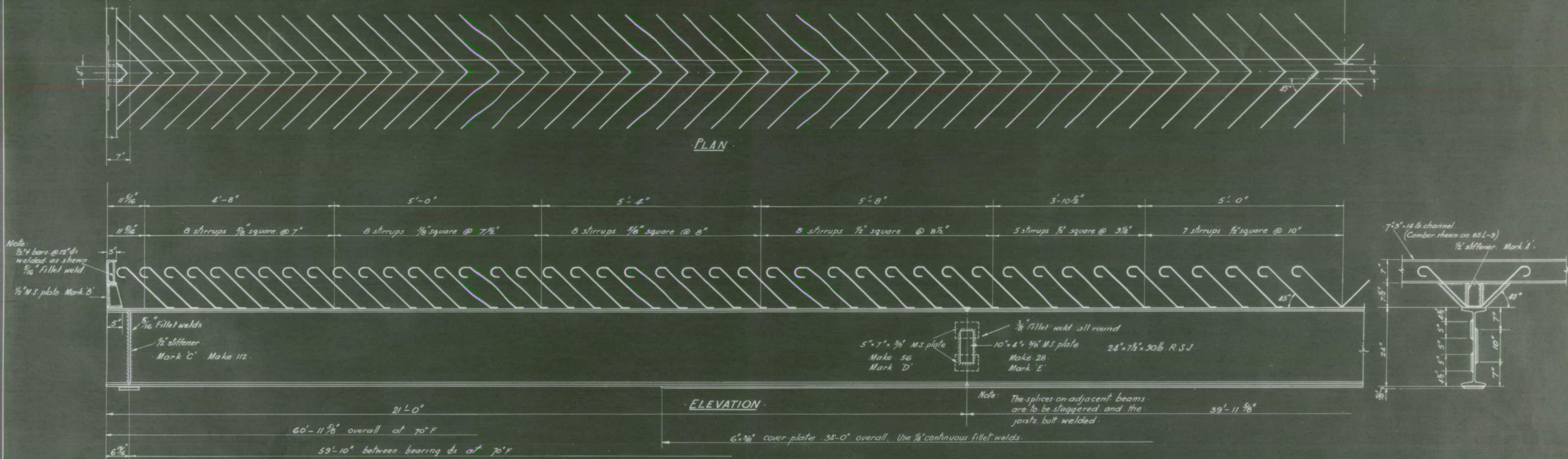
No	Revisions	
		Designed
		Examined
		Checked
		Approved

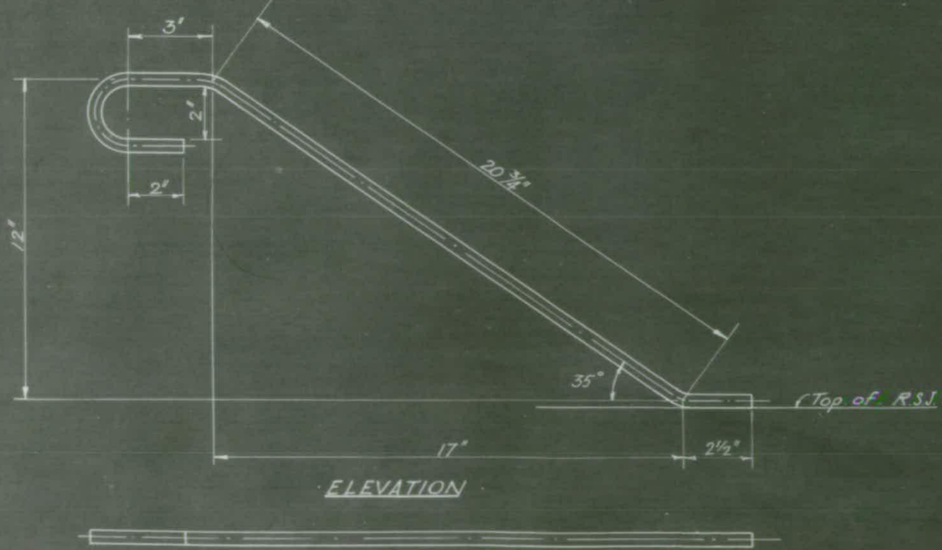
65L-16

DIR. OF PUB. WKS.

Note: Stirrups are to be bent to shape as shown in the detail drawing then welded in pairs at the spacing indicated in the plan and at an angle of 45° in both directions from the flange of the joist.

Note: 1/2" ø bars bent to form hooks to be welded to top of 7"×3" channel under kerb.

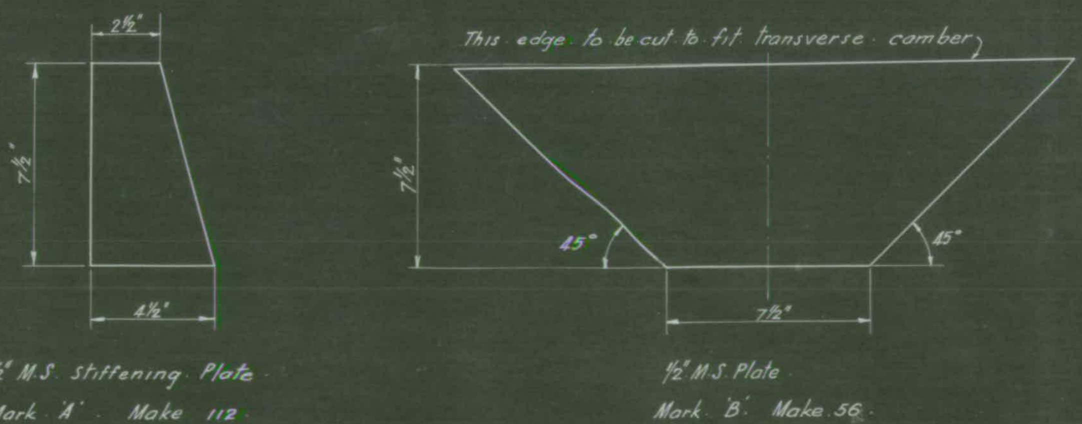




DETAILS OF STIRRUPS (Approx length 32¼")

Make 2688 5/8" square and 2240 ½" square.

Note: 5/8" stirrups to be attached to R.S.J. by 4 lin ins of 5/16" fillet weld.
½" " " " " " " 4 " " ¼" " "



DETAIL OF M.S PLATES

MATERIAL LIST

	Length	Number
24" × 7½" × 90 lb R.S.J.	30'-11 5/8"	28
" "	21'-0"	28
7" × 3" × 14 lb channels	31'-0"	14
M.S rods 5/8" square	32¼"	2688
" ½" "	32¼"	2240
M.S cover plate 6" × 3/8"	34'-0"	28
M.S plate Mark 'A'		112
" " 'B'		56
" " 'C'		112
" 5"×7"×3/8" " 'D'		56
" 10"×4"×3/8" " 'E'		28

¼ OF AN INCH = ONE FOOT

LEVEN BRIDGE

DETAILS OF BEAMS & STIRRUPS

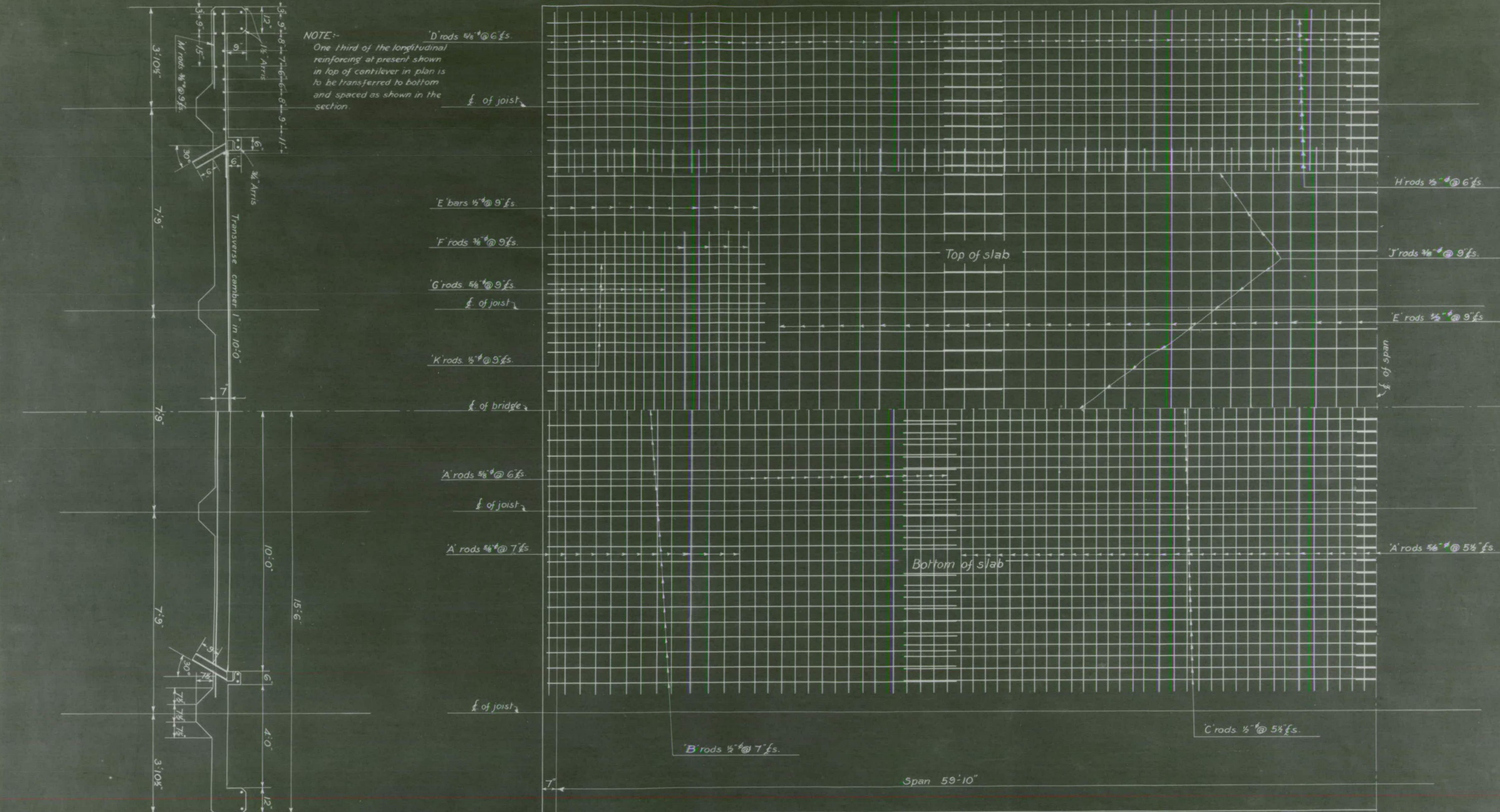
Scale 3/8" = 1'-0"

PUBLIC WORKS DEPT. TAS.

Nº	Revisions		
		Designed	A. Knight
		Examined	R.C. Sharp
		Checked	A. Knight
		Approved	G. S. Batville

Dr of Pub Wks

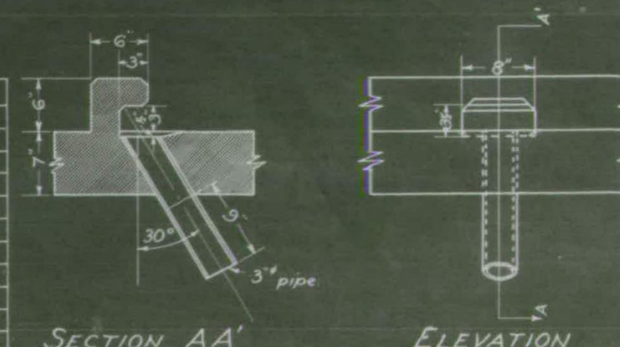
65L-17



STEEL LIST

Mark	Size	Nº Off	Length	Wt of each	Total Wt	Shape	Remarks
A	5/8"ϕ	123	22'0"	22·90 lbs	2820 lbs	Straight	Bottom of slab (transverse)
B	1/2"ϕ	74	15'0"	10·05 "	743 "	"	" " " (longitudinal)
C	1/2"ϕ	94	18'0"	12·05 "	1132 "	"	" " " (")
D	5/8"ϕ	244	6'0"	6·25 "	1525 "	"	Top of slab (transverse)
E	1/2"ϕ	80	20'0"	13·39 "	1070 "	"	" " " (")
F	3/8"ϕ	8	14'0"	5·33 "	43 "	"	" " " (")
G	5/8"ϕ	12	14'0"	14·58 "	175 "	"	" " " (")
H	1/2"ϕ	96	17'0"	11·38 "	1092 "	"	" " " (longitudinal)
J	3/8"ϕ	100	17'0"	6·46 "	646 "	"	" " " (")
K	1/2"ϕ	20	8'0"	5·36 "	107 "	"	" " " (")
L	5/8"ϕ	32	17'0"	17·70 "	566 "	"	Kerbs.
M	3/8"ϕ	163	3'0"	1·13	184 "	"	Deck slab. Bottom of cantilever.

Total weight of steel = 4·51 tons.



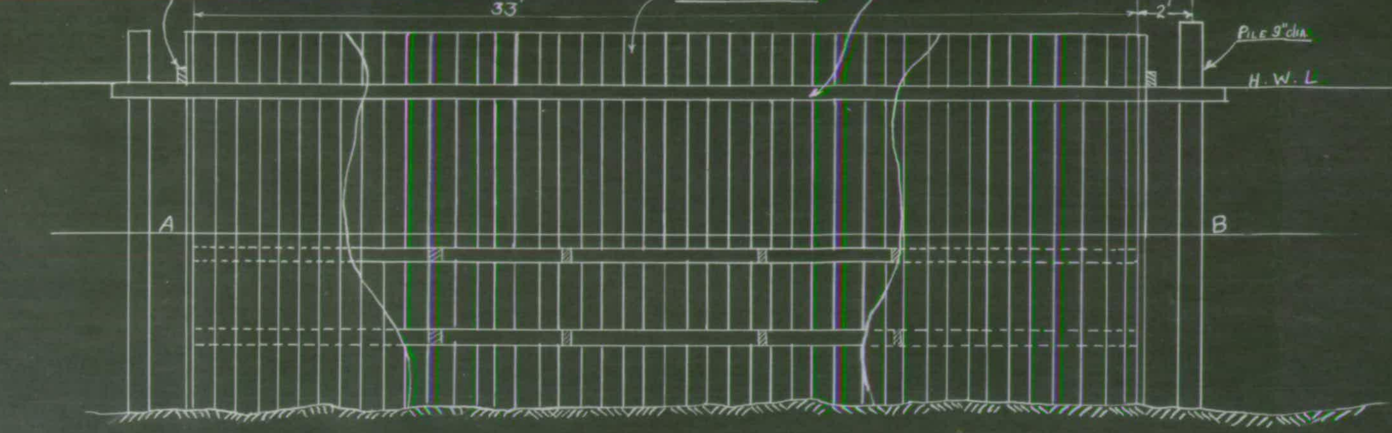
DETAIL OF SCUPPERS

LEVEN BRIDGE

ARRANGEMENT OF DECK REINFORCEMENT

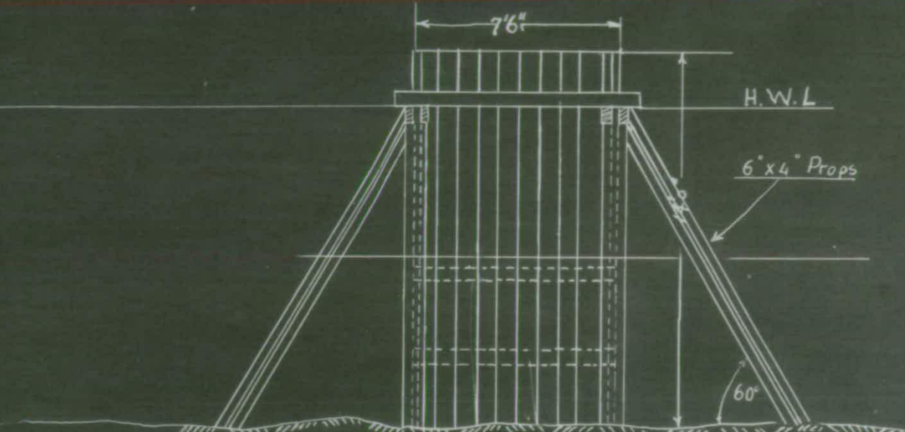
Scale:- 1/2" = 1'	PUBLIC WORKS DEPT TAS
Designed	[signature]
Examined	[signature]
Checked	[signature]
Approved	[signature] Dir of Pub Works

65L-18

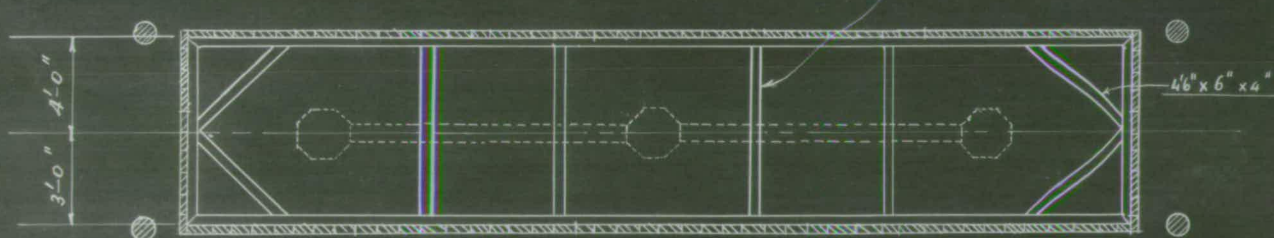


SIDE ELEVATION

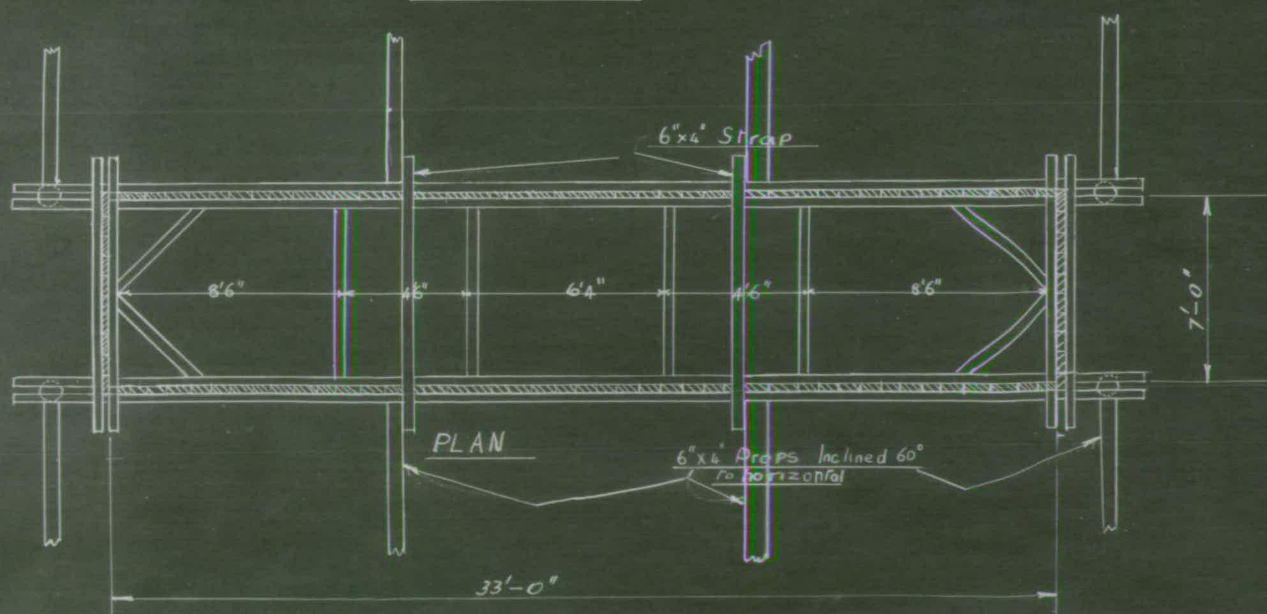
Planks to be driven 2' below bottom of Clay



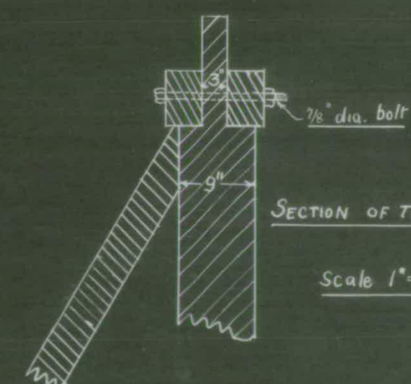
END ELEVATION



SECTION ON A.B



PLAN



SECTION OF TOP OF PILE

Scale 1" = 1'

MATERIAL LIST

SAWN HARDWOOD

SIZE	LENGTH	No.
6"x4"	14 ft	25
6"x4"	20·	7
6"x4"	16·	8
9"x3"	16·	108

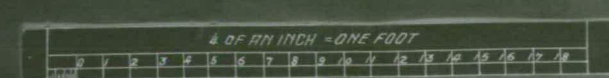
BOLTS

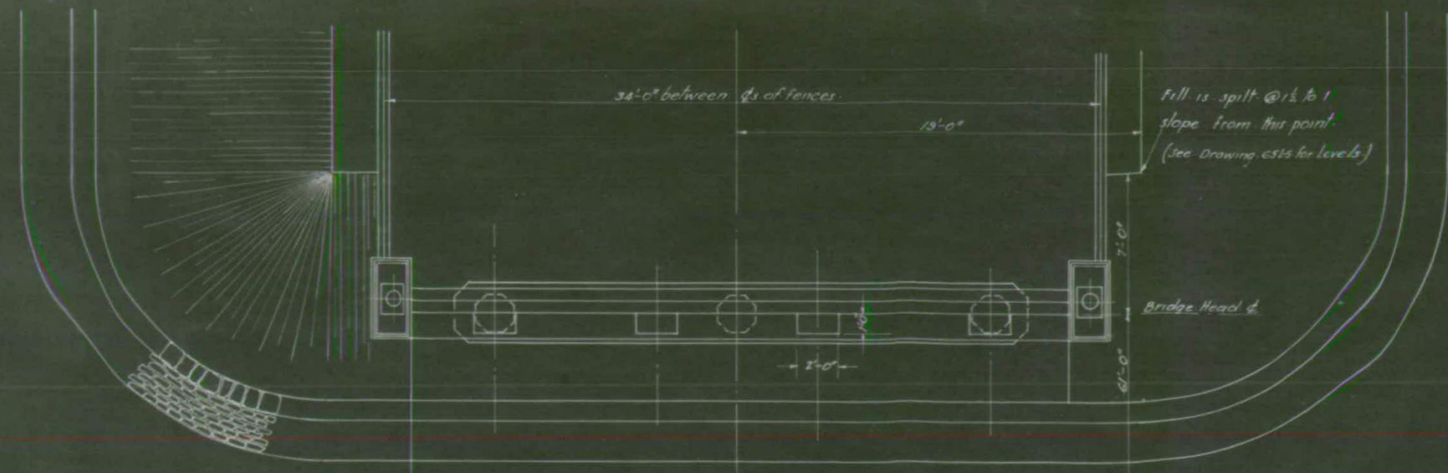
SIZE	LENGTH	No.
7/8"ϕ	14"	4
3/4"ϕ	12"	8

LEVEN BRIDGE

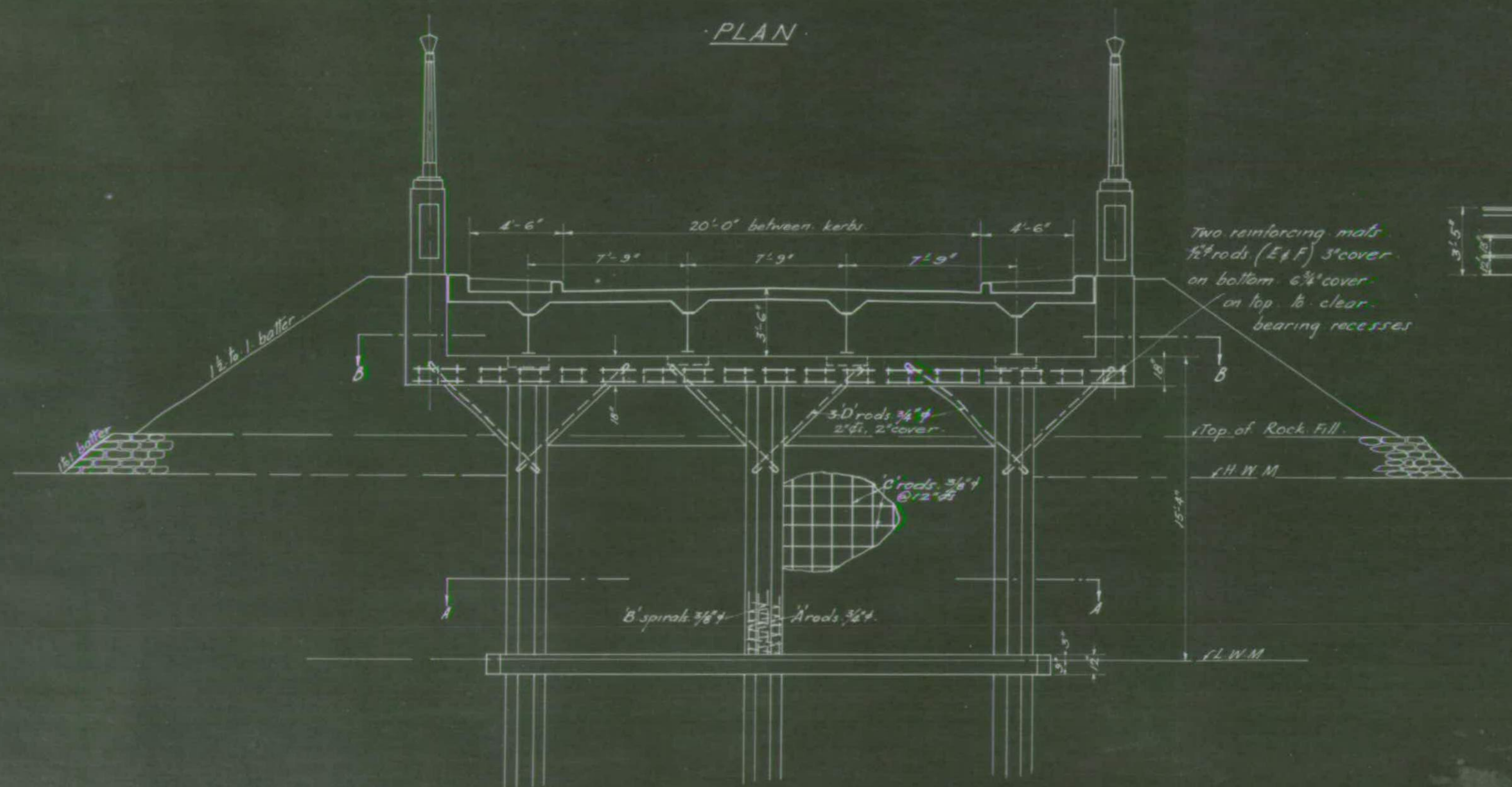
CAISSON - EASTERN ABUTMT.

SCALE 1/4" = 1 Foot	PUBLIC WORKS DEPT. TAS	
No. REVISIONS	DESIGNED [signature]	65L-19
	EXAMINED [signature] Knight	
	CHECKED [signature] Knight	
	APP.D G. W. Balsille Dir. of Pub. Wks.	





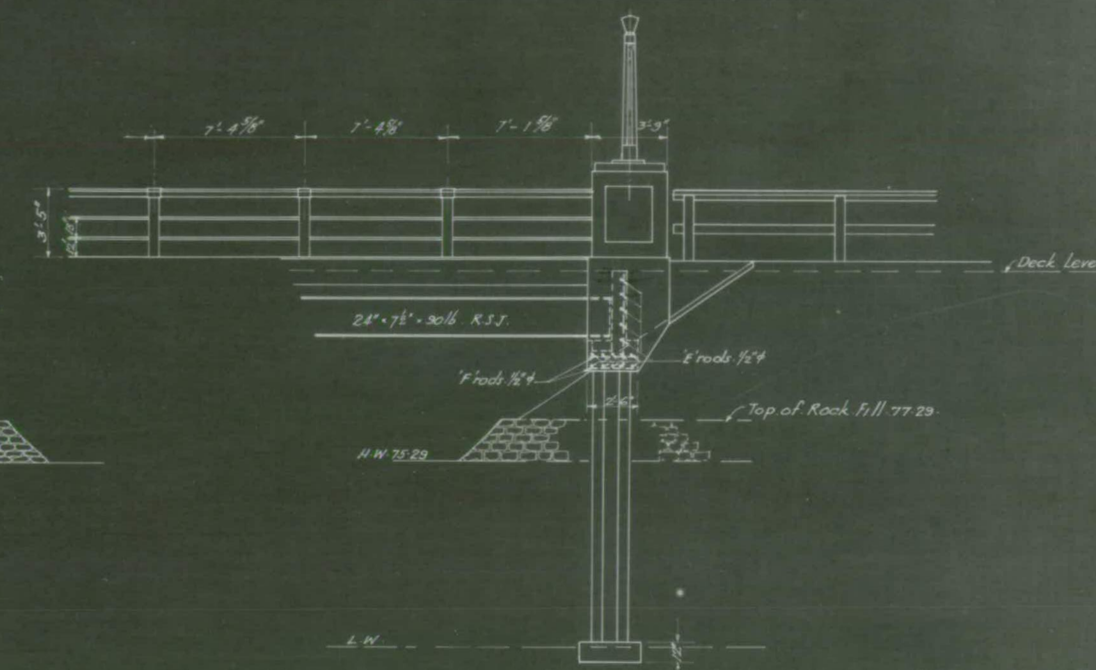
PLAN



ELEVATION

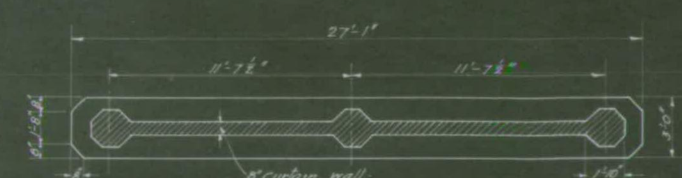
STEEL LIST

Mark	Size	Length	Nº off	Wt of Each	Total Wt	Shape	Remarks
A	¾"⌀	15'-4"	24	23.0 lbs.	782 lbs.	Straight	Verticals in Columns - See Drwg 65L-11 for detail
B	⅜"⌀	20'	18	7.5 "	135 "	Spiral	Spirals " " " " " " "
C	⅜"⌀	10'	42	3.76 "	158 "	Straight	Verticals & horizontals in Curtain wall — lower
D	¾"⌀	8'	18	12 "	216 "	Bent as shewn	In Fillets between Cap & Columns
E	½"⌀	18'	30	12 "	360 "	Straight	Longitudinal in Cap & Curtain wall — upper - See Drwg 65L-11
F	½"⌀	2'	50	1.33 "	67 "	"	Transverse " " " " "
G	½"⌀	16'	16	10.7 "	171 "	"	Top & bottom of Foundation slab (longitudinal) For detail see Drwg 65L-15
H	½"⌀	3'-9"	8	2.5 "	20 "	"	" " " " (" - by end piles) "
J	½"⌀	5'	4	3.34 "	13 "	"	" " " " (" - " central ") "
K	½"⌀	9'-3"	20	6.18 "	124 "	"	" " " " (") "
L	½"⌀	2'-10"	88	1.89 "	166 "	"	" " " " (Transverse) "
M	½"⌀	2'-3"	24	1.5 "	36 "	"	" " " " (diagonals) "
Q	⅜"⌀	10'	10	3.76 "	38 "	see detail	Stirrups to " " "
R	⅜"⌀	2'-3"	16	0.85 "	14 "	straight	Braces " " " "
S	⅜"⌀	2'-9"	4	1.05 "	4 "	"	" " " " "
N	½"⌀	14'	20	9.36 "	187 "	"	To cut Pylon reinforcement
T	½"⌀	4'-6"	31	3 "	93 "	"	Verticals in Curtain wall — upper
W	½"⌀	6'	31	4 "	124 "	4'-6"	" " " " "

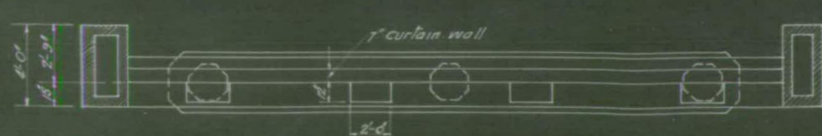


SIDE ELEVATION

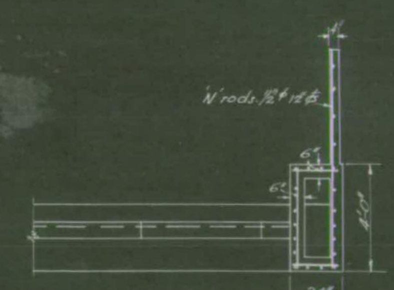
NOTE:- For details of Precast Foundation slab, see Drawing 65L-15



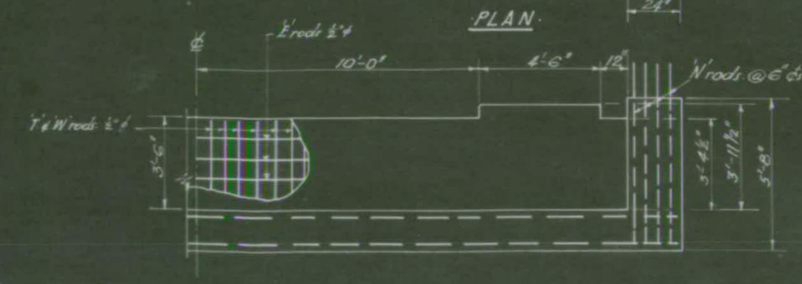
SECTION A-A



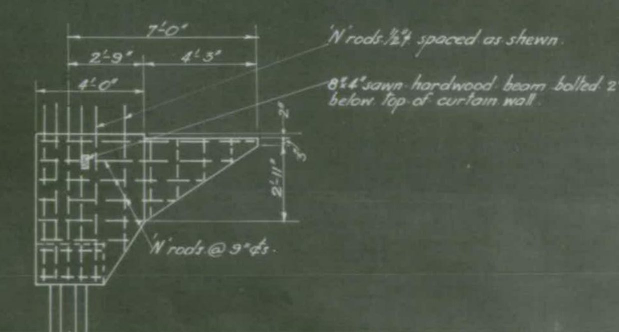
SECTION B-B



PLAN

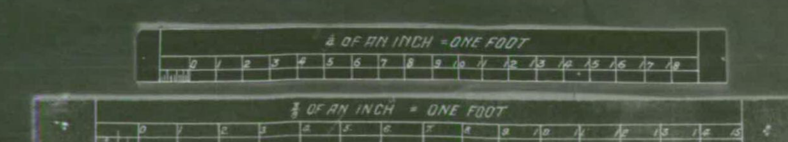


END ELEVATION



SIDE ELEVATION

DETAILS OF CURTAIN WALLS (PYLONS REMOVED)
Scale ⅛" = 1'-0"



LEVEN BRIDGE
DETAILS OF WESTERN ABUTMENT

SCALES 3/16" & ⅛" = 1'-0" — PUBLIC WORKS DEPT. TAS.

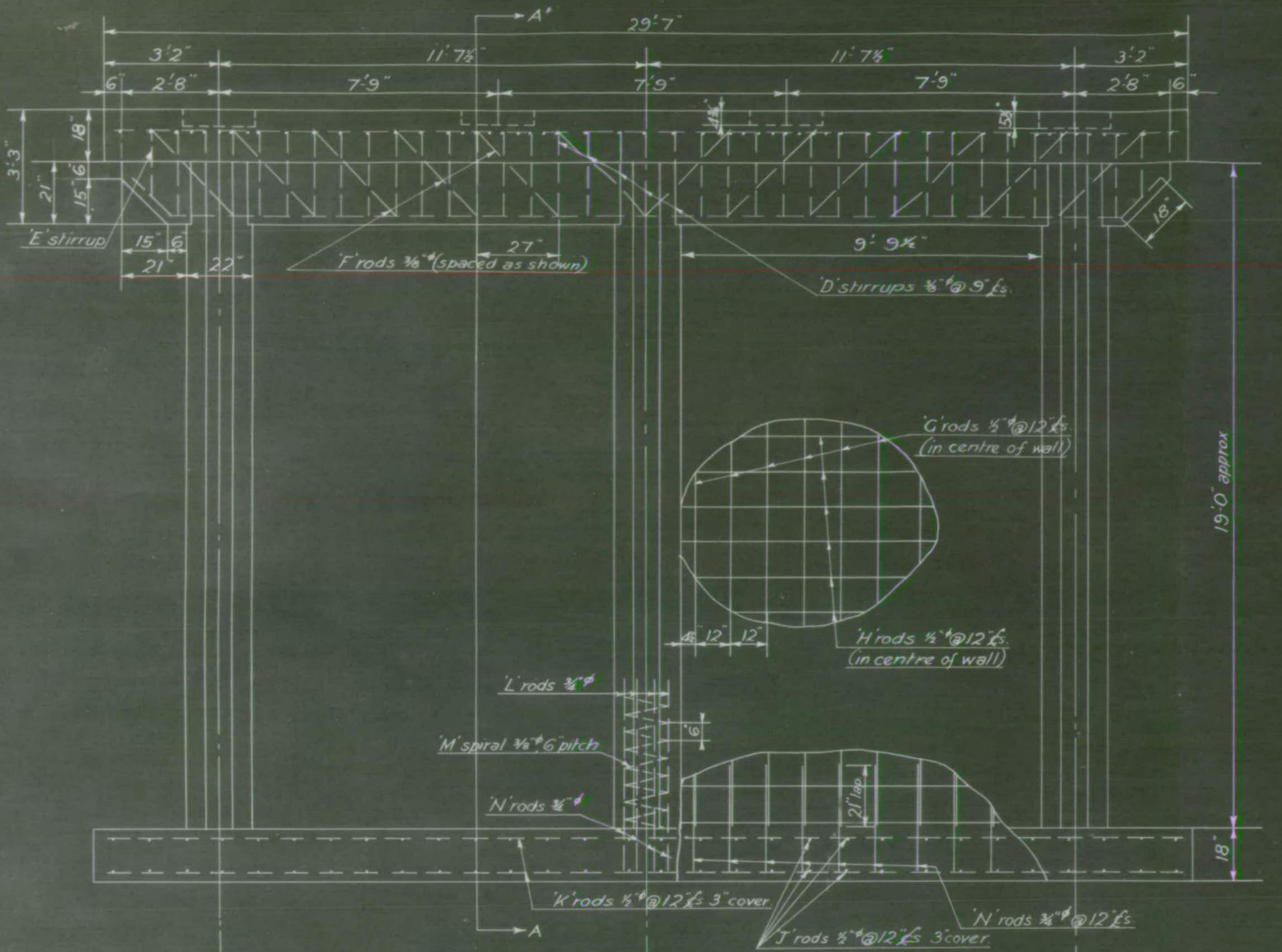
Nº	Revisions		
		Designed	G. W. Knight
		Examined	R. G. Knapp
		Checked	G. Donnelly
		Approved	G. O. Batchelor

DIR. OF PUB. WKS.

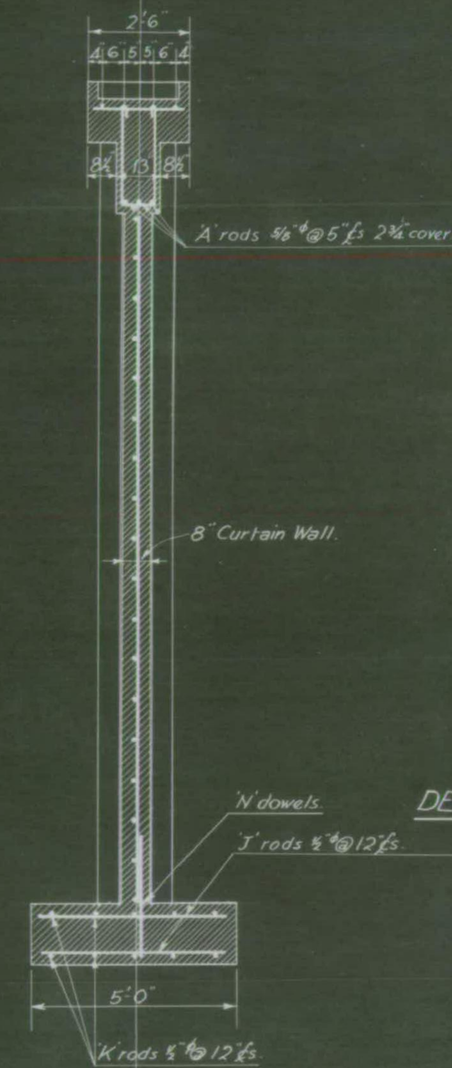
65L-20

A.S. 12.6.34

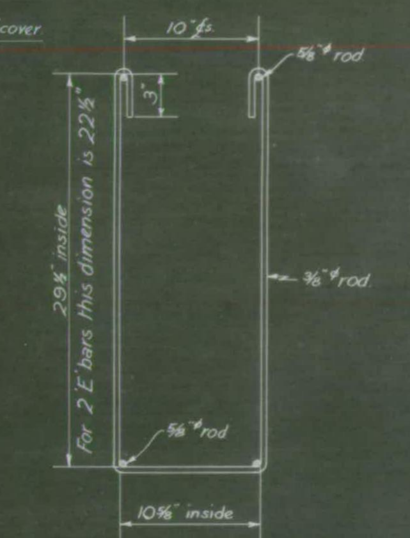
NOTE :-
For levels see drawing 65L-5



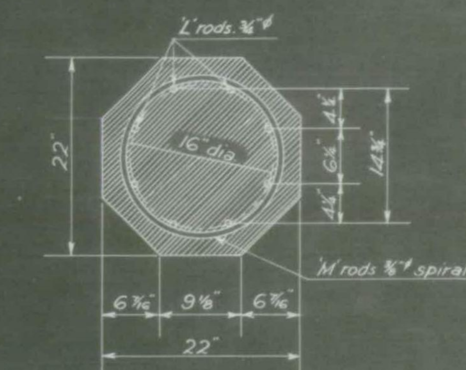
ELEVATION



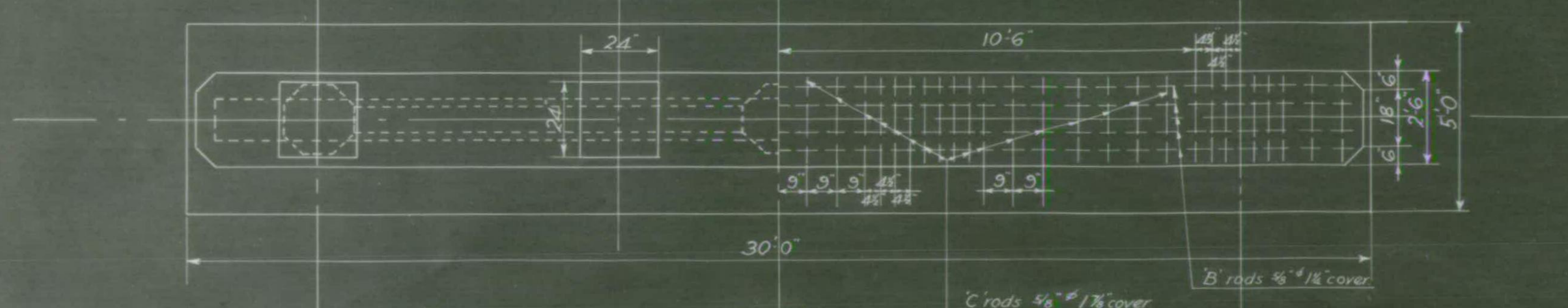
SECTION AA'



DETAIL OF CAP STIRRUP 'D'
1" = 1'
Make 35



DETAIL OF COLUMN REINFORCEMENT
1" = 1'



PLAN
3/16" = 1'

STEEL LIST

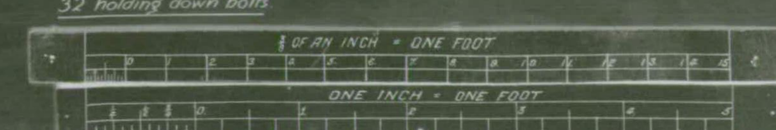
Mark	Size	Nº Off	Length	Wt of ea.	Total Wt	Shape	Remarks.
A	⅝"⌀	6	16'-0"	16·69 lbs	100 lbs	Bent as shown	Bottom of cap (longitudinal)
B	⅝"⌀	10	16'-0"	16·69 "	167 "	Straight	Top of cap (")
C	⅝"⌀	53	2'-3"	2·35 "	124 "	"	" " " (transverse)
D	⅜"⌀	35	6'-6"	2·44 "	86 "	See detail	Cap stirrups.
E	⅜"⌀	2	5'-3"	1·97 "	4 "	" "	" "
F	⅜"⌀	24	3'-6"	1·32 "	32 "	Straight	Bracing in cap.
G	½"⌀	20	17'-9"	11·85 "	237 "	"	Curtain walls (vertical)
H	½"⌀	36	9'-6"	6·35	229 "	"	" " (horizontal)
J	½"⌀	60	4'-6"	3·01	181 "	"	Foundation slab (Transverse)
K	½"⌀	20	16'-0"	10·70 "	214 "	"	" " (longitudinal)
L	¾"⌀	24	20'-0"	30·05 "	721 "	"	Columns (Vertical)
M	⅜"⌀	35	15'-0"	5·65 "	198 "	Spiral 6" pitch	"
N	¾"⌀	44	3'-0"	4·51 "	199 "	Straight	Splice rods

Total weight = 1 ton 2·2 cwt

QUANTITIES

CONCRETE :- Cap .... 6·00 cyds.
Curtain wall .... 8·35 " "
Foundation slab ..... 8·35 " "
Columns .... 5·70 " "
Total = 28·4 " "

STRENGTH :- 3000 #/□"
STEEL :- 1 ton 2·2 cwt.
32 holding down bolts.



LEVEN BRIDGE

DETAIL OF PIER Nº 1

Scales :- 3/16" & 1" = 1'	PUBLIC WORKS DEPT TAS	
Nº / Revision	Designed: A. W. Knight	65L-21
	Examined: [signature]	
	Checked: A. W. Knight	
	Approved: G. H. Balsille (for) DIR OF PUB WKS.	

LEVEN BRIDGE

WESTERN APPROACH

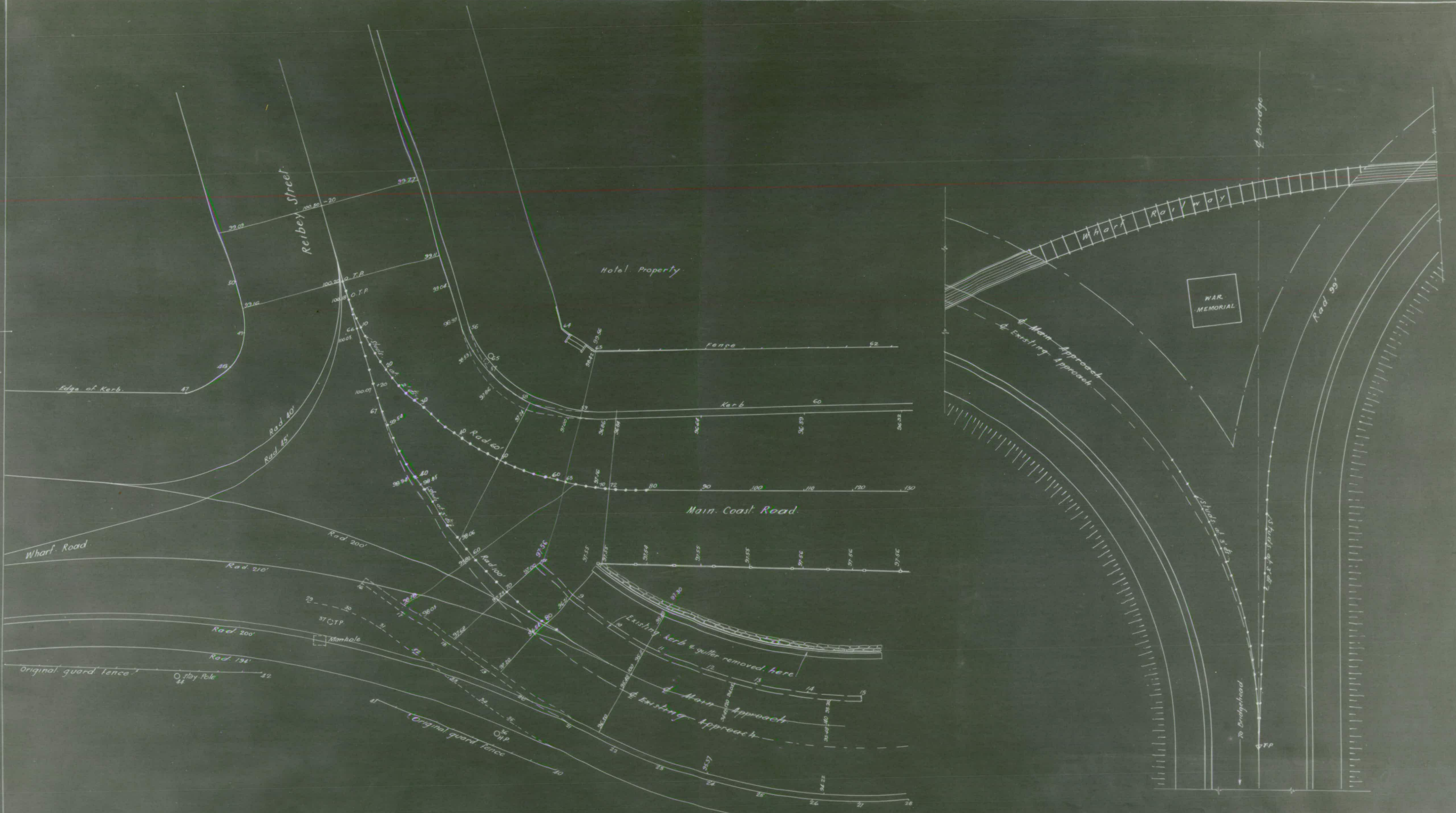
SCALE:- 20' = 1"

PUBLIC WORKS DEPT, TAS.

Nº	REVISIONS			
		Designed	Sw Knight	65L-22
		Examined	[illegible]	
		Checked	Sw Knight	
		Approved-	G. D Balsille	

DIR. of PUB. WKS.

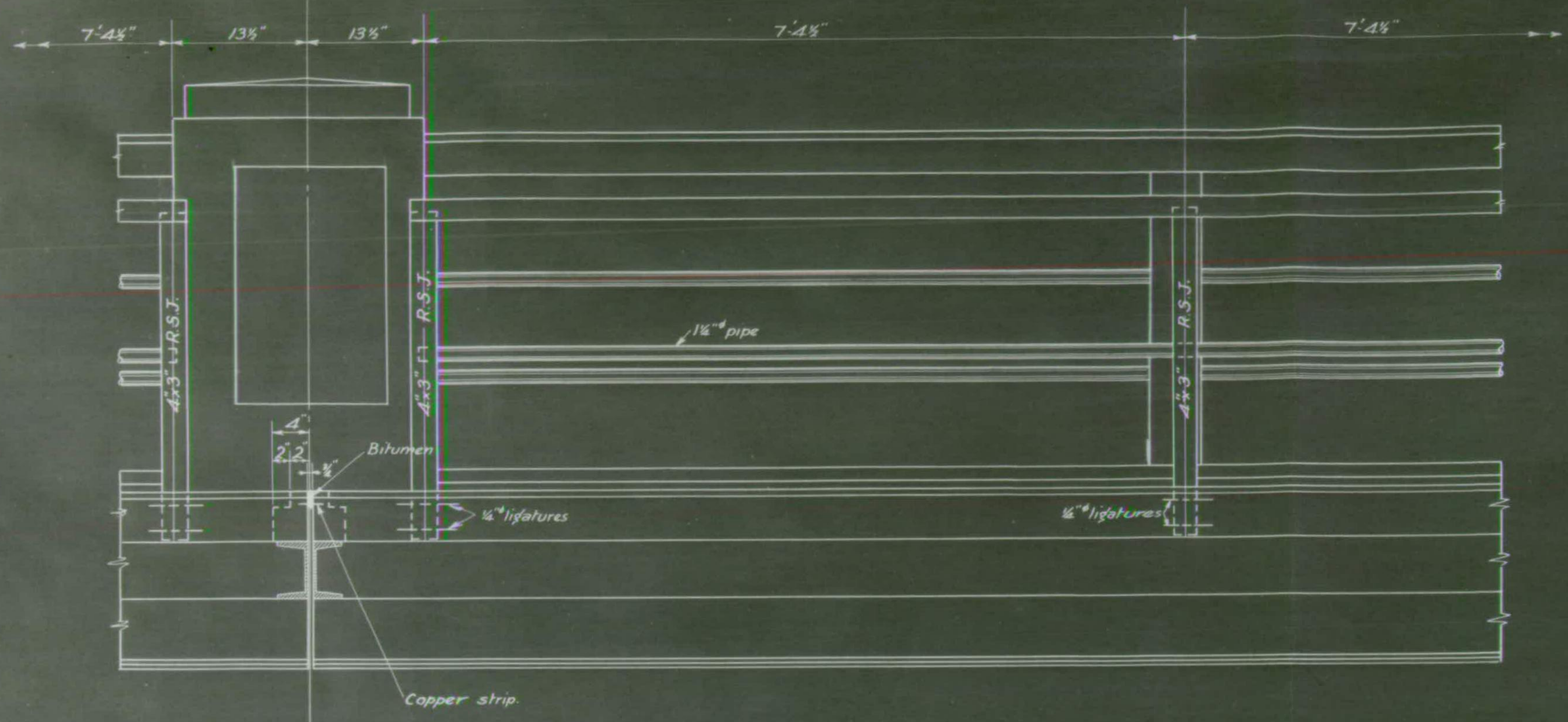
J.B. 1/6/34



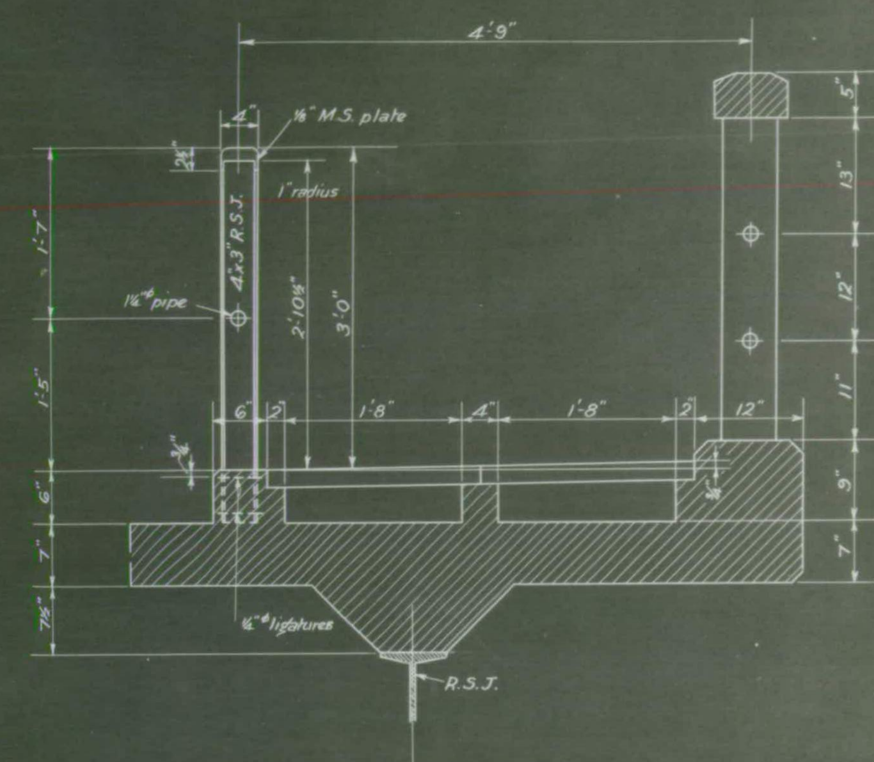
LEVEN BRIDGE
EASTERN APPROACHES

SCALE 1" = 10'-0"		PUBLIC WORKS DEPT TAS	
Nº	Revisions	Designed	
		Examined	
		Checked	
		Approved	

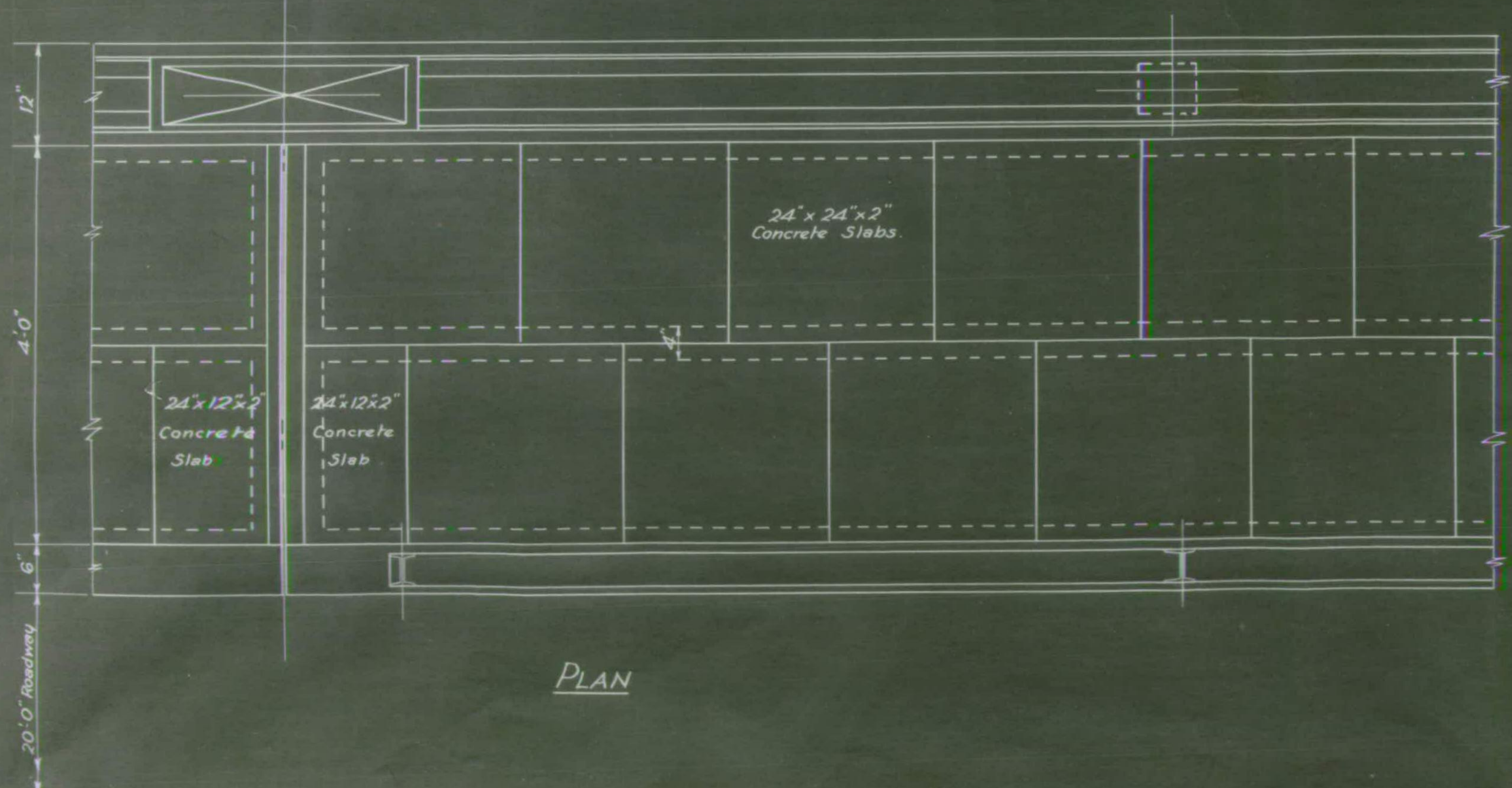
65L-23



ELEVATION



SECTION

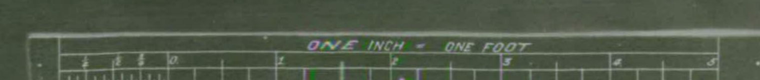


PLAN

MATERIAL LIST

Item	Size	Number
Concrete slabs	24"x24"x2"	840
" "	24"x12"x2"	28
R.S.J's.	4"x3"x10#x3'-4½"	63
1¼"ᶲ pipe		413 lin. ft.
Pressed steel caps	4"x2¼"x7'-6" long	56
Sheet copper	4"x5'-0"x20 gauge	12

NOTE :- Footway fence to be placed on upstream side only

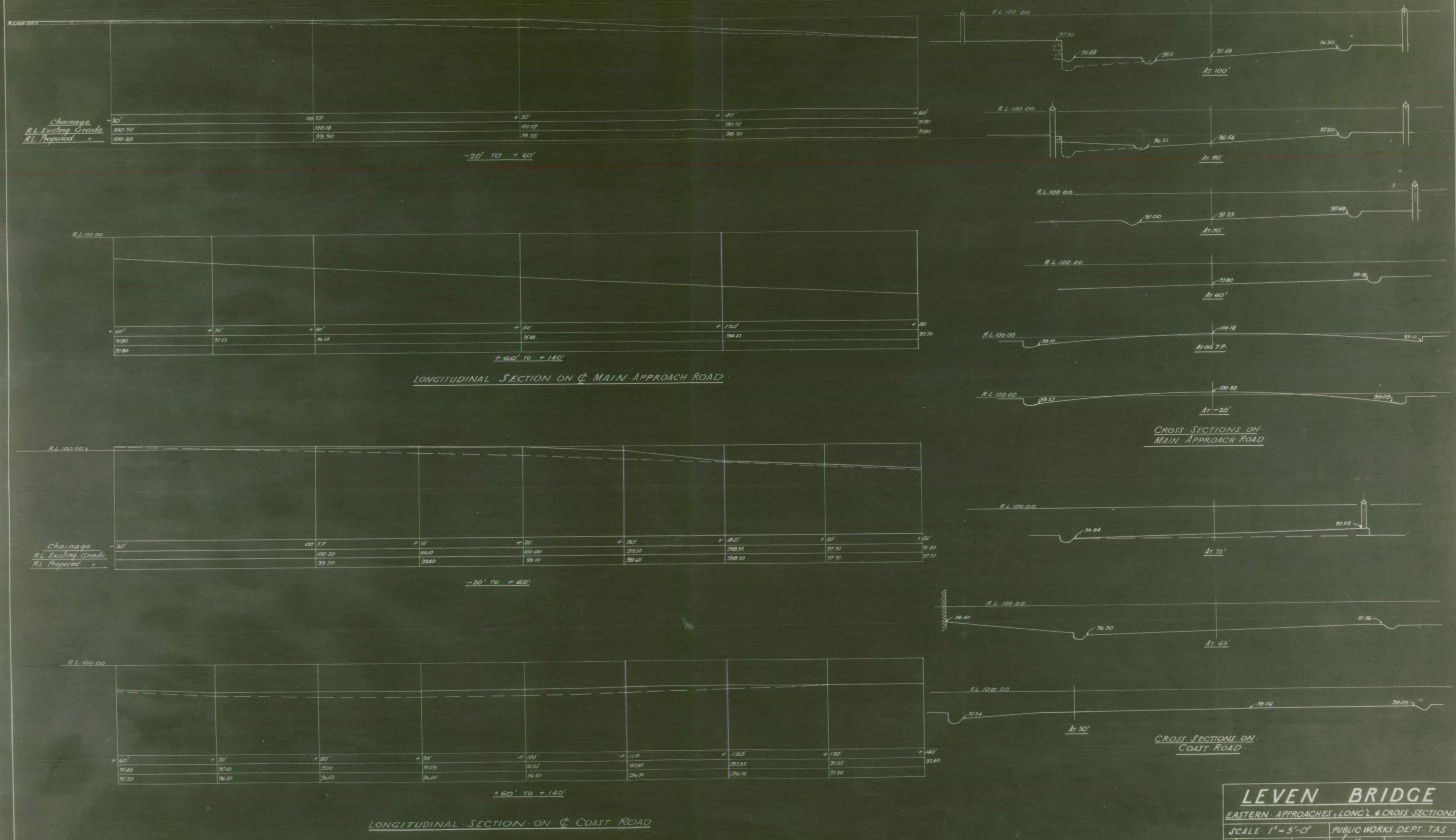


LEVEN BRIDGE

Detail of footway and guard fence.

SCALE = 1" = 1'	PUBLIC WORKS DEPT TAS.	
No. Revisions	Designed [signature]	65L-24
	Examined [signature]	
	Checked [signature]	
	Approved [signature] DIR. OF PUB. WKS.	

N.E.W. Oct '34



LEVEN BRIDGE

EASTERN APPROACHES, LONG'L & CROSS SECTIONS

SCALE 1" = 5'-0"		PUBLIC WORKS DEPT. TAS.	
Nº	Revisions	Designed	65L-25
		Examined	
		Checked	
		Approved	DIR OF PUB WKS

"THE DESIGN AND CONSTRUCTION OF COMPOSITE SLAB AND GIRDER BRIDGES WITH PARTICULAR REFERENCE TO THE LEVEN BRIDGE AT ULVERSTONE".

1. INTRODUCTION.

The theory on which this particular type of structure is based is given in the January issue of the "Journal of the Institute of Engineers" in a paper entitled "The design and construction of composite slab and girder bridges" and reference will be made to the subject matter of this paper. The paper can be regarded as part of this thesis.

Experiments on test pieces designed to investigate the possibility of reinforcing a structure consisting of a concrete deck slab supported by steel joists for shear between the top of the steel section and the concrete, were first conducted and indicated that the theoretical expectations were realised in practice A 34ft span two beam bridge with a 10ft roadway was designed and constructed for testing the type of structure on a larger scale (Commonwealth Engineer, April 1933) Later a 1/6 scale model of one span of the proposed Leven bridge was also constructed for testing purposes, but rather from the point of view of load distribution between the beams than from strength considerations.

Two, three, four and five beam highway bridges of this type have now been constructed and it is of interest to note that the structural and economic advantage to be gained is likely to lead to a very extensive use in the future of this class of construction.

The subject matter of the thesis, outlines the principles of the design and construction of composite beam bridges in general, and describes the Leven Bridge, which is a four beam bridge of this type, having 7 spans each 61 feet long. The deck of the Leven bridge carries a roadway 20ft between the kerbs, with a 4ft footway on each side. Reference is made, in describing the work, to the Journal paper and to the working drawings for the bridge.

The Leven River rises in the vicinity of Mt. Pearce about 35 miles from the coast, and flows into Bass Strait at Ulverstone on the North West Coast of Tasmania; the Coast Highway passes through the town less than a mile from the sea. The existing bridge is a timber structure 49 years old, and is in such a state of decay that it was considered to have reached the end of its useful life. Consideration was therefore given to the renewal of the structure which is a vital link in the communication system in this part of the State. A survey, of the locality, was therefore made and consideration given to five sites on which the bridge could be renewed; these are shown on Drawing 65L-2. Eventually the site marked 1.B. was selected by the Parliamentary Committee investigating the proposals and a decision made to renew the bridge with a permanent structure at an estimated cost of £13,700, exclusive of land resumption. Bore holes 66 feet apart were put down on the centre line of this site and indicated that a suitable foundation could not be obtained less than 35 feet below the river bed. A 10ft rise and fall of tide had also to be reckoned with. A number of designs were examined for the superstructure, including a three span high through truss bridge, several arrangements of welded pony trusses, and several arrangements of welded plate girders. Of these proposals a 7 span cantilever welded plate girder bridge was recommended as most suitable; the above designs were all prepared for a 18ft roadway and one 4ft footway. Later, however, an examination of the composite slab and girder type showed a marked saving over the latter bridge, but as the necessary funds had already been voted by Parliament, it was decided to increase the roadway width to 20feet, and add another footway, as it was estimated that the additional work could be undertaken for the difference in costs of the two types.

2. DESIGN.

SUBSTRUCTURE.

An examination of 65L-9 will show the varigated nature of the material forming the river bed and shows the difficulty of obtaining a type of substructure suitable for the full width of the river. On the eastern side serpentine carrying hard kernels of the original basalt was out-cropping, and extended to a considerable depth. At the eastern abutment four to five feet of clay and mud covered the serpentine, but at No.1 pier the rock was bare. The serpentine disappeared on the western side of No.2 pier, and in midstream the formation was mud and sand followed by clay, more sand, and finally the micacious schist at a depth of about 36 feet. Towards the western bank the blue clay disappeared altogether, and the material overlying the rock was almost wholly sand. The sand however was mixed with river shingle, which in some cases reached the dimensions of boulders, the shingle was more pronounced in some parts than in others, and in places occured in layers several feet thick. Near the western abutment the sand and shingle was cemented with a rich yellow-coloured clay which, however, responded easily to the action of the water jet.

The eastern abutment was designed as an ordinary spill abutment, the concrete foundation being spread and founded on the serpentine.

The main columns, as shown in 65L-11, are octagonal in cross section, 22" between the flats supported by a slab 7ft X 33ft, and connected at the top by a cross beam 30" X 15". To avoid any chance of damage by settlement of the earth filling, which is 9 feet high at the abutment, the pylons are directly connected to this cap and carried on the main foundation. The base of the slab is 7 feet below ground level and 12 feet below high water level.

The first pier is also supported by a spread foundation placed directly on the serpentine. The concrete slab 26ft X 5ft X 2ft carries three octagonal columns of the same dimensions as those of the abutment, but the columns are connected by an 8" curtain wall and a cap 26ft X 30" X 15"; the details of construction are given in 65L-9.

The remainder of the piers and the western abutment are supported by 22" octagonal concrete piles. There are three piles in each pier, connected at low water level by a concrete waling and above this waling by an 8" concrete curtain wall capped in the same manner as No 1. pier.

The longest piles were 65 feet in length and weighed 13 tons. The manufacture, handling and driving of these piles constitued the major portion of the substructure work. Various other types of substructure were investigated, the form finally adopted being regarded as the most economical for the site conditions indicated by the survey borings.

DESIGN OF CONCRETE PILES.

Design Stresses.

A compressive stress in the concrete of 33% of the ultimate strength as indicated by test blocks was allowed. The handling and driving stresses are usually the most severe for piles, it is only with piles having a considerable unsupported length above ground, that the working stresses require investigation on account of long column action; in such cases the unsupported length can usually be reduced by suitable bracing. The concrete mix was designed to give an ultimate strength of 5,000 lbs. per square inch, at 28 days, which gives an allowable working stress of 1,500 lbs. per square inch. The average strength of test blocks for the piles was actually as follows.

Mix	Age	Average Strength	No. of blocks
$1:1\frac{3}{4}:3\frac{1}{2}$	21 days	5,100 lbs. sq. inch	16
"	28 "	5,247 " " "	16
"	66 "	5,808 " " "	16

66 days was the average age of the piles at driving.

Modular Ratio.

Since strain is not proportional to stress for concrete, the value of the modular ratio depends on the stress at which the slope of the stress strain curve is measured. The L.C.C. regulations (1915) give the formula -

$$n = \frac{9,000}{c} \quad \text{where } c = \text{allowable compressive stress.}$$

Dr. Faber (Proc. Inst. C.E.- vol. 225) suggests that Young's Modulas for concrete is approximately equal ultimate strength X 1,000. For 5,000 lb concrete and E_s = 30,000,000 n by the above method is 6 in both cases. This value was therefore assumed in calculating the modulas of the pile section.

Tension Stress.

There is a diversity of opinion as to the allowable tension stresses for design of reinforced concrete piles. Usually a steel stress up to 24,000 lbs. per sq. inch due to handling loads is allowed, but in other works a maximum tension stress of 100 lbs. per sq. inch in the concrete is specified which for $n = 6$ limits the steel stress to 600 lbs. per sq. inch, a marked variation. For this particular work the latter figure was adhered to for the reasons given later.

The stresses for which a pile must be designed are as follows.

1. Bending and shear stresses due to handling.
2. Compression, shear and possibly buckling stresses due to driving.
3. Compression stress under working load.

The relative importance of these items depends to a large extent on the nature of the material through which the pile is to be driven. If this material provides any serious resistance to the pile in the early stages of the driving it is safe to say that the stresses for item 2. will be the most severe, and, therefore, the ones which should be given most consideration in design. Unfortunately the values of these stresses are difficult to determine, and this is apparently the reason for the development of various empirical rules for pile design. The formula commonly quoted for determining the size of the pile is the column formula.

$$P = A_c.f_c. + A_s.n.f_c.$$

where
P = safe load in lbs.
A_c = net core area of concrete in square inches.
A_s = area of main reinforcing steel in square inches.
f_c = allowable compressive stress of concrete.
n = modulus of elasticity $\frac{\text{steel}}{\text{concrete}}$.

Use was made of this formula together with the reduction formula

$$r = 1.75 - \frac{L}{20d.}$$

where
L = unsupported length in feet.
d = least dimension of the core section in inches.
r = the coefficient by which f_c in the short column formula must be reduced to give the safe stress for long columns.

The value of L can only be fixed by making allowance for the supporting effect of the material into which the pile is driven and of the curtain wall and cap of the pier. It should be noted that P, the safe load referred to in the formula is the maximum load which the pile can reasonably be expected to carry without damage; as it is necessary to allow a factor of safety on this load, against settlement of the pile, of from 2 to 5 according to the nature of the ground, the value of P used in the formula should be 2 to 5 times the working load. Allowing 15% for impact on live loads, the total working load per pile is 65 tons. The test bores indicated that a rock foundation could be reached, it was therefore considered safe to adopt a factor of safety of 2, for which the concrete stress on the pile section adopted is approximately 730 lbs. per square inch. The Leven Bridge is only a short distance from the sea, it was therefore necessary to provide adequate concrete cover on the steel rods to protect the steel from the effect of sea water. A minimum of $2\frac{3}{4}$" on the main rods was adopted for this. For the above section the allowable unsupported length is 25 feet and as this is not exceeded, consideration of the pile as a long column is unnecessary.

Observations made while driving piles for the Leven Bridge, and on other piling works, suggests that the concrete cover of the main rods has an important bearing on the allowable tension steel stresses caused by lifting and pitching the piles. It is obvious that concrete must fail if the tension stress exceeds its ultimate value, which, for first class concrete, is about 500 lbs. per square inch, but the result of this failure may not be apparent at the tension face of the concrete if the cracks are very small, owing to their distribution by the reinforcing effect of steel rods in close proximity to the concrete surface. If, however, the rods are some distance from the surface their effect in distrubuting the cracks in the concrete at the surface is negligable, with the result that one large crack of perhaps a serious nature developes in the place of many small ones that are not of serious consequence. An appreciation of this point is necessary in fixing the steel stresses by which the handling system is regulated.

In this case a maximum tension stress of 100 lbs per sq. inch in the concrete, and consequently n times this in the steel, was selected and a method of handling devised to keep the stress within this limit. In computing the modulus of the pile section, the concrete was taken to be effective in tension giving a value of Z = 2554. A stiff back, which consisted of a 24" X $7\frac{1}{2}$" X 90lb R.S.J. 61ft long, was strapped to the pile to give additional support, and in calculating stresses the bending moments were assumed to be distributed in proportion to the moments of inertia of the two sections. Various arrangements of lifting gear were investigated and finally that shewn in the sketch was adopted. For some of the shorter piles the stiffback was dispensed with and the same gear used on the pile alone.

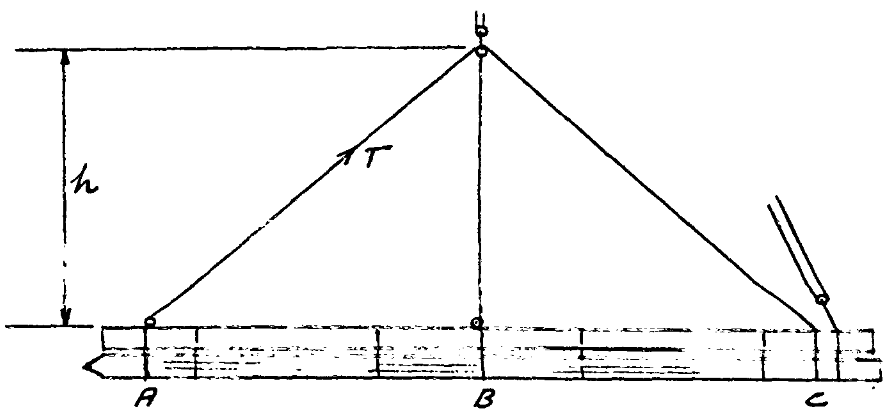


FIGURE 1.

For a given height h, which was actually 40 feet, the uniform tension T in the bridle can be resolved to find the vertical forces at A.B. and C. the sum of which equals the weight of the pile and stiff back. The distribution of bending moment can then be simply obtained and the position of A. and C. fixed by trial and error to give an equal positive and negative moment in the pile. Immediately the pile is swung to an inclined position it becomes unstable and tends to hang vertically, so that a guy is necessary to control this tendency once the pile is moved from the horizontal position by the head gear.

The longitudinal reinforcing must be designed to carry tension stresses in the pile caused by handling or driving. In this case the tension stresses due to handling were very low and the latter factor became of major importance. The quantity of steel for this purpose has been determined by experience with concrete piles of various design, and a minimum of 2% of the total area is recognised as a satisfactory amount. In this case it amounts to 2.45%, and consists of eight 1-1/8" diam. round rods spaced at the corners of the octagon and bent in to fit the form of the pile at the shoe and butt welded to the cast steel shoe, as shown on sheet 65L-14.

Generally speaking reinforcing is not required to resist shear stresses in the pile due to bending under its own weight, it is however necessary to resist impact stresses, to give resilience, to resist the bursting pressure of the concrete and to prevent buckling of the main rode; there is nothing gained by increasing the volume of the lateral reinforcement beyond that required to resist these forces. The lateral reinforcement is fixed by regulations in an empircal manner, but an application of Navier's theory* and the value of shear stress, which has been derived from it, enables the volume of the lateral reinforcing necessary to resist the bursting pressure of the concrete - its most important function - to be calculated in terms of the working compression stress.

Considering the shearing plane of a short concrete column to be inclined at 60° to the plane of loading, assume such a square column of side 'a' and height $\sqrt{3}a$ to be part of the core of a pile. If the maximum working stress is c.lbs per sq. inch then for $\theta = 60^\circ$ and $\phi = 30^\circ$ the working shear stress = .30 c approx.

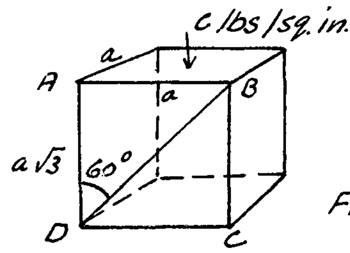


Figure II

Tangential stress on plane

$$BD = c \sin\theta \cos\theta = c\,\frac{\sqrt{3}}{4} = .433\ c.$$

There is therefore a shearing stress on plane BD of .133c greater than the material can sustain with a factor of safety of 3. This produces a tangential force = $.133\ c\ a^2\sqrt{3}$ lbs.

Resolving this force vertically and horizontally, the vertical component can be carried as a compression in the concrete; the horizontal component must be resisted by the hoop reinforcing if the factor of safety against shear within the core is not to be reduced below 3.

The horizontal component of the slipping force

$$= 0.133\ c\ a^2 \text{ lbs.}$$

Assume the links to form a square instead of an octogan, the side of the square being the same as the distance across the flats of the octogan. For a column hooped with these square links there are links in two planes to support the above force, therefore the required total sectional area of links on one plane

$$= \frac{.133\ c\ a^2}{2 \times 18{,}000} \text{ sq. inches}$$

But the total length of links = 4 a inches
therefore volume of links $= \dfrac{4\ c\ a^3 \times .133}{2 \times 18{,}000}$

The volume of the core ABCD = $\sqrt{3}\,a^3$ cu. inches.
therefore volume of links as a percentage of the core

$$= \frac{4\ c\ a^3 \times .133 \times 100}{2 \times 18{,}000 \times \sqrt{3}\,a^3}$$

$$= .00085\ c.$$

For c = 730 lbs per sq. inch the percentage is .62% and the spacing of $\frac{3}{8}$" diam. links to give this percentage for the pile is approximately 4.5". By this method an indication is given of the quantity of hoop reinforcing required, but the actual quantity used was fixed after an inspection of various designs for concrete piles which had proved satisfactory in practice. The hoop reinforcing is wrapped round the main rods in the form of a spiral, the spacing being decreased at the head and the shoe, to allow for the higher stresses at these points due to the effects of driving. At intervals of 4 feet bridles are located; these

* Note. Navier's theory proves that the tangential stress on the plane of rupture of a brittle compression specimen is compounded of the shearing stress plus a frictio[n] stress, the latter depending on the angle of internal friction of the material ϕ, and that the angle of the plane of rupture with the vertical axis is $\theta = 45^\circ + \phi/2$

are required to facilitate fabrication and also serve the same function as the hoop reinforcing. The details of the hoop reinforcing and other features of the pile design are shown on drawing 65L - 14.

The greater part of the material through which the piles were to be driven was of a sandy nature and this suggested that the water jet might be used to advantage. The water jet is eminently suitable for sinking piles in clean sand, a material which affords considerable resistance to penetration by piles under the hammer alone, its efficiency, however, is very much reduced if the sand contains river shingle or layers of clay which will block the jet. The difficulties due to the presence of shingle can be overcome by increasing the volume and pressure of the flow of water through the jet, in one instance piles penetrated through a rock filling twenty feet deep by this means. Under normal circumstances a volume of 10,000 gallons per hour at 150 lbs. per sq. inch pressure is sufficient to facilitate the penetration of the pile provided the material is in any way suitable for the method, and this quantity was adopted in this case. Sometimes the water is carried through the centre of the pile and through a nozzle formed in the shoe, or as an alternative two jets can be used externally and operated one each side of the pile. The chief factor claimed in favour of this arrangement is that a tendency for the pile to run at the shoe can be corrected by breaking up the material at the shoe of the pile with the hand jet. While this is to some extent true, any movement of the pile due to this process will generally jamb the hand jet between the pile and the material obstructing its path, and necessitate the use of a winch to withdraw it. In mud or sand, free from shingle, either arrangement is satisfactory. In designing the piles for the Leven Bridge it was recognised that the driving would be difficult and provision was therefore made for the centrel jet pipe, which could be used in conjunction with external jets if necessary. It is $2\frac{1}{2}$" diameter with a right angle bend 5 feet from the head of the pile and connected to a nozzle of $\frac{3}{4}$" diam. formed in the cast steel shoe. Although this arrangement has been used to advantage on other works it did not give good results at the Leven, and after some experimenting the return outlets were blocked up and the central outlet increased to $1\frac{1}{2}$" diam. This decreased the pressure to some extent but where the material was really suitable for jetting, the piles would sink steadily under their own weight.

P I E R S.

Excepting No. 1 pier which is supported by a slab foundation, all the piers are supported by piles. As the effect of the sea water on steel reinforcing is particularly severe, in the Leven River, the design of the piers was arranged to avoid breaking into the piles below high water level for the purpose of connecting the piles by a curtain wall. A concrete waling 27 feet x 3 feet x 9" thick was precast with holes to fit the three piles and slipped down over the piles to a point below low water level. It was supported at this level by timber clamps fixed at the correct level on the piles by a diver. The 8" curtain wall was then cast upwards from the waling, monolithic with the cross beam The cross beam, details of which are given on 65L - 15, is designed to distribute the loads from the four main members of the superstructure to the three piles; provision is made for recesses to carry bearing plates, the levels of which are conveniently adjusted by means of the lower nuts carried by the holding down bolts - see 65L - 12.

ABUTMENTS.

The design of the abutment calls for little comment. As stated previously the eastern abutment - see 65L-11 - is of the spill type and is supported by a spread foundation. Account was taken of the end reaction from the first span and of the pressure of the filling on the curtain wall in examining the stability of the structure. A feature of the design is the method of supporting the pylons from the main abutment foundation in order to avoid any difficulties due to settlement of the filling. The western abutment is similar except for the fact that it is supported by three piles of the same dimensions as those used for the piers. The piles are connected by a concrete waling and curtain wall which reaches to high water level. Above this the design is the same as for the eastern abutment.

A timber retaining wall had previously been erected on the western bank of the river apposite the wharfs and it was decided to continue this wall round the face of the western bridge abutment and reclaim the area enclosed. A two-fold purpose was thus served, a foundation for the western approach was provided, and the scour in the river increased. Continuous dredging is necessary to remove sand deposited in the channel by tidal water and thus maintain the required depth of water for ships proceeding to the wharf immediately below the bridge, and this fact required consideration in arriving at the most suitable form of abutment. The pressure of the filling on the sides of the curtain wall of the abutment also provided longitudinal stability not only to the abutment but to the bridge as a whole.

SUPERSTRUCTURE.

The superstructure provides a roadway 20 feet wide and two footpaths each 4 feet wide, the overall width is 31 feet. It is on a slight grade, being 21" lower at the western end, which improves the approach grade to this end of the bridge. The bridge has a parabolic camber with a mid ordinate of 12", the deck is also cambered transversely to facilitate drainage. As stated previously various types of superstructure were examined and a decision made in favour of the composite beam type which showed a marked saving over other arrangements. The loading adopted for design purposes was the standard used by the Department for a bridge providing two traffic lanes and consists of a crusher train of a total weight of 34½ tons in one lane and a 10 ton motor truck in the other. The loads were not taken to be in the centre of the respective traffic lanes but were placed in the position to give the maximum reaction to any one of the supporting members and this reaction increased by 15% to allow for impact effect. The distribution of live loads to beams carrying a concrete deck slab of considerable stiffness presents a problem of which very little information is available and in the interests of accuracy and economy an effort was made to analyse this distributing effect of the slab. An account of this work will be found in the paper published in the Journal, and the calculation of the stresses in the members of the superstructure of the Leven Bridge is given as an example of the method of procedure for the design of composite slab and girder structures. In arriving at a suitable thickness of deck slab and size of steel member it will be found best to select what appears to be a suitable section, a simple matter after some experience with these designs and then take out the stresses due to the dead and live loads. The section can then be adjusted to give the required working stress in the steel and concrete. This method was adopted in arriving at the dimensions of themembers of the superstructure for the Leven Bridge and for which the design calculations are given in the paper referred to. As this section of the work is covered in the paper, to which reference should be made, it will not be repeated here. In conjunction with the work on distribution described in the paper for which a 1/6th scale model of one span of the bridge was constructed and tested it is interesting to refer to the results of a similar test to which the first span of the Leven Bridge was subjected the results of which are given in Appendix 11.

By assuming that the stresses in the steel are uniform throughout the section, a favourable condition for maximum loading, Prof. Burn has derived the propping moment necessary for this stress distribution and shown that the stresses in the concrete and steel can be expressed by simple formulae which are independent of the propping system. The method provides a convenient means for designing a suitable section quickly. The formulae are obtained as follows.

The properties of the section may be calculated by replacing the concrete by its equivalent steel area. In figure 111.

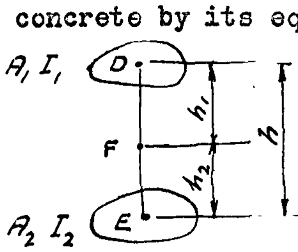


Figure III

D and E are the centroids of areas A_1 and A_2 with individual moments of inertia l_1 and l_2. If F is the centroid of the whole area then

$$h_1 = \frac{A_2 h}{A_1 + A_2} \qquad h_2 = \frac{A_1 h}{A_1 + A_2}$$

The moment of inertia of the whole is

$$\begin{aligned} 1 &= l_1 + l_2 + A_1 h_1{}^2 + A_2 h_2{}^2 \\ &= l_1 + l_2 + \frac{A_1 A_2 h^2}{A_1 + A_2} \end{aligned}$$

Suppose the moments at some point in the span (generally the centre) are:-

M_D due to dead loads)
M_F due to formwork) All taken as positive.
M_L due to live loads)
M_P due to propping system)

Then the steel alone has to carry a bending moment $M_D + M_F - M_P$ which will generally be negative as the propping moment exceeds the others.

If V_1 and V_2 are the fibre distances to the top and bottom of the steel, the former negative and the latter positive and I_s the moment of inertia of the joist the tensile stresses are:-

At top of steel $(M_D+M_F-M_P)\dfrac{V_1}{I_s}$ At bottom of steel $(M_D+M_F-M_P)\dfrac{V_2}{I_s}$

Generally the former is tension (positive) and the latter compression (negative). When the concrete is set the props and formwork are removed, which is in effect equivalent to applying a positive moment M_P-M_F to the composite section. If the live load is then applied, the total bending moment acting on the section is $M_P - M_F+M_L$.

If the steel fibre distances are V_3 and V_4 to the top and bottom and I_c is the moment of inertia of the composite section, the tensile stresses are

At top of steel $(M_P - M_F + M_L)\dfrac{V_3}{I_c}$

At bottom of steel $(M_P-M_F+M_L)\dfrac{V_4}{I_c}$

Adding the stresses before and after propping the total steel stresses are

At top of steel $(M_D+M_F-M_P)\dfrac{V_1}{I_s}+(M_P - M_F+M_L)\dfrac{V_3}{I_c}$

At bottom of steel $(M_D+M_F-M_P)\dfrac{V_2}{I_s}+(M_P- M_F+M_L)\dfrac{V_4}{I_c}$

By equating the above expressions the propping moment to give equal stresses throughout the steel can be found

$$(M_P-M_F)\left(\frac{V_2-V_1}{I_s}-\frac{V_4-V_3}{I_c}\right)=M_D\left(\frac{V_2-V_1}{I_s}\right)+M_L\left(\frac{V_4-V_3}{I_c}\right)$$

But $V_2 - V_1 = V_4-V_3 = d$ the depth of the steel

$$\therefore\ (M_P-M_F)\left(\frac{1}{I_s}-\frac{1}{I_c}\right)=\frac{M_D}{I_s}+\frac{M_L}{I_c}$$

Giving $M_P = M_F + \dfrac{M_D I_c + M_L I_s}{I_c - I_s}$

Substituting $M_P - M_F$ from this expression in either of the expressions above gives the final steel stresses when equalised.

$$=\frac{M_D V_2}{I_s}+\frac{M_L V_4}{I_c}+\frac{M_D I_c+M_L I_s}{I_c-I_s}\cdot\left\{\frac{V_4}{I_c}-\frac{V_2}{I_s}\right\}$$

On reduction of this expression

$$fs = (M_D+M_L)\cdot\frac{V_4-V_2}{I_c-I_s}$$ ✓

The final concrete stresses are those resulting form the moment $M_P-M_F+M_L$

applied to the composite section. If V_5 and V_6 (generally both negative) are the fibre distances to the top and bottom of the slab, the concrete stresses are

top of slab $(M_P - M_F + M_L)\dfrac{V_5}{n \cdot I_c}$

bottom " $(M_P - M_F + M_L)\dfrac{V_6}{n \cdot I_c}$

substituting in these for the value of $M_P - M_F$ required to equalise the steel stresses

$$fc\ (top) = \frac{(M_D + M_L)\ V_5}{n\ (I_c - I_s)}$$

$$fs\ (bottom) = \frac{(M_D + M_L)\ V_6}{n(I_c - I_s)}$$

Although the actual shear force between the steel and concrete is given in the Journal also the bending moment in the deck slab, the actual nature of the reinforcing is not commented on. The shear reinforcing is shown on 65L-17 and consists of square reinforcing rods bent into the shape of hooks as illustrated and electrically welded to the top flange of the steel joist. The distribution of shear force S along the longitudinal member is shown diagramatically in Fig. 20C, and it is shown that the horizontal shear force at the top of the steel section is for this section .0317S. The force for which the stirrups must be designed is found in this way and the usual practice of bending the stirrups up at an angle of 45° in two directions and designing them to carry this horizontal force as a tension stress on the cross sectional area was followed. $\frac{1}{2}$" and $\frac{5}{8}$" square stirrups were used and the spacing varied to provide the necessary reinforcement.

The 24 x $7\frac{1}{2}$" R.S.Joists required splicing to make the length of 61ft. This splice was designed as a butt weld the webs and flanges being bevelled before welding and cover plates applied in the web to give an excess strength of 25% at the splice over the strength of the joist itself. These plates were spaced to distribute as far as possible the stresses in the web due to the attachment of the plate to the webs. More recent information indicates the importance of shaping cover plates to reduce fatigue stresses due to alternating loads, experiments having shown that the strength of a member under this class of loading may even be reduced by attaching cover plates of improper design. The cover plate on the lower flange is attached by a continuous weld designed to prevent the effect of weathering and which gives ample strength; in this connection it is of interest to note that intermittent welding is subject to far higher fatigue stresses under alternating loading than the continuous weld run and for this reason should be avoided. The length of this cover plate was determined by examining the stresses on the section without the cover plate at points some distance from the centre line. Provision is made for expansion and contraction due to temperature changes in each span. On each pier there are four rocker bearings carrying the ends of the joists of the next. The design of the bearings is shown on 65L-12 and 65L-13 The reaction at each bearing is $26\frac{1}{4}$ tons and is transferred to a reinforcing mat of $\frac{1}{2}$" diam. reinforcing rods cast in the concrete of the pier; a 10" length of a standard 100 lb. rail with welded stiffeners was used for the fixed bearing, a one inch diameter pin serving to fix the joist to the bearing. The size of the rocker of the movable bearing was obtained by using the formula P =600 D where P = load in lbs per lineal inch of rocker, D = diameter of roller required. The rocker and also the shoe and bearing plate are of cast steel, the height of the bearing plates could be conveniently and accurately fixed by adjusting the lower set of nuts on the holding down bolts before grouting in the plates and screwing down the top set of nuts. A $\frac{1}{2}$" stiffener is placed between the flanges of the joist directly over each bearing.

The elasticity of the supporting beams was taken into account in designing the reinforcing for the deck slab and the live and dead load bending moments for different parts of the slab calculated by the method outlined in Section 4 of the Journal paper. The variation in stiffness of the beams for different parts of the span accounts for the variation in live load bending moment which are expressed in terms of Wl - and summarised in the graph of Figure 11. Mention is made of the method of distributing this moment over the effective width of the slab. This was determined as follows.

From test results an effective width, e, of slabs on rigid supports we find that at the support e = .71. In the following table the positive bending moment in the centre span of the deck slab due to a concentrated load is given in terms of Wl - the value of the moment is obtained by takingmoments of the external forces which are the reactions, as obtained from the corrected reaction diagram, due to the applied load W. The ratio of these moments to the moment at the support is calculated and it is assumed that the effective width of the slab increases in proportion to this ratio.

	M	Ratio $\frac{M}{M \text{ at support}}$	Effective Width e	
Support	.175Wl	1	.7x 93	= 65"
1/16 span	.213 "	1.22	.7x113	= 79"
1/8 "	.276 "	1.59	.7x148	= 104"
1/4 "	.362 "	2.06	.7x192	= 134"
1/2 "	.404 "	2.31	.7x215	= 151"

e for different parts of the deck slab was obtained by this means and the quantity of steel reinforcing proportioned in terms of this effective width and the dead and live load moments. An inspection of the quantities obtained enabled the reinforcing system detailed on drawing 65L-18 to be evolved. At the centre of the deck slab the maximum concrete stress in thetop of the slab is 680 lbs. per sq. inch dueto longitudinal bending giving a principle stress in the vicinity of 1090 lbs. per sq. inch. The concrete mix was designed for an ultimate strength of 3600 lbs.per sq. inch at 28 days and thefollowing results actually obtained.

Mix	Age	Average Strength	Number of Blocks
1:2:4	14	3267 lbs. per sq.inch	24
	21	4219 "	14
	28	4294 "	14

An average of 5.93 cub.ft. of cement was used per cub.yd. of concrete.

The deck slab is cast directly into a 7" x 3½" channel at the end of each span. Some form of cross member is necessary at this point to carry wheel loads at the edge of the slab the assumption being made that the member takes half and the slab half - a 7" x 3½" channel is found to serve this purpose and forms a convenient finish to the slab. The channel is bent to conform to the transverse camber of the deck and supported on stools made up from ½" plate and welded to the top flange of the joist. Since the steel girders in this composite type of bridge are rigidly connected to the deck slab there is no need to provide bracing to carry wind loads and the practice has been followed with this bridge. The concrete deck of thebridge carries a bitumastic wearing surfact ½" thick, the footways are built up above the road level and surfaced with precast concrete slabs 2" thick. This type of footway allows of a saving in dead load to be made. A light fence as protection from cattle crossing the bridge is provided between the foot and roadway on the upstream side of the bridge only. The main fence consists of reinforced precast concrete posts carrying a concrete coping on the top of two 1¼" diameter pipe rails. The method of attaching the coping* in a simple manner and at the same time to provide a means of adjusting minor errors in alignment of the posts. The general principle of precasting fence units etc. has much to commend it as it provides a simple method of disposing of small quantities of concrete left in the mixer from time to time and eliminates delay in constructing the fence when the rest of the bridge is completed. Lights are provided on each side of thebridge at the abutments and the second and fifth piers; the standards are of concrete and are precast.

*is designed to allow for expansion in the coping

APPROACHES.

The site for the new Leven Bridge has been the subject of keen discussion and although the site eventually adopted has advantages from the point of view of local traffic between the parts of the town on the two banks of the river, it has serious disadvantages in that existing roads and railways made it impossible to fix the position of the bridge to provide a good approach on the eastern side at a reasonable cost. Local interests pressed for the bridge to be made a continuation of Reiby Street. This proposal however was considered inadvisable owing to the proximity of shipping and the danger of boats striking the bridge in the strong tide. Failing this site the nearest practicable one to it was suggested as the most acceptable. At any point further upstream a level crossing in the wharf railway line was necessary in place of the overhead crossing at the Reiby St. site. With this condition for a level crossing, and that the maximum approach grade and minimum curvature on the approach road should be 1 in 20 and $1\frac{1}{2}$ chains respectively, the position of the bridge became a matter of location. A subsidiary approach in a southerly direction at the eastern end was also provided, but this was of minor importance. On the western side the approach is straight with easy grades and involved the removal of a number of buildings and the construction of a new street to connect with the Main Coast Road approximately $\frac{1}{4}$ mile from the western abutment.

Details of the arrangements of the approach on the Western side as originally planned are shown on 65L-4,5 & 6. After construction work was commenced some alterations were made to provide easier grades at the entrance to Reiby Street, and these are shown on 65L-8 & 22. The stone filling in the banks of the approach roads was carried up 2 feet above high water mark to obviate any chance of erosion of the filling by the tidal waters.

3. CONSTRUCTION.

1. GENERAL.

Owing to difficulties connected with the practice of carrying out bridge foundation work by contract, it has become the established practice in the Department to do this work by day labour, in fact contracts are only let for work which can be specified defintely and for which a reasonable price is tendered. The Levon Bridge however received special consideration as far as the contract versus day labour question was concerned owing to the fact that particularly accurate and careful work of an unfamiliar nature was required in the erection of the superstructure and also that the falsework required in connection with the substructure work could be made to serve also for the superstructure. These and other considerations influenced the Department in a decision to carry out the whole of the construction work by day labour, contracts being let only for the supply of materials. The work was therefore organised to allow construction of the substructure to commence from the eastern side followed by the construction of the superstructure from the same end as the piers were made ready for the beams, by thi means the construction period was considerably reduced occupying only $10\frac{1}{2}$ months from its commencement on January 9th 1934 to the official opening on November 26th 1934.

Two methods of driving the concrete piles of the substructure suggested themselves, one to drive the piles from a floating plant and the other from a fixed falsework. The former idea was rejected owing to the difficulty due to a 10 feet rise and fall of tide and the strong current; a timber falsework was therefore erected to carry the main pile driving frame and equipment, the piles in the falsework being driven in a convenient position to carry the screw jacks which supplied the propping forces necessary for the erection of the composite beams of the superstructure. The falsework was designed to carry the load from the jacks and support the pile frame as it was moved from pier to pier. At each pier the falsework was strengthened considerably to allow for the extra weight of a pile, the heaviest weighing 13 tons, and also to resist the effects of driving.

Plant.

The method of handling and driving the piles was influenced to some extent by the fact that although the piles were heavy and of considerable length necessitating heavy equipment the total number to be driven was only 18; it was obvious therefore, that the cost of the actual driving would be of minor importance compared with the cost of equipment and the cost of erecting and moving it so that speed in handling and driving could well be sacrificed if the cost of plant was thereby reduced. The piles were cast on the western bank of the river, rolled on to a punt and the punt floated across to the falsework. A steel pile frame 65 feet high carrying a 4 ton drop hammer and a double drum friction winch, operated by a 60 H.P. electric motor, was erected on the falsework and gear rigged to lift the pile off the punt from the frame itself using one drum of the winch to lift the pil and the other for the gear to pitch it. As the punt was available free of charge the cost of the pile driving plant was thus reduced to a minimum.

Electric power was supplied from a 6,600 volt line and transformed to 415 volts by a transformer at the welding bay on the western side of the river. An insulated 3 phase line was run across the river parallel to the bridge from this point and tapped at various points to supply power to the electric motor driving the winch on the pile frame and to a welding machine used at a later stage on the bridge deck. A second welding machine of sufficient capacity to provide for two welders was located at the welding bay itself.

Other machinery used on the work was part of the normal plant carried by the Department and arranged as self-contained units.

Material.

Contracts were let for the supply of all materials delivered to Ulverstone. Sand, cement andtimber were delivered by rail and the steel by boat to the wharf. This steel was picked up from the wharf by the punt, shipped across the river and transported on a light rail track to the welding bay, where it was stacked ready for fabrication. The steam crane on the punt was of sufficient capacity to load the steel beams from the wharf and directly to trucks after transportation to the western side of the river. A timber derrick of 4 ton capacity, hand operated, was provided in the welding bay for unloading this steel from the trucks and also for moving the beams about in the process of fabrication. The same derrick served to load the beams on to trucks on a line running across the falsework used for placing the beam in position on the bridge.

Falsework.

The falsework, as stated previously, served the dual purpose of carrying the pile driving gear for the concrete piles and the screw jacks which provided the propping forces at the centre and the quarter points of each span. The weight of the driving equipment was approximately 30 tons, exclusive of the pile which had a maximum weight of 13 tons, the prop load to each beam was 13.30 tons at the centre and 8.57 tons at the quarter points of each span; the general arrangement of the falsework to carry these loads is shown on plan 65L-16, but some additional piles were driven to facilitate the removal of the frame from pier to pier. All the piles in the falsework were driven from the punt by a gang of four men at an average rate of 7 per day, some difficulty was experienced in keeping them accurately in position owing to the strong current and this accounted for some delay. The piles were all driven about 10 feet with a 30 cwt. drop hammer. They were not shod but simply pointed with an axe. One set of piles in the sixth bay was omitted to allow for river traffic to use this opening and was not driven until the falsework had been removed from the second bay to allow boats to pass under the bridge at this point.

A second gang worked across the river bracing the piles of the falsework and sawing off the piles to the correct level. The falsework between the piers was not all braced at-once to allow the frame to be shifted, two bays were constructed to allow for this operation and after the frame had been moved the timber was transferred to the next unbraced bay and so on. Eventually when the falsework had to be removed, the braces were unbolted and taken ashore and a small charge placed in a hole bored in the pile at ground level by a diver. The charge was exploded by a submarine detonator andthe pile removed. This was found to be a simpler method than drawing the pile or sawing it off. A few of the piles were with-drawn, however, and used a second time.

Boring.

When the original examination of the bridge site was made test bores at one chain intervals were put down along the centre line of the bridge. A hand boring plant consistingof a hollow drill through which water was pumped at a pressure of approximately 100 lbs per sq. inch, was used for this purpose. Where the material was of a sandy nature or contained fine shingle a steel casing was used to prevent the drill from jaming. Different bits were used on the drill according to the nature of the material, but the calyx bit was generally found to be most suitable. This preliminary boring was done from a punt; it served to indicate in a general way the nature of the material in the river bed and from the information obtained the best type of substructure could be selected with confidence. It might also be mentioned that the results obtained indicated the recognised importance of using suitable equipment in testing the foundation material for bridges. A previous attempt to bore the river on a site close to the one adopted apparently showed solid rock within a few feet of the surface - an erroneous conclusion, for which unsuitable equipment was responsible.

From the evidence obtained as a result of this work the concrete pile substructure was adopted but there was so much variation in the material of the river bed that additional boring to determine the actual lengths of piles required was undertaken after the position of the piers had been fixed. At least two test bores were put down at each pier, one in the position of the up-stream pile, and the other in the position of the down-stream pile. If any marked difference was obtained another bore was put down between these two i.e. on the centre line of the bridge. All these holes reached a solid rock bottom, accurate records being kept of the various strata and plotted for each bore. Although this information was accurate it was not easy to know just how far a 22" pile could be expected to penetrate before reaching the required set. It was decided to make use of the water jet to facilitate driving in sandy materials and taking this fact into account the lengths of the piles were fixed on the assumption that they would reach their set when the head was level with the falsework and so allow the frame to be moved to the next pier. However if there was any doubt as to the length of pile required they were made longer rather than find when they came to be driven that they were too short. Generally speaking the lengths adopted were satisfactory. At No2. pier the piles could not be driven through the serpentine and about 15 feet had to be cut from each of the three piles. On all the other piers they drove approximately to the expected depth, in one or two instances the guides ofthe frame were extended and the pile driven a foot or so below the falsework before the set was obtained.

The extent of the boring that can be undertaken to advantage in and concrete pile work varies with the nature of the material through which the piles are to be driven and the uniformity in levels of any layer to which the poin is driven. The nature of the ground warranted a thorough investigation in this case the information being of value both in determining the lengths of the piles and during the driving. The same hand plant was used as for the preliminary boring but in this case the work was done from the falsework and a reciprocating jetting pump of 3,500 gallons per hour capacity at 150 lbs per sq. inch pressure was used. The cost including all items amounted to 4/7 per foot of bore hole.

SUBSTRUCTURE.

1. Eastern Abutment.

At high tide the water was 5 feet deep at the easternn abutment and 7 feet of soft clay and mud overlay the level of the foundation. A cofferdam 33 x 7' was constructed by driving 9" x 3" hardwood timbers round the outside of the foundation with a 10 cwt drop hammer from a timber frame. The cofferdam was suitably braced and the material excavated by hand from the inside. A 3" centrifugal pump unit was used to keep the cofferdam dry while the tide was high but towards the end ofoperations it become almost watertight due to swelling of the timber and the effect of clay being forced into the cracks from the outside. The excavation was carried about a foot into the serpentine and the concrete slab foundation then cast. Steel reinforcing dowels were cast into the slab to fix the columns and curtain wall and these members built up from the slab in the usual way.

No 1 Pier.

The serpentine was outcropping at this pier as shown on 65L-9 and at low tide was about 2 feet out of the water. It was only necessary to excavate deep enough to prevent any possibility of damage due to scour and this work was done between tides. As many men as possible were put on the excavation in order to reduce to a minimum the number of times the hole had to be dewatered. The foundation slab was cast and the columns and curtain walls constructed without any trouble. The reinforcing grids were electrical welded in the steel yard and placed in position as complete units. Provided the reinforcing is not too heavy the steel yard is the best place to do this work, as the time of fabricatio can be reduced by that means particularly if a number of similar units have to be made. There is very little to choose between tying and welding for holding the reinforcing in position. If a grid is to be moved about it will be a simpler job to use the welding - it should be noted too that additional reinforcing is often necessary to enable the grid to be moved without damage - but fabrication with the tie wire is as cheap and is to be preferred in that the section of the reinforcing rod is not reduced as is generally the case with the welded connection. A combination of tying and welding was used on the Leven Bridge according to the nature and situation ofthe work.

Piles - Manufacture.

The main rods for the pile reinforcing grids were bought in 30 feet lengths and had therefore at least one join in the length of the pile. A butt weld was used for this join and in addition some rods that had been cut to waste were put in at the splice. The main rods were spaced on jigs and the hoop reinforcing and stirrups, which had previously been bent to shape were slipped on and tack welded at the correct spacing along the length of the pile. The ends of the main rods were butt welded to the cast steel shoe care being taken to fix the shoe symmetrically on the end of the rods. The whole of the fabrication of the reinforcing grids was done by electric welding special electrodes suitable for striking an arc quickly being used. This is important because with some classes of electrods practically as much time is taken up trying to strike an arc as in actual welding. Care is necessary to see that the grids are not damaged when they are moved into the forms before concreting, either they should be designed to resist the stresses involved when they are lifted at one or two points only or adequate supports along the whole length of the grid should be provided during every stage of their transfer from the welding bay to the forms.

The casting bay for the piles was located on the reclaimed land on the western bank of the river; the reclaimed material was a mixture of sand and river shingle and therefore provided an excellent foundation. 9" x 3" timbers previously used for the cofferdam round the eastern abutment were founded on the sand at 2 feet centres and levelled with a surveyors level. 6" x 4" timbers at 2 feet centres were then placed at right angles on these and the forms for the piles built up on this foundation. The boxing for each pile was made as a separate unit although the opposite sides of the vertical posts between the piles carried the boxing for adjacent piles.

The concrete aggregates used for the piles and all other parts of the work consisted of crushed beach shingle and sand obtained from the Blyth River 10 miles away. The course aggregate was crushed from the particularly hard round stone which abounds on the N.W.Coast beaches, only pieces which would be retained on a 6" screen being used. It was crushed to 2" to $\frac{3}{4}$" "crusher run" for everything except the piles for which the specification called for crusher run metal of $1\frac{1}{4}$" to $\frac{1}{4}$". Although the reinforcing in the piles was closely spaced it was found possible to use the large aggregate and as higher strength was possible with this, at least a proportion was used and the surplus $1\frac{1}{4}$" aggregate placed elsewhere. The sand is recognised as of first class quality and calls for little comment. The mixture used was approximately $1:1\frac{3}{4}:3\frac{1}{2}$ and considering that no mechanical aids for tamping were available the high strengths indicated by the test blocks are very satisfactory. A number of cylindrical blocks either 10" x 5" or 12" x 6" were cast whenever concrete was placed and details of the mix, age, compressive strength etc. recorded. A synopsis of the results of test blocks taken from the piles is given on P. 2.

The slump test was not used in any of the work as the mixing, placing etc. was in the hands of experienced men who were conversent not only with the methods in use but also with the aggregates. The quantity of water was cut to a minimum and particular attention given to tamping and ramming the concrete into the forms. The latter feature no doubt accounts to some extent for the high strengths obtained but extensive testing on a large number of other bridges with various aggregates and proceedure in mixing and placing has indicated that the crusher run course aggregates and Blythe River sand used on the Leven Bridge are the best obtainable locally and if these are used under close supervision, 28 day strengths of 4,000 lbs per sq. inch with a nominal 1:2:4 mix can be guaranteed. The piles were covered with jute bags after the concrete was placed and hosed three times a day for a week after casting.

As only a limited number of piles were required there was no necessity to shift them until they were to be driven. Owing to their length and weight particular care was necessary to avoid the development of tension cracks in the concrete due to handling stresses, the punt previously used for driving the timber piles of the falsework was fitted with supporting bearers and skids placed from the edge of the pile casting bay to the bearers on the punt. Two hand winches were rigged on the side of the punt farthest from the pile and when the tide brought the top of the bearers to the same level as the bearers on the casting bay the pile was rolled on to the punt by winding in wire ropes run from the winches to the pile, a turn being taken round the latter. To prevent the pile from bumping as it was rolled towards the punt two additional guys were taken from the pile to be moved to the adjacent pile on the casting bay. The guys were passed round this pile and payed out as the other was rolled forward by the winch ropes. The general arrangement is clearly indicated in one of the photos of Appendix 111. As the lengths of the piles had previously been determined from the borings and the piles cast on the casting bay in the order they would be required for driving no difficulty was experienced in obtaining the pile required without the services of a lifting crane.

A pile having been transferred to the punt the latter was warped across the river and moored against the falsework at the pier over which the pile frame was situated. As stated in the section on design, a 24 x 7½" steel joist was used as a stiff back for lifting the piles; it was kept on the punt and lifted on the uppermost face of the octagonal pile by a rope from the pile frame where it was secured by four wire ropes long enough to pass completely round the pile and stiff back twice and spaced at intervals along the length of the pile. Each rope was clamped back on itself and tightened by driving timber wedges under the rope at the top flange of the stiff back. Pieces of timber two inches thick were placed between the pile and stiff back to allow the head gear which was attacked to the pile alone to be fixed without fouling the stiff back. These details can be seen in photo No 26.

The steel pile frame, 65 feet in height and 16ft x 20ft at the bas was erected on two 24 x 7½" steel joists 50 feet in length placed symetrically on either side of the centre line of No 2 Pier. Rails were welded to the top flanges of these joists and steel shoes put on the rails under the main members of the base of the frame. The winch and gear operating it was mounted on the back of the frame itself and by rigging the necessary gear the whole frame could be traversed on the rails by winding in a rope running from the friction winch on the frame to the joists supporting it. Originally this frame was equiped with wheels which were attached to the base of the frame and served to traverse it on rails but experience has shown that it is more satisfactory when driving piles to have the frame solidly supported and depend on sliding it when it has to be moved. The frame then was traversed to the downstream side of the falsework and after lifting and fixing the monkey and dolley at the top of the frame one gear was attached to the bridge for lifting the pile, and the second directly to the head of the pile for pitching it. Three heavy strops were passed completely round the pile and stiff back and shackled to one end of the bridle and to the pulleys through which the bridle passed at the points of support. The weight of the pile was then taken on the bridle gear, the punt floated clear and the pile swung into the vertical position by winding in the head rope. Additional guys were necessary on the head and point of the pile to steady its motion while it was being pitched. On reaching the vertical position the weight of the pile was taken on the head rope alone and the bridle and stiff back lowered again to the punt. The pile was then swung into the leaders where it was fixed by clamps, the frame carrying the pile traversed across the falsework to the position for driving and the dolley fitted in position on the head of the pile. After the three piles had been driven at No2 pier the frame and supporting beams were moved bodily by sliding them on rails running along the two sides of the falsework. Three hand winches supplied the power for this purpose, the frame being moved in stages by this means to the western abutment where it was dismantled and taken ashore.

Piles - Driving.

It has been pointed out elsewhere concerning the choice of hammer for driving concrete piles that the drop hammer is to be preferred to the steam hammer for most circumstances on the grounds of economy. In many cases the actual driving time is only a small percentage of the total time required for the work so that even a saving of half the driving time does not represent a large proportional saving in total time, this fact combined with the higher capital cost of the steam hammer, the large amount of steam required and the heavy tackle and equipment required to handle almost double the weight of hammer made the decision in favour of the drop hammer for the Leven Bridge piles an obvious one. Although there are some thirty or fourty pile formulae in existence and at least half a dozen in common use, the subject having received a great deal of consideration, there is still much information to be desired on the subject. It is not proposed to treat the subject in anyway exhustively here but the points which received consideration in deciding the set required to ensure that the piles would carry a 65 ton load will be indicated.

In the first place the weight of the drop hammer necessary to drive the heaviest piles of 13 tons is required. It is important to notice the effect on the efficiency of the driving of hammers of different weight. There are two main stages in pile driving, in the first the pile is hit by a hammer and set in motion, and in the second the resistance of the earth stops the pile. The effectiveness of the impact is a function not so much of the energy as of the momentum per blow.

If W = weight of hammer
P = " " pile
V = velocity of hammer at moment of impact
v = resulting velocity of pile and hammer together

Then
momentum of the hammer before striking = WV., after the impact the total mass in motion is $(W+P)$ and the momentum is $(W+P)v$. Equating the momenta gives

$$WV = (W+P)v \quad \text{or} \quad \frac{v}{V} = \frac{W}{W+P}$$

, that is the ratio of the velocities before and after impact is the inverse of the masses in motion. The kinetic energy before impact is $\frac{WV^2}{2g}$, and the energy of the total moving mass after impact is $\frac{(W+P)v^2}{2g}$. Therefore the kinetic energy is reduced by the impact in the ratio $\frac{W}{W+P}$, and the resulting kinetic energy of the blow is $\frac{WV^2}{2g} \cdot \frac{W}{W+P}$. In actual practice the available energy will be even less than this. The following analysis relates to the 13 ton pile being driven by drop hammers of various weights from one to ten tons, the fall being that necessary to give a blow of 20 ft tons in each case.

The figures show that the driving effect is much greater with a heavy hammer than with a light one for the same energy of blow, and the risk of cracking the pile is considerably greater with a light hammer and a high velocity than with a heavy hammer and a small fall.

Hammer weight tons	Fall feet	W + P tons	V ft/sec.	v ft/sec.	K.E. after impact ft tons	%	K.E. lost by impact. %
1	20	14	35.8	2.6	1.43	7.1	92.9
2	10	15	25.3	3.4	2.66	13.3	86.7
4	5	17	17.9	4.2	4.70	23.5	76.5
6	3.33	19	14.6	4.6	6.31	31.5	68.5
8	2.50	21	12.6	4.8	7.62	38.1	61.9

The figures also show that any pile formula based directly on the product of hammer weight and fall is misleading. Fig. 4 shows a graph of driving energy plotted against the weight of the hammer expressed as a percentage of the weight of the pile. In practice the driving energy is decreased by the energy required to overcome friction and the elastic compression in the pile itself so that the resulting effective driving energy at the pile point is indicated by some such curve as the dotted one, which shows that over a certain range of weight ratio of hammer and pile there can be no driving, and that increase in the weight of the hammer has a very great effect.

The Brix pile formula is a driving resistance formula which taken account of the weight of the pile but no allowance is made for work done in compressing the dolley and packing or in compressing the pile itself but to offset this the available energy of the drop hammer after impact is neglected. This formula gives good results for sets greater than $\frac{1}{4}$". Hiley has gone a step further and makes allowance for these items, also for loss of energy due to errors in centring the pile and due to longitudinal vibrations.

The formulae is

$$R = \frac{12\ Wh}{s + \frac{c}{2}} \quad x \quad \frac{W + P\,\mathcal{E}^2}{W + P}$$

where R = ultimate driving resistance (tons)
h = equivalent height of free fall of hammer.
s = set per blow in driving (inches)
c = equivalent compression in pile and dolley.
$\mathcal{E}$ = coefficient of restitution of dolley.

A 4 ton hammer was used at the Leven and the piles driven to a set of 1 inch for the last 7 blows with a 4 foot drop; inserting in the formula the other constants applicable for the conditions of driving

$$R = \frac{12 \times 4 \times 36}{.14 + .18} \quad x \quad .24.$$
$$= \underline{108 \text{ tons}}.$$

It is recognised that this formulae gives a reliable estimate of the driving resistance for all values of the setwhich more recent work has shown to tend, if anything, to the conservative side for small values. As the piles were driven to a rock bottom with the assistance of the water-jet, and the set measured before the effects of the water had entirely disappeared it was considered that there was no justification in driving the piles to a smaller set than the average "adopted of .14 " for each of the last seven blows. The piles were all driven to the same set although the shorter piles would indicate a somewhat higher driving resistance from the formula.

Particular care was necessary to see that the dolley retained a symmetrical position on the head of the pile. The dolley was of cast steel with a blue gum block in the top section and packing between the central diaphram and the head of the pile. The most effective arrangement of the packing was found to consist of two layers of pine separated by one of hardwood with woodwool

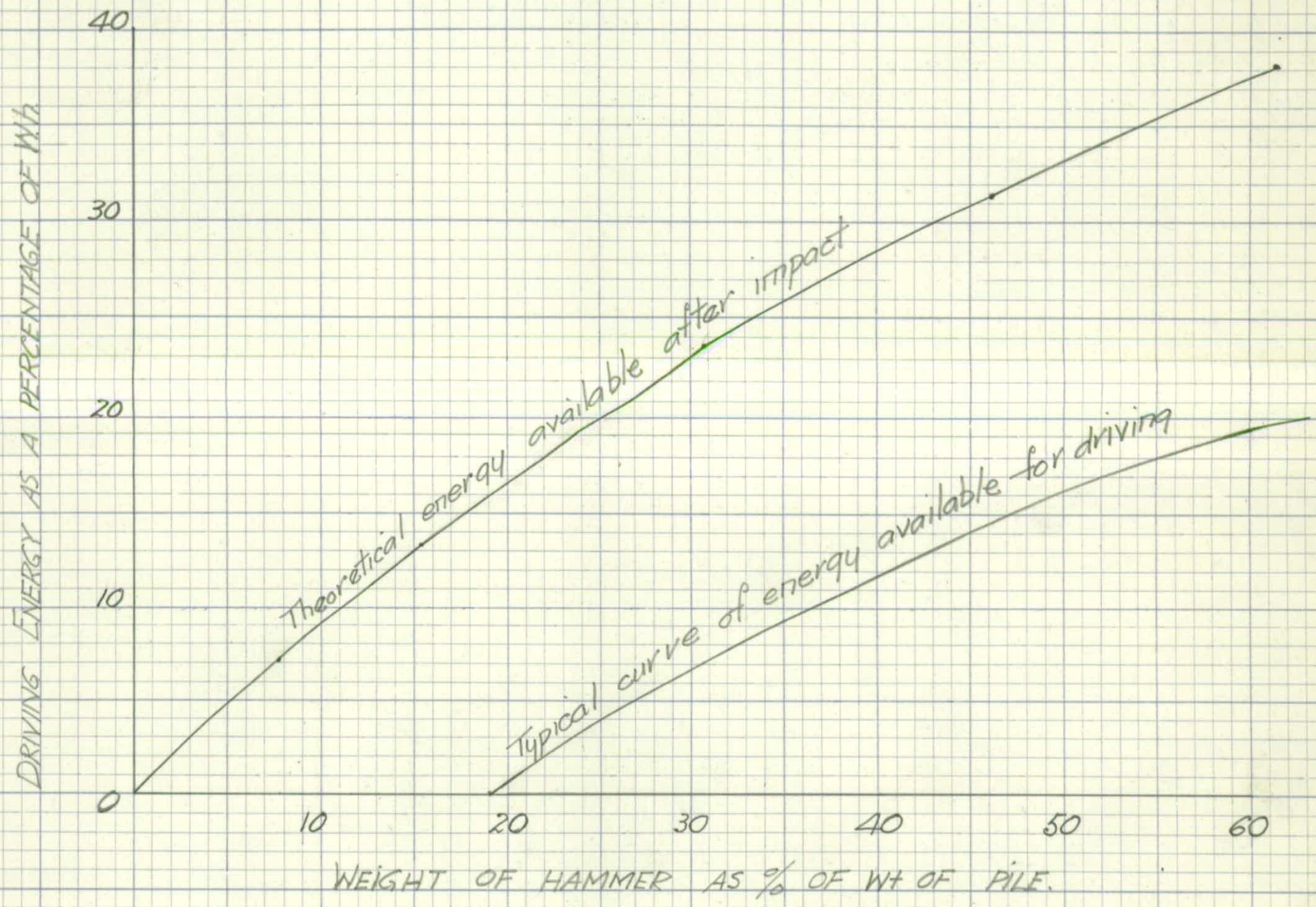

DRIVING ENERGY AS A PERCENTAGE OF Wh
40
30
20
10
0
10
20
30
40
50
60
Theoretical energy available after impact
Typical curve of energy available for driving
WEIGHT OF HAMMER AS % OF Wt OF PILE.

FIG. 4.

between the pine and the head of the pile. This woodwool was only used to drive one or two piles and was then replaced by another lot. Two lugs cast on the sides of the dolley served to hold it in position, the lugs fitting round the leaders of the frame.

The nozzle shown in the plans was used in driving the first few piles but some alterations were made to the remainder. A reciprocating pump is generally used to supply the water to the jet, a safety valve being used to relieve the excess pressure should the jet become blocked. As it was intended to use the pump on various other works as a general purpose pump a centrifigal pump was used in this case with the idea of reducing weight. It was specially designed for the job with four stages, head 350 feet and capacity 10,000 gallons per hour and proved entirbly satisfactory. Care however is necessary to ensure that the delivery through the pile and nozzle is not unduly restricted otherwise the quantity of water is reduced below that required for effective jetting. The unit was belt driven by a Stutz petrol engine the whole outfit being mounted on the punt, connection being made to the pile through a fire hose and coupling. The nozzle as originally designed was not satisfactory although it may have been if a greater supply of water had been used, the four return outlets were therefore blocked up but without any appreciable improvement. The size of the central nozzle was then experimented with, a diameter of $1\frac{1}{2}$" finally being adopted as the best. The pump runner was of stainless steel to resist the corrosion of the salt water which was pumped from the river through the jet.

The work of driving the piles proved a particularly difficult proposition owing to the nature of the material to be negotiated. At the second pie: with the water jet in operation the piles only needed light blows from the hammer to reach the level of the serpentine where after a tendency to run at the toe they soon reached the specified set. This tendency for the piles to run was difficult to prevent without damaging the piles. Heavy timber frames were constructed at low water level, the piles of the falsework being used as supports and the pile driven down through this frame. If the obstruction in the path of the pile was a boulder of any appreciable size the bending moment induced in the pile, due to the restraining influence of the frame, would either cause the concrete to crack or else force the frame out of position. To relieve this situation frequent resort was made to the hand jet the vicinity of the obstruction being thoroughly explored with the jet before any further driving was attempted. The extra lengths of the piles were cut off at the second pier when it was obvious that the expected penetration would not be attained and the pile-frame moved to the third pier. The surface of the river bed at this pier was covered by several feet of sand and heavy shingle with the result that the jet water forced the sand away and left a solid mass of stone for the pile to penetrate. This hard patch on the surface made it difficult to keep the pile in the correct position only a small error at this stage of the driving causing considerable difficulty later on. The fact that it was necessary to keep the piles within a few inches of the correct position was the chief cause of trouble as once the best size of the jet had been obtained no great difficulty was met in obtaining the penetration. This ranged between 26 and 30 feet at this pier. At the fourth pier the driving was good, the penetration ranging between 34 and 37 feet. At the fifth pier boulders caused a great deal of trouble, three days being taken in driving one pile. The pile was withdrawn several times to facilitate the work of removing the obstructions which were of such a nature that on the downstream side of the bridge the pile only reached a penetration of 20' - 2" whereas on the upstream side the penetration amounted to 34' - 11"for the same set. The material at the sixth and seventh pier and at the western abutment was of a more consistent nature, mainly sand carrying fine to course shingle and the driging did not give very much trouble.

As the timber retaining wall on the western sideof the river was erected before the piles for the western abutment were driven it was necessary to transfer the piles from the punt to the filling behind the wall again before pitching them. This was done by hauling the pile off skids placed on the punt on to rollers carried by a 24" X $7\frac{1}{2}$" joist on it's flat, the joist being supporte so that the rollers were just above the level of the timber sheathing. This was done on a falling tide and the piles moved at such a rate that the rise in th level of the punt due to the removal of the weight of the pile equalled the fall in the water level. By this means the pile was removed without difficulty, sufficient rollers being used to reduce the bending moment in the pile to the required limit, and brought opposite the pile frame by which it was lifted from its supports and pitched ready for driving.

An accurate driving record was kept for each pile showing the number of blows, height of drop, jet pressure at various stages of the driving and so on. Curves have been plotted from these results showing penetration of pile per foot lb of energy delivered for various sizes of jet but the material was of such a varigated nature that no useful conclusions can be derived from them, About 2,000 blows of 3 feet drop was an average value for the number of blows required before the specified set was obtained but very often half the blows were spent in negotiating thin layers of shingle or boulders. The water jet was kept in action until the pile had reached a penetration sufficient to bring it on to a sound foundation as indicated by the test bore, it was then turned off and the driving continued under the hammer alone. For some cases, where the point of the pile had not reached solid rock, the additional penetration after the water was turned off was only a few inches, which illustrates the effectiveness of the jet.

A number of the piles developed small cracks during the driving particularly in the early stages of the work. The hard driving required to force the pile through the surface shingle was responsible for some of these but the chief cause was from the restraint placed on the piles to prevent them from running. This can be overcome im most instances by the use of external jets but here they were only of limited value. Even if the pile is allowed to follow a direction other than the vertical without restriction damage can be expected near the head of the pile owing to the fact that the frame is fixed. Both the position and the direction of the guides would need to be adjustable to overcome this trouble. However the cracks were not serious and had it not been for the large cover allowed on the steel would probably not have been visable, regular inspection both above and below water level shows that most of these have now disappeared. In no case was it considered necessary to sleeve the pile.

Any length of pile projecting above the level of the falsework was removed by chipping the concrete away from the main reinforcing rods with gads and then cutting the rods with an oxy torch. The piece was then easily pulled off. Instead of casting the concrete walings on the shore as was originally intended these were cast directly over the three piles of the pier, a piece of the curtain wall about three feet in height being cast on the waling between the piles. Sufficient clearance was left round the piles to allow the waling to be lowered by two chain blocks on to three timber clamps placed at the required level, one on each pile, by a diver. The space between the waling and the piles was filled with concrete placed through the water at lowtide and the curtain wall extended and cast monolithic with the cross beam. When the concrete was set the clamps were removed from the piles and transferred to the next pier. Considerable advantage was gained by pre-casting some of the curtain wall on the slab as only the spring tides were low enough to allow work at this level to be done in the dry. The reinforcing grids for the cross beams were fabricated in the steel yard but for convenience in handling were made in two pieces. $1\frac{1}{8}$" diam. rods were welded to those of the same size in the piles to run into the crossbeams to ensure adequate bond between these members. The holding down bolts for the bearing plates were cast in position in the top of the pier the lower of the two nuts on each bolt providing a particularly convenient method of adjusting the levels of the plates. The level of the plate having been obtained it was removed and the space underneath filled with cement mortar, on replacing the plate and screwing down the top set of nuts the excess mortar was squeezed round the sides of the plate.

SUPERSTRUCTURE.

The whole of the structural and reinforcing steel required for the superstructure was stacked adjacent to the crane in the steel yard. A welding bay was constructed on a timber foundation and two lengths of the steel beams set up in position on the bay ready for splicing. The beams were cut to length and the ends prepared for welding by a mechanically operated oxy-acetylene torch; the cuts were hand finished before being welded. The cover plates, splice plates etc, were all cut to shape with the torch and welded in position while the beams were on the welding bay. The square stirrups were bent by hand with the use of a jig and set in position by a template, first just being tacked in position and finally welded afterwards. It was found advantageous to use extra fluxed electrodes for this work the E.M.F. electrodes of this type giving good satisfaction. For all other work excepting the reinforcing grids for which rods designed for ease in striking the arc were used, the New Era electrodes manufactured by the E.M.F. Co. were used. The stirrups were put on while the beam was still on the welding bay, no difficulty being encountered in handling the beams afterwards. On completion of this phase of the work the member was placed on two bogey trucks carried by a line running across the falsework and hauled into position on the various spans. By fabricating the steel work and placing the four beams of each span in position as each pier was completed the difficulties attached to moving the beams over the tops of a number of piers were avoided. Only one track was used for taking out the beams, greased timber placed on the top of the piers serving as a base to slide the beams from this track to their respective lateral positions in the span.

Having welded the fixed bearing in position on the bearing plate it was a simple matter to drop this end of the beam into position and thus set the other end on the rocker. The four beams were then connected by the 7 x 3" channel by welding the channel to the stools which were already in position on the webs of the joist. When the deck concrete had set the bolts were removed and theholes filled up with weld metal. In addition to these bolts reinforcing rods were welded to the flange of one beam near a support and to another at a point ten or fifteen feet from the pier, a few of these braces was sufficient to fix the direction of the beams until the concrete was set.

Heavy timber beams across the top of each set of four piles provided the propping forces calculated in the section on design. The four jacks at the centre of the span were placed in position first and the beams jacked up the required amount; the ends of the beams were held on the bearing plates by steel clamps attached to the concrete cap of the pier and all measurements taken from a datum level established by stretching 22 gauge piano wires between the piers a few inches directly beneath each beam. The jacks at the quarter points were brought into contact with the bottom flange of the beams and then thedistance between datum and the flange at each jack checked and tabulated for reference. These measurements were checked at intervals while the deck concrete was being placed and any alterations necessary were made by adjusting the jacks. It was found better to set the jacks a little higher in the first instance as the load was sufficient to cause a loss in height of about $\frac{1}{4}$" in the supports, in any case it was easier to lower the jack than to raise it. All the jacks were of the screw type of 15 tons load capacity.

The boxing for the under side of the deck was made up in the form of shutters from $\frac{7}{8}$" hardwood flooring supported from bearers placed on the lower flanges of the beams. By handling the shutters carefully and painting with oil each time they were used the one complete set lasted for the seven spans, as a result the cost of this work was kept at a minimum. The jacks were not removed until the test blocks indicated a strength of 3,000 lbs per sq. inch in the deck concrete, the precast fence posts were then set in position and cast into the kerb. In each panel of the fence a break was made in the kerb to relieve the compressive strength at the top of the kerb due to bending under live load. If the pipe rail is threaded through the holes in the posts before the kerb is cast it saves any difficulty in doing this afterwards due to slight errors in alignment. The posts in the footway fence were also set up in position before any of the kerb was cast.

48 cubic yards of concrete were placed in each span of the deck in one operation. Two petrol driven concrete mixers and a gang of twenty mem placed this concrete in about seven hours, the whole of the work being done

from the eastern side to avoid lifting the concrete from the level of the falsework to the level of the deck which would have been necessary from the other end. The mixing plant was moved out on to the deck after two spans had been cast in order to reduce the distance over which the concrete had to be barrowed. Although the reinforcing system in the deck slab was fairly complicated no difficulty was experienced in placing concrete containing 2" metal; use was made of heavy tamping rods to assist in placing the concrete the continual ramming of these heavy rods giving a very satisfactory job.

As successive spans of the deck were cast the beams were cleaned and painted and the temporary falsework removed. A primer known commercially as "Fishoilene" was applied to the steel first and then followed by a coat of aluminium paint. This paint is exclusively used by the Department for either high or pony type through trusses owing to the excellent lighting effect given the trusses by reflection from car lights and as it has given satisfactory service as a paint it is also used for steelwork in other bridges.

APPROACHES.

The stone filling of the eastern approach was placed by contract, also some of the earth filling at both ends of the bridge. These contracts were schedule rate contracts the material being paid for at so much per cubic yard measured either in position or in trucks as specified. Other than some stone for the retaining wall on the eastern side and for the road foundation which was obtained by contract the whole of the remaining work was done by day labour. This proved a convenient method as it formed a stock job for a number of labourers which were only required on the bridge deck when concrete was being placed. The work of demolition of buildings, road construction etc. was all straight forward and calls for little comment.

C O S T S.

An accurate costing system was maintained throughout the whole of the period of construction as it was realised that the information obtained would be of particular value in estimating the cost of future work. This information however is not available for inclusion in this thesis, but it is of interest to note that the actual cost of the bridge was slightly less than the estimate.

A P P E N D I X 11.

As the design of the longitudinal members and the reinforcing of the deck slab was based on the test results obtained from the 1/6th scale model of the bridge it was of interest to check these results by making a similar test on the bridge after it has been constructed. The first span was selected for this test and the 4 ton hammer applied as a concentrated load at points on the centre line of the span the deflections of the longitudinal members for each position of the load being measured by gauges.

The gauges were set up, the hammer moved into the first position and the resulting deflections indicated by the gauges recorded. Owing to the weight of the hammer and the consequent difficulty in moving it about it was necessary to reduce this movement to a minimum, the hammer was therefore moved across the deck in steps, the gauges being read at each position, and finally removed from the span. It was found then that the zero error in the gauges was sufficient to render the previous readings valueless. This error was traced to the effect of atmospheric temperature changes above and below the deck, the deflections produced by the normal temperature changes in a few hours being greater than those due to the hammer itself. This difficulty was partly overcome by setting the gauges at zero, loading the hammer in one particular position and then removing the hammer from the span to check up any zero error. This process was slow but sufficient information was obtained by this means to indicate the form of the influence lines for reactions. The influence line diagram plotted from the results is substantially the same as that obtained from a similar test on the model the results of which are given in Table 11. in the Journal. The diagrams are superimposed in Figure 5.

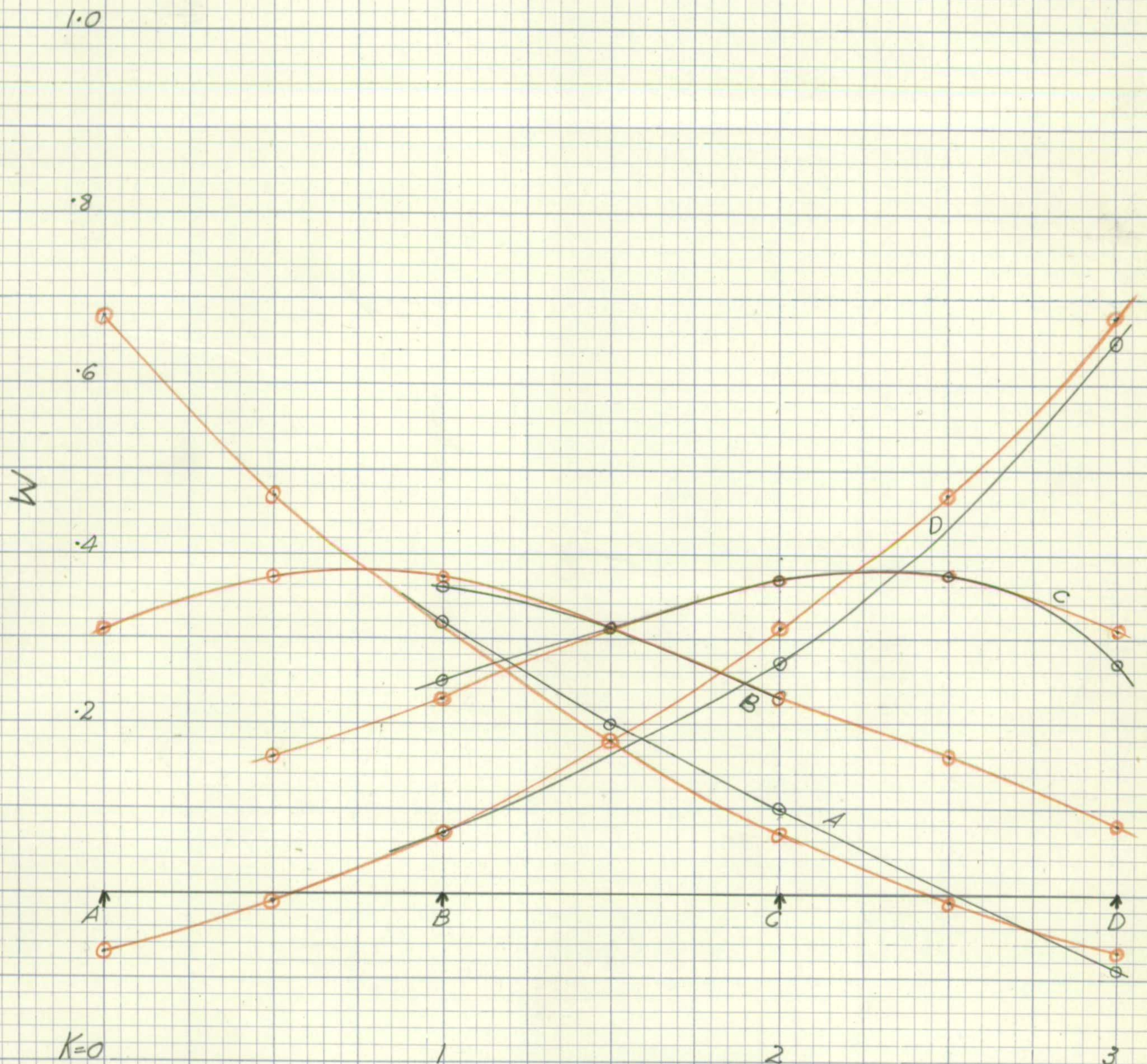
Very little useful information can be obtained from a test which involves the use of a load at a number of points, the complicated distribution of these loads rendering the results unintelligable; this fact accounts for the use of the concentrated load in the above test. It might be noted that the deflections measured for this type of structure for a given load are invariably less than those indicated by deflection formula. The elasticity of the structure under load was a feature of the Leven Bridge test.

An attempt was made to obtain some check on the deflections caused by atmospheric temperature changes. Figure 6. shows a graph of deflections of the two inside beams plotted against time and shows how the deck slab rises as the surface concrete expands due to the increased temperature. It is worth noting that these deflections exceed those due to the 4 ton hammer used for the distribution test but whether they involve any stress in the material of the superstructure depends on the distribution of temperature through the concrete deck and steel beams. If the temperature distribution between the deck surface and the bottom of the steel beam is linear then there is no stress but if the temperature distribution from the top to the bottom of the slab is linear and the temperature in the steel constant - a more probable arrangement - then temperature stresses are involved and these can be calculated. Observations indicate that the difference in temperature between the top and bottom of the slab might be as much as 20° F.

LEVEN BRIDGE TEST

Influence line for Reactions

½ span



Position of unit load

FIG 5

Model test thus

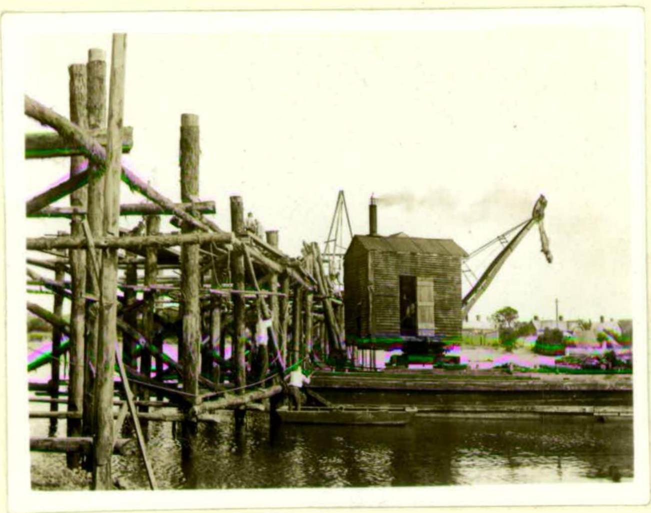
Leven Bridge test thus

FIG. 6

1. The bridge site.

2. Falsework piles and boring plant.

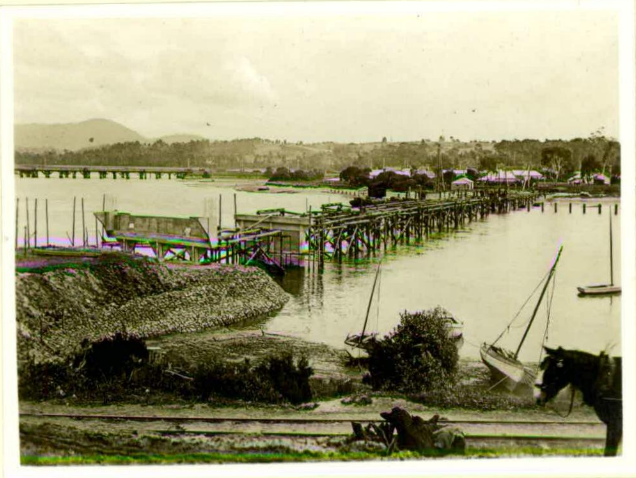
3. The eastern abutment caisson and stone filling.



4. Bracing the falsework.

5. View of the falsework from the western end.

6. The falsework ready for the longitudinal rails.



7. General view showing eastern abutment and No.1 pier.

8. Another view of the falsework. The stone filling on the eastern approach completed.

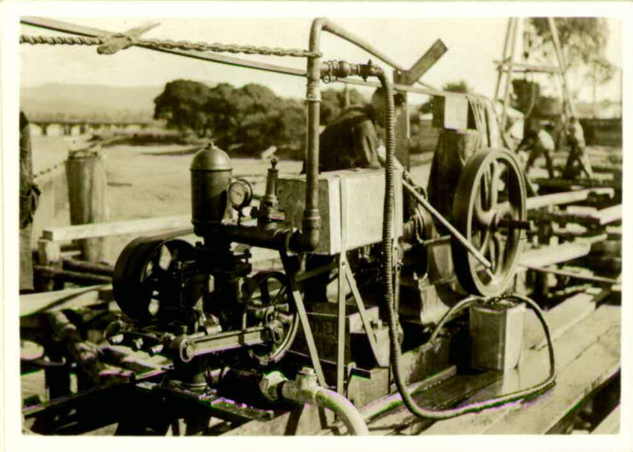
9. A later view of the work on the eastern approach.

10. No.1 pier with the abutment in the back ground.

11. Another view of No.1 pier.

12. The eastern abutment before the earth filling was placed.

13. Boring operations in progress from the falsework.



14. The reciprocating jetting pump which supplied water to the drill.

15. Preliminary work on the eastern approach.

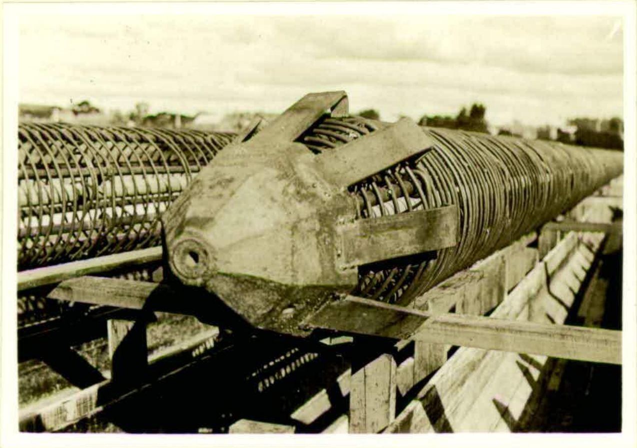
16. Work in progress on the western approach road.

17. The western approach from the pile frame.

18. The timber retaining wall on the western bank.

19. Setting up pile reinforcing grids.

20. Welding the spiral hoop reinforcing on the main rods.

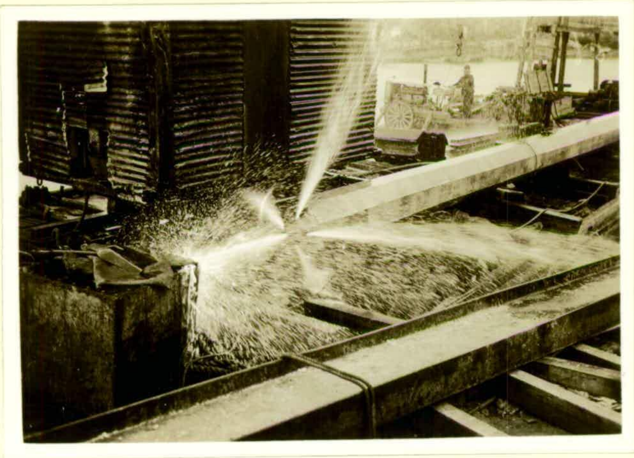


21. Completed grids ready for placing in the forms.

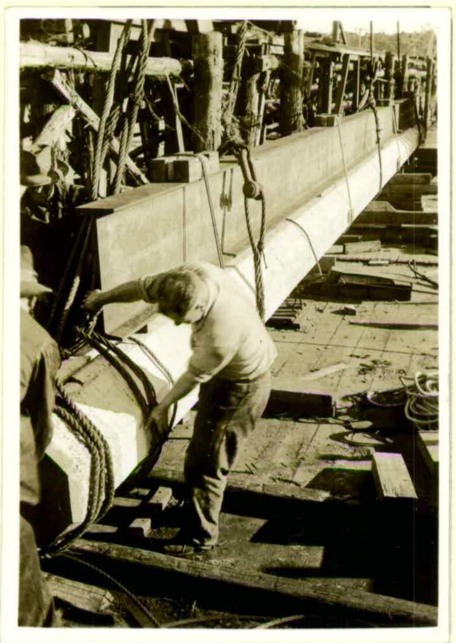
22. Placing concrete in the pile forms.

23. The pile casting bay on the western bank.

24. Rolling a pile from the casting bay to the punt.



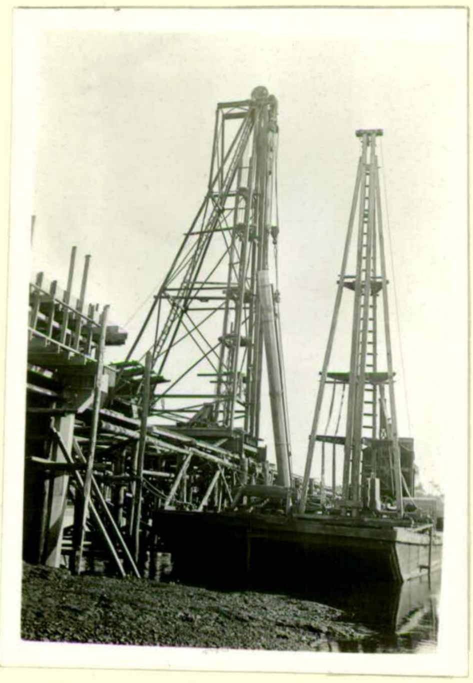
25. The pile water jet in action.



26. Fixing the stiff-back on the pile preparatory to pitching it.

27. Erecting the steel pile frame over No.2 pier.

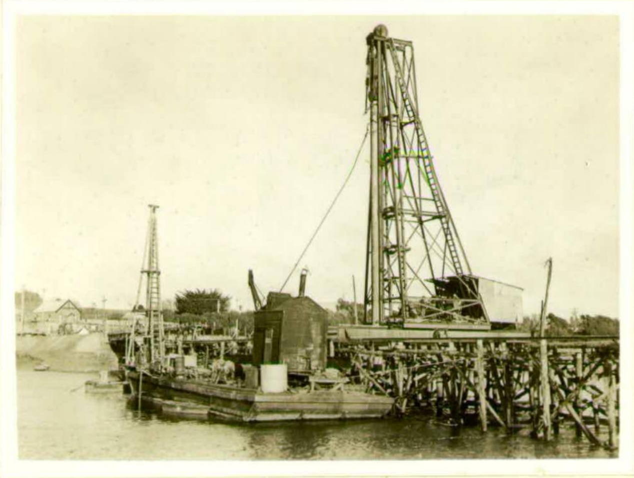
28. Pitching one of the concrete piles



29. A later stage in the pitching operation.

30. Removing the bridle and stiff-back.

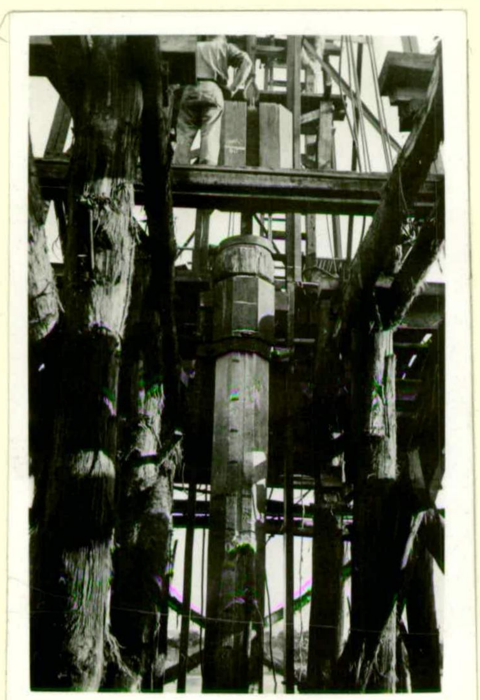
31. The pile ready to be swung into the guides.



32. Ready to drive. The jetting hose can be seen in this photo.

33. One of the piles at No.6 pier before driving.

34. A pile about to be lowered with the water jet in action.



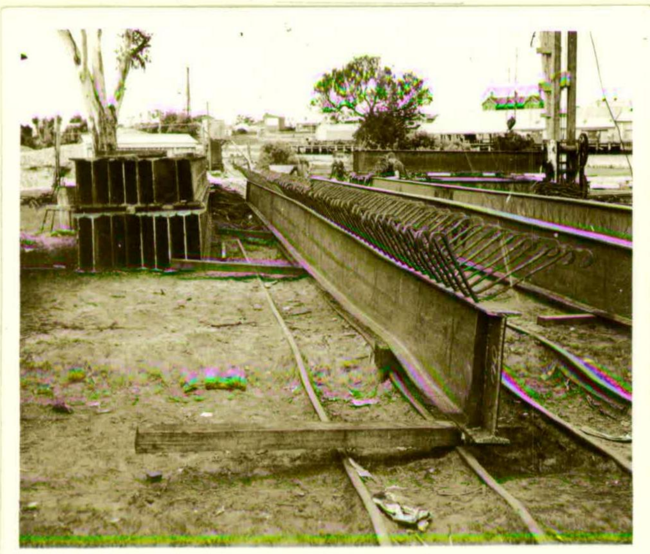
35. Driving in progress under hammer and water jet.

36. Driving completed and dolley removed.

37. This pile was withdrawn and the timber pile on the right which had fouled the concrete pile was removed.

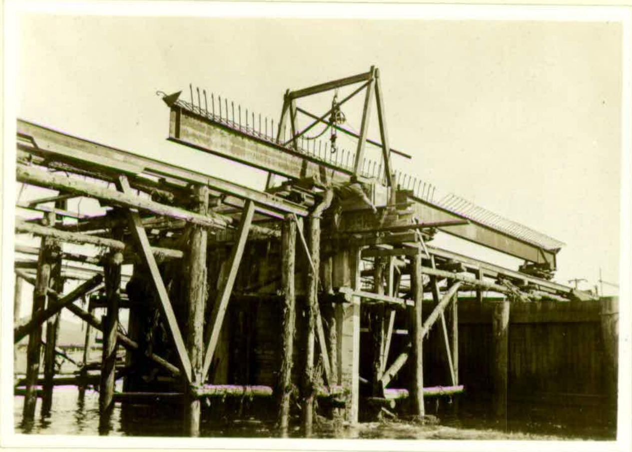
38. Cutting one of the piles at No.2 pier.

39. Loading steel beams to the punt from the wharf.

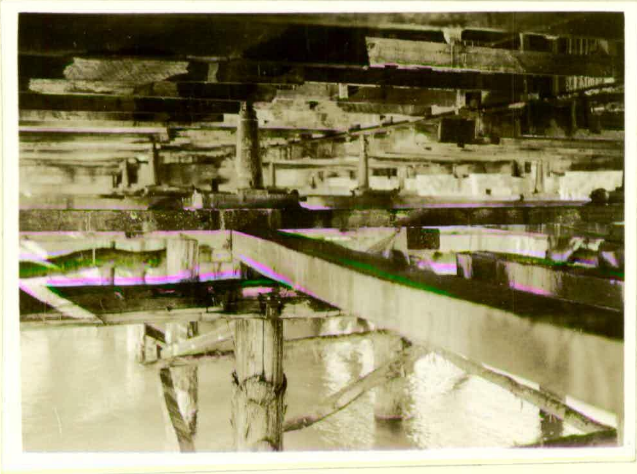


40. Beams in the steel yard in course of fabrication.

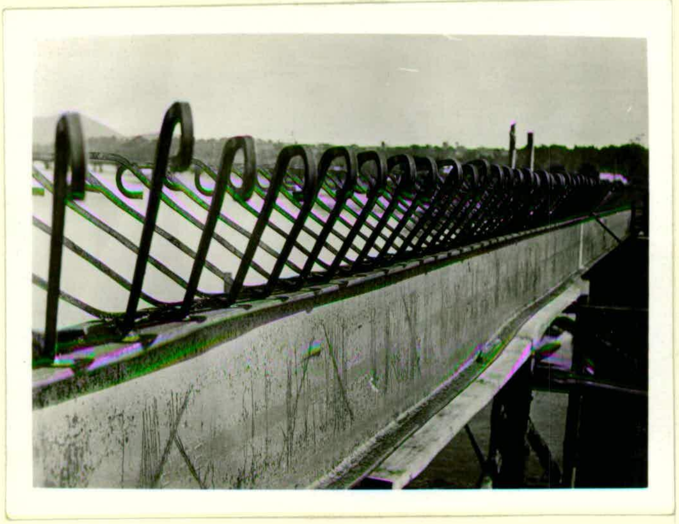
41. Placing a beam on the first span.



42. A beam with the stirrups already welded in position on its way from the steel yard to its place on the bridge.

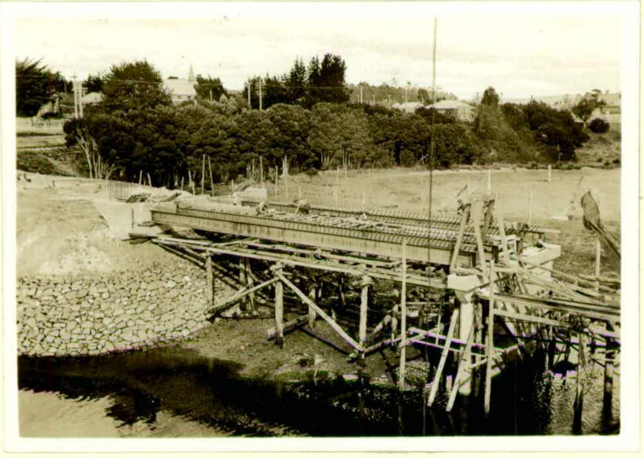


43. A view showing the jacks in position under the deck. The concrete has been placed.



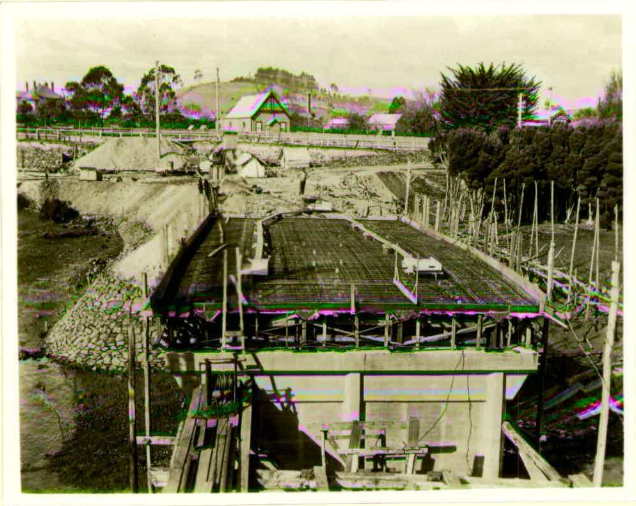
44. The shear reinforcing which develops the composite beam action in the deck.

45. A span of the deck with part of the formwork in position.

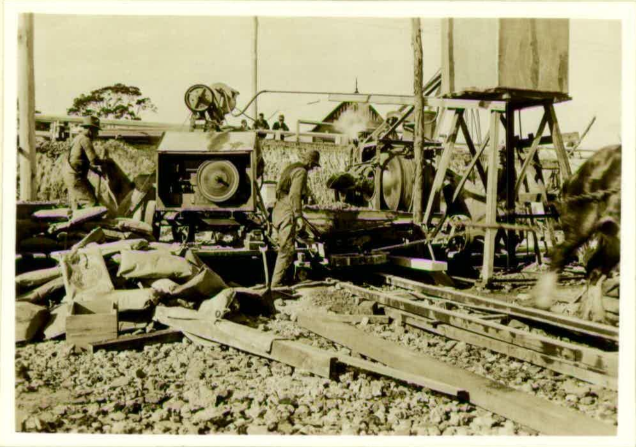


46. The first span with the beams in position and ready for the formwork.

47. The reinforcing system in the deck slab.



48. The first span ready for concreting.

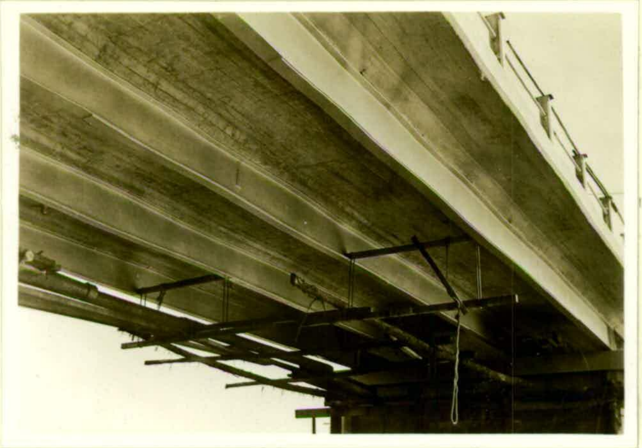


49. The two mechanical concrete mixers which mixed the concrete for the deck slab.

50. Placing concrete on the deck of the bridge

51. The first and second spans with the form-work removed. The splice in the steel joist is plainly visible.

52. Another view after part of the super-structure was completed.



53. Underneath the second span.

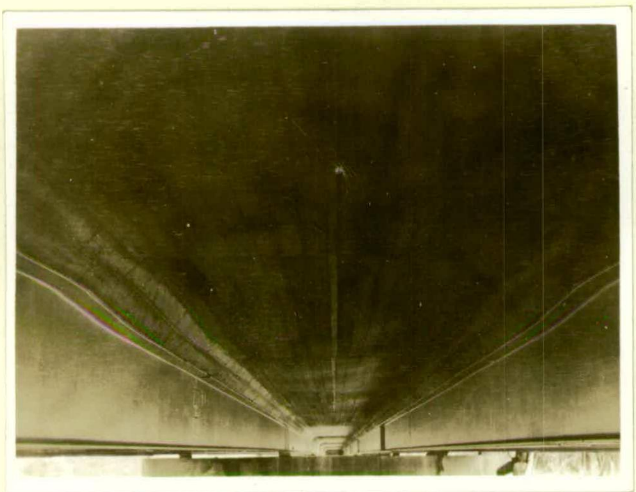
54. A view of the second span with the old road and railway bridges in the distance.

55. At this stage the falsework was removed from the second span to provide a passage for river traffic.

56. The bridge nearing completion- a view from the west on the upstream side.

57. The deck before surfacing with the fences and footways under construction.

58. The footway ready for the concrete slabs.



59. This photo was taken from the western abutment underneath the deck slab.

60. Moving the 4 ton hammer during testing operations.

61. The bridge completed -a view of the deck.

62. A view from the upstream side.

63. Another view from the upstream side.

64. From the eastern approach.