

## **Port Mann Bridge, Vancouver, Canada**

by

**N. Hilton, D.L.C.(Hon.), M.I.C.E.**

and

**G. Hardenburg**

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**Mr W. Kirby Laing** (John Laing and Son Ltd) said that the bridge and the highway connected with it were a fine example to people in Great Britain of the advantages of urban motorways. This bridge, although several miles from the centre of Vancouver, allowed the motorist to reach the city centre in approximately half an hour in all traffic conditions.

152. The piers in the northern approach viaduct were hollow, and where they were more than 65 ft in height an intermediate cross beam was introduced. The piers had a transverse width of about 8 ft and were longitudinally tapered from 14 ft to 6 ft, with a wall thickness of 18 in. They were cast as a single unit from foundation to cross beam and from cross beam to capping beam and using this method the maximum pour involved on the highest pier was approximately 100 cu. yd.

153. As indicated in § 97, the pile caps of these foundations, for piers 7 and 8 on the north side of the river, were constructed of injection concrete placed under water at a depth of approximately 30 ft.

154. Difficulty had been experienced with the grouting of some of these pile caps and it appeared that the sequence of pumping of the grout could produce conditions which resulted in pockets of either laitance or silt.

155. After pumping out the cofferdam the pile caps were counter-weighted while the 25-ft high pedestal was being constructed, by using cylindrical tanks filled with sand and water. These tanks provided a loading of 1000 tons dead weight and two layers of tanks were used. The lower layer acted as formwork for the circular voids which had to be constructed in the pedestals so as to reduce the final loading on the piles. There were six' voids in each pedestal, about 10 ft in diameter and over 20 ft high, which were filled with clean fresh water after the concrete had set. A light timber lining was used to prevent bonding between the tank and the concrete, and after removing the sand and water from the tanks, water was introduced into the area occupied by the timber lining and provided the necessary uplift to allow the tanks to float free for removal by the crawler cranes.

156. The specification required protection to be provided against scour attack on the river piers. It was decided to provide this protection by means of rip-rap which was placed into previously prepared depressions around the sheet of piling. In some of the piers, however, brush mattresses were sunk to provide the protection, and it would be interesting to know whether the Authors could give any indication as to the comparative effectiveness of the two methods of protection used.

157. As indicated in § 87, a novel form of illumination had been used. On a recent visit it had been found that, contrary to what Mr Hilton stated, there was some adverse reaction to this lighting by motorists. Most of it was due to the unaccustomed intensity and location of the lighting, although there was general favourable comment with regard to the consistency of illumination provided. Perhaps a lower intensity might have been used with advantage.

**Mr P. A. St C. Grant** (Senior Engineer, Messrs Mott, Hay and Anderson) pointed out that the Fraser River freshet was very variable in its character, and, whilst it was allowed for in the design of the bridge, it would be interesting to know if it produced any hazards during construction.

159. With regard to the approach spans, he understood from other published material that very effective use was made of retarding agents in the concrete decks of the composite structures. He would like to hear more about the application of these retarders and to what extent they assisted effective construction.

160. In the matter of welding of the orthotropic deck, he noted the use made of shop-welding, and was aware of the very high reliability of mechanically executed submerged arc-welding. Would the Authors say whether in fact there was any appreciable incidence of defective first-run welds requiring remedial action in the shops?

**Mr P. S. A. Berridge** (formerly Assistant Engineer (Bridges) British Railways, W.R.) referred to the remarkable economy resulting from the orthotropic plate deck. The principle of orthogonal-anisotropic plate design used the deck as an integral part of the main structure. By its use the dead weight of steel in long spans could be cut by between 25% and 55% compared with bridges where the main girders received no stress relief from the floor system. The orthotropic plate carried a share of the principal stresses and contributed to the strength of the structure as a whole; it was not to be confused with the battledeck floor of the 1930's in which the floor plate was considered to be part of the top flange of the stringer. But although improved methods of stress analysis were no doubt helping designers to make better use of floor-interaction, the idea was not new. Stephenson had certainly used the top and bottom decks to assist in carrying the flange stresses of his through-type plate girder spans of the Conway and Britannia tubular bridges in 1850. And according to the Design Manual for Orthotropic Steel Plate Deck Bridges published by the American Institute of Steel Construction in 1963, the problem was first studied by F. Gehung in 1860.

162. Noting the trouble experienced in matching corresponding stiffener webs and the large amount of site drilling needed at the Port Mann Bridge, (§§ 112 and 113), he thought flanged joints with high tensile steel bolts in direct tension would have simplified erection. He had used internal flange joints for the site connexions between the five prefabricated parts of the box-section top chords of the 300-ft truss spans for the reconstruction of the railway bridge over the River Wye at Chepstow in 1962 (Fig. 17). These top chords had been designed on the principle of the orthotropic

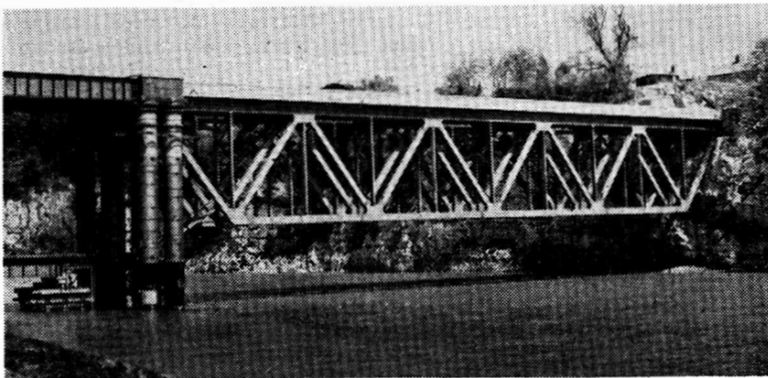


FIG. 17: THE TWIN TRUSSES OF THE RAILWAY BRIDGE AT CHEPSTOW

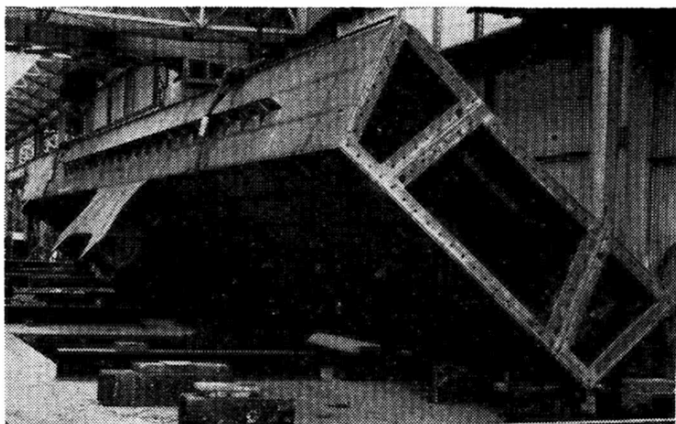


FIG. 18: SLINGING A TOP CHORD UNIT FOR THE CHEPSTOW BRIDGE. THE INTERNAL FLANGES AND THE UNDERSIDE OF THE ORTHOTROPIC PLATE DECK ARE SHOWN

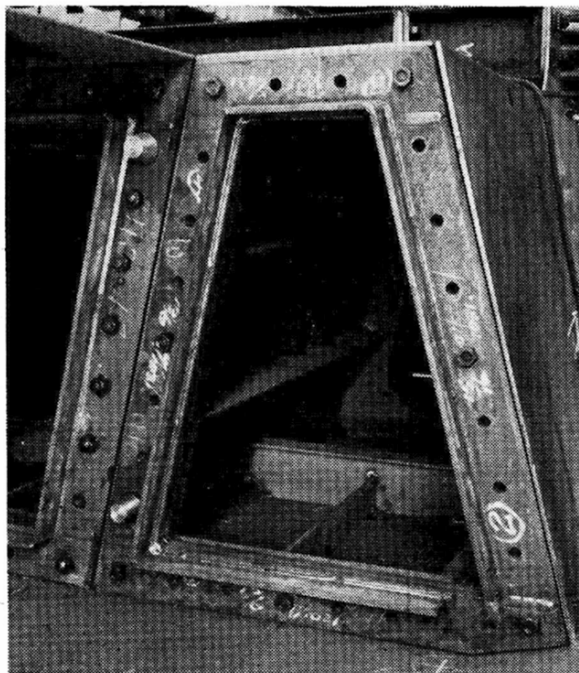


FIG. 19: THE FLANGES OF A UNIT BOLTED AND DOWELLED TO ITS ADJOINING UNIT DURING FABRICATION, UPSIDE DOWN ON A JIG

plate; each was common to two truss systems; and the resulting weight of 0.97 ton of mild steel per foot run of railway track made them among the lightest bridge spans of that length on any 4 ft 8½ in. gauge railway. As at Port Mann, the steelwork for the Chepstow girders had not had to be shop-assembled before erection on site. The units had been prefabricated and the correct fit between the internal flanges of one unit and those of the adjacent unit had been ensured by bolting together temporarily in the shops while welding (Fig. 18); the set-up on the jig had given the correct camber to the chord as a whole; a few fitted dowels had kept the parts in correct alignment; and the removal of mill scale following the usual machine-straightening of the angles without any costly machining sufficed to give perfect contact between adjoining flanges (Fig. 19) without any drilling or reaming on site.

Mr C. D. Brown (Partner, Messrs Mott, Hay and Anderson) said that he had hitherto doubted the economy of an arched construction compared with a cable-suspended construction for spans of this nature. This was because of the unfavourable cost/strength/weight ratio of steelwork compared with steel wire, in the form of ropes or otherwise. It would be interesting, therefore, to have some breakdown of cost in relation to the main span, separating, if possible, the foundations and the superstructure. He was also interested in the detail of the self-anchorage because he was not aware of a span as large as this on the self-anchored principle, and certainly with smaller spans he had found physical difficulties in detailing and building the anchorage and the distribution of the very large forces which must be involved. Would the Authors give an idea of the magnitude of these forces and whether these caused special difficulties. It would also be useful to have a breakdown of the time of construction, and if the costs could be translated into English money equivalents it would be most useful.

164. Referring to § 33, he realized that the foundation difficulties which had been so ably overcome had probably dictated the choice of orthotropic deck for this particular bridge. He did not agree, however, with the statements made concerning the relative weights of deck structures. He had looked up the weight of the deck structure on the Tamar bridge, which was a not dissimilar span. This was a concrete deck 6 in. thick with asphalt surfacing and it worked out at just under 100 lb/sq. ft. The weight of the deck structure on the Port Mann bridge must be between 55 and 60 lb/sq. ft. He therefore suggested that the multiplying factor was less than 2 and not 3 as indicated in the Paper.

165. Turning to § 36, apart from aesthetic reasons, he was not clear why high towers were not favoured in connexion with a self-anchored suspension bridge design, or even a stayed girder bridge design. The bridge was obviously built in the form of a stayed girder. Might it not have been cheaper to leave it as a stayed girder bridge, albeit not so beautiful? He did not think that the depth of the stiffening girder could have influenced the decision because he felt confident that the depth of stiffening girder required for a self-anchored suspension bridge or a stayed girder bridge would be very nearly the same: 12 ft, as had been adopted at Port Mann.

166. Referring to the deck system, he asked the Authors to give details of the stresses in the deck, the principal longitudinal tensile stress and the lateral stresses, subdivided if possible between dead and live load stresses.

167. In relation to the longitudinal trough stiffening members, he wondered to what extent it was necessary to provide the steel area to take the overall tension since the provision of longitudinal stiffeners in a deck which had its principal compression transversely inevitably complicated the site joining of the orthotropic deck. In the details of the deck would the Authors give some further information as to why 6 ft 3 in. spacing was chosen for the cross girders? The trough stiffening was approximately 10 in. deep and certainly much of the work done in Great Britain had adopted spacing roughly twice that of 6 ft 3 in. used on the Port Mann bridge.

168. In relation to the detail of the joint he was a little uneasy about the combination of torque bolts and rivets. The rivets seemed fairly inaccessible. They were countersunk and he wondered whether the torque bolts were doing all the work. Were any tests done to show how the load was spread between the two? Were there eccentricities here which had to be taken into account in the design of the deck? Bearing in mind the elegance of the deck, he wondered also why welded boxes were not used for the main chords and for the stiffening girder.

169. With regard to the approach spans, for similar schemes in which he had been engaged he had found it uneconomical to carry the deck structure three ways in plate girder construction. By this he meant main girders, cross girders, and stringers. It was, of course, inevitable very often to do this with truss construction, but for this type of bridge in plate girder construction, acting compositely with the concrete deck, he had favoured longitudinal girders and cross girders only—with apparent overall steel economy.

170. Would the Authors give a little further information concerning the corrosion problems in concrete bridge decks mentioned in § 67? Had they looked into other methods of protection against corrosion than the bonded concrete layer? Was an epoxy or watertight coating underneath a normal asphalt surfacing considered? In view of the very high adhesion would it be easy to repair the concrete decking when it deteriorated?

171. What was the source of the wind loadings specified in the Appendix? They seemed rather large, certainly by standards in Great Britain.

**Mr W. C. Dyne** (Holdyne Ltd, Winchester, Hants) congratulated the Authors on a Paper which contained many advanced and difficult engineering solutions, which had been tackled in a most original manner.

173. He had intended to raise the same question as the previous speaker with regard to the use of trusses for the 225-ft spans. He thought they would probably offer less dead weight than the welded plate girders. The Authors did not make clear why it was decided not to use an orthotropic deck on the 225-ft spans.

174. With regard to the concrete overlay on the approach spans, which he understood had been very successful, what were the reasons why this was not used on the main span?

175. He understood that it was considered that with the use of epoxy adhesives and good concrete quality control, joints could be dispensed with. He felt that on such a large area it would have been an ideal opportunity to lay down a test section to test this.

176. With regard to the 24-in. diameter piles, the Authors mentioned the use of strain gauges. It would be most interesting to know from the interpretation of these results what in fact was the point load on the pile.

**Mr J. F. Pain** (Dorman Long & Co.) said that the bridge described in the Paper seemed to provide a most interesting comparison with the bridge over the Mersey between Runcorn and Widnes which had been discussed at the Institution.<sup>1</sup> The overall dimensions of the bridges were comparable and the difference in the solutions adopted to the design problems was striking. The section of the Paper which particularly impressed him was that dealing with the scheme of erection adopted.

178. Any self-anchored structure inevitably posed erection problems of its own, but the scheme adopted in this case was both bold and complicated and clearly involved first-class preparation and engineering for its successful completion. He would be glad to learn from the Authors whether the scheme of construction adopted was developed as an integral part of the design of the bridge or whether it was evolved subsequently by the Engineers or Contractors after the design was completed. It would be most interesting to know what weight of permanent material had to be added to the arch structure to enable it to sustain the erection stresses.

179. In most through-type arches of considerable span which had been erected as cantilevers in the past it had proved necessary to omit the greater part of the deck steelwork until the arch had been completed so as to avoid the need for large increases in the arch sections to cater for erection stresses. Here it had proved possible to cantilever both arch and deck simultaneously, including the steel deck plate, and to all appearances this had not involved any great increase in the permanent sections of the arch members. It would be interesting to know whether much material had had to be added to carry the erection loadings.

180. A considerable cost must have been involved in the supply, erection, and dismantling of all the temporary works involved in the construction of the arch, and some figures for the relative costs of the arch steelwork as delivered on the site and after erection would be interesting, to bring out what proportion of the cost was involved in putting the steelwork together.

181. From some association with the erection of a number of through-type arches he envied the Authors the facility provided by the steel deck as a platform for their erection plant and materials, so as to cut out the quite severe problems involved in making the cranes climb up the back of the arch.

**Mr A. K. Chattopadhyay\*** (late Freeman, Fox & Partners), referring to Appendix I, said that the stresses used were normal, in accordance with A.A.S.H.O. The basic allowable stresses for medium tensile steel A373, was 20 000 lb/sq. in. here but the American code gave it as 18 000. The same thing followed for the pile steels. The compression stresses there would be the same.

183. Shear in high-strength bolts, in Appendix 1(e), was given as 15 000 lb/sq. in., but again, according to A.A.S.H.O., it should be 13 500.

184. The Authors must have had some reason to deviate from these American code figures and it would be interesting to know how they managed to get permission from the highway authorities.

185. It would have been interesting also to have the stresses used in the deck slab for concrete and reinforcement.

186. With regard to § 65, the Authors had said they used medium steel, A7 and A373. Had they considered using A242 or A441 as an alternative to this?

187. In § 106 he suggested that '25 × 5/16 in.' should read '25½ × 5/16 in.'

188. Would the Authors also comment on the use of A7 and A373 steel for the same job? For similar steel in Great Britain the price difference was about 2s. 6d. per ton. Did the use of these two steels of almost identical quality have anything to do with the pricing of the total job?

189. The Authors' findings with regard to local reinforcement of the steel for continuous girders near the supports was quite contrary to the idea of prestressing currently held in Great Britain. Had the Authors thought of using a steel like T1 in certain places for highly stressed members?

190. The loading used was H25-S20 and Mr Chattopadhyay asked the Authors the reason for this high loading.

The Authors were gratified by the reception given to the Paper.

192. In reply to Mr Kirby Laing on scour protection (§ 156), only one brush mattress was used at Port Mann and that was around the south main pier on the river bank. The bank at this location was composed of soft mud, peat, etc, and if heavy rock protection had been used without support from brush mattresses it would have been lost in the soft material. Brush mattresses were made upstream of the bridge site, floated down, and sunk into position at the change of tide. The heavy rock protection was placed by grab. The whole operation worked very well although

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\* It is regretted that Mr Chattopadhyay died before publication of his contribution.

the Authors believed the cost for making the mattresses was considerably higher than for normal rock protection. For the other piers, a graded rock filter system was used with the top of the protection on or below the river bed level. Recent soundings had indicated that there had been no scour.

193. With regard to the bridge lighting, the Authors had not encountered the criticism referred to by Mr Laing (§ 157). In fact, the overall comment had been one of approval, in particular, of the ability of the system to maintain overall safe illumination in fog. However, there had been adverse reaction from the public concerning insufficient transition lighting from a well-lighted bridge to no lighting on the approach highway on the south side. Respecting the intensity of light, this could only have been lessened by taking out alternate lamps or by spacing the fixtures further apart. This had been tried on a small bridge in New Jersey and there had been a 'flickering' or 'stroboscopic' effect. It was felt that in a structure the length of Port Mann Bridge, this flickering could mesmerize the motorist and cause accidents. Since the lights first came into operation there had been a reduction in the brightness due to reduced output from the tubes and dirt collecting in the fixtures. The fixtures were cleaned at least twice a year and all tubes would be replaced every two years.

194. Mr Grant (§ 158) had mentioned the hazards of the Fraser River. Fortunately no severe freshets were encountered during construction. The Contractors were fully aware of the condition and were well prepared to accommodate any eventuality.

195. With respect to the retarding agents in concrete (§ 159), the concrete deck of the approaches was poured continuously between expansion joints. To allow revibration of the concrete over the intermediate supports, retarding agents were added to the area over these supports. The revibration was required to avoid tensile cracks induced by wet concrete.

196. Before starting the production of the deck panels, welding tests were undertaken by the Contractor to determine the best conditions under which the deck welds could be carried out.

197. As the quality of submerged arch welds was generally either consistently good or bad depending on whether the proper conditions for laying such a weld were present or not, the Consultants decided to have one X-ray photograph taken of each weld in the first panels. If any defects showed up the Contractor could, at his own expense, either make more photographs of the adjacent zones to prove that the remainder of the weld was sound or replace the whole weld. In general, it was found that whenever an X-ray check detected other than very minor defects the weld was indeed inferior throughout its whole length. On investigation the reason for these defects proved to be insufficient cleaning of the copper backing bar which was used when laying the first pass. After this was realized and given due attention no more weld defects were discovered and the number of X-ray shots was gradually reduced to two per panel, taken at random.

198. The Authors were grateful to Mr Berridge for his comments (§ 162) on the design of the Chepstow Bridge but disagreed that the system of field-assembling the individual deck panels of the Port Mann Bridge could have been achieved economically in the same manner as in the Chepstow design. From the data on the Chepstow Bridge, as far as was known to the Authors, the forces between the top chords were transmitted by contact of adjoining flanges which was possible because only compression forces could occur. Such was not the case in Port Mann, where the deck system had to transmit tension forces. Analysis would show that the amount of material required for a flange connexion would be quite considerable. Furthermore, a flanged joint of a deck panel about 65 ft wide would require a much higher (and costlier) degree of accuracy in shop fabrication than would be required for the Chepstow chords of about 12 ft width.

199. Another difference from the Chepstow Bridge was that at Port Mann the erection of the stiffening girders of necessity preceded the erection of the deck panels. As was pointed out in § 112 this in turn required the possibility of adjusting the

accumulated length of the deck to that of the stiffening girders. With a 'flange' design this could be quite difficult. Not only would fillers undoubtedly be required between the flanges, but furthermore such fillers would have to vary in thickness over the width of the panel, as it was virtually impossible to guarantee that deck panels 65 ft wide, had absolutely parallel edges.

200. Replying to Mr Brown (§ 163) the Authors agreed in general with his statement on the economy of steel wire versus steelwork. However, in the case of the Port Mann design the high clearance under the bridge made it possible to give a relatively high rise to the arch (about 230 ft) more or less evenly distributed below and above deck level, thus avoiding the need for unduly long posts or hangers. With a self-anchored suspension bridge or similar structure, the sag in the cables would not have been more than about  $\frac{1}{4}$ th of the span or about 150 ft. Correspondingly, the cable thrust would have been larger than was presently the case, about in the ratio of 230:150. This increase in forces would more than offset the saving obtained by the basically higher allowable stresses in wire construction. The amount of steelwork for the required towers should not be underestimated either.

201. In this regard, the temporary cable system used for the erection should not be confused with the construction that would have been required for a permanent supporting structure. Firstly, the bridge strands used in the erection system were obtained from the dismantling of a demolished suspension bridge in northern British Columbia. While the price of these cables was much lower than for new material, their condition, although sufficient for a temporary structure, would not have been acceptable for a permanent design. Secondly, the forces for which the whole system had to be designed were much lower than would have been required for a permanent cable-supported design, because on the one hand the weight to be carried during erection was only about 85% of the final weight due to the absence of paving material and miscellaneous items, and, of course, the absence of live load. On the other hand, and of even greater influence, it should be kept in mind that the arch sections underneath the deck level already gave complete support in the later stages, when the 800-ft central part was being erected, i.e. in those stages when the demands on the cable supports become the largest. Following these considerations the Authors concluded that, in this particular case at least, an arch-supported structure proved to be the most economic solution.

202. Details of the connexion of the arch to the stiffening girder at the ends of the side spans as well as the design of the supports at the anchor piers were shown in Figs 20 and 21. The latter were designed for an uplift reaction but a small positive reaction which could occur in extreme loading cases had also to be taken into account.

203. In the connexion arch-stiffening girder, the arch force was lead into the stiffening girder, which was duly reinforced for the purpose at this location. From there on proceeding towards the centre span this force was evenly distributed over the combined cross section of girder and deck. The cross section of the stiffening girder was accordingly reduced.

204. Analysis showed that the 'effective width' of the deck for this loading originated at the point of intersection of girder and arch and increased from there on under an angle of about 30° with the stiffening girder: roughly 50 ft, or the length of one panel, was therefore required to develop a uniform distribution of the thrust force. The arch thrust to be transmitted attains approximately 5800 kips under combined dead and live load per arch.

205. With regard to time of construction and the costs thereof the following data could be given. Fabrication of the main span started towards the end of 1961, and the erection took place between July 1962 and October 1963. The contract for the superstructure of the main span was let for an amount of about \$6 900 000-00 of which \$6 500 000 was required for the supply and erection of the steelwork proper, the remainder for miscellaneous items like drain pipes, inspection carriage, catwalks, field painting, insurance, etc. The contract for the foundation of the four piers



supporting the main span, together with three adjacent approach piers on the south side, was let for about \$8 100 000-00, approximately \$5 500 000-00 of which covered the cost of the two main piers and the two anchor piers of the main span.

206. Contracts for the installation of fences, guard rails, sidewalks, epoxy coating, and asphalt paving were let separately for a price of about \$556 000-00. Certain materials for the construction were made available by the owner to the contractors to a value of about \$350 000-00. The complete cost of the main structure could thus be calculated at about \$13 306 000-00 (in Canadian currency) or £4 435 000. Considering the differences in the prices of labour and materials between British Columbia and the U.K. the cost of this structure built in England would of course be different. Due to lack of knowledge of the amount of these differences, the Authors would not be able to state what the total costs in such a case would have been.

207. The remarks made in § 164 must be the result of a misunderstanding. A comparison was made here between the weights of total steelwork required for orthotropic and conventional design, including main girders and deck system in both cases. In order to compare the complete dead load of both solutions the only items to be added were the asphalt paving in the one case and the concrete slab, possibly topped by asphalt as well, in the other. As the asphalt paving had a weight of about 20–25 lb/sq. ft, the Authors were confident of their opinion that concrete decks would weigh at least three times this amount.

208. The stresses in the deck system, referred to in p. 166, were a combination of main girder stresses (System I), floorbeam and longitudinal stiffener bending-moment stresses (System II) and deck plate bending-moment stresses due to local wheel loads (System III). Extreme cases of stresses in System I were given in Fig. 6 of the Paper (combination of arch thrust and main girder bending moments). Typical

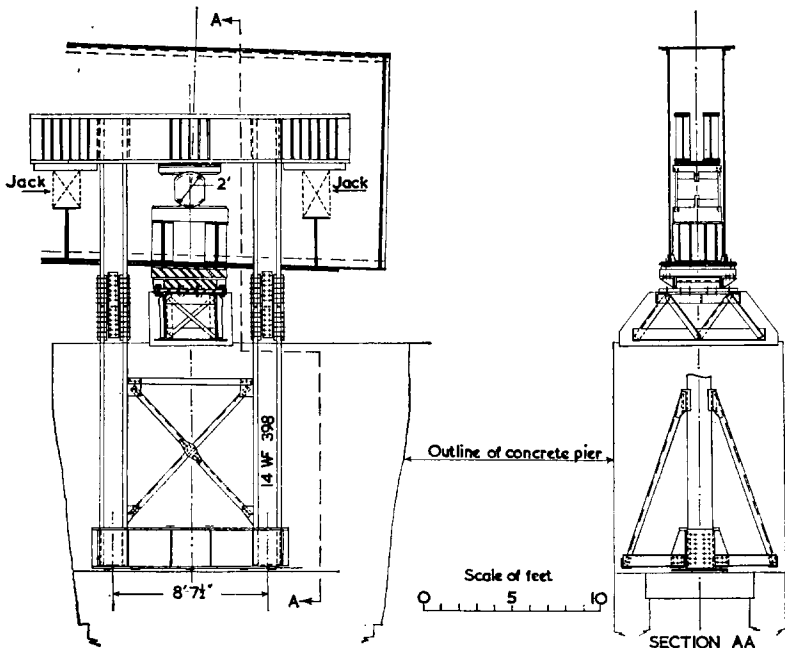


FIG. 21: TIEDOWN ARRANGEMENT AT ANCHOR PIERS

stresses in System II were: transverse (compression of floor beam top flange), 6600 psi; longitudinal (compression of stiffener top flange) 5200 psi. Typical wheel load stresses in System III were: transverse 28 500 psi; longitudinal 8600 psi. The loadings used to produce extreme stresses in each of the individual systems were not quite identical. It would not be correct, therefore, to combine all extreme cases as given above. By correctly combining stresses that could occur simultaneously, an extreme stress (according to Huber-Hencky) of 37 200 psi was found.

209. With regard to the question raised in § 167, it was quite obvious that the deckplate had to be stiffened in two directions. With a main girder spacing of 68 ft, provision of transverse stiffeners only would result in an extremely uneconomical design. It was also quite obvious that the transverse stiffeners (the floor beams) should be relatively few and heavy, and the longitudinal stiffeners (the trough sections) relatively light and narrowly spaced. The fact that these stiffeners had to be field-joined was a problem that had to be taken in one's stride. The stresses in the stiffeners were a combination of (a) bending-moment stresses produced by the wheel loadings, (b) bending-moment stresses brought about by participation of the stiffeners in the main girder action, and (c) tensile stresses produced by the arch thrust, evenly distributed over the complete cross section.

210. The distribution of material between the cross sections of the deck and the main girders should be such that stresses in each part of the cross section remained within the allowable values for all possible combinations. It was not possible to consider deck and stiffening girders separately; proportioning the deck for local loading only and stiffening girders for 'main girder' loading only would result in overstressing one part while the other was understressed.

211. The principle of considering the orthotropic deck as an integral part of the main girders was the prime reason that this type of construction proved to be so economical, particularly for long span bridges; namely the fact that a steel deck, strong but also rigid enough to support the traffic loads, had a reserve in strength which could be put to work in main girder action.

212. The spacing of the floor beams (6 ft 3 in.) was obtained by dividing the length of deck panel (25 ft) into four equal parts. While the economic length of any span could never be absolutely pinned down to an accuracy of an inch, the comparative design investigations showed that with the specified loading and design stresses, the span of the trough stiffeners could certainly not have been increased to, say, 7 or 8 ft without increasing the weight of the structure.

213. Field-joining the panels had not caused any problems: contrary to Mr Brown's belief (§ 168), the rivets were very easily accessible. Riveting took place at high speed and the number of rejected and replaced rivets was remarkably low. No tests were done on the joint construction. In Western Europe this type of joint had already been used for a number of years with satisfactory results.

214. The preliminary plans for the arches and stiffening girders showed welded boxes and the Authors agreed that generally speaking, welded construction should be entertained as being more economical. Investigation of the existing fabrication facilities proved, however, that with the available equipment and fabrication experience, riveted construction would provide a less expensive structure for the locale.

215. In reply to Mr Brown's question on corrosion of bridge decks in Canada (§ 170), the Authors stated that such corrosion had occurred mainly in eastern Canada where the concrete bridge decks were 'protected' by 2 or 3 in. of asphalt surfacing. In recent years the asphalt had cracked, and the acids and salts used in winter for keeping the streets free of ice, etc, had penetrated these cracks and 'pooled' on the concrete deck. This concrete, placed many years ago and not designed to resist acid attack, had corroded rapidly, and, in turn, seriously affected the reinforcement. Unfortunately, this corrosion was not detected in its early stages since the surface was under the asphalt. In some cases, the first indication of trouble was when a hole appeared through the asphalt and concrete deck.

216. The original design of the approaches of the Port Mann Bridge called for a 2-in. asphalt surface; however, by the time it was decided to change to a different surfacing, the structural concrete decking had been placed and provision made at the gutters and expansion joint to take the 2 in. of asphalt. It was, therefore, necessary to 'build up' a 2-in. thickness and not just spray on an epoxy protective coating. It was also felt that if a concrete overlay was correctly designed and supervised carefully during construction, it would provide a better and more maintenance-free surface than the asphalt. A point of interest was that the concrete overlay had been in place 18 months before it received any amount of traffic. This created the very best conditions for curing and it was felt that this resulted in the surfacing withstanding a very vigorous test during the winter of 1964-65. A great many streets in Vancouver were severely damaged by snow, ice, and chains, but no deterioration had been experienced in the bridge surfacing. It would indeed be very difficult to break the adhesion between the overlay and the original concrete deck to repair the surfacing. However, for any repair work the two concrete layers would be considered as monolithic and patching done as for normal concrete.

217. The wind loading specified by the A.A.S.H.O. Code was 50 lb/sq. ft on girders and 75 lb/sq. ft on arches. For the stiffening girders this loading was retained, but for the arches the loading was increased, since it was felt that the ratio of distance between arches (68 ft 4 in.) to depth of arches (4 ft 8 in.) was too high to assume the leeward arch to be completely shielded by the windward arch. Except in the wind-bearing members themselves, the wind forces did not govern the design of any member when taking into account the usual increase in allowable stress when combinations of live load and wind load were considered.

218. Replying to Mr Dyne (§ 173) the Authors agreed that the use of trusses for the 225-ft approach spans might have resulted in a somewhat more economical construction than that offered by the adopted design. Such was not the case, however, for the 125-ft and 175-ft approach spans. The total length of these latter spans (12 × 125 ft and 12 × 175 ft) was much greater than that of the 225-ft spans (6 × 225-ft) and it was decided that the slight saving in cost by using trusses for the 225-ft spans would not outweigh the fact that the total appearance of the bridge would be thoroughly spoiled by the intermixing of a few trusses between the single and straight line formed by the plate girders of the shorter approach spans on the one hand and the main span stiffening girder on the other.

219. Generally speaking, orthotropic deck construction might compete with other designs for span lengths of 225 ft. However, in the present case these spans varied considerably in geometry due to the horizontal alignment of the bridge. Within the length of the 225-ft approach spans, the bridge contained straight and curved parts and the required transition curves. Furthermore, the geometry of the cross section varied as a result of the super-elevation of the highway in the curved parts. Designing these spans with an orthotropic deck would have resulted in a great variety of different panels making the cost thereof prohibitive.

220. Mr Dyne (§ 174) asked about the possibility of using a concrete overlay on the main span. This had been given serious consideration but it was felt that the flexibility of the main span steel would probably be detrimental to the concrete overlay.

221. With regard to dispensing with contraction joints in the overlay, the Authors were of the opinion that when epoxies were used for bonding an overlay in a manner similar to that at Port Mann, contraction joints could possibly be dispensed with or greatly reduced in number. This opinion was not reached until towards the end of the laying of the concrete surfacing at Port Mann. One short length of the approach, 375 ft in length and 12 ft 4 in. wide, was left without sawn joints for a period of 3 months. No cracking was noticed, but the temperatures during this period were not high and there was considerable rain to aid a slow cure and reduce shrinkage. It was believed that the lack of cracking of the overlay was due to several combined

factors. In the first place the epoxy adhesive did not cure completely for several days, mainly due to temperature under which the curing took place. Most of the shrinkage in the concrete took place between initial and final cure of the epoxy and the stress relief was therefore possible by internal movement of the adhesive. Secondly the water:cement ratio was low, and an almost 'no slump' concrete, made workable by the use of an air entraining agent was used. Finally efficient moist curing was started early, and helped to dissipate exothermic heat from the epoxy and heat of hydration of the concrete.

222. Mr Dyne's question on the point loading on the 24-in. dia. piles (§ 176) was best answered by a study of the following figures:

| <i>Load on pile</i> | <i>Point settlement</i> | <i>Point load</i> |
|---------------------|-------------------------|-------------------|
| 150 tons            | 0.207 in.               | 120 tons          |
| 200 tons            | 0.238 in.               | 170 tons          |
| 300 tons            | 0.394 in.               | 275 tons          |
| 400 tons            | 0.545 in.               | 370 tons          |

223. Referring to Mr Pain's remarks on the erection (§§ 178, 179), the Authors pointed out that the designer of any large bridge should check whether his plans could be translated into reality without unduly large difficulties. This might have an adverse effect on the apparent economy of the proposal. Keeping this in mind, the consultants studied a possible erection scheme which was supplied to the bidding contractors as supplementary information to the contract. This scheme made use of a number of falsework bents. However, the contractor chose to adopt the system with temporary supporting cables. These stay cables were divided so that no erection material had to be added to the arches and stiffening girders to allow for erection conditions. To lead the falsework and cable forces into the structure, a number of vertical members had to be reinforced, as specified in § 147. For the reinforcing of these members approximately 85 tons of extra material were required.

224. The Authors agreed that it would be interesting to know the cost specification referred to in § 180 but as the contract was let with one total price for supply, fabrication and erection of the steelwork, these figures are only known to the Contractor who had not seen fit to reveal them.

225. With regard to Mr Chattopadhyay's remarks on the allowable stresses used in the Port Mann design (§§ 183, 184) the Authors observed that the bridge was built in Canada where the American A.A.S.H.O. Code was frequently used but freedom was allowed by the Provincial Governments to deviate from this Code. As the A.A.S.H.O. Code could be considered as very conservative, if not outdated, with regard to many design criteria, notably design stresses, the Consultants had proposed the use of stresses generally in accordance with modern (West European) bridge specifications.

226. The stresses used in the deck slab were: concrete 1600 psi; reinforcing steel (intermediate grade), 20 000 psi. The use of high strength steel had not been considered for the approach spans. Where medium steel was used in the design, A7 steel was specified for non-welded construction and A373 steel for welded construction. In this way the lower price of A7 steel was taken advantage of wherever the design permitted its use.

227. The Authors agreed with Mr Chattopadhyay (§ 189) that prestressing the concrete near the supports of continuous composite girder design was in principle a better solution. However, in the opinion of the consultants, the experience of the local contractors with these techniques at the time the design was prepared was not sufficiently extensive to warrant trouble-free construction. The use of T1 steel had been in the consultants mind but the locations in the bridge, both approach and main spans, where this steel might have been used economically proved to be too few and too small to warrant its introduction.

228. The H25-S20 loading—an increase of 25% above the A.A.S.H.O. H20-S16

loading—was the standard loading specified by the Department of Highways of British Columbia for all bridges on main highways. The Authors agreed with the Department of Highways that the H20-S16 loading seemed on the low side when compared with the demands of present (and future) traffic. In particular, the concentrated truck loading, of such importance for the design of the deck structure, when taken at 36 short tons only, was quite low when compared with vehicles actually existing. The Authors were tempted to think that bridges in the United States nevertheless appeared to sustain the demands of modern traffic because of the low stresses used in their design.

#### REFERENCE

1. ANDERSON J. K. Runcorn-Widnes bridge. *Proc. Instn civ. Engrs*, 1964, 29, (Nov.), 535-570.