*Discussion on Paper No. 6153**

Composite steel and concrete bridge construction in Southern Rhodesia

by

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and

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Dr Goschy Béla (Budapest) welcomed the Authors' attempt to describe modern design standards for composite bridge constructions, despite the existence of Codes e.g. DIN 1078, C.P. 117, etc. He had only two comments to make.

98. First, the computation of the ultimate moment was based on the assumption of pure bending. Modern interpretation of the problem of ultimate strength should be deduced from the failure condition of the composite section under combined bending and shear.

99. Considering

$$\left(\frac{f}{f_y}\right)^2+\left(\frac{v}{v_y}\right)^2 = 1; \quad v_y = \sqrt{3f_y}$$

yielding condition for steel and

$$\left(\frac{u}{u_u}\right)^2+\left(\frac{v_c}{v_{cu}}\right)^2 = 1; \quad u_u = 0{\cdot}667u_w, \quad v_{cu} = 0{\cdot}08u_w$$

rupture condition for concrete; the moment-shear interaction function could be written in the general form as:

$$C_1\left(\frac{M}{M_{uo}}\right)^\alpha+C_2\left(\frac{V}{V_{uo}}\right)^\beta = 1$$

where

M_{uo}, V_{uo} are the ultimate moment and shear respectively in pure bending and shear,

M, V are the ultimate moment and shear respectively in combined bending and shear,

α, β are constants, and

C_1, C_2 are shape factors.

100. Safety against failures should be checked as follows: external moment taken for

$$M_u = 1{\cdot}5\sum M_{dl}+2{\cdot}5\sum M_{ll} = M;$$

ultimate vertical shear should be greater than or equal to

$$V \geqq V_u = 1{\cdot}5\sum V_{dl}+2{\cdot}5\sum V_{ll}.$$

101. Second, the deduction of shear at the interface at ultimate moment (§ 72) was based on the principles of the theory of elasticity by assuming that the shape of the compression block in the plastic region remained square and only the stress intensity was changed. That was contradictory to the principles of the theory of plasticity adopted and also to Figs 7c and 7d. So long as the interface was in the

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* *Proc. Instn civ. Engrs*, vol. 21, April 1962, pp. 785–810.

plastic stress region (Fig. 21) there was no shear at the interface; furthermore, shear region in plastic stage was pushed towards supports and total horizontal shear force

$$F = 0{\cdot}667u_w b_s t_s$$

had to be carried by the connectors placed between support and point B (Fig. 21).

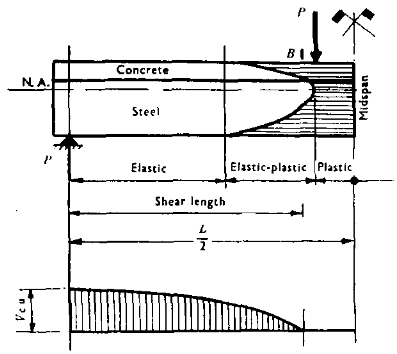


FIG. 21: SHEAR-STRESS DISTRIBUTION AT THE INTERFACE

Mr P. L. Easterbrook (Senior Executive Engineer, Design and Research Branch, Ministry of Works and Transport, Western Nigeria) was very interested in the Paper because composite steel and concrete bridges had recently been introduced into Western Nigeria and many of the problems associated with this form of construction had been discussed. The Western Nigeria Government had introduced a series of standard composite-action bridges using universal beams and they covered the range from 30 ft to 80 ft span. Several welded-plate girder spans of composite construction had also been designed for spans up to 120 ft. With all spans a general saving in construction costs of up to 20% had been made over the usual non-composite steel and concrete construction.

103. It was noted that the bridges in Southern Rhodesia had been designed for H.A. loading to B.S.153. Consideration was now being given in Western Nigeria to designing to the H20–S16 loading of the A.A.S.H.O. specification. A study of vehicles at present in Nigeria and of the laws affecting import of future vehicles had shown that H.A. loading was very conservative, especially for spans above about 60 ft. For spans below this figure, the difference between the two loadings was small and the difference in total cost of a bridge even smaller. Above 60 ft, however, the difference in loadings became more marked and needed careful investigation. A recent study by the Road Research Laboratory had shown that axle loads of vehicles in Southern Rhodesia were similar to those in use in Western Nigeria. The writer would like the Authors' comments on this question of design loadings.

104. It was fortunate that in Western Nigeria there was a prestressed concrete factory, manufacturing precast, prestressed, bridge-beam units, as well as a steel fabricating workshop. This led to highly competitive prices for the two forms of construction. The criteria of erection facilities and transportation costs usually governed the choice of the form of construction. With bridge sites near the prestressed concrete factory and with good erection facilities the prestressed-concrete spans were usually cheaper. Where long haulage distances were involved and erection facilities were not good the more robust and lighter steel beams were used.

105. The Authors suggested that the concrete haunches should extend 1½ in.

beyond the edge of the steel flange. It would be interesting to know the reason for this as it only seemed to be an unnecessary complication of the shuttering.

106. No comment was made in the Paper on the relative merits of the different types of shear connectors. Channel shear connectors were at present being used in Western Nigeria mainly because no facilities for stud welding were available locally and the spiral type seemed too delicate for use on remote sites where the beams might have rough usage.

107. Where the concrete deck of large span bridges had to be poured in several sections the final dead load stresses would obviously depend upon the sequence of casting the sections of the concrete deck. Had the Authors considered it necessary to specify any definite sequence of casting the deck?

Mr S. D. Gogoi, Mr J. S. Teraszkiewicz, and Dr J. C. Chapman (Imperial College, London) observed that the Authors stated that complete interaction should be maintained until the ultimate moment was reached, and this was the basis of the A.A.S.H.O. specification which defined a useful capacity of shear connector as that load for which the slip was sufficiently small. In the case of many types of shear connector this restriction would result in less than half the ultimate shear connector capacity being mobilized when the ultimate moment of the beam is reached.

109. Experiments at Lehigh University and at Imperial College had shown that, even where the shear connectors were designed to reach their full ultimate capacity when the beam reached its ultimate moment, the fully plastic moment of the beam was still developed, although by this stage considerable slip had taken place. It appeared therefore that the A.A.S.H.O. specification was unduly conservative in respect of static loading, and this was recognized in the new A.I.S.C. specification for steelwork in buildings in which the shear connectors were allowed to carry 80% of their ultimate capacity at ultimate moment.

110. The presence of the flange plate in Fig. 19 raised an interesting question. The flange plate acted as a very rigid shear connector which was juxtaposed to the relatively flexible shear connectors, which therefore did not develop any shear and acted only as vertical anchors in this region. It would be interesting to know whether the presence of this large concentrated shear force had any adverse effects on the performance of the slab in practice.

111. In § 72 the Authors derived an expression for the interface shear in the plastic region. In this derivation two sections δx apart, subjected to moments M and $M+\delta M$, were considered and the stress distributions at both sections were assumed to be rectangular. This was inconsistent with the trapezoidal stress strain curve

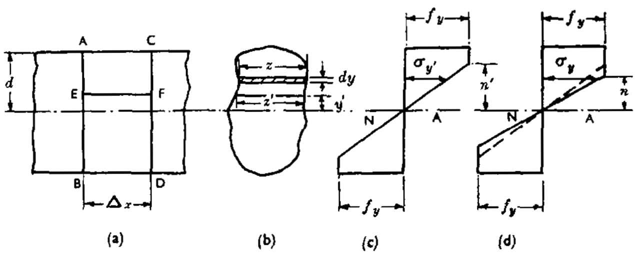


FIG. 22

f_y = Yield stress
q = Horizontal shear stress
z' = Width of section at y' from N.A. where shear stress is q
z = Width of section at a distance y from N.A.

(which resulted in a rectangular stress block), since for equilibrium the stress magnitude at the extreme fibre could not then be constant.

112. An expression for the horizontal shear in the elasto-plastic region of a homogeneous beam, where the stress strain curve is trapezoidal, could be obtained in the following way (see Fig. 22):

$$qz'\Delta x = \int_{n}^{n'} z(f_y - \sigma_y)dy + \int_{y'}^{n} z(\sigma_y - \sigma_{y'})\,dy \quad . \quad . \quad . \quad . \quad . \quad (1)$$

113. This equation was easily solved for a rectangular section but in the case of the T-beam the equation did not lead to a simple expression for q. In the composite beam, where two different stress strain curves were involved the analytical expressions became much more complicated, and the integrations had to be performed numerically.

114. The shear flow at the interface did not vary linearly along the beam, and in the particular case of a central point load, and with the neutral axis coincident with the interface, the shear at the centre tended to infinity. The Authors' suggestion of calculating interface shear by linear interpolation between the end and centre sections might therefore lead to serious error.

115. The following methods could be used for calculating the shear flow along the length of the beam at failure:

(*a*) Calculate the interface shear in the inelastic region by numerical integration, and in the elastic region by the elastic equation. The area of this shear diagram was the total horizontal shear and was equal to the total axial force in the steel section at the point of maximum moment.

(*b*) Calculate the interface shear in the elastic zone from the elastic equation, and in the elasto-plastic zone assume a trapezoidal diagram such that the total area of the shear diagram was equal to the total axial force in the steel section.

(*c*) A rectangular shear diagram could be assumed such that the total area was equal to the total axial force in the steel section. The three alternative methods were illustrated in Fig. 23.

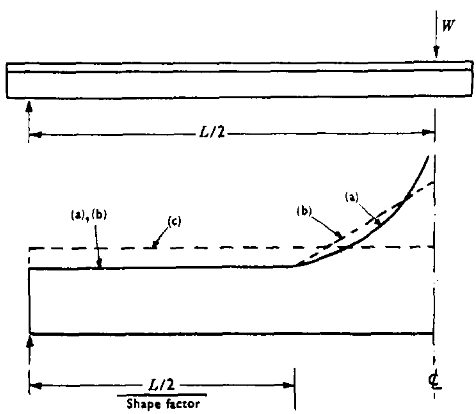


FIG. 23

116. The total number of shear connectors had to be sufficient to ensure development of the fully plastic moment. The spacing might be according to the elastic shear diagram, according to the ultimate shear diagram, or, for example, uniform.

Experiments at Imperial College on beams with a central point load did not show any significant difference in behaviour between beams with shear connectors spaced according to the first two alternatives. Experiments on beams with uniformly distributed loading showed little difference between the first and third alternatives.

The Authors, in reply, thanked Messrs Gogoi, Teraszkiewicz, Dr Chapman and Dr Béla for drawing their attention to the incorrect derivation of the formula for interface shear at Ultimate Moment. *Although, for the sake of simplicity, the* Authors agreed with the assumption that concrete behaved as an ideal plastic material,[41,42] they considered that the possibility of interface slip could only be neglected in the case of a shear connexion provided by rigid connectors, an artificial bond such as epoxy resin, or chequered top flange-plates acting in conjunction with mechanical anchors to resist vertical lifting of the slab.[43]

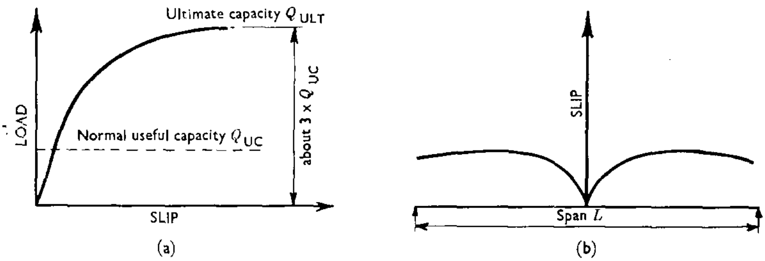


FIG. 24: (a) LOAD-SLIP CHARACTERISTIC OF FLEXIBLE CONNECTOR; (b) TYPICAL INTERFACE SLIP ALONG LENGTH OF BEAM

118. In the case of flexible shear connectors it had recently been found that under a gradually increasing load the natural bond at the steel-concrete interface was broken at about the design load.[44–47]. The relatively small amount of slip which occurred at this stage increased as the load was increased until at failure the differential movement at the end was of the order of 0·1 in. At the same time the connectors behaved in the manner indicated in Fig. 24 where the ultimate strength was approximately three times that given by A.A.S.H.O. regulations.[52] The compressive force in the concrete slab was thus distributed more or less equally between the connectors spaced between the support and point of maximum moment. This theory formed the basis of the new A.I.S.C. regulations which required that the number of shear connectors working near their ultimate capacity should be sufficient to ensure an ultimate moment equal to twice the sum of the dead load and the live load moments. These regulations also recommended that the shear connectors be spaced evenly along the beam.

119. While this provided a ready method for the design of shear connectors in composite beams subjected to static loads, it would be premature to apply them to bridges carrying dynamic loads because of fatigue considerations.

120. Once the natural bond had been broken, all the interface shear had to be carried by mechanical anchorages. It had been shown that provided the end slip was kept below 0·003 in. there was practically no loss of composite action.[48,49] At normal working loads, therefore, the interface shear could be determined on an elastic basis, and the connectors given a variable spacing. It had been demonstrated in small scale tests on evenly spaced connectors that those nearest the support were the first to yield.[48] Beside this, at least one static test at Lehigh had indicated that variable spacing gave a slightly improved performance.[47]

121. In view of this, the interface shear under normal working conditions should be computed on an elastic basis at all points using the Useful Capacity of Shear

Connectors and Load Factor given by $V_{ult.}$ in § 35. At ultimate moment the number of connectors, working at their ultimate capacity within the shear length L_s, should be sufficient to resist the compressive force in the concrete slab at midspan: viz.

$$\sum_{x=0}^{x=0\cdot5L} N.Q_{ult.} = \tfrac{2}{3}\, u_w.b_s.c. \quad (1a) \text{ Neutral Axis in Concrete}$$

$$\sum_{x=0}^{x=0\cdot4L} N.Q_{ult.} = \tfrac{2}{3}\, u_w.b_s.t_s \quad (1b) \text{ Neutral Axis in Steel}$$

where N, u_w, b_s, c were as given previously on pp. 797–798 of the Paper. $Q_{ult.}$ was the ultimate capacity of connectors,[48] see Table 1.

TABLE 1

Connector	Ultimate capacity
Channels	$3{\cdot}06\ Q_{uc}$
Spirals	$2{\cdot}09\ Q_{uc}$
Headed and L-Studs ($h/d \geqslant 4{\cdot}2$)	$2{\cdot}82\ Q_{uc}$
($h/d < 4{\cdot}2$)	$2{\cdot}75\ Q_{uc}$

122. This would ensure that in the event of collapse the shear connectors would not snap off suddenly, and failure due to crushing of concrete or buckling and yielding of the steel girder would be gradual.

123. When the neutral axis lay within the steel section the shear length L_s had been reduced to 0·4 L for the following considerations.

124. In the case of simply-supported spans the Knife Edge Load maximum bending moment envelope could be replaced by an equivalent uniformly distributed load w_e

where $$w_e = \frac{2P}{L}$$

P = Knife Edge Load

Hence $$M_{ult.} \nless 1{\cdot}5\ M_{dl} + 2{\cdot}5 M_{ll} \text{ became}$$

$$M_{ult.} \nless \frac{L^2}{8}\{1{\cdot}5\ w_{dl} + 2{\cdot}5\ w_{ll} + 2{\cdot}5\ w_e\}$$

and the maximum stress envelope for any position of the Knife Edge Load could be obtained directly from consideration of a uniform load. The length of flange stressed beyond the elastic limit was given by the formula

$$\text{Plastic Length,} \quad L_P = \frac{L}{2}\sqrt{1-\frac{1}{\eta}} \quad \text{for U.D.L.}$$

where η was the shape factor referred to the lower steel flange and varied from 1·3 to 1·6.

125. The distance R could be approximated from L_P using the ratio of yield stress/working stress of steel, to that for concrete

$$\text{i.e.} \quad R \approx \frac{1{\cdot}7}{2{\cdot}0} L_P$$

126. The position of point B in Fig. 25 could then be approximated by linear interpolation. However, since the depth c' of the neutral axis below the interface was seldom large, the shear length was underestimated if a value of $L_s = 0{\cdot}8\ L/2$ was used.

127. As regards the splice detail indicated in Fig. 19, one of three things could happen:

(*a*) the concrete immediately in front of the splice plate and bolt heads would crush; the shear load would then be carried by the connectors;

(*b*) the horizontal shear force on the plate, bolts and shear connectors would exceed the frictional slip resistance of the splice which would then move about 0·003 in;

(*c*) the natural bond might not have been overcome, as appeared to be the case after three years' service.

128. While the problem was hardly serious enough to warrant the introduction of flexible pads at the ends of the splice plate and bolts, the Authors considered that an experimental investigation of such splices was overdue.

129. Turning to Dr Béla's interaction formula, it had been shown that in the case of a steel beam the problem of reduction due to shear could be approached in one of two ways.

130. In the first method[50, 51] the steel girder was assumed to be stressed in the manner shown in Fig. 26 (a), where the central elastic core carried all the vertical shear.

131. Since the shear stress varied parabolically, the depth

$$z = \frac{3V}{4\tau_0 t_w}.$$

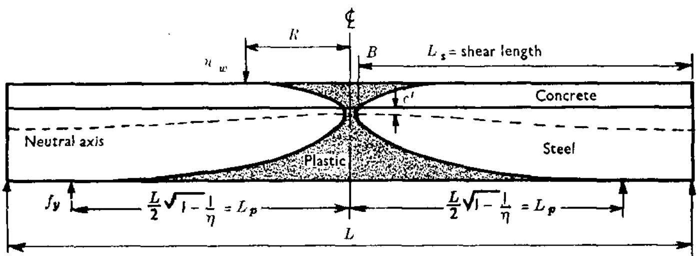


Fig. 25: Elasto-plastic diagram for uniform load

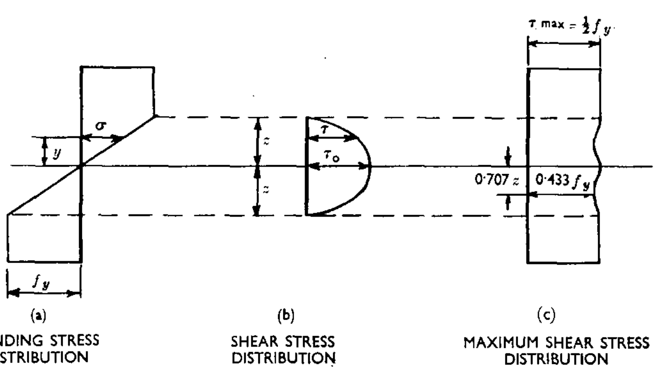


Fig. 26: Stress distribution in cantilever at failure

Using Tresca's maximum shear stress criterion
and

$$\tau_{\max} = \tfrac{1}{2}\sqrt{\sigma^2+4\tau^2} \quad \text{hence } \tau_0 = \tfrac{1}{2}f_y$$

thus
$$z = \frac{3}{2}\frac{f_y t_w}{V}$$

and
$$\tau_{\max} = \tfrac{1}{2}f_y\sqrt{\left(1-\frac{y^2}{z^2}+\frac{y^4}{z^4}\right)} \leqslant \tfrac{1}{2}f_y \text{ for } -z \leqslant y \leqslant z$$

as plotted in Fig. 26(c).

132. Although this theory had since been extended it could be seen that several complications immediately occurred when applying it to a composite steel-concrete beam:

(*a*) the problem of vertical shear stress distribution in the concrete flange had not been satisfactorily solved although attention had recently been drawn to the high axial tension present in stud type connectors;[52]

(*b*) concrete was assumed to take no tension, and to have a rectangular stress distribution;

(*c*) in the case of flexible shear connectors slip would occur at the interface.

133. In view of this the Authors preferred to consider the total vertical shear to be uniformly distributed over the web and then use the approach of Heyman and Dutton.[51,53]

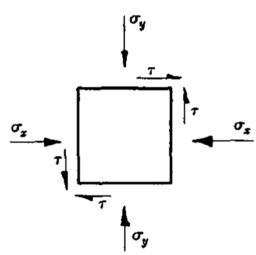


FIG. 27: NORMAL AND SHEAR FORCES ACTING ON AN ELEMENT

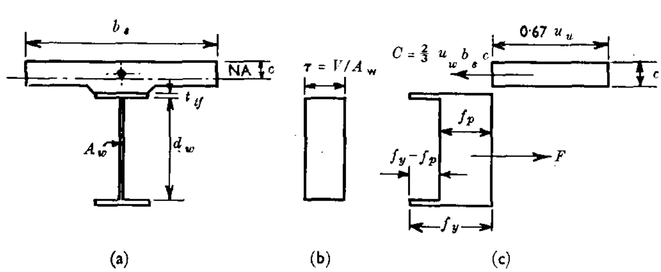


FIG. 28: IDEALISED STRESS DISTRIBUTION AT FAILURE

134. Assuming that the normal stress σ_y (Fig. 27) was negligible, the Huber–von Mises–Henky interaction formula

$$f_e = \sqrt{(\sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_y + 3\tau^2)}$$

reduced to

$$f_e = \sqrt{(\sigma_x^2 + 3\tau^2)}, \quad \text{i.e.} \quad f_e \leqslant f_{\text{yield}};$$

hence, when $\sigma_x = 0 \quad \tau_{\max} = f_y/\sqrt{3}$

and rearranging the permissible plastic bending stress

$$\sigma_x = f_p = \sqrt{(f_y^2 - 3\tau^2)} \quad \text{where} \quad \tau = V/A_w.$$

135. The bending stress distribution could be seen in Fig. 28(c) and the ultimate moment determined as previously from consideration of horizontal equilibrium.

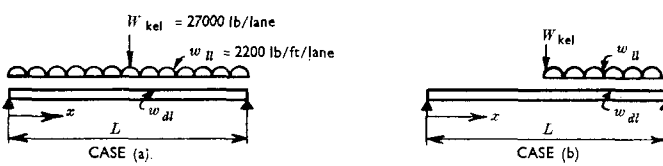
$$c = \frac{A_s f_y - A_w(f_y - f_p)}{\frac{2}{3}\, u_w b_s}$$

$$M_{\text{ult.}} = A_s f_y\left(e_c + \frac{t_s}{2} + Y_{ts} - \frac{c}{2}\right) - A_w(f_y - f_p) \times \left(e_c + \frac{t_s}{2} + t_{tf} - \frac{c}{2} + \frac{d_w}{2}\right)$$

136. It would be found that the composite section could carry a reduced moment when the web was fully stressed in shear, although this was not recommended.

137. Using this approach, a number of simply-supported spans between 30 and 100 ft had been investigated when loaded according to BS143 Part 3A, as shown in Fig. 29. In both loading cases it was found that the shear reduction of Ultimate Moment at midspan was less than 1%. The reduction would obviously become more pronounced at intermediate supports on continuous spans where the shear forces were high although the contribution of reinforcing steel embedded within the concrete might be small. While the steel girder should still be designed in the normal way the Authors considered that the shear connectors, when present in negative regions, should be designed elastically using the factors given in § 35. Consideration should be given to the formation of plastic hinges at both midspan and supports when computing the shear length at ultimate loads.

138. So far as the Authors were aware, DIN 1078 'Composite girders for highway bridges—directions for calculation and design', 1955, had not been translated, although its successor DIN 4239, Parts 1 and 2, 'Composite beams for structures' had been translated by G. B. Godfrey.[54]



CASE (a):

$$M_{x=L/2} = \frac{2{\cdot}5\, w_{ll} L^2}{8} + \frac{2{\cdot}5\, W_{kel} L}{4} + \frac{1{\cdot}5\, w_{dl} L^2}{8}$$

$$V_{x=L/2} = \frac{2{\cdot}5\, W_{kel}}{2}$$

CASE (b):

$$M_{x=L/2} = \frac{2{\cdot}5\, w_{ll} L^2}{16} + \frac{2{\cdot}5\, W_{kel} L}{4} + \frac{1{\cdot}5\, w_{dl} L^2}{8}$$

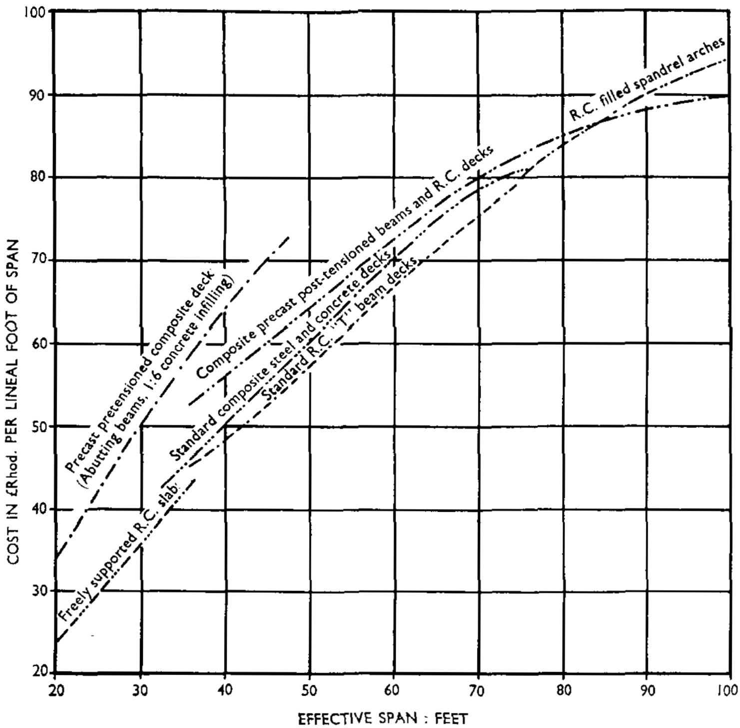
$$V_{x=L/2} = \frac{2{\cdot}5\, W_{kel}}{2} + \frac{2{\cdot}5\, w_{ll} L}{8}$$

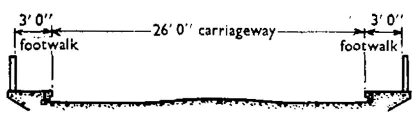
FIG. 29: SIMPLE SPAN LOADING DIAGRAMS

139. The Authors had no knowledge of CP.117, but only of CP.116 in its draft form, which referred to the structural use of precast concrete including composite construction, and not to composite steel-concrete construction.

140. In reply to Mr Easterbrook, the Authors agreed that in the 35–70 ft-span range the composite steel-concrete form of deck compared very favourably with other types of construction. Fig. 30 showed the approximate relationship between common types of bridge decks found in Southern Rhodesia. It would be seen that the composite steel and concrete deck compared very favourably. In fact, it became the most economical form of construction incorporating precast or prefabricated deck units.

141. The H.A. loading to BS153:1954 had been retained in view of the steady increase in heavy indivisible loads using the highway system. Though the Authors agreed with Mr Easterbrook's contention that above 60 ft the H.A. loading departed





TYPICAL CROSS-SECTION

FIG. 30: COST PER LINEAL FOOT OF VARIOUS BRIDGE SUPERSTRUCTURES, INCLUDING PARAPETS BUT EXCLUDING BEARINGS, 1960–62

rapidly from the A.A.S.H.O. H20–S16 loading specifications, it had been considered that the H.A. loading supplemented with 30 units of H.B. loading was more in line with prevailing conditions in Southern Rhodesia.

142. The A.A.S.H.O. loading regulations did not cover heavy indivisible loads in the H20–S16 specifications, and this fact, coupled with the introduction of two different load systems (truck or U.D. and K.E. loading), appeared to the Authors to complicate matters in spans below 120 ft, particularly in the design of statically indeterminate structures. It was interesting to note that it had been suggested that the H20–S16 truck loading should be increased to meet the increasing demands of the transport industry for heavier payloads.[55]

143. The extension to the concrete haunches $1\frac{1}{2}$ in. beyond the edges of the top flange had been introduced primarily to ensure adequate cover to the shear connectors which crushed the concrete immediately above the steel flange. They also strengthened the flange against unpredictable lateral forces due to camber, superelevation, and so on besides ensuring effective lateral restraint to the compression flange in the event of collapse. The Authors agreed that they slightly complicated the soffit shutters but thought that the advantages outweighed the small additional cost.

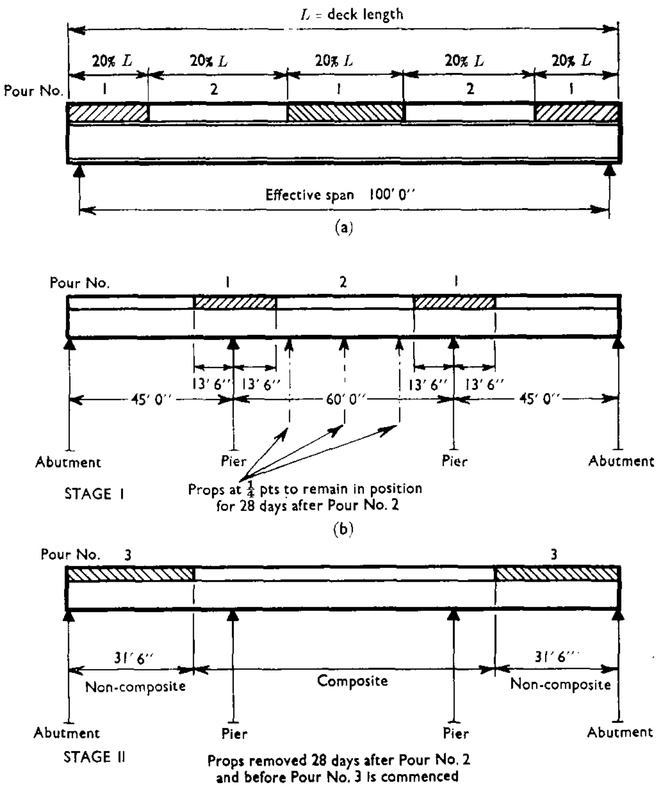


FIG. 31: EXAMPLE OF POUR-SEQUENCE

144. The relative merits of the different types of shear connectors had been based more upon a geographical than technical viewpoint. In Southern Rhodesia, stud-welding was expensive and not available in normal circumstances. Standard welding equipment and labour, which were reasonably cheap, had led to the general use of the L-stud connector. The cost of these studs varied between 1*s*. and 1*s*. 3*d*., according to size.

145. Where labour costs were high, however, the automatic welded stud on sand-blasted or mechanically-cleaned surfaces was undoubtedly the answer, on account of its simplicity and speed of installation.

146. With regard to the necessity to specify a definite pour-sequence in the decks of larger spans, the Authors had found that in certain instances decided advantages could be realized. The following examples might be of interest:

(*a*) 100 ft span, simply-supported deck, see Fig. 31(a). The two support and midspan sections, each amounting to 20% of the span length, were cast first and allowed to cure for 28 days. This improved the compression flange stability for the second pour which was carried by a composite section at midspan. Shrinkage would also be reduced.

(*b*) Continuous three-span 45 ft–60 ft–45 ft composite. In this case, advantage was taken of a particular pour sequence to control deflexions and so eliminate precambering. The total length was subdivided into five sections as shown in Fig. 31(b), the dead load inflexion points determining their lengths. The centre span was propped at the quarter-points. The sequence was to pour simultaneously Sections 1, followed after 7 days by Section 2, thus providing full dead load composite action throughout the centre span. Pours 3 were carried out simultaneously 28 days after the completion of Pour 2, and after the props had been removed in the centre span. The side spans which were non-composite for dead load tended to counteract the central span deflexion and leave the girder level throughout its length. (Fig. 31(c)). Again, this could be a question of geographical location since the delay between pours would be a serious disadvantage where labour costs were high, particularly in areas where ready-mix concrete was available.

147. The Authors wished to thank the contributors for an interesting discussion. They looked forward to the publication of results of the Imperial College Research Team which deserved every encouragement in this difficult task.

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CORRIGENDA

The following are corrections to the original Paper.

(*a*) Delete §§ 32, 33, 72 to 76 inclusive.

(*b*) § 60, lines 5 and 6 to read

'Area of concrete below the neutral axis is neglected providing $\frac{d}{t_s} \geqslant \frac{A_c}{3A_s}$.

(*c*) § 63, line 7 to read

'The plastic modulus $S_{p1} = A_s(y_{ts}+e_c)-2A_s'(c+e_c)$'.

(*d*) § 78 (b) Channel shear connectors, p. 805, line 5 to read

'Flanges should be rotated away from the direction of horizontal shear.'

(*e*) § 78 (d) L-Stud shear connectors.

These have now been found to have the same strength as headed studs.[44]