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Autor: Kerensky, O.A.

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Contributions libres sur des développements nouveaux importants
Freie Beiträge zu wichtigen neuen Entwicklungen
Free Contributions to Important New Developments

VI

Progrès importants de l'art de l'ingénieur. Constructions mixtes
Bedeutende Fortschritte der Baukunst. Verbundbauten
Important Progress in Bridge and Constructional Engineering
Composite Structures

General Report

O. A. KERENSKY
B. Sc., M. I. C. E., M. I. Struct. E., London

Introduction

The eight papers accepted under Theme VI deal with Atomic Power Stations, Dams, Bridges, Plate Girders, Composite Construction, and Battle Decks. The papers have little in common except that all describe new constructional features. It is, therefore, impossible to make a proper appreciation of the latest developments in the fields covered by them. Instead, the report gives reasonably comprehensive summaries, underlining important features in each paper and suggests subjects for discussion at the Session.

Summaries

1. The two papers by Professor KURT BILLIG are complimentary and give some factual data for, and a general appreciation of, the factors affecting design and construction of the Nuclear Power Stations, with particular reference to the civil and structural engineering problems associated with the Nuclear Reactor.

The Author gives a considerable amount of statistical data relating to six nuclear stations constructed, or under construction, in Great Britain and indicates the lines upon which the development of nuclear reactors is progressing. He states that the capital cost per K. W. for present day nuclear plant is between two and three times that of conventional stations, but the total generating costs per K. W. are very little different.

Development work in nuclear power station reactors has concentrated on the gas cooled graphite moderated reactor and on the possible advantages of heavy water as a moderator.

The main difference between a nuclear power station and a conventional one lies in the reactor unit itself, which is composed of a pressure vessel, a biological shield and steam raising units. Although the gravity loads in nuclear structures are considerable, the most important problems arise from temperature changes, creep, shrinkage, and variations in moisture content of th econcrete.

The raft has to withstand gravity loads and temperature stresses, together with moments and shears due to the fixity of the shield walls. It is imperative that differential settlements between units of plant are kept to a minimum.

In addition to many of the usual requirements regarding the siting of a conventional thermal power station, in nuclear work consideration must be given to the protection of the public from the hazards of possible radiation due to accidentally released fission products. Nuclear reactors have either to be located in non-populous areas or provided with a container sufficient to prevent the escape of radio active materials in the event of an accident.

The Author suggests that the containment vessel design be based on a limit of energy release equivalent to that just sufficient to melt all the uranium fuel in the core.

As a compromise solution the idea of semi containment is now being considered.

By siting reactors underground advantage can probably be obtained from the point of view of containment as well as appearance and protection in war. At Halden in Norway it has been found that there is little difference in cost between a conventional reactor building and an underground containment.

Due to irradiation most metals become hardened, and limited experiments have shown a slight decrease in creep strength. In soft metals the yield stress is raised, whilst ductility is reduced. Neutron irradiation, in general, tends to raise the transition temperature of a metal, thus rendering it more liable to brittle fracture. Research is being carried out on embrittlement of mild steel under prolonged strong irradiation. Many of the effects of irradiation on metals can be reduced or removed by annealing.

In the case of concrete, radiation damage is not so serious. Tests on concrete specimens carried out at Harwell lead Professor Billig to believe that it is unlikely that there will be any significant change in the properties of concrete in shields already built during the operational life of the reactors.

The Author briefly describes a prestressed concrete pressure vessel used for the reactor at Marcoule where the concrete also acts as a shield, gas tightness being assured by lining the vessel with sheet steel which acted as formwork during the casting.

The action of the shield is to slow down fast neutrons by collision with atoms of light elements and to absorb thermal neutrons and gamma radiation, for which purpose heavy elements are needed. These processes result in a temperature rise within the shield.

The thermal shield should have high density, melting point, high atomic number and good thermal conductivity, be stable under radiation and cheap and easy to fabricate. In practice the choice lies between boron and cadium bearing materials.

Considering the practical aspects of concrete construction the Author states that certain forms of cracking do not reduce the effectiveness of concrete as a shield, and in addition suggests that in the past specifications have been too stringent in respect of uniformity of density of shield concrete.

It appears that experimental evidence is lacking in support of many assumptions used in shield analysis, but measuring instruments are now being included in the shields under construction. At Hinkley Point this work is being supplemented by a comprehensive investigation of the site concrete.

In full scale work, only conditions arising from normal reactor operation can be investigated. Whilst laboratory work is not limited in this way, shrinkage, creep, moisture and thermal effects cannot be truly simulated in scale work. By comparing the information obtained from the full scale measurements of these phenomena with the behaviour of models, it is hoped to discover methods of using models to predict full size behaviour.

2. In the paper on "Le barrage de Tourtemagne en Valais (Suisse)" Professor F. PANCHAUD describes the application of prestressing adopted in the construction of a 30 metres high Thin Arch Dam which retains the glacier waters in the 850,000 cubic metres capacity reservoir.

The necessity for prestressing was mainly due to three factors:

1. The decision to use a thin structure.
2. The location of the dam in an area subject to severe climatic conditions.
3. The function of the reservoir which involves its being emptied and filled at all seasons with glacial water.

The dam closes a narrow gorge and is composed of a thin vertical cylindrical arch 1.20 m thick and with a maximum height of 28.5 resting on a pedestal of concrete built into the bottom of the gorge. At the level of the coping, the length of the structure is about 115 m. On the right bank the arch is prolonged by a rectilinear wall of no great height forming a gravity dam about 30 m long. Thus the developed length of the arch portion of the dam is 85 m. The directrix of this arch is formed by a series of segmental arches whose radii increase from the crown to the springings from 20 to 50 m. A gallery 1.20 m wide located on the foundation rock, runs around the periphery of the dam embedment.

When the lake is empty only thrusts due to thermal effects are developed in the arch.

A network of vertical cables starting from the coping and ending in the gallery at the foot of the dam compress the horizontal sections of the vault and assist in combating the bending forces. The trace of the cables in each vertical section was chosen so that no tension appears in these sections however full the lake may be.

The horizontal prestressing was obtained by means of curved horizontal cables following the form of the arches and ending in the galleries at the abutment in each bank. The cables were supplied with sheaths and placed on the horizontal concrete construction joints at 1 m centres.

Four vertical temporary expansion joints were provided starting from the foundations and dividing the vault of the dam into five voussoirs of 15 and 17 m long. Moreover, in the neighbourhood of the coping two construction slots were left open which will only be closed after the prestressing has been completed and after a cold season.

The expansion joints were provided with Freyssinet flat jacks of 42 cm diameter which enable the shortening of the voussoirs, due to the prestressing, to be compensated for by opening the expansion joints proportionally thus maintaining the horizontal prestressing force. The force exerted by the jacks at each level was chosen to be slightly above that produced in the same section by the horizontal cables, so that at the springings a permanent compression is opposed to all forces which would uproot the foundation.

The Author claims that the construction of the Tourtemagne Dam is the first application of prestressing to thin arch dams.

For combating the tensions due to intense thermal effects the solution adopted is technically more satisfactory than that which consists of strengthening the sections by passive reinforcement.

Prestressing cables and flat jacks have been used in other dams to assist in resisting the primary forces due to impounded water but this dam employs a unique combination of prestressing cables and flat jacks to resist the secondary stresses due to temperature changes.

Prestressing cables were used to keep the arch homogeneous and watertight by neutralising tensile stresses due to thermal action and to control stresses due to boundary effects adjacent to foundations. Flat jacks inserted in expansion joints were used to:

a) Maintain the geometric shape of the structure by "taking up" the shortening of the concrete due to horizontal prestressing and thus eliminating unwanted secondary effects of the prestressing.
b) Push the sides of the dam into the rock abutments.
c) Enable the expansion joints to be kept open until the best closure temperature of the dam was reached.

The Author does not state how the actual construction worked out in practice and whether the scheme proved economical.

3. The paper by Mr. ROBERT SAILER describes the Colorado River Bridge, completed in February 1959. The bridge is spectacular in appearance, crossing Glen Canyon 700-ft. above water level with a single span of 1028-ft. It is the highest arch bridge in the world and the second longest of its type in the U.S.A., but it incorporates few new features which call for discussion.

The bridge is designed to A.A.S.H.O. Specification to carry two lanes of H. 20 – S. 16 loading on the 30-ft. wide roadway. Wind load was taken as 75 lb./sq.ft. without live load, and 25 lb./sq.ft. with live load. A temperature range of $\pm 60°$ F. was adopted.

Studies were made of fixed, 2-hinged, and 3-hinged arches, and, in the case of the fixed arches, solid ribs were compared with trussed ribs. Although somewhat less economical, trussed ribs were preferred as they were less flexible, and subject to smaller wind loads. The 2- and 3-hinged arches were both about 14% lighter than the considered fixed arch. Finally, a 2-hinge trussed arch was selected.

The deck system consists of a 6-in. R.C. roadway slab (4-in. footway) on continuous wide flange beam stringers. But it is not clear whether composite action has been considered.

At the crown the floor beams (cross girders) rest directly on the ribs and the floor system is held longitudinally at this point. In order to reduce secondary stresses due to interaction of floor system and ribs, disc bearings allowing rotation longitudinally and transversely are used for these floor beams and also for the columns supporting the floor beams over the central portion of the arch. The longer columns have riveted connections.

Horizontal wind bracing is provided between abutments and Panel Point 12 primarily for wind stresses during erection, the slab being effective for transverse winds in the final structure.

Field connections of trusses and bracing are riveted, and those of columns and floor are bolted with high tensile bolts — presumably grip bolts.

The 10-ft. wide concrete skewbacks are set 16-ft. into the sandstone and are designed so that under service conditions tension never occurs at the backface but anchor rods are provided to cater for any tensile forces which might occur during erection.

The bridge was erected by the cantilever method with two sets of tie-back cables, using a 25-ton capacity cableway spanning 1540 feet. An auxiliary cableway for personnel was also installed. The cables were moved forward panel by panel as erection proceeded, as far as Panel Point 15, the final 130 feet of each side being cantilevered out.

The arch was closed on 20-in. diameter steel pins in the upper chords, the tie-back cables were then slackened off. The erection of the floor system proceeded from the centre outwards. The whole of the steelwork was erected in7 months.

The concrete deck was completed in 12 days using metal forms which were left in place. To avoid overstressing the centre section of deck was placed first, followed by the end sections and finally the sections above the quarter points.

It would be interesting to know:

1. Whether composite action of R.C. deck slab and steel stringers has been considered and, if not, why not.
2. Why some field connections were made with rivets and others with bolts, and what kind of rivets and bolts were used.
3. Are the riveted box chord members considered to be sealed against weather, or will they be painted in the same manner as the rest of the steelwork and what protective system has been adopted for the exposed surfaces.
4. Cost of construction.

4. In his paper on the Tancarville Bridge, Mr. A. Delcamp briefly describes the main features of the bridge, which was opened to traffic in July 1959, and discusses several novel features of design and especially the solution of the aerodynamic problem.

The designers and constructors of the bridge have shown originality in conception and adopted most modern techniques in reinforced concrete, prestressed concrete and steel construction.

The bridge and approach viaducts carry a 40-ft. roadway and two 4-ft. footways. The suspension bridge has a central span of 1995-ft. and two side spans of 577-ft. with a clearance for shipping of 167-ft. The approach viaduct on the left bank consists of 8 spans of 164-ft. each, constructed in prestressed concrete.

The left bank anchorage is a gravity one consisting essentially of two walls, in line with the bridge cable, supported on caissons founded on a bed of sand and gravel. The walls are joined at the back by a reinforced concrete box containing ballast, and are capable of slight articulation at their junction with the caissons. In view of the difficult ground conditions the designers are to be congratulated on having evolved a relatively economic anchorage.

The right bank anchorage consists of two separate 160-ft. long prestressed concrete tunnel anchorages in the calcareous rock.

The towers are constructed in reinforced concrete and, at 390-ft. are by far the tallest concrete towers yet built.

It is in the design of the suspended structure that the biggest departure has been made from previous practice in long span suspension bridges. Since the failure, from wind oscillation of the first Tacoma Narrows bridge in 1940 American and British designers have kept the conventionally flexible features of suspension bridge design and directed their efforts towards producing a cross section which was aerodynamically stable. In contrast with this the designers of the Tancarville bridge, whilst keeping the open web stiffening truss which is the most important feature of an aerodynamically stable cross section, have done all they could to stiffen their suspension system. This has been done by interconnecting systems which in most previous designs have been independent. The side span and main span trusses have been made continuous at the towers, the deck has been made to act integrally with the top chord of the stiffening truss, the two trusses have been braced to form a torsion box and the cables have been clamped to the tops of the stiffening trusses at the centre of the main span to prevent relative longitudinal movement. These measures greatly complicated the analysis of the structure as it increased the number of static indeterminances from 1 to 4.

It is a pity that the account of the aerodynamic tests is incomplete and rather confused. The ignoring of the effects of symmetrical oscillations seems to require further justification.

The paper concludes with a very interesting description of some of the special construction details in the bridge. Apparently all the site joints were made with rivets; presumably in France riveting is cheaper than grip bolting.

5. Messrs. TROTT and WILSON's paper describes tests carried out over a period of eight years to find a suitable form of asphalt surfacing for the decks of Severn and Forth suspension bridges.

All the test panels were laid in a heavily trafficked road and some have now been in service for six years. A 12½ ton test load was applied to each panel and no cracks appeared in the asphalt. The Authors also investigated the contribution of the asphalt surfacing to the rigidity of the deck plates; it was found that there was very little contribution at summer temperatures, but under winter conditions the 1½ inch asphalt increased by about 80% the rigidity of the steel decking.

In the first experiment four steel panels ½ inch thick were set in the roadway and surfaced with mastic asphalt. Three of the panels were smooth and had different thicknesses of asphalt: 1″, 1½″ and 2″, the fourth panel had a chequered pattern and 1½″ of asphalt. The steel panels had previously been sand blasted and coated with bituminous paint. During the first year of service the bond failed at the edges of the panels, and rust had penetrated 7″ in from the edge. Over the next four years cracks appeared in the asphalt over the deck stiffeners, penetrating the 1″ asphalt right through to the steel deck, but in the case of the 1½″ asphalt the cracks appeared only in the surface. Apparently there was no evidence of the asphalt pushing on either the smooth or chequered panels except where the bond had failed at the edges.

The second experiment commenced when the first two panels were removed and replaced by a new larger panel consisting of three steel plates of different thickness, ½″, $^9/_{16}$″ and $^5/_8$″, welded together. The panel was sprayed with 0.002 inches of zinc to gain improved protection from corrosion, and then treated with bituminous paint. Half of the panel was surfaced with 1½″ of stone filled mastic asphalt and the other half with $1^1/_8$″ of stone filled mastic on $^3/_8$″ of damp proofing mastic. The $^3/_8$″ mastic was returned into an angle welded to the edges of the panel. The edges of the $1^1/_2''$ asphalt being sealed against the edge angles by a rubber-bitumen compound. Apparently there was little difference between the condition of the asphalt on the $1/2''$, $^9/_{16}''$ and $^5/_8''$ deck panels after $5\frac{1}{2}$ years of service, but there was some corrosion at the edge joint between the angle and the two course mastic. The rubber-bitumen joint was found to be entirely successful.

6. Messrs. BASLER and THÜRLIMANN of Lehigh University give and discuss results of 15 tests on 7 full-size welded Mild Steel plate girders. The paper makes an important contribution to the factual data on which the design of slender webs should be based. Girders with webs of slenderness ratio of 185 and 388 were tested in bending and with ratio of 255 and 259 in shear, with vertical stiffeners spaced apart at $\frac{1}{2}$, $\frac{3}{4}$ and $1\frac{1}{2}$ times depth of web. No horizontal stiffeners were used.

The tests show yet once again, that the web critical loads in bending or shear have no real significance in assessing the actual load carrying capacity of slender plate girders.

The Authors do not attempt to develop a new design method, but in discussing the results they conclude that the classical critical load theory based on an elastic behaviour of a perfect web is unable to predict the carrying capacity of girders subject to shear or moment. They assert that a panel of web should not be considered as an isolated element, but that it is of the utmost importance to investigate the strength of the supporting frame, consisting of the flanges and the transverse stiffeners.

As far as your reporter knows of all modern Specifications only the British Standard (B.S. 153) published in 1958 takes into account the postbuckling strength of effectively stiffened webs and further study of the strength of plate girders about to be undertaken by Lehigh University should be welcomed by all concerned with the economic design of deep plate girders. There is no doubt that more tests are required to determine the effects of combined shear and bending into the buckled range and also the influences of the web on the stability of thin compression flanges.

7. Professor GIBSCHMANN describes the use of precast reinforced concrete units in composite action construction in the U.S.S.R. He states that combined action construction is considered to be very economical and is much used. To expedite construction, particularly in the winter, and to reduce shrinkage and creep effects, the R.C. slab is often made up of precast units. Many large span bridges have already been built using this form of construction, notable among them being the 3-span continuous girder bridge in Moscow with spans of 72.6 m, 108 m and 72.6 m. A further and novel development is the provision of a precast concrete bottom flange in the compression zone of continuous girders. The precast units are attached to the supporting girders either by steel shear connectors which are welded to the steel flanges and concreted in situ into the specially provided holes or grooves in the precast units or by

steel lugs or plates cast into the precast units and welded on site to the sides of the steel flanges.

The edges of grooves or holes formed in the precast units must be castelated to ensure a good bond between the in situ concrete filling and the precast units. A great deal of research has been carried out in the U.S.S.R. on the behaviour of composite decks and in particular on the design of shear connectors. The best type of connector was found to be of the rigid dowel type with oblique tying in rods welded to it. It was found that the anchoring of the concrete to the flange against vertical uplifts improves the carrying capacity of the composite unit. These findings seem to be in agreement with the German ones. When welding the cast in connectors to the steel girders, care must be taken not to damage the concrete by the heat from the arc and by the changes in length of the steel elements. To eliminate both these dangers the distance of the weld from the concrete must be appreciable and the welds as thin as possible.

Tests on a complete bridge show that precast concrete combined action deck behaves very similarly to the cast in situ one. In the calculations due account is taken of the shrinkage, creep and temperature effects. This again conforms to German regulations, but the American Specification suggests that these factors can be ignored.

Comments and Subjects for Discussion

The two papers by Professor BILLIG survey the state of Atomic Power Station developments in Great Britain, and deal with the structural problems only in general terms. They should give a useful background to the workers in this new field.

Points of particular interest, suitable for discussion are:

1. The siting of reactors underground.
2. Relative merits of prestressed concrete and steel pressure vessels.
3. Design and construction of reactor shields.

In his paper on the Tourtemagne Dam, Professor PANCHAUD describes the use of flat jacks and prestressing cables to control the effects of temperature changes and shrinkage in mass concrete structures, which should have useful application in structures other than dams, the atomic shield coming immediately to mind.

Mr. R. SAILER describes the considerable engineering achievement in bridging the 1000-ft. wide and 800-ft. deep cannion, but the design of the bridge itself is orthodox and hardly calls for further discussion.

Mr. A. DELCAMP's paper on Tancarville bridge contains a brief description of many bold and novel features in the design of long span suspension bridges. Although the adopted solution of the aerodynamic problem would appear to be somewhat elaborate it is certainly thorough. However, its cost should be compared with that for other large bridges. The adoption of a composite steel-concrete deck acting in combination with the stiffening trusses, and the problem encountered in the design and construction of the 390-ft. high reinforced concrete towers, should be of great interest to all bridge engineers and merits further discussion.

With this can be linked the factual paper by Messrs. TROTT and WILSON on the development of asphalt surfacings laid directly on the steel plate (as has already been successfully done on many German bridges) instead of on an intermediate layer of concrete provided in the Tancarville bridge.

A collection of data on the behaviour under traffic of mastic asphalt laid on steel battle decks would make a valuable contribution to the solution of this problem.

Messrs. BASLER and THÜRLIMANN's paper describing some buckling tests on plate girders makes a useful addition to our knowledge of the behaviour of slender webs. Since the war much work has been done in this field, but entirely satisfactory design rules, based on the actual behaviour of web and stiffeners, rather than on theoretical critical loads, have not yet been formulated. The main difficulty arises from the unavoidable presence of initial imperfections, which rather paradoxially make all rules based on the behaviour of a perfect plate, somewhat unrealistic and wastefull.

It would appear that the ultimate strength method of design of slender plate girders should be more generally accepted and a discussion of this problem could make a valuable contribution at this juncture.

Professor E. GIBSCHMANN's paper on the use of precast reinforced concrete units in composite constructions is the only one directly dealing with this form of construction, which is especially mentioned as a desirable theme for the Free Session. Composite construction is still in a state of development and has not been universally adopted.

Although a few countries have produced tentative Specifications during the last few years, often the method of design and even the allowable stresses are left to the discretion of the designer.

The various fundamental differences between, say, the American and German Specifications clearly indicate that full exchange of information and a critical appraisal of the different practices is a matter of some urgency.

The following items should form a basis for a useful discussion:

1. Best type of connectors.
2. Allowable stresses at connectors.
3. Effects of, and allowances, if any, for shrinkage, creep and temperature gradients.
4. Relative merits of precast and cast in situ composite section decks, especially with reference to continuity over supports and possible corrosion of members in contact and of exposed shear connectors.