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Le nouveau pont de Waterloo à Londres

Die neue Waterloo-Brücke in London

New Waterloo bridge at London

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Introduction

This paper gives details of the design of the highway bridge across the Thames in London.

The new bridge, shown in figures 1 and 2 was commenced in 1937, and was opened to traffic in 1942. It replaces Old Waterloo Bridge, built to the designs of John Rennie, which was begun in 1811 and opened in 1817. The old bridge consisted of a heavy structure in masonry having nine arches, each of 120 feet clear span, carried on piers 20 feet in thickness and massive end abutments. The width of the structure was 45 feet. In 1923 foundation failure commenced at one of its piers in consequence of which restrictions were imposed on traffic and, as a further precaution, a temporary bridge was erected 30 feet clear downstream. With the completion of the new bridge, the temporary bridge has been removed. Between the years 1923, when foundation trouble commenced, and 1934, when it was finally decided to remove the old bridge, there had been much controversy as to whether the old bridge should be reconditioned and widened or removed. The decision for its removal rendered it possible to design in reinforced concrete a new bridge incorporating the existing approaches, having five spans each nearly 240 feet clear, an overall width of 83 feet and 3 feet more headroom for the passage of river traffic.

General description

The new Bridge has a roadway 58 ft wide, and two footpaths each 11 ft wide. It has been designed to carry the Standard Load for Highway Bridges of the Ministry of Transport, the equivalent loading curve being used. No reduction on account of the six-line width of roadway was made. In addition, the design of the cross girders and deck slab was controlled by the assumption of a 40-ton axle load (two wheels of 20-tons each, including impact) or a special lorry on multiple wheels having a gross weight of 150 tons including impact allowance.

The main members throughout are of reinforced concrete. Fabricated steelwork has, however, been used for details, such as those at expansion joints and for the inclined struts to the pier shells, wherever it was advantageous.

The relation of the primary structural elements is shown in figure 3. Changes in length are taken care of by expansion joints, which also permit of angular movement in a vertical plane. A simple knuckle joint is provided at the bridge end of the hinged approach slab so that changes in level and slope of the cantilever are accommodated by the hinge action of this slab. The suspended portion of the centre span is similarly adapted by the expansion joints at its ends. The bearing walls of the piers and abutments, by reason of their flexibility in the direction of the bridge, offer little restraint to horizontal movement and change of slope of the superstructure, and so constitute in effect an articulated support.

Piers

The pier construction is also shown in figures 3 and 4. The foundation consists of a solid block of concrete 6 ft thick, reinforced at the bottom with transverse and longitudinal bars. Alternate bar-ends are welded to the steel sheet piling, and welded anchors are also provided near the top of the block, thus effectively securing the piling against relative movement. The reinforcement has been designed to suit the bending stresses resulting from the excess of upward over downward pressure on the footing projection and from the skin frictional support afforded by the piling. Projecting bars are cast into the foundation to bond with the cellular base to the bearing wall and pier shells.

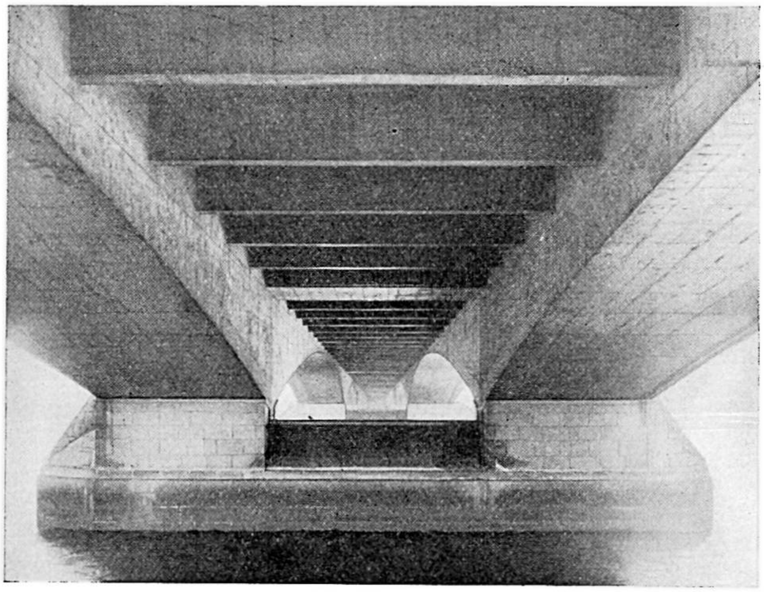


Fig. 1. Underside view of the new Waterloo bridge.

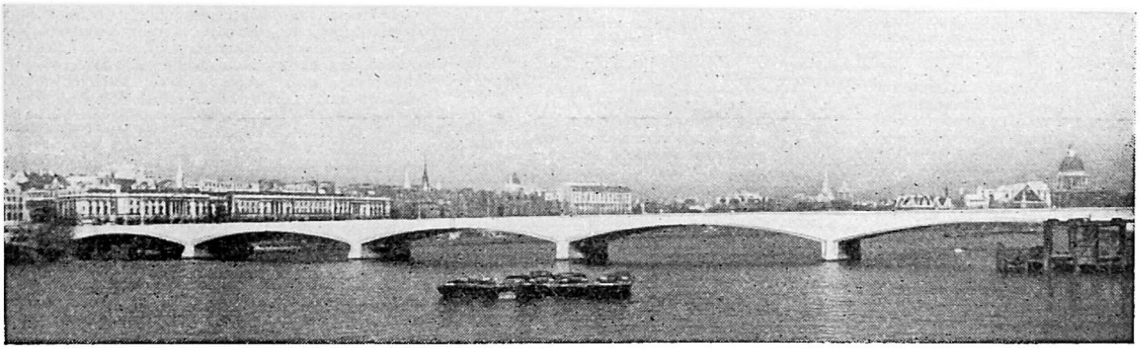


Fig. 2. New Waterloo bridge.

The cellular base is provided to spread transversely the load from the bearing wall, and to form a transition between the solid block and the narrow wall. The reinforcement to the cross walls of the base has been arranged to take the principal tensile stresses arising from this action.

The bearing wall carries the whole weight of the superstructure and, in addition to the direct stress resulting therefrom, is subject to flexural stresses due to the various movements. The wall is rigidly connected to the cellular base and the superstructure and serves to fix the main box members against torsion; it also distributes the loading longitudinally over the foundation and lends itself to a convenient scheme of jacking. The lower portion of the wall is comparatively lightly reinforced, whilst the upper portion, above the jacking gap, is heavily reinforced, particularly under the inner ribs of the superstructure; in this region transverse reinforcement in the form of bars welded at their ends to the vertical and the longitudinal horizontal bars is also provided. The proportion of direct stress to bending stress is such that tension cannot occur in any portion of the wall.

The bearing wall is surrounded by, but completely separated from, a granite and Portland stone faced reinforced concrete shell. This shell, apart from appearance, protects the main supporting member from damage by shipping, provides the substance of the stops preventing excess movements of the superstructure, and constitutes a permanent cofferdam which would, if necessary, facilitate inspection and repair of the bearing wall.

The vertical ribs on opposite sides of the bearing wall are connected by steel diagonal members passing through clearance holes, thus forming in effect a truss the full width of the pier to resist impact and forces applied at the stops. The steel diagonals, where not embedded in the rib concrete, are protected against corrosion by zinc spraying and « gunned » bitumen.

The stops at the top of the vertical ribs are opposed to the haunch of the bearing wall. Design to resist traction forces only presented no difficulty, but it was thought desirable to develop, if possible without any great additional cost, the full strength of the shell construction (its scantlings being as determined by its other functions) in resisting indeterminate forces such as might be occasioned by an exceptionally heavy impact deflecting the shell against the superstructure. The solution was seen in developing the maximum possible friction between the haunch and the stop so that virtual A-frames would be set up by the bearing wall and the ribs resisting

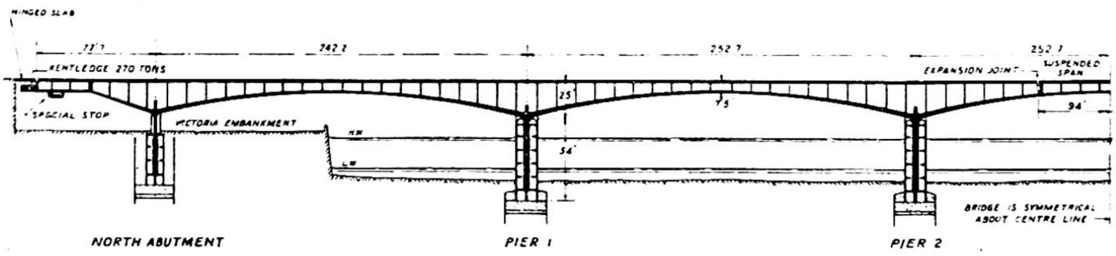


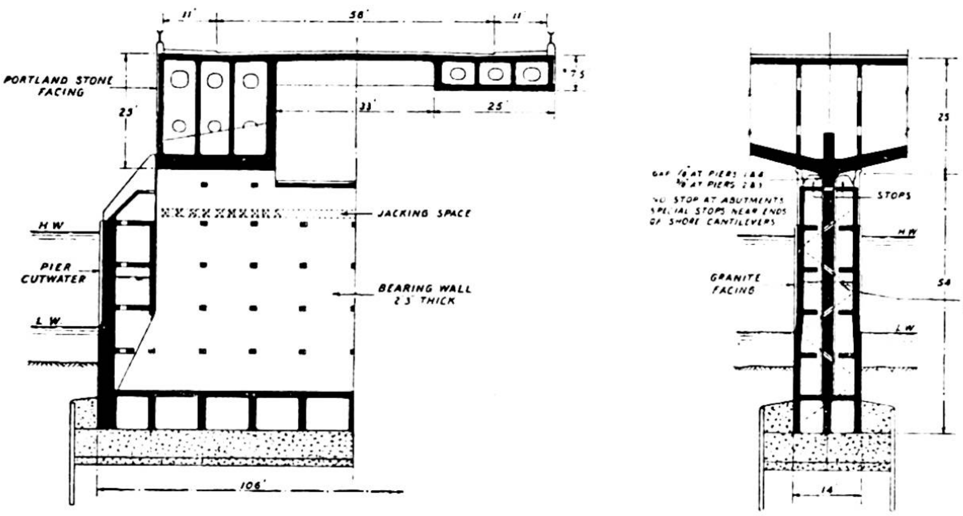
Fig. 3. Longitudinal section through box girder.

pressure. Experiment showed that with concrete cast against hardened concrete and separated after setting, a friction angle of about 45 deg. was developed on further contact. The stops at piers Nos. 1 and 4 were therefore formed by pouring them against the roughened haunch, the superstructure being jacked over as necessary. At piers Nos. 2 and 3, where clearances are greater, it was inconvenient, in relation to the contractor's programme, to jack over the superstructure, and the stop faces were formed by a slab of hard Portland stone fitted to the haunch and then drawn away and cast into the stop.

The design of the abutments is generally on similar lines to that of the piers, but no stops are incorporated in the shell structure.

Main girders

The superstructure consists throughout of two box girders; subdivided by internal ribs and diaphragms, carrying a central strip of decking which is integral with the main members. Considered longitudinally, the bridge is symmetrical about its centre and each half consists of a twin two-span girder continuous over the first river pier (pier No. 1 or pier No. 4) and cantilevering shorewards from the abutments and into the centre span from the second pier. The gap in the centre span, between the cantilevers



Half section at pier. Half section at crown. **Section of pier.**

Fig. 4.

extending from the north and south, is filled by a suspended section, whilst each shore end cantilever carries a short hinged approach slab.

A cross section of the girder at the crown is shown in fig. 5. The lower flange is comprised by the curved vault slab 25 ft wide. The web member is made up by four ribs, the inner being about double width throughout compared with the others; the upper flange is formed by the deck slab.

The moment of inertia was calculated at ten sections on the span and the analysis was carried out by ordinary slope-deflection means. It was decided to consider the full concrete section as acting, but with a varying modulus of elasticity for concrete. On the compression side the full value was taken from the extreme fibre to the neutral axis, except near the centre-line of the deck, whilst on the tension side modulus was assumed to vary parabolically from the full value at the neutral axis to half value at the extreme fibre. The reinforcement (tension and compression) was taken into account at the appropriate modular ratio. The central portion of the deck slab, being monolithic with the box members, will act with them, but near the centre line, owing to shear strain, the full stress will not be developed, and to allow for this the slab depth and steel area at the centre were taken at half value, increasing uniformly to full value at a point 4 ft from the inner face of rib D. A modulus value for concrete of $E = 4\ 500\ 000$ lb per square inch was taken for live load effects, and of $E = 1\ 800\ 000$ per square inch for dead load moments and deflection. The initial deflections on jacking showed $E = 5\ 500\ 000$ lb per square inch approximately, whilst subsequent creep deflections indicate $E = 1\ 800\ 000$ lb per square inch to be about the correct asymptotic value.

Longitudinal reinforcement in the deck and vault consists of straight bars only, their number and sectional area being varied to suit the bending moment. Shear reinforcement is generally in the form of transverse bars, but in the ribs horizontal shear steel is also used. Stirrups and transverse bars are generally at constant pitch, diameters of bars being adjusted as necessary.

The cross girders carrying the deck slab between the main members are of T-beam type. The external dimensions were determined by architectural considerations, and the webs were made hollow to avoid unnecessary weight.

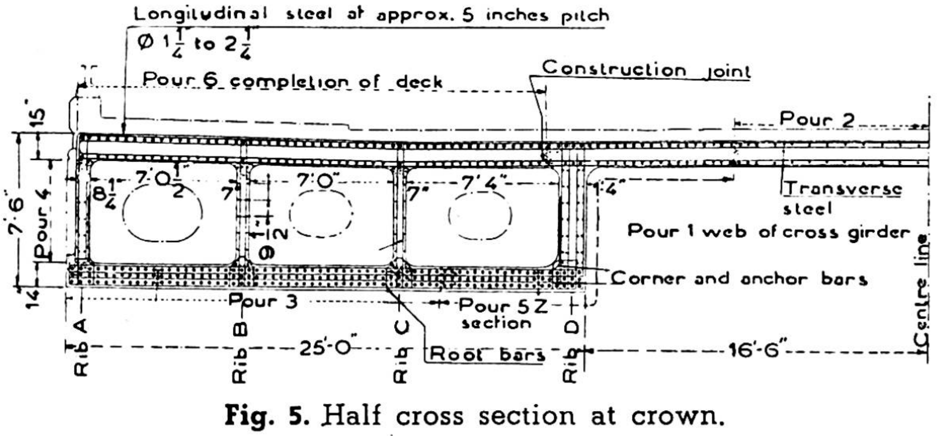


Fig. 5. Half cross section at crown.

Cantilevers and suspended span

In the case of the shore cantilever the negative bending moment at the abutment resulting from its self-weight was insufficient to reduce the positive moment near the centre of the first span to a practicable value. Considerations of the effect of relative settlement of supports showed it to be undesirable to anchor down the end, as the additional span thereby introduced would be too short. It was decided, therefore, to load the end of each member with kentledge, and 270 tons of cast iron were provided in a special box at each corner of the bridge.

Calculations showed the ideal length of the suspended span to be 88 ft. This length, however, would have caused the joint to clash with a cross girder and diaphragm, the uniform pitch of which it was not desired to upset; the suspended length was therefore increased to about 94 ft and to preserve the « balance » at the adjacent piers 40 tons of kentledge were provided inside each cantilever member near the joint.

Torsion

By reason of the disposition of the loading on a cross section of the superstructure and the comparative flexibility of the cross girders, the main box members are subjected to torsion. In computing the value of this, together with the degree of fixity at the ends of the various cross girders, a first assumption was made that the shape factor for torsion rigidity for a hollow box member with an eccentric core was as given by St. Venant for a solid rectangular member of similar external proportions. To confirm this assumption, and to obtain the relation between the shear and flexural moduli of the concrete, experiments were made on both solid rectangular specimens and twin hollow specimens of the mean proportions of the bridge members, with wall thicknesses to scale. All specimens were of the same concrete mix as the bridge members and were carefully vibrated during manufacture. The box specimens were tested with and without a connecting slab simulating the strutting action of the deck slab. Bending tests were also made. It was found that the shape factors for solid and hollow specimens were practically identical; the shear modulus was two fifths of Young's modulus (that is, for the very high-grade concrete used, a Poisson's ratio of 0.25 obtained as for steel). The torsional rigidity was the same whether or not the members were strutted, and, by test to destruction, it was confirmed that the greatest shear stress occurred in the thinnest wall independently of its distance from the centroid of the section.

Expansion joints

At the extreme ends, and in the centre span at the bearings of the suspended section, expansion joints are provided. All joints are of the single segmental roller-bearing type, and they are arranged to cater for a total change of length of 6 in, corresponding to a range of body temperature of 60 deg. F. The centre span joints have several features worthy of mention

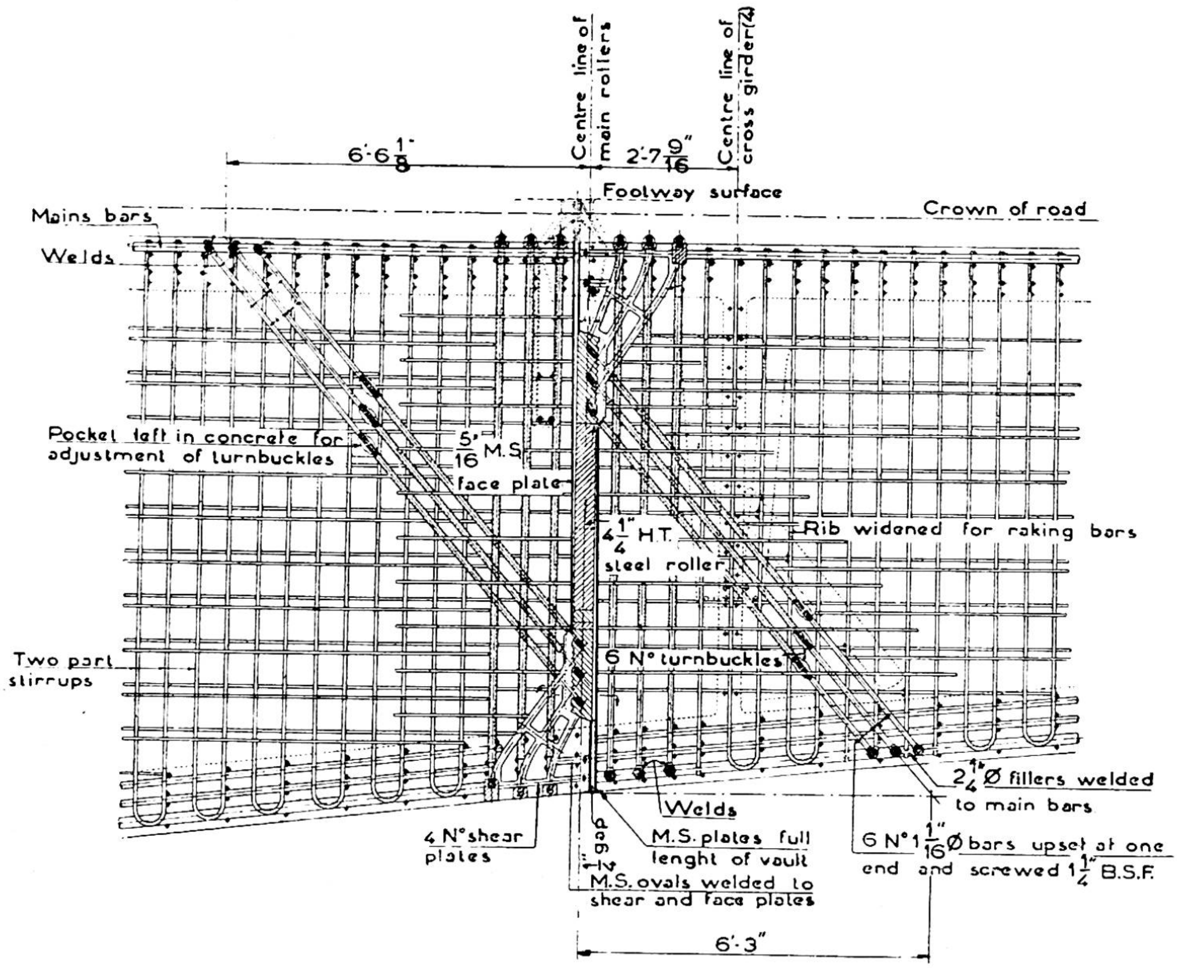


Fig. 6. Centre span (expansion joint details).

(fig. 6). It was necessary to avoid projections at the joints, and the main rollers were therefore limited in width to the thickness of the ribs. Loading conditions were such that, using medium high-tensile steel, a diameter of 40 in, was required for the rollers, and this, together with the inevitable division of the rib depth, so reduced the corbel above and below that a shear value in excess of 1.000 lb per square inch was called for. Such a high figure could not be realized with normal reinforcing; accordingly the principal compression component was transferred from the bearing billets by means of special reinforcement, or « shear » plates, whilst the principal tensions were taken care of by medium high-tensile bars which were prestressed as described later.

Considerations of transverse deflection showed that, with the main bearing rollers only, the major portion of the loading would be thrown on to the inner rib. Although this action could have been compensated to a large extent, in the case of dead loading, by the initial setting, the greater live load effect could not conveniently be eliminated. Secondary rollers in the deck and vault slab were therefore provided which ensured the transmission of the torsion arising from transverse deflection across the joint, and the whole span was thus enabled to act in relation to torsion as if monolithic.

Stops

At the top of the vertical ribs of the pier shells and near the end of the shore cantilevers, stops are provided to limit the movements of the superstructure resulting from various causes. The stops at piers Nos. 1 and 4, being practically at the centre of expansion of the north and south halves respectively, are provided with only 1/8-in mean clearance each side; those at the other piers and the end stops have about 3/8-in mean clearance, the amount being adjusted so that contact is made, one side or the other, at the extremes of temperature. The end stops consist of heavy reinforced concrete walling, founded generally on cored piles, and are of such robust construction that should any pier develop a tendency to tilt sufficient resistance would be available for prevention or correction. The stop detail is comprised of an elongated steel-shod nib projecting horizontally from the vault slab of the cantilever into a steel-lined recess in the walling. In addition to the stops described, ties are arranged across the centre span expansion joints to limit independently the amount by which they may open.

Should the bridge ever be called upon to carry its full design load there would be, in the absence of preventive means, first a downward and then an upward deflection of about 1 in at the end of the shore cantilever as the load passed over and on the first span. This would cause damage to the road and footpath surfacing at the approach and might give rise to an undesirable feeling of deflection. Dampers were therefore provided in the form of reinforced concrete columns containing a central rod which passes through a hole in the vault slab. Adjustable stops were formed between the top of the column and the vault to limit downward deflection, and between a pad on the top of the vault and a nut at the end of the rod to limit upward movement. These dampers were so proportioned that whilst their strength was adequate to control live-load deflections they would yield before any damage to the bridge structure proper could result from failure to readjust their setting following an appreciable settlement of the abutment or pier foundations.

Construction joints

The desirability of avoiding joints in zone of high shear stress, the effect of the settlement of centering piles during construction, and the contractor's scheme for carrying out the work had to be taken into account. The arrangement finally evolved consisted in providing temporary transverse gaps at about one-sixth span from the piers and at the centre, thus dividing each span-length into three sections, one pier section, and two centre sections, each capable of slight relative settlement during concreting. The sections were sub-divided by longitudinal joints in pre-determined positions, to give reasonable pours without introducing additional transverse joints, except as afforded in the centre span by the expansion joints.

Pre-stressing

In the shore cantilevers, at the top of the bearing walls, and in the vicinity of the centre-span expansion joints, where exceptionally high shear stresses are obtained, certain reinforcing bars were required to be pre-stressed. Such bars were of medium high-tensile steel with ends upset and screwed, and contained in steel tubes fitted with projecting end connections. After the concrete had been poured and had hardened the bars were stressed by passing steam through the tubes and taking up the thermal extension by turning the end nuts or turn-buckles so that, on cooling, the required stress was induced. A final stress of 30 000 lb per square inch was required in the bars, and the initial apparent stress, as measured by the turns of the screws, to ensure this was found by calibration with a hydraulic jack to be 45 000 lb per square inch, the difference being due to elastic and creep compression of the concrete and to shrinkage, the latter factor being small on account of the concrete being many months old when stressing was carried out. The bars were finally grouted up solid in the tubes, the steam connections being used for this purpose.

Welding

To realize the desired slimness of the construction a high percentage of reinforcement was necessary, and it was evident that welding would afford many advantages. The elimination of laps, splice bars, and hooks would enable scantlings, and therefore dead weight, to be reduced to the minimum, and within the limitations of the design would permit the simplest arrangement of the steel in relation to concrete placing. Welding also offered a rigid reinforcement cage true to dimensions and cover and resistant to displacement during concreting, and provided a means whereby efficient crack-control could be ensured. Electric arc welding was used.

Concrete

The superstructure, the bearing walls, and the cellular bases to the piers and abutments were specified to be of concrete quality A, the mix being : cement, 112 lb minimum and 140 lb maximum; aggregates (sum of volumes measured separately), 5 5/8 cub. ft. Strength requirements on 6-in cubes were : preliminary tests at 28 days, 5 600 lb per square inch; works tests at 28 days, 4 200 lb per square inch, and at 3 months 5 000 lb per square inch.

The pier and abutment shells were of concrete quality B, which mix was similar to quality A but with strength requirements reduced by 20 per cent. In practice it was found that the strengths corresponding to quality A could be realized with the minimum quantity of cement, so that the mixes became, in fact, identical. Generally for vibrated reinforced work a slump of 3 in was permitted. Concrete with 1-in slump was used in many heavily reinforced parts of the work, and there was no evidence to show that a higher value was generally necessary. The water/cement ratio ranged from 54 per cent. to 60 per cent. by weight.

The whole of the concrete in the bridge structure, except the foundation block, was vibrated, a minimum frequency of 5 000 cycles per minute being specified. Electric vibrators, of 160-watt and 250-watt capacity, clamped to the shuttering, were used generally. Immersion vibrators were employed in situations such as at junctions of members where the external machines could not be fully effective.

Surface treatment

No treatment could be adopted which would impair the strength and life of the structure, and accordingly bush hammering and tooling, which not only remove the surface but also cause damage to the underlying concrete, were excluded from general adoption. The stone facing has been arranged in vertical strips so that the joints in no way imply heavy stones functioning as an arch or wall. It was specified that the forms for surfaces exposed to public view should be lined with a smooth-faced non-absorbent plywood and, after stripping, the concrete face should be mechanically ground by carborundum disks. The « arch ring », where special cover to the reinforcement had been provided, was lightly hammered with an electric hammer, whilst the underside of the structure, except the curved vaults of the end spans, received an « engineering » finish, consisting of grinding down the worst board marks and blemishes resulting from welding splash, cleaning, and then stippling with and rubbing in a thin coat of neat cement mortar. The end span vaults were lightly sandblasted in two stages, the first to expose flaws for making good and the second an all-over treatment to give uniform texture.

Stresses

The maximum values of stresses in the reinforced concrete construction were broadly governed by the desire to provide factors of safety equal to those implicit in the Code of Practice, Section 3. The concrete generally corresponded to the nominal 1 : 1.5 : 3 Special Grade mix; the reinforcement was generally mild steel, but medium high-tensile steel was used for pre-stressed bars. The maximum working stresses are given in table I. Preliminary and works crushing tests on 6-in cubes at 28 days called for minimum strengths of 5 600 lb per square inch (4 x) and 4 200 lb per square inch (3 x) respectively. The compressive stress in the main girders, although a bending stress, was limited to the mean of the bending and direct values on account of the cellular form of these members. The shear on a reinforced section was limited to the value given (the Code would permit 550 lb per square inch) for reasons of crack control. The figure was arrived at as a result of tests to determine the ultimate tensile strength of the concrete in bending and shear. A general minimum of 450 lb per square inch was found from both bending and torsion tests on unreinforced specimens.

The flexural reinforcement to the main girders consisted generally of bars about 2 in diameter; tests showed a yield-point of 36 000 lb per square inch for this size of bar, so that to preserve the normal factor of safety (about 2.2), which is based on the yield-point of smaller diameter bars, the working stress was reduced.

In the superstructure the calculated maximum stresses correspond closely to the working stresses given in table I, except in the case of bond, where the maximum stress, computed in the ordinary manner, amounts only to 70 lb per square inch. The bearing pressures on the foundations are given in Table II. The figure of 8 cwt per square foot for the skin friction on the piling, taken on the projected area, is given as a probable mean value. On first loading the friction would be some 50 per cent. higher, but as a result of shear « creep » in the clay there would be a transference of load to direct bearing, and the ultimate value would be small.

Concrete stresses : lb per square inch		Steel stresses : lb square inch	
Bending	1 400 (x)	Beam tension	16 500
Direct	1 100	Web	18 000
Main Girders	1 250	Pre-stressed bar	30 000
Shear	140	Compression	Modular ratio basis
Bond	140		
Shear (reinforced) . . .	450		

TABLE I

Skin friction on piling	Gross pressure : tons per square foot		Net pressure : tons per square foot	
	Abutment	Pier	Abutment	Pier
Nil	4.76	4.45	2.54	3.12
8 cwt per square foot	3.77	3.68	1.55	2.35

TABLE II

The new bridge was designed for the London County Council by Messrs. Rendel, Palmer and Tritton, Consulting Engineers; in association with the Council's then Chief Engineer, Sir Peirson Frank, M. I. C. E., F. S. I., the collaborating architect being Sir Giles Gilbert Scott, O. M., R. A. The contractors were Messrs. Peter Lind & Co. Ltd.

Résumé

Ce mémoire donne des indications concernant la construction du nouveau pont-route sur la Tamise, à Londres.

Ce nouveau pont, dont la construction débuta en 1937 a été ouvert à la circulation en 1942. Il remplace l'ancien pont de Waterloo construit de 1811 à 1817 d'après les plans de John Rennie. Le vieux pont était constitué par

neuf arches lourdes en maçonnerie de 120 pieds de portée, reposant sur des piles de 6 mètres d'épaisseur et des culées massives. La largeur était de 13^m50. En 1923 la fondation d'une pile commença à s'affaisser, nécessitant une réduction du trafic. Un pont provisoire fut érigé 9 mètres en aval, pont qui fut enlevé après l'achèvement du nouveau pont. En 1934, la construction du nouveau pont fut décidée. Pendant la période 1923-1934, il y eut de nombreuses discussions pour savoir s'il fallait renforcer et élargir l'ancien pont ou le démolir. La décision intervenue en 1934 permit de concevoir un nouveau pont en béton armé pour lequel les voies d'accès étaient maintenues, comportant cinq arches d'environ 240 pieds de portée intérieure et un tirant d'air dépassant le tirant existant de 3 pieds.

La caractéristique principale du nouveau pont consiste dans ses lignes élancées. Les voies d'accès, d'une hauteur considérable au-dessus des deux rives, sont reliées par une construction d'une longueur de 375 mètres reposant sur des piles dont la hauteur au-dessus du niveau de l'eau n'est que de 0^m30.

La superstructure supportant le tablier, comporte deux maîtresses-poutres de hauteur variable de 7^m50 de largeur, écartées de 10 mètres. Celles-ci supportent une dalle intermédiaire d'une construction monolithique. Les appuis sont constitués par quatre piles reposant dans le lit du fleuve et deux culées, non apparentes sur les berges. La travée centrale est en cantilever.

Les caractéristique techniques principales sont :

Béton vibré.

Un système de renforcement soudé sur chantier éliminant tous les accessoires tels que recouvrement, crochets, etc.

Précontrainte et renforcements locaux.

Poutres jumelées en caisson continues au-dessus de trois appuis. Ces poutres sont supportées sur des voiles minces flexibles les préservant de tout effet de torsion pouvant résulter d'une charge excentrée sur la dalle de 33 pieds de largeur entre ces deux poutres.

Répartition des charges sur toute la largeur des piles par l'intermédiaire de voiles comportant toute la largeur de ces piles.

Piles constituées par des caissons donnant une protection suffisante aux voiles portants et limitant le déplacement longitudinal de ceux-ci.

Pendant la construction, la répartition correcte des charges et la mise à niveau furent exécutées au moyen de vérins hydrauliques placés dans les voiles portants.

Zusammenfassung

Dieser Bericht gibt Aufschluss über die Konstruktion der neuen Strassenbrücke über die Themse in London.

Die neue, 1937 in Angriff genommene Brücke wurde 1942 dem Verkehr übergeben. Sie ersetzt die alte Waterloo Brücke, die, 1811 begonnen und 1817 dem Verkehr übergeben, nach den Plänen von John Rennie gebaut wurde. Die alte Brücke bestand aus einer schweren Mauerkonstruktion mit neun Bögen von je 120 Fuss der inneren Oeffnung, die von 20 Fuss dicken Pfeilern und massiven End-Widerlagern getragen wurde. Die Brücke

hatte eine Breite von 45 Fuss. Im Jahre 1923 begann das Fundament unter einem der Pfeiler zu versagen, sodass der Verkehr auf der Brücke eingeschränkt werden musste. Als weitere Vorbeugungsmassnahme wurde 30 Fuss (ca. 10 Meter) stromabwärts eine Notbrücke erstellt. Nach Fertigstellung der neuen Brücke ist die Notbrücke entfernt worden. Zwischen 1923, dem Jahre, in dem die Schwierigkeiten mit dem Fundament einsetzten, und 1934, in dem der Beschluss endgültig gefasst wurde, die alte Brücke abzubrechen, war es wiederholt zu Meinungsverschiedenheiten darüber gekommen, ob die alte Brücke renoviert und verbreitert, oder aber abgebrochen werden sollte. Durch den Beschluss, die alte Brücke zu entfernen, wurde die Möglichkeit geschaffen, eine neue Brücke in Eisenbetonkonstruktion zu bauen, die die bestehenden Zufahrtswege beibehielt, fünf Bögen von je beinahe 240 Fuss der inneren Oeffnung besass und den Wasserfahrzeugen einen 3 Fuss höheren Passierraum gestattete.

Das Hauptmerkmal der neuen Brücke besteht in ihren schlanken Linien. Die bestehenden Zufahrtswege, die eine beträchtliche Höhe über den beiden Flussufern erreichen, werden durch eine Konstruktion miteinander verbunden, die über ihre 1 250 Fuss Gesamtlänge eine Pfeilhöhe von nur einem Fuss aufweist. Die Tragelemente befinden sich unterhalb des Fahrdammes und bestehen aus doppelten, vierkantigen Trägern verschiedener Höhe und von je 25 Fuss Breite; diese Tragelemente liegen in 33 Fuss Abstand voneinander. Sie tragen eine mit ihnen monolithartig gebaute Zwischendecke. Im Fluss stehen vier Pfeiler und auf jedem Ufer ein verstecktes Widerlager. Die vierkantigen Träger ragen über ihre Stützen hinaus und tragen in der Mitte einen Hängebogen.

Die technischen Besonderheiten dieser Brücke sind :

Vibrierter Beton.

Eine an Ort und Stelle geschweisste Verstärkung, wodurch alle Ueberlappungen, hakenförmige Endteile usw. wegfallen.

Oertliche Vorspannung der Verstärkungsglieder.

Doppelte, hohle, vierkantige Träger, die ununterbrochen über drei Stützen verlaufen. Diese Träger sind von dünnen, biegsamen Stützwänden getragen, die sie gegen Torsion festhalten, welche aus der durch die den 33 Fuss weiten Zwischenraum abdeckende Zwischendecke bedingten exzentrischen Belastung entsteht.

Tragmauern, die sich über die ganze Breite der Konstruktion erstrecken und deren Gewicht über die Länge der Pfeiler verteilen.

Pfeiler, bestehend aus kräftigen, hohlen Tragwänden, die den unabhängigen Tragmauern jeden notwendigen Schutz und, in ihren Oberteilen, den notwendigen Rückhalt bieten, um Längsverschiebungen zu begrenzen.

Bei der eigentlichen Bauarbeit erzielte man die notwendige Gewichtsverteilung und Höhenberichtigung mittels hydraulischer Pressen, die in Aussparungen in den Tragmauern untergebracht wurden.

Summary

This paper gives details of the design of the new highway bridge across the Thames in London.

The new bridge, commenced in 1937, was opened to traffic in 1942.

It replaces old Waterloo Bridge, built to the designs of John Rennie, which was begun in 1811 and opened in 1817. The old bridge consisted of a heavy structure in masonry having nine arches, each of 120 feet clear span, carried on piers 20 feet in thickness, and massive end abutments. The width of the structure was 45 feet. In 1923 foundation failure commenced at one of its piers in consequence of which restrictions were imposed on traffic and as a further precaution a temporary bridge was erected 30 feet clear downstream. With the completion of the new bridge, the temporary bridge has been removed. Between the years 1923 when foundation trouble commenced and 1934 when it was finally decided to remove the old bridge, there had been much controversy as to whether the old bridge should be reconditioned and widened or removed. The decision for its removal rendered it possible to design in reinforced concrete a new bridge incorporating the existing approaches, having five spans each nearly 240 feet clear, an overall width of 83 feet and 3 feet more headroom for the passage of river traffic.

The main feature in the new bridge consists in its slender outline whereby the existing approaches at a level of some considerable height above the banks on either side of the river are joined by a structure having only one foot of camber in its 1 250 feet of length. The load-carrying members are situated below road surface and consist of twin box girders of variable depth each 25 feet in width situated 33 feet apart. These carry an intermediate deck, built monolithic with them. There are four piers in the river and an abutment hidden on each bank. The box members cantilever beyond their supports and carry in the centre a suspended span.

The principal technical features comprised in the design of this bridge include :

Vibrated concrete.

A system of reinforcement welded in situ whereby all necessity for laps, hooked ends & c. was eliminated.

Prestressing of reinforcement in places.

Twin hollow box girders continuous over three supports. These girders are supported on thin flexible bearing walls which fix them against the torsion resulting from the eccentric loading caused by the deck over the 33 foot space between.

Load-bearing walls extending the full width of the structure and distributing its weight over the length of the piers.

Piers consisting of strong hollow shells which provide all necessary protection to the independent bearing walls and, at their tops, the restraints necessary to limit longitudinal movement.

In construction, decentering and correction of levels was affected by means of hydraulic jacks situated in emplacements formed in the load-bearing walls.