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Steel Bridge Design

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Civil Engineering III
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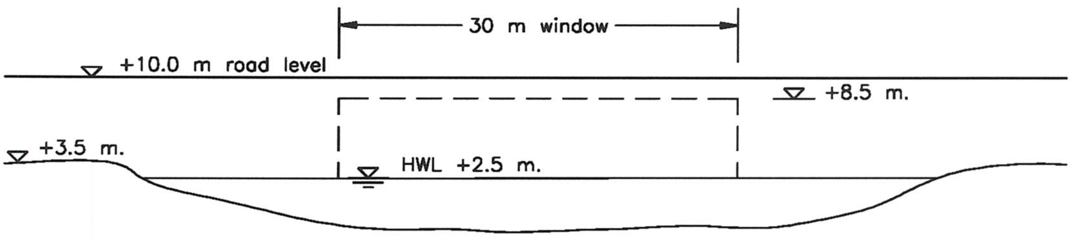
DESIGN BRIEF

Objective:

- to provide a new steel bridge across a navigable tidal river in the Midlands
- bridge is to be of modern structural form and to be aesthetically pleasing
- bridge will connect with existing roads at an interchange on each bank of the river; the level of the bridge at each of the interchanges is fixed at +10.0 m

General Diagram of Bridge Site



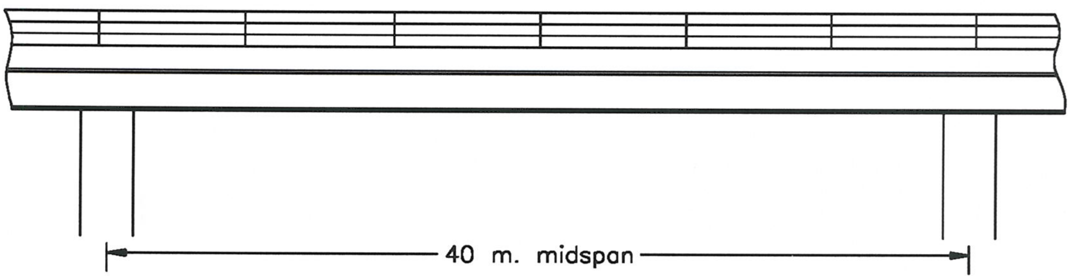
Design Constraints:

- the bridge is to have a single carriageway with two lanes, each of which is 3.65 m. wide
- a 2 m. wide footpath is to be provided on each side of the carriageway
- existing road levels at the interchange on each bank of the river govern the road levels of the bridge
- the level at each interchange is fixed at +10.0 m.

- the river is navigable and tidal and is 60 m. wide between the high water mark on each bank
- the level of the existing ground on each bank is +3.50 m.
- two navigation channels, each 10 m. wide, are to be provided in the river and must be contained within a central 30 m. wide window in the river
- the high water level (HWL) of the river is +2.50 m. and a minimum clearance of 6 m. is required between the HWL and the underside of the bridge structure in each of the navigation channels
- a nominal footpath is provided on each bank of the river
- the design life of the bridge is to be 120 years
- the bridge is designed for HA loading only
- the footpaths are designed to carry a load of 5 kN/m^2
- the bridge is designed in accordance with BS 5400

A site investigation consisting of three boreholes along the line of the bridge indicates that piled foundations are required and by using 750 mm diameter bored piles, each pile would be capable of supporting a *factored* axial force of 1000 kN.

Bridge Elevation (General)



FITTING INTO THE COUNTRYSIDE

This section deals with how to make the bridge 'look right' with the surrounding countryside since the aesthetics of the bridge is a major factor in the design.

Looking at existing bridges suggest that there are at least four factors that contribute to the success of a particular design:

1) The relationship of the bridge to the existing footpath, since the bridge can be seen as a continuation of the footpath, and since many bridges are set higher than the original ground level, a connection must be made either by steps or by a stepped ramp keeping changes in level as gradual as possible for comfort and convenience, or by raising the existing footpath up on an embankment.

2) Rich earthy colours which fit into the countryside can be used to paint the bridge.

3) The standard of workmanship is important.

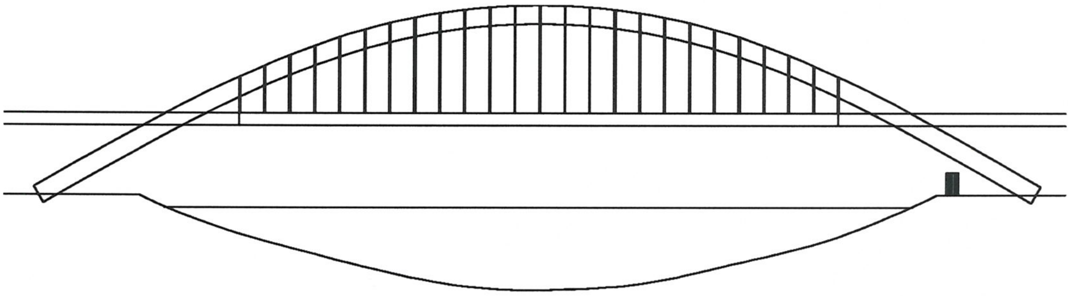
4) Building operations destroy some of the existing ground cover of a site. Restoration and replanting should therefore be allowed for.

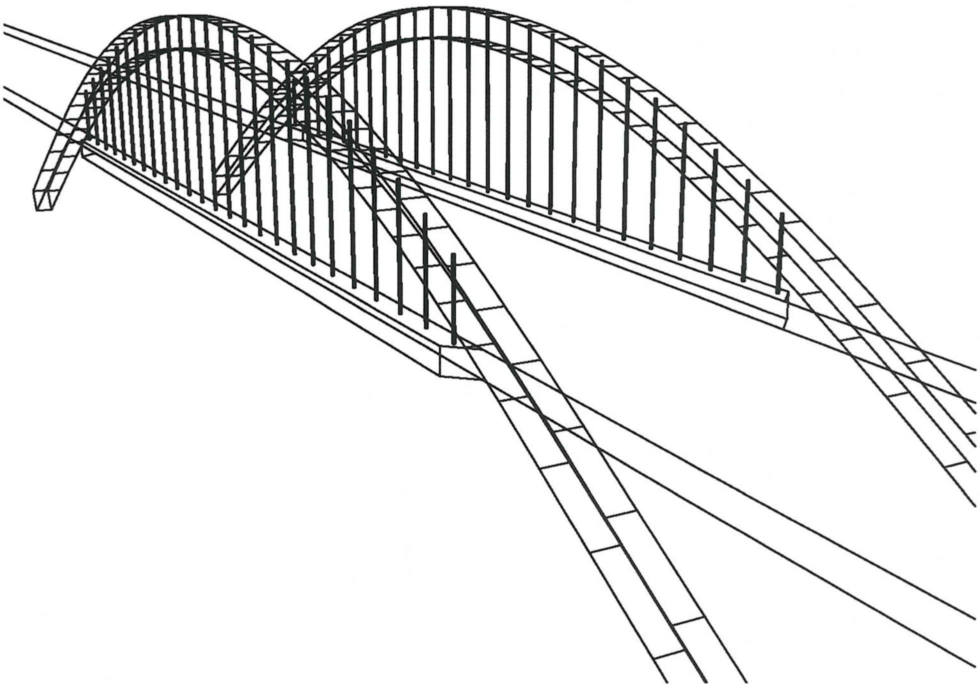
BRIDGE DESIGNS CONSIDERED

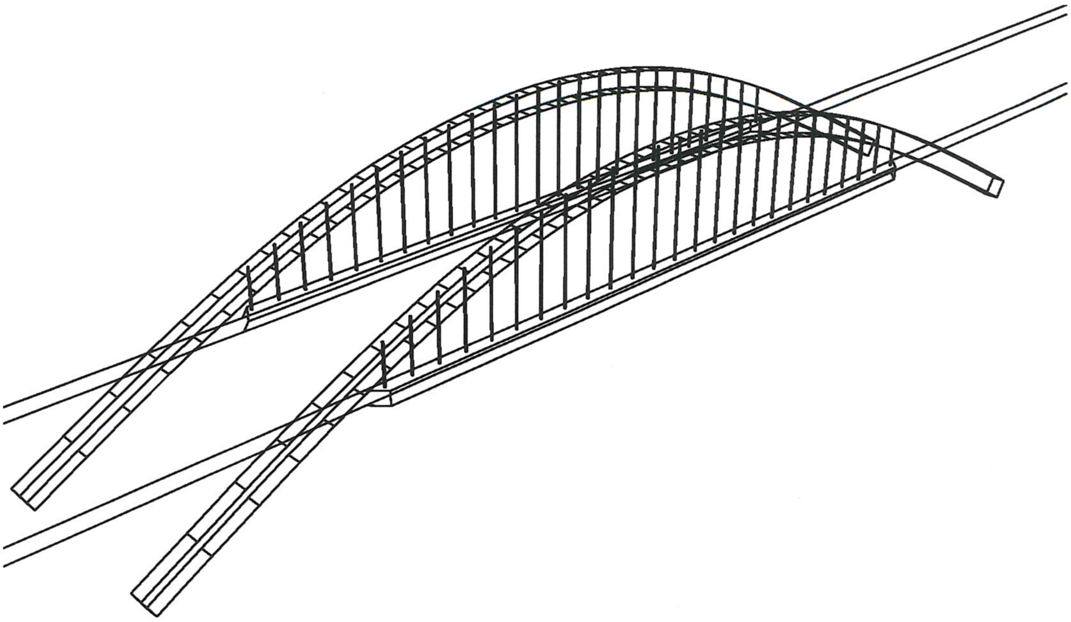
Throughout the design project, every aspect of the bridge was discussed by all three members of the group so as to be sure that every member was in agreement. In this way, we eliminated the possibility of a mistake or something being overlooked by any one member of the group and it also gave the opportunity for each group member to contribute to every discussion that took place. As a result of using teamwork rather than each group member individually, we believe that the final bridge design is much better than that would have otherwise been achieved.

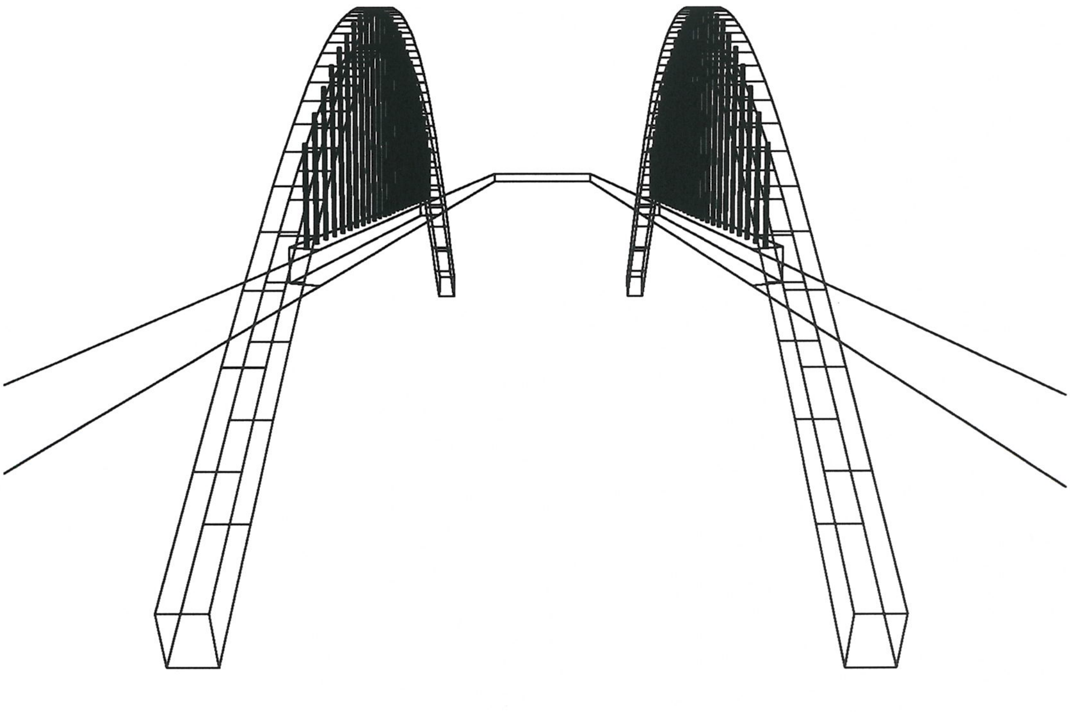
The initial design phase for the bridge was to consider the shape and type of bridge to be built in order to establish, in our minds, how the finished product would look like.

Three general bridge types were looked into including a cable-stayed, steel bowstring girder, and composite girder deck bridge. The design of the cable-stayed bridge is very complex with a very large degree of indeterminality. Wind bracing of the bridge would be required and careful attention would have to be paid to the cable anchorage points to avoid corrosion. This bridge design was discarded at quite an early stage. The second design involved a steel bowstring girder bridge not wholly unlike the design of the Sydney Harbour Bridge. Design sketches of this bridge are shown in the report. However, due to the complexity of design, major construction costs, and the insufficient time provided to complete the design calculations, a simple deck-type composite girder bridge with three spans has been designed with a low-profile aesthetic effect as compared to our original intended design (i.e. the bowstring bridge).









BRIDGE DESIGN

Bridge Details

The bridge span is to be 40 m and the bridge is to be simply supported. As specified in the design constraints each lane is to be 3.65 m wide, and so the carriageway width is to be 7.3 m.

The bridge is to be designed for HA loading only, which is a formula loading representing normal traffic in Great Britain.

From BS 5400 Part 2 Table 13, for a 40 m loaded length, the HA uniformly distributed load is 26.2 kN/m.

From BS 5400 Part 2 Clause 6.2.2, the nominal knife edge load (KEL) per notional lane is 120 kN.

The load on the footpath is to be 5 kN/m^2 as specified in the design brief.

Design of Carriageway Girder

There is no universal beam capable of carrying highway loading on the span of 40 m whilst complying with the requirements of BS 5400. A built-up girder will therefore be necessary and, for the design under consideration, a plate girder acting compositely with a concrete deck is both suitable and economical. Also on the grounds of economy the girder should be made as deep as possible and grade 50 steel is to be used. Before deciding on the size of the girder, however, it is necessary to calculate the loads to be carried by the bridge.

The determination of the amount of load carried by each longitudinal beam may most simply be made by assuming that the deck consists of a number of longitudinal strips. The load carried by each beam is then the static reaction from the strip edges which it supports. It will be apparent, however, that the effect of a concentrated load placed at some random point on the deck is not confined to the beams immediately adjacent to the load; the deck distributes the load over all beams and so reduces the static reaction. It is possible by more complex analytical techniques of load distribution to obtain a closer estimate of the actual bending moment in each beam.

Loading

Dead load, non-composite:

At this stage in the design, the dimensions of the steel plate girder are not as yet known (although an approximation can be made) so a value for its contribution to the dead load has been assumed as follows:

Assuming the density of concrete to be 24 kN/m^3, and the thickness of the concrete decking to be 225 mm,

dead load due to slab: $1.66 \times 0.225 \times 40 \times 24$ = 359 kN

For the plate girder, a weight of 5 kN/m is assumed and so the dead load is

$$5 \times 40 = 200 \text{ kN}$$

Therefore, total non-composite dead load = 559 kN

Dead load, composite:

Assuming the surfacing has the same density as concrete and its thickness is to be 100 mm,

$$\text{dead load} = 1.66 \times 0.100 \times 40 \times 24 = 159 \text{ kN}$$

Live load

HA loading

HA rate on 40 m loaded length = 26.2 kN/m of lane
KEL = 120 kN per lane

Load on an internal girder assuming simple static distribution is thus:

UDL: $26.2 \times 1.66/3.65$ = 11.92 kN/m
KEL: $120 \times 1.66/3.65$ = 54.58 kN

Bending Moments

Using WL/8,

Dead load:

non-composite:	559 × 40/8 = 2795 kNm
composite:	159 × 40/8 = 795 kNm

Live load (HA):

composite:	$11.92 \times 40^2/8$ = 2384 kNm
KEL at midspan:	54.58 × 40/4 = 546 kNm

∴ Total bending moment = 6520 kNm

Shear

The shear at any point is calculated from the influence line diagram for that point using the relevant intensity of live loading for the length of the positive or negative portion of the diagram.

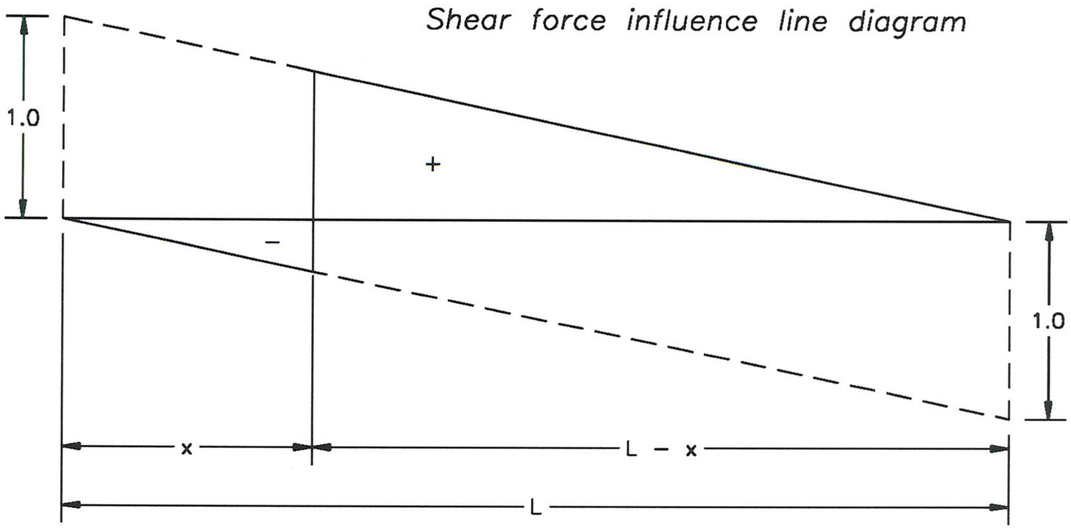
It is necessary to consider the various components of vertical shear separately when calculating the horizontal shear force on shear connectors because composite dead load shear is evaluated using section properties which are different from those of live load. Dead load produces a shear of a fixed sign at any given point; live load on the other hand produces, at points on the span other than the extreme ends, two values of shear of opposite sign (see shear force influence line diagram on next page). This fact must be taken into account when designing shear connectors as it is the shear range which is significant, unless there is no change in the sign of the combined shear forces.

Dead load:

Non-composite:	559/40 = 13.98 kN/m
Composite:	159/40 = 3.98 kN/m

Live load (HA):

UDL:	Loaded length up to 23.0 m:	31.5 × 1.66/3.65 = 14.33 kN/m
	Loaded length up to 30.0 m:	28.9 × 1.66/3.65 = 13.14 kN/m
	Loaded length up to 40.0 m:	25.7 × 1.66/3.65 = 11.69 kN/m
KEL:		120 × 1.66/3.65 = 54.58 kN



The shear forces at any point x from one end are represented by the areas of the positive and negative triangles in the diagram above and are tabulated below.

x (m)	Area of diagram		Ordinate	
	+	-	+	-
0	20.0	0.0	1.0	0.0
10	11.25	1.25	0.75	0.25
20	5.0	5.0	0.50	0.50
30	1.25	11.25	0.25	0.75
40	0.0	20.0	0.0	1.0

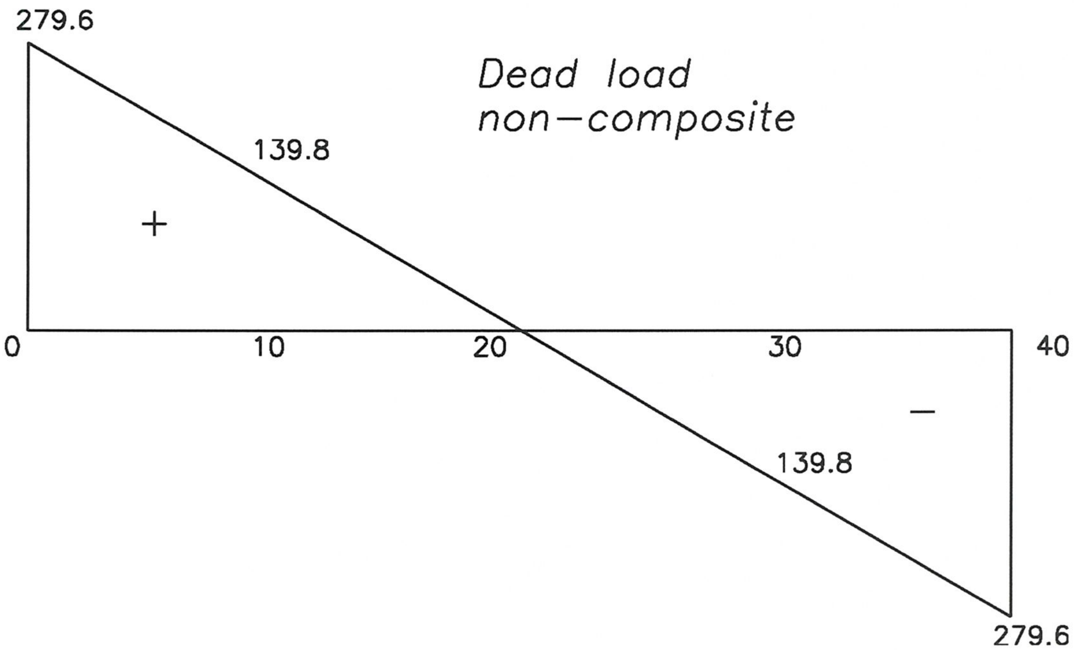
The UDL shear forces are determined by multiplying these areas by the load/m for the appropriate loaded length.

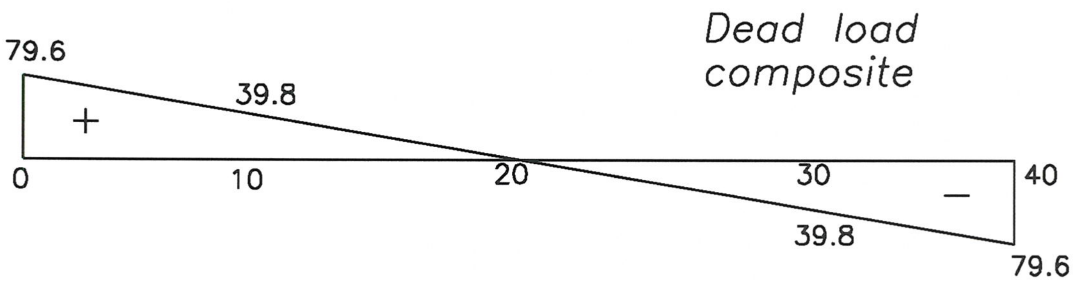
The KEL values are obtained by multiplying the ordinates of the influence line diagram by the KEL.

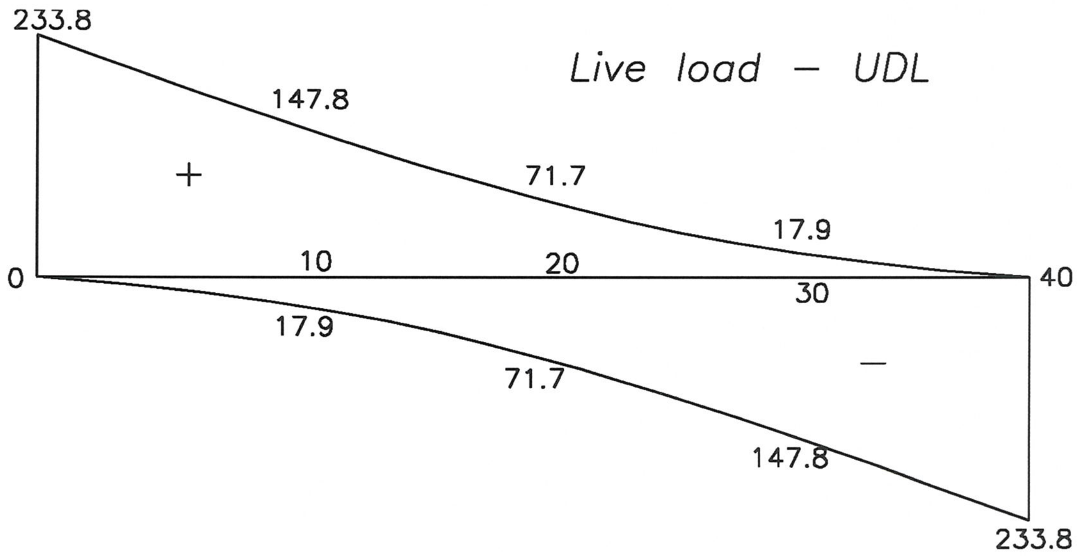
Vertical Shears (kN)								
x	Dead load				Live load			
	Non-composite		Composite		UDL		KEL	
(m)	+	-	+	-	+	-	+	-
0	279.6	0.0	79.6	0.0	233.8	0.0	54.6	0.0
10	139.8	0.0	39.8	0.0	147.8	17.9	40.9	13.6
20	0.0	0.0	0.0	0.0	71.7	71.7	27.3	27.3
30	0.0	139.8	0.0	39.8	17.9	147.8	13.6	40.9
40	0.0	279.6	0.0	79.6	0.0	233.8	0.0	54.6

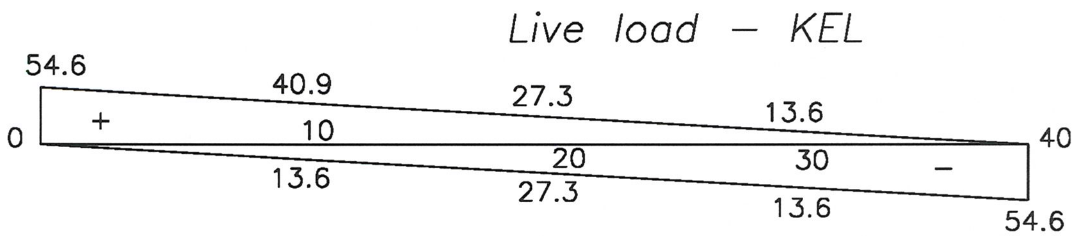
Maximum vertical shear force to be used in designing the girder web is:

$$279.6 + 79.6 + 233.8 + 54.6 = 647.6 \text{ kN}$$




Dead load
composite
79.6
39.8
+
0
10
20
30
40
−
39.8
79.6

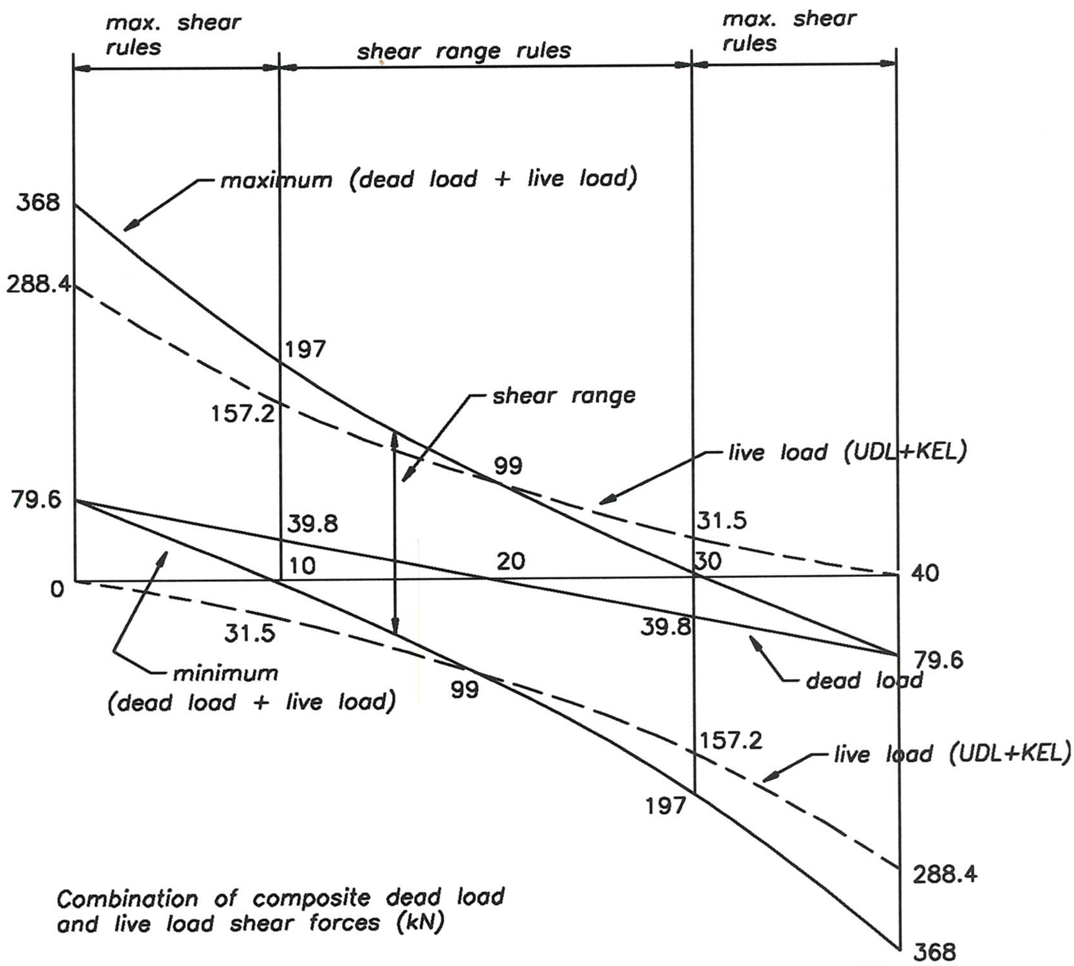

233.8
Live load − UDL
147.8
+
71.7
17.9
0
10
20
40
30
17.9
71.7
−
147.8
233.8


Live load − KEL
54.6
40.9
27.3
13.6
0
+
10
40
20
30
13.6
27.3
−
13.6
54.6

For shear connector design, shear forces due to composite loads only are considered and the value used is the maximum shear or the maximum shear range, whichever is the greater.

Vertical shear forces for shear connector design (kN)				
x (m)	DL comp.	LL	Design values DL	Design values LL
0	+79.6	233.8 + 54.6	79.6	288.4
10	+39.8	147.8 + 40.9 - 17.9 - 13.6	39.8	+75.4
20	+0.0	71.7 + 27.3 - 71.7 - 27.3	+0.0	198*

* Shear range is design criterion



Combination of composite dead load and live load shear forces (kN)

Initial Estimate of Steel Girder Size

For a composite girder in which the concrete deck acts as an extra top flange, an unsymmetrical steel girder is appropriate in which the top flange is smaller than the bottom one. The role of the steel top flange is as much to carry the shear connectors as to resist bending.

For a symmetrical plate girder without composite action it can be shown that the most economical arrangement is for each flange to have an area equal to half that of the web. The same argument cannot be applied to a composite girder but it will give some direction to the designer's first estimate.

For economy, the girder should be as deep as possible but the depth depends on several criteria including the constraints of the site and aesthetics, and the web depth/thickness (d/t) ratio which is limited to ensure that the web will not buckle prematurely. In practice, provided headroom permits, it is found that a span/depth ratio of about 20 is satisfactory and this will be adopted here. The steel girder will be about 2 m deep overall. The thickness of the web can be determined from the limiting d/t ratio which for grade 50 steel is 75 for unstiffened webs and 180 for webs with vertical stiffeners. Although there is more fabrication involved in applying stiffeners, the saving in steel which results from using a thinner web generally offsets the additional cost. The girder will therefore be designed with a vertically stiffened web.

In order to obtain a horizontal carriageway across the whole length of the bridge, the total depth of the bridge must be 1.5 m. Taking the depth of the road surface to be 100 mm and the depth of the concrete slab to be 225 mm, the depth of the steel beams can be calculated as,

$$1500 \text{ mm} - 100 \text{ mm} - 225 \text{ mm} = 1175 \text{ mm}$$

The d/t ratio is 1175/180 = 6.53 mm

The standard thickness of plate of 10 mm is to be used, as we are trying to increase the steel area of the beam, whilst limiting the beam depth to 1175 mm. For this reason, a value of 6.53 mm for web thickness is increased in our design.

Assume a web of 1175 mm × 10 mm which gives an area of 117.5 cm^2

For a symmetrical girder, the total area would be 235 cm^2 plus an allowance of 100 cm^2 for stiffeners. Assuming that the mass of the steel is 78 kN/m^3 the girder mass would be

$$(235 + 100)10^{-4} \times 40 \times 78 = 104.52 \text{ kN}$$

In the initial calculations for the dead load, 200 kN was assumed so the resulting bending moments and shears are slightly overestimated.

In this example the girder is to be unsymmetrical as it acts compositely with the deck slab. A method for assessing the flange areas required in these circumstances is as follows:

Divide the total applied bending moment by the web depth and the allowable stress in bending. The resulting area when reduced by 10% gives a trial tension flange area. The trial compression flange is then assumed to have an area of half this value. When the stresses in the initial trial section have been determined, the assumed sizes can be suitably modified to bring the working stresses as close as possible to the maximum allowable stresses.

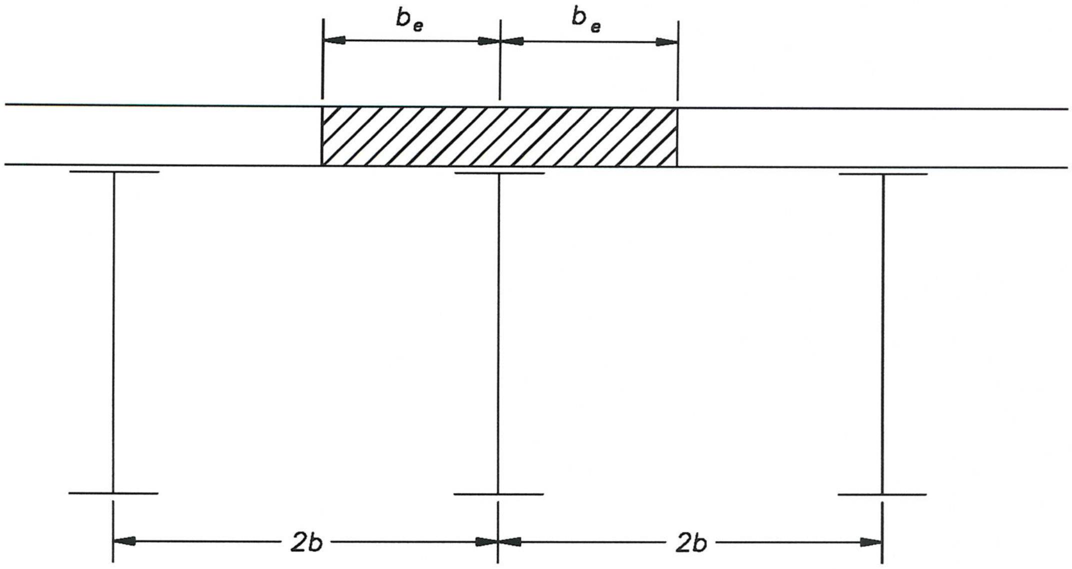
Effective width:

A composite beam, consisting of a steel girder with a concrete top flange, does not always fall into the class of beam which can be analyzed by the simple bending theory that sections plane before bending remain so after the load has been applied. The cause is the relatively large width of the concrete flange. To take account of the resulting non-uniform longitudinal stress distribution the actual width of concrete $2b$ between beams is reduced to an effective flange width $2b_e$ whenever b is more than one twentieth of the beam's span L by the formula:

$$\frac{b_e}{b} = \frac{1}{\sqrt{1 + 12\left(\frac{2b}{L}\right)^2}}$$

provided the resulting value of b_e is not less than $L/20$.

When $b \leq L/20$, b_e is taken as equal to b.



In the example under consideration, the girders are at 1.66 m centres.

$\therefore$ $b = 0.83$ and $b/L = 0.83/40 < 1/20$

$\therefore$ $b_e = b$ and the effective slab width = 1.66 m

Modular ratio:

In order to determine the stresses in a composite beam it is necessary to fix the value of the ratio of the moduli of elasticity of steel and concrete. Because the modulus for the latter is dependent on the duration of loading two modular ratios are specified; one for transient loads of short duration and another to take account of long-term loading and the creep of concrete caused by it. The modular ratios depend on the concrete cube strength and the values adopted are:

For permanent loads, $m = 83/\sqrt{\text{cube strength}}$

For transient loads, $m = 41.5/\sqrt{\text{cube strength}}$

where the cube strength is in N/mm^2.

Hence, for concrete having a 28 day cube strength of 40 N/mm^2,

$m = 83/\sqrt{40} \simeq 13.2$ for permanent loads
$m = 41.5/\sqrt{40} \simeq 6.6$ for transient loads

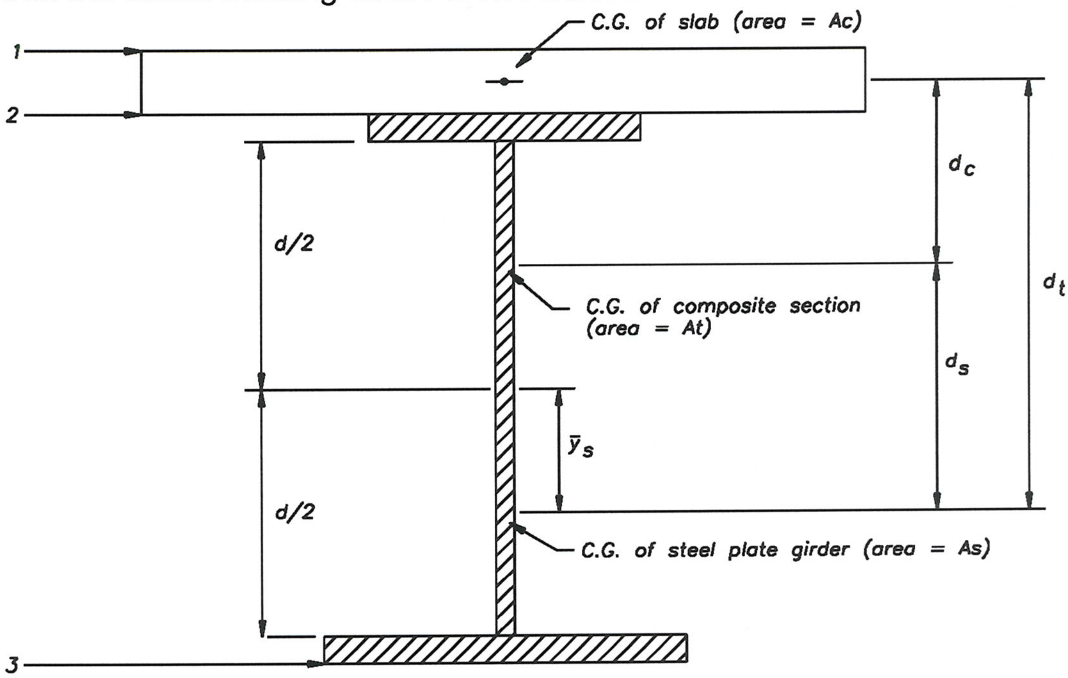
Allowable stresses:

From BS 4360, Table 7

Steel, grades 50B and 50C					
Thickness (mm)	Yield stress (N/mm^2)	Bending stress (N/mm^2)	Shear stress (N/mm^2) Max	Shear stress (N/mm^2) Min	Bearing stress (N/mm^2)
Up to 16	355	209	154	131	280
Over 16 and up to 40	345	203			

For concrete with a 28 day works cube strength of 30 N/mm^2, the allowable compressive bending stress is 13.3 N/mm^2.

The bending stresses for steel given above are maxima and assume lateral stability of the compression flange. They will apply when the deck and girders are acting compositely but lower values may apply during construction and temporary bracing may be needed to provide restraints so that the actual bending stress is not excessive.



Notation for section properties (see previous figure):

A_c	=	cross-sectional area of slab
A_s	=	cross-sectional area of plate girder
A_t	=	cross-sectional area of composite girder
I_c	=	moment of inertia of slab about major axis – in terms of steel
I_s	=	moment of inertia of plate girder about major axis
I_t	=	moment of inertia of composite section about major axis
	=	$I_c + I_s + d_c^2 A_c/m + A_s d_s^2$
Z_{s2}	=	elastic modulus of plate girder about its top
Z_{s3}	=	elastic modulus of plate girder about its bottom
Z_{t1}	=	elastic modulus of composite section about top of slab
Z_{t2}	=	elastic modulus of composite section about top of plate girder
Z_{t3}	=	elastic modulus of composite section about bottom of plate girder
d_c	=	distance between centroids of slab and composite section
	=	$A_s d_t / A_t$
d_s	=	distance between centroids of plate girder and composite section
d_t	=	$d_c + d_s$
S_t	=	$d_c A_c / m$

Trial Size for Plate Girder

Total applied bending moment = 6520 kNm

Allowable stress (flanges between 16 and 40 mm) = 203 N/mm^2

Area of tension flange
= $0.9 \times 6520 \times 10^6 \div 1175 \times 203$ = 2.46×10^4 mm^2

Area of top flange = $0.5 \times 2.46 \times 10^4$ = 1.23×10^4 mm^2

Try top flange of 400 mm × 40 mm = 1.6×10^4 mm^2

and bottom flange, 800 mm × 40 mm = 3.2×10^4 mm^2

Web area = 1095 × 10 = 1.095×10^4 mm^2

$A_s = (1.095 + 3.2 + 1.6)\ 10^4$ = 5.895×10^4 mm^2

I_s = 1.516×10^{10} mm^4

top Z = 2.043×10^7 mm^3

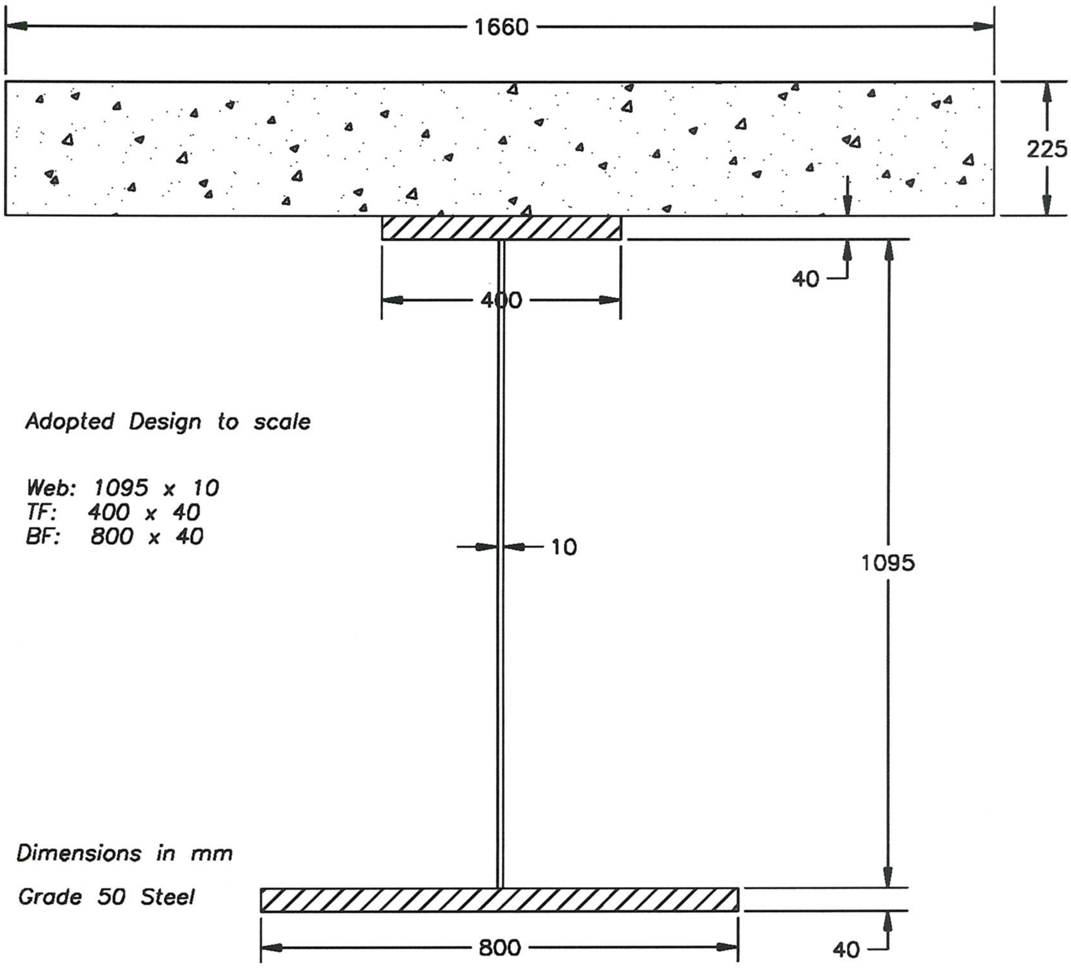
bottom Z = 3.501×10^7 mm^3

		$m = 7.5$	$m = 1.5$
S_t	=	2.307×10^7 mm^3	1.496×10^7 mm^3
I_t	=	3.487×10^{10} mm^4	2.794×10^{10} mm^4
d_s	=	391.3 mm	253.8 mm
d_c	=	463.2 mm	600.7 mm
d_t	=	854.5 mm	854.5 mm
Z_{t1}	=	6.057×10^7 mm^3	3.918×10^7 mm^3
Z_{t2}	=	9.943×10^7 mm^3	5.723×10^7 mm^3
Z_{t3}	=	4.23×10^7 mm^3	4.068×10^7 mm^3

Bending stresses

The tabulated values below have been calculated by dividing the appropriate bending moments by the elastic modulus concerned, remembering that for concrete stresses the elastic moduli, which are in terms of steel, must be multiplied by the modular ratio (7.5 for live load 15 for dead load).

Material and level	Bending stress (N/mm^2) due to			
	DL non-comp.	DL composite	Live load	Total
Concrete 1	-	1.35	6.45	7.80
Concrete 2	-	0.93	3.93	4.86
Steel 2	48.84	13.89	29.47	92.20
Steel 3	68.71	19.54	69.27	157.52



Temperature stresses:

Stresses due to differential temperature between concrete slab and steel girder are as follows:

Level 1: top of concrete slab
Level 2: bottom of concrete slab
Level 3: bottom of steel girder

At level 1:

$$f_{c1} = -\frac{Q_c}{A_c} + \frac{M_c}{Z_{c1}}$$

At level 2:

$$f_{c2} = -\frac{Q_c}{A_c} - \frac{M_c}{Z_{c2}}$$

At level 2:

$$f_{s2} = \frac{Q_s}{A_s} + \frac{M_s}{Z_{s2}}$$

At level 3:

$$f_{s3} = \frac{Q_s}{A_s} - \frac{M_s}{Z_{s3}}$$

Where

$$Q_c = Q_s = A'\beta T E_s \left(\frac{I_c}{m} + I_s \right)$$

$$M_c = A'\beta T d_t E_s \frac{I_c}{m}$$

$$M_s = A'\beta T d_t E_s I_s$$

$$\frac{1}{A'} = d_t^2 + \left(\frac{I_c}{m} + I_s \right)\left(\frac{m}{A_c} + \frac{1}{A_s} \right)$$

β = linear coefficient of expansion of steel = 1.1×10^{-5}/°C

T = temperature differential between slab and beam = 10°C

E_s = elastic modulus of steel = 205×10^3 N/mm^2

I_c = moment of inertia of concrete alone = 1.8×10^9 mm^4

I_s = moment of inertia of steel beam alone = 1.516×10^{10} mm^4

A_c = area of concrete slab = 3.735×10^5 mm^2

A_s = area of steel beam = 5.895×10^4 mm^2

m = modular ratio = 7.5

d_t = distance between centroids of slab and steel beam = 854.5 mm

$$A' = \frac{1}{854.5^2 + \left(\dfrac{1.58\times10^9}{7.5} + 1.516\times10^{10}\right)\left(\dfrac{7.5}{3.735\times10^5} + \dfrac{1}{5.895\times10^4}\right)} = 7.7\times10^{-7}\text{mm}^{-2}$$

Z_{c1}; Z_{c2} = moduli for top and bottom of slab alone

Z_{s2}; Z_{s3} = moduli for top and bottom of steel beam alone

$$M_c = (7.7\times10^{-7})(1.1\times10^{-5})(10)(854.5)(205\times10^3)\frac{1.58\times10^9}{7.5} = 3.126 \text{ kNm}$$

$$M_s = (7.7\times10^{-7})(1.1\times10^{-5})(10)(854.5)(205\times10^3)(1.516\times10^{10}) = 224.93 \text{ kNm}$$

$Q_c = Q_s =$

$$(7.7\times10^{-7})(1.1\times10^{-5})(10)(205\times10^3)\left[\frac{1.58\times10^9}{7.5} + 1.516\times10^{10}\right] = 266.89 \text{ kN}$$

$$Z_{c1} = Z_{c2} = \frac{I_c}{\text{(depth of slab)/2}} = \frac{1.58\times10^9}{112.5} = 1.4\times10^7 \text{ mm}^3$$

top $Z = Z_{s2} = 2.043 \times 10^7$ mm^3

bottom $Z = Z_{s3} = 3.501 \times 10^7$ mm^3

$$f_{c1} = -\frac{266.89}{3.735 \times 10^5} + \frac{3126}{1.4 \times 10^7} = -0.491 \text{ N/mm}^2$$

$$f_{c2} = -\frac{266.89}{3.735 \times 10^5} - \frac{3126}{1.4 \times 10^7} = -0.938 \text{ N/mm}^2$$

$$f_{s2} = \frac{266.89}{5.895 \times 10^4} + \frac{224.93 \times 1000}{2.043 \times 10^7} = 15.54 \text{ N/mm}^2$$

$$f_{s3} = \frac{266.89}{5.895 \times 10^4} - \frac{224.93 \times 1000}{3.501 \times 10^7} = -1.9 \text{ N/mm}^2$$

(Negative stress is tension and positive, compression)

When temperature effects are included the basic working stress may be increased for combination 2 by 25%. Inspection of the bending stresses tabulated above shows that the temperature stresses fall within this allowance and may therefore be ignored.

Stiffeners

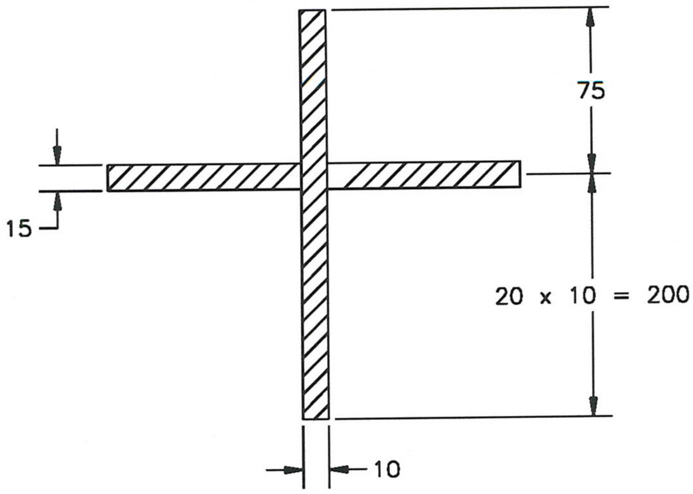
Web stiffeners:

These will be designed using the same grade of steel as the main girders, grade 50 steel. In addition to the vertical stiffeners required to stiffen the web, it is necessary to provide load bearing stiffeners at the girder supports. These transmit the reaction force through the flange into the web.

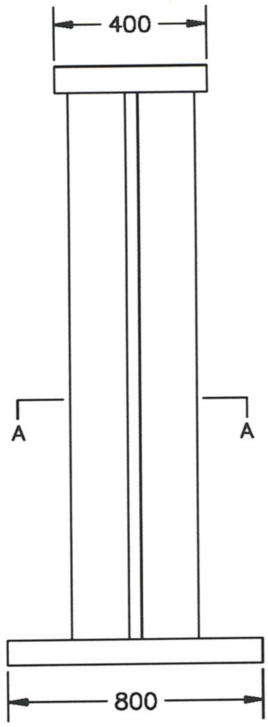
Intermediate stiffeners do not require to be attached to the flanges but normal practice is to weld them to the compression flange and stop them short of the tension flange. It is also usual to provide intermediate stiffeners on one side only of the web.

Design of load bearing stiffeners:

Assuming that the centre of the bridge bearing is to be 75 mm from the end of the girders, the centreline of the stiffeners will be at this position and the arrangement is shown in the figure on the next page.



Section AA



The width of the stiffeners, chosen to come within the width of the top flange will be taken as 150 mm.

The thickness of the stiffeners must be such that the bearing stress on the bottom flange does not exceed the allowable value of 280 N/mm^2.

Maximum vertical reaction = 647.6 kN

$$\therefore \text{ thickness required } = \frac{647.6 \times 10^3}{2(150 - 10)280} = 8.26 \text{ mm}$$

The thickness, *t*, must also satisfy the requirement that the outstand from the web must not exceed 12*t*.

$$\text{i.e.} \quad t \not< \frac{150}{10} = 15 \text{ mm}$$

$\therefore$ a thickness of 15 mm will be used.

To check for buckling, the stiffeners together with that part of the web shaded in the above figure are treated as a strut with effective length of 0.7 times the length of the stiffeners.

$$I_{xx} = \left(\frac{15 \times (150 + 150 + 10)}{12} - \frac{15 \times 10^3}{12}\right) + \frac{(75 + 200)\,10^3}{12} = 22.05 \times 10^6 \text{ mm}^4$$

$A = (300 \times 15) + (315 \times 10) = 7650 \text{ mm}^2$

$$\therefore\ r_{xx} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{22.05 \times 10^6}{7650}} = 53.69 \text{ mm}$$

effective length, $l = 0.7 \times 1175 = 822.5$ mm

$$\therefore\ \frac{l}{r_{xx}} = \frac{822.5}{53.69} = 15.32$$

and allowable axial stress = 196.26 N/mm^2

actual axial stress = $(647.6 \times 10^3)/7650 = 84.65$ N/mm^2

$\therefore$ flat stiffeners 150 × 15 are satisfactory in all respects.

Design of intermediate stiffeners:

depth of web, $d_1 = 1175$ mm

maximum spacing, S, of stiffeners = $1.5d_1$ = 1762.5 mm

The maximum size of stiffened panel of web is thus 1175mm × 1762.5mm and the minimum thickness, t, of the web required for such a panel is:

$1/180 \times 1175 = 6.53$ mm or $1/270 \times 1762.5 = 6.53$ mm

6.53 mm is therefore the web thickness to be used in designing the stiffeners.

$$I_{min} = 1.5\frac{d_1^3 t^3}{S^2} = 1.5\frac{1175^3 \times 6.53^3}{1762.5^2} = 2.18 \times 10^5 \text{ mm}^4$$

Assuming a flat stiffener 50 mm wide and t_s thick,

$$I = 2.18 \times 10^5 = \frac{t_s \times 50^3}{3}$$

(I = moment of inertia of stiffener about face of web)

$$\text{hence} \quad t_s = \frac{2.18 \times 10^5 \times 3}{50^3} = 5.232 \text{ mm}$$

The outstand of the stiffener must not exceed 12 × thickness

$$\text{i.e.} \quad t_s \nless \frac{50}{10} = 5 \text{ mm}$$

$\therefore$ use 5 mm thick flats

CROSS–SECTION