Artist's conception of bird's-eye view of future Haringvliet Sluices.

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INVESTIGATION OF A MODEL OF THE NABLA GIRDER

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The construction and the method of testing a model (scale 1 :15) of a box girder of triangular cross-section are described. Apart from dead weight, the girder is subjected mainly to concentrated loads. Some of the results obtained are reported by way of illustration.
The prestressed concrete girder is assembled from precast units. As the behaviour of the structure had to be investigated not only in the elastic range but also in the cracked state and at failure, it was necessary to construct a so-called realistic model, i.e., a model which resembles the actual structure as closely as possible both in construction and in material properties.

0 Introduction

One of the works being carried out in connection with the Delta Scheme,[1]) under the direction of the Rijkswaterstaat (Netherlands Government Department of Waterways and Highways), is the construction of an outlet sluice, approximately 1000 m in length, in the dam across the Haringvliet near Hellevoetsluis. The sluice has 17 discharge openings, the piers separating these openings being spaced about 60 m apart. Each opening is spanned by a prestressed concrete "box" girder of triangular hollow section and can be closed by two segmental gates which have a stand-by function in relation to each other (see Fig. 1). These two gates are each connected to the girder by means of four steel "arms". The arms are attached to the gates by rigid joints, whereas their connections to the girder are pivoted. The gates can be raised by means of hydraulically operated machinery mounted on the piers.

Having regard to the direction of the forces to be resisted, and in order to reduce the impact effect of the waves, the triangular cross-sectional shape, with the apex pointing downwards, was chosen for the concrete girder. It was this shape that suggested the name "Nabla" girder.[2]) Also, this shape would enable the using top of the girder (i.e., the "base" of the triangle) as a roadway for traffic.

The water pressure acting on the two gates is transmitted through the arms and produces concentrated compressive or tensile forces on the girder. The lines of action of these forces form an angle of about 30° with the horizontal. A mainly uniformly distributed loading due to dead weight and live load (rolling loads from traffic) must furthermore be taken into account. Evidently a large

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[1]) The object of the Delta Scheme is to prevent the recurrence of the floods that ravaged the south-western part of the Netherlands in February 1953. It involves the damming of the mouths of the tidal estuaries (one of which is the Haringvliet) constituting the combined delta of the Rhine and Meuse.

[2]) I.e., the name is derived from the mathematical symbol "nabla" (also known as "del"): ∇.

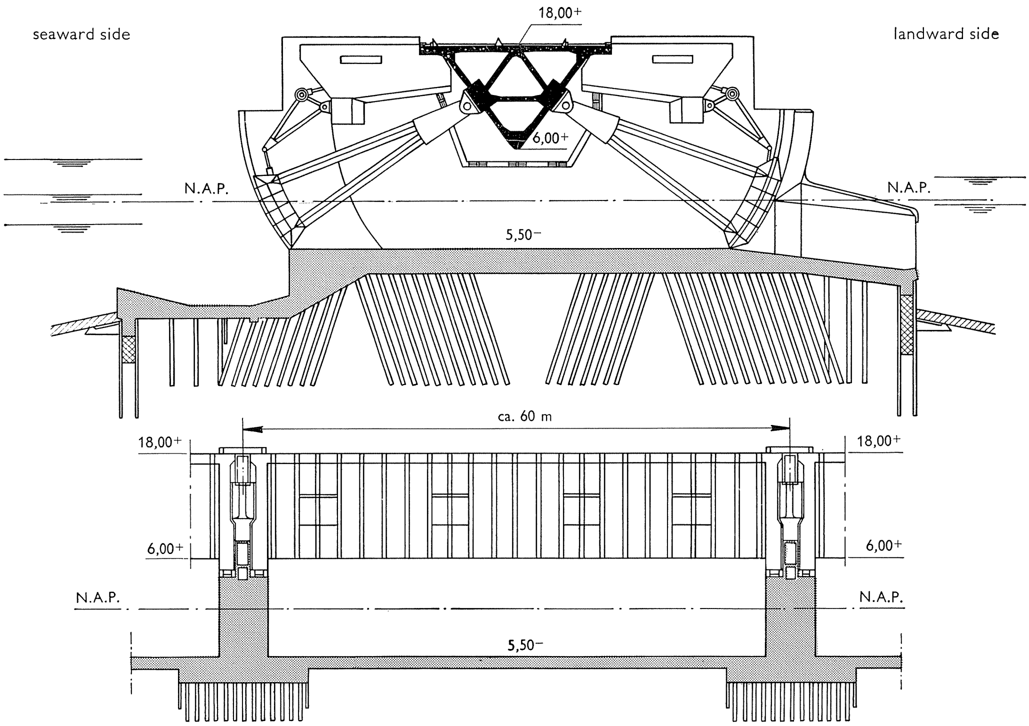


Fig. 1. Diagram representing one discharge opening of the outlet sluice.
N.A.P. = Netherlands Standard Datum.

number of load combinations are possible. One of the requirements was that, under the design loads, none of the possible load combinations should cause tensile stresses in the girder. A further requirement was that for a loading corresponding to 1.3 times these design loads there should on no account be tensile stresses exceeding 10 kg/cm^2. In addition, failure of the structure, when subjected to the most unfavourable loading condition, was not allowed to occur at less than 2.2 times the design load.

It is, of course, by no means a simple matter to determine the distribution of forces in such a girder. When at last the design, after evolving through various stages, reached its final form, it was felt necessary to verify the effects of various load combinations with the aid of a model. The construction and the method of testing this model will be described in this article. Some results will, by way of illustration, also be reported.

1 Description of the girder

As shown in Fig. 1, the cross-section is composed of several triangles. The maximum width is 22.40 m and the depth is 12 m. The external walls are 60 cm and the internal walls are 50 cm thick. The girder is assembled from 22 precast units ("slices") and two end diaphragms. Each unit is approximately 2 m in length and weighs 250 tons.[1]) In between the units are "joints" consisting of 50 cm wide gaps filled with concrete. In order to obtain good resistance to shear between one unit and another, the faces coming into contact with the joint-filling concrete have been given a special profile. In addition, the mild steel reinforcement (deformed bars) installed in the precast units continues into the joints.

The girder is prestressed both longitudinally and transversely. In the longitudinal direction 193 cables (B.B.R.V. prestressing system) are provided. Each cable comprises 54 wires of 6 mm diameter, developing a permanent prestressing force of 137 tons. Straight as well as parabolically curved cables are employed. In the transverse direction the prestress is applied by means of the FREYSSINET system, each cable comprising 12 wires of 7 mm diameter with a total prestressing force of about 43 tons. Each girder (not counting the end diaphragms) is provided with about 1100 transverse cables in all. The end diaphragms are each prestressed with about 150 cables (in part B.B.R.V. and in part FREYSSINET cables). Figs. 2 and 3 show the location of the prestressing cables in the girder.

The order of magnitude of the main forces acting upon the girder is:

pressure on seaward side	220 t/m
tension on seaward side	44 t/m
pressure on landward side	30 t/m
tension on landward side	100 t/m

The dead weight of a girder is about 125 t/m, and the uniformly distributed load is 1 t/m^2. Some important load combinations are indicated in Fig. 4.

[1]) By "ton" (abbreviated "t") is always meant the metric ton of 1000 kg (=2205 lb.).

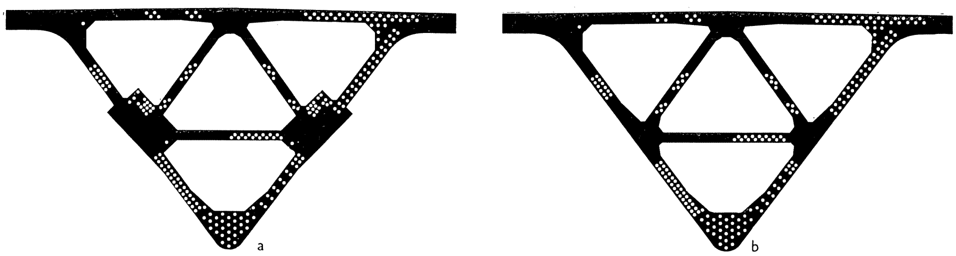


Fig. 2. Location of the prestressing cables in the longitudinal direction:
a. in those precast units provided with thickened portions (at the gate pivots) for resisting the concentrated loads acting upon them;
b. in the other precast units.

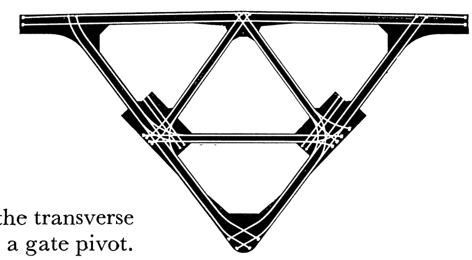


Fig. 3. Location of the prestressing cables in the transverse direction in one of the precast units at a gate pivot.

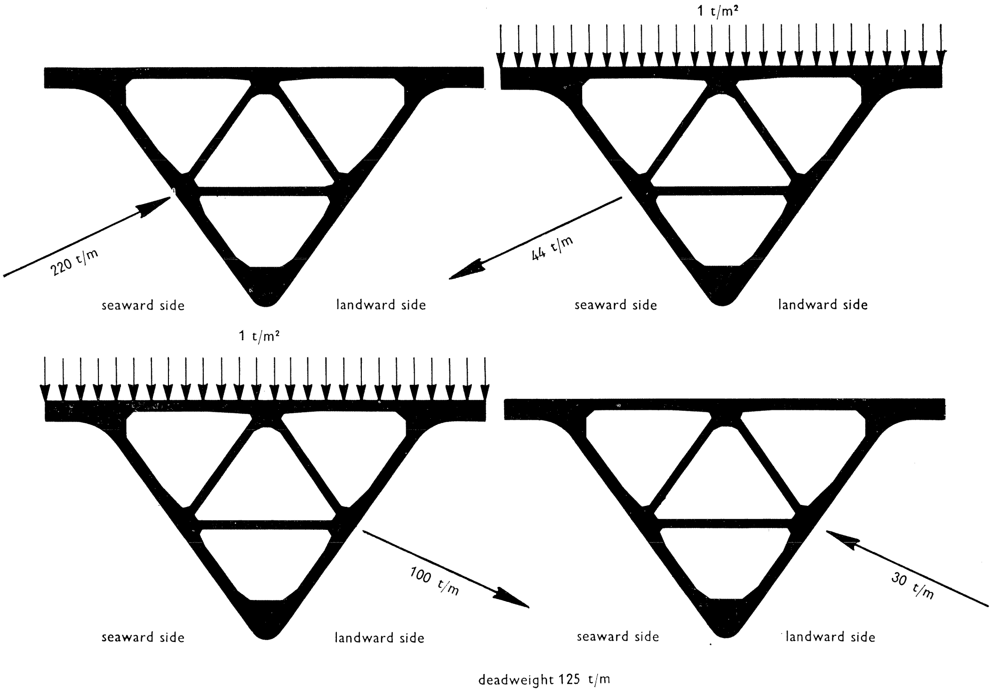


Fig. 4. Some important load combinations.

2 Model similarity rules and scale factor

To investigate the behaviour of a structure under the action of a loading it is often possible to make use of a model. If it is desired to observe this behaviour not only in the elastic range but also in the cracked range and at failure, it will be necessary to use a so-called "realistic" model, i.e., the model must be a geometrical reduction of the actual structure besides having material properties which are as similar as possible to those in reality. Provided that these conditions are satisfied, such phenomena as cracking, yielding of the reinforcement, fracture, etc. will occur in the model of a concrete structure in the same places and at the same stresses as they would in reality.

If the model is a $1 : \lambda$ scale reduction of the actual structure, then, for stresses equal to those occurring in the latter, the following ratios for the principal quantities will exist between the model and the structure:

strains	$1/1$
forces, moments per unit length	$1/\lambda^2$
displacements, distance and crack width . . .	$1/\lambda$

The weight of the model is $1/\lambda^3$ times that of the actual structure. From the rules of model similarity it follows that the stresses due to deadweight in the model will have their correct value only if the total load on the model is equal to $1/\lambda^2$ times the weight of the full-sized actual structure. This signifies that additional loading equal to $(\lambda-1)$ times the weight of the model must be applied to the model. On the one hand the application of this loading constitutes a complicating feature; on the other hand, it enables the behaviour of the structure due to its own weight to be studied.

The size of the model, and therefore the value of λ, is determined by two considerations. For one thing, the model must not be too small to enable it to be properly constructed, but on the other hand, in view of the magnitude of the forces to be applied to it, the model must not be too large either. The model described here was found to satisfy these conditions reasonably well for a value of $\lambda = 15$.

3 Construction of the model

In order to expedite the manufacture of the precast units, eleven moulds were constructed. Each mould was therefore used twice. The material used for making the moulds was 2 cm thick plywood of the kind employed in concrete formwork construction. A cross-section through the formwork of one of the external walls is illustrated in Fig. 5. In order to obtain good dimensional

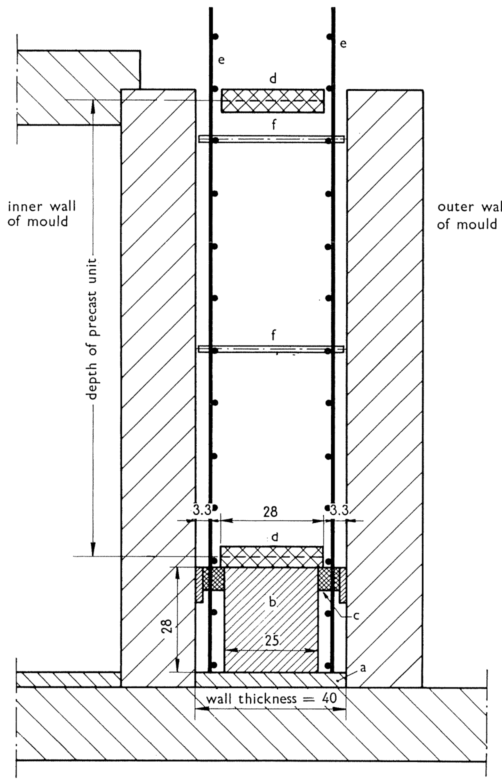


Fig. 5. Cross-section through the formwork for an external wall (dimensions in mm).

a. 4 mm thick plywood template having the cross-sectional shape of the girder;
b. timber block for providing sufficient length for the projecting ends of the mild steel reinforcement;
c. rubber sealing strips;
d. profiled perspex for producing the correct profile for the interface at the joint;
e. mild steel reinforcement;
f. spacers for the mild steel reinforcement.

accuracy, a template was nailed to a 2 cm thick base panel. This template had the same shape as the cross-section of the girder and was enclosed by the mould (see Fig. 6). The external panels of the mould were secured in position by means of wooden chocks fixed to the base panel. The internal panels were pressed against the template on the base panel by the insertion of suitably fitted pieces of plywood. At the top the mould was enclosed in the grooves of a "cover", which was secured to the base panel by bolts.

The ducts in the concrete for threading the prestressing wires were formed both in the longitudinal and in the transverse direction by embedded plastic tubes which were each provided with a steel wire core so as to have the necessary stiffness to resist bending. Fig. 7 shows how the ducts for the longitudinal cables (which are straight within the depth of each precast unit) were held in position: at the base plate they were located in holes in the profiled perspex and they passed through a steel template mounted approximately 35 cm above it. The positions for the transverse ducts were fixed by means of spot-welded spacers inserted into holes in the perspex (see Fig. 8). Fig. 9 is an overall view of one of the end diaphragms showing both the mild steel reinforcement and the devices for forming the longitudinal and transverse ducts fixed in position.

The stiff concrete mix employed had the following composition (parts by weight):

1 part of Class A blast-furnace slag cement (winter grade)
4 parts of aggregate (fineness modulus 2.9)
0.46 part of water

The average 28-day cube strength, as determined from 7 cm cubes, was 439 kg/cm^2 (the specified 28-day strength was 400–500 kg/cm^2). At the time of testing the model (the corresponding age of the concrete being 100–200 days) the average cube strength was 502 kg/cm^2. The flexural tensile strength at that time was 59 kg/cm^2 and the modulus of elasticity was approximately 350,000 kg/cm^2.

The concrete was compacted by a poker type vibrator attached to the underside of the base panel. After two days the concrete units were freed from their forms and the steel cores together with the plastic tubes were removed. Fig. 10 shows a number of completed units, while Fig. 11 shows one of the end diaphragms as viewed from the centre of the girder. For comparison some photographs of the actual girder are also reproduced (see Figs. 12 and 13).

After a so-called erection prestress (which was necessary for the erection of the precast units of the actual structure) had been applied to the units in the transverse direction, the joints between them were concreted. The procedure to be employed on the site – i.e., placing all the units in position one next to the other and concreting the joints almost simultaneously – was not possible in constructing the model because of the inaccessibility of the joints in the internal triangle. For this reason the following system was applied. After an

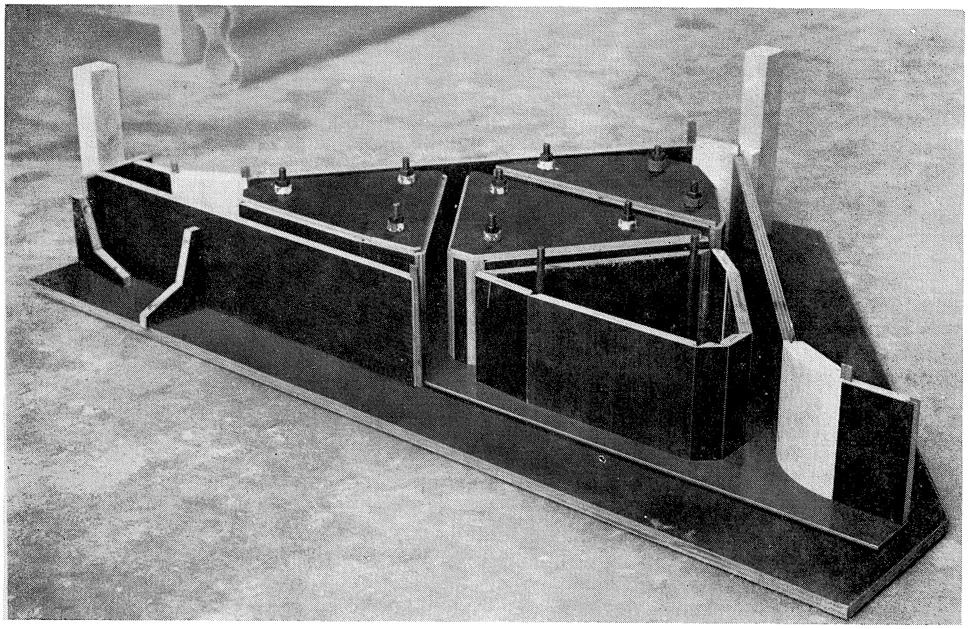
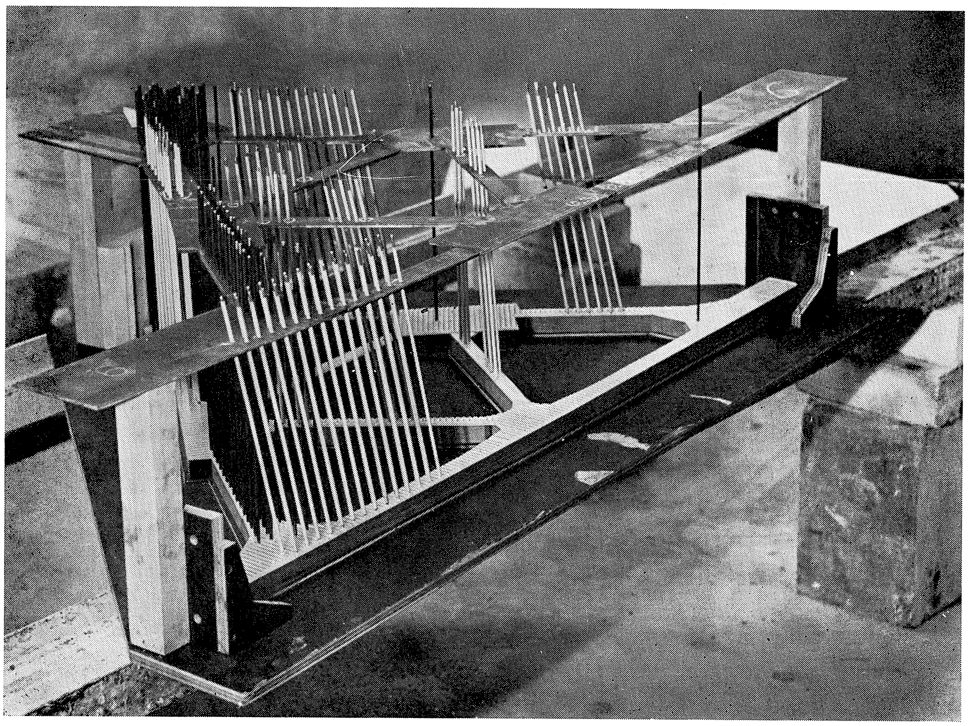


Fig. 6. Overall view of the mould.

Fig. 7. The method of forming and locating the ducts for the longitudinal prestressing wires.



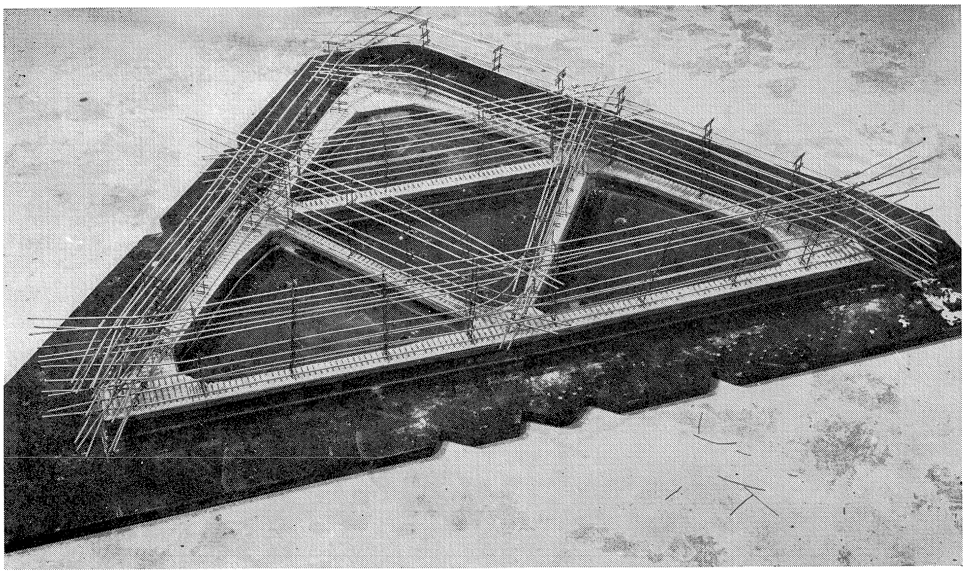


Fig. 8. The method of forming and locating the ducts for the transverse prestressing wires.

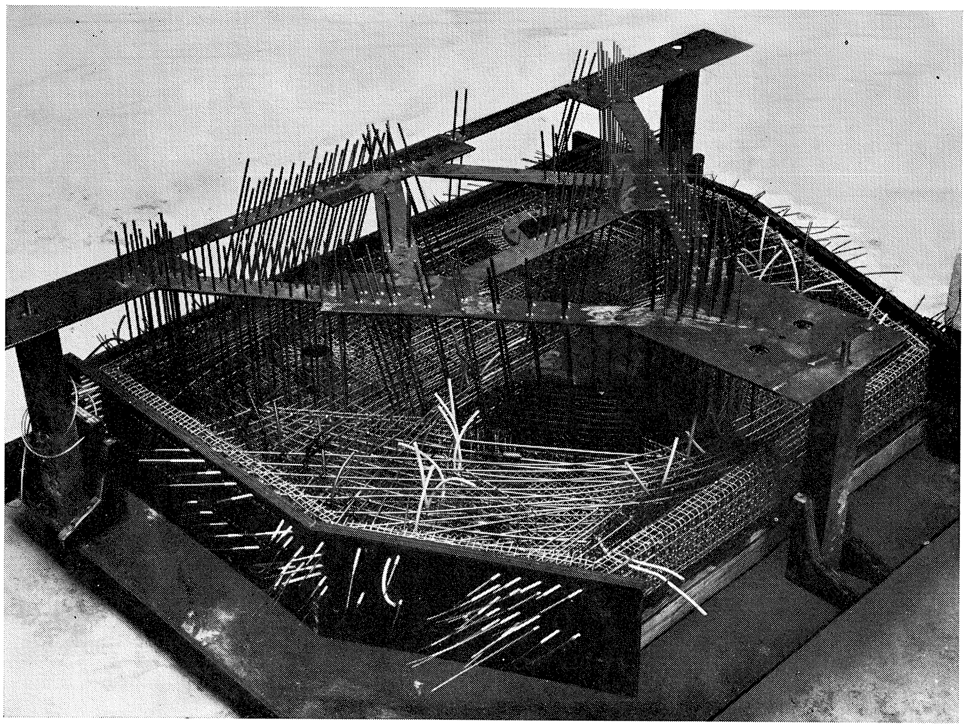
end diaphragm and the adjacent unit of the girder had been correctly positioned in relation to each other, the joint between them was concreted. At intervals of two days a fresh unit was added in this way to the portion of the girder that had already been assembled. The ducts for the longitudinal cables in the joints were formed by drawing rubber tubes through the holes in the units. At the joints these tubes were provided with steel cores (see Fig. 14). In concreting the joints, shuttering was employed on one side only, viz., on the outside or underside. The concrete in the joints was compacted by means of a poker type vibrator.

A transverse FREYSSINET prestressing cable (comprising 12 wires of 7 mm dia.) in the actual girder was replaced in the model by one wire possessing practically the same material properties as the 7 mm dia. wires. The method of anchoring the wire and wedging it after tensioning is illustrated in Fig. 15. Tensioning the wires was performed with the aid of a mechanical device, which is illustrated in Fig. 16. The wire is shown in the tensioned, but not yet wedged, condition. After the wires had been tensioned, the ducts were grouted at a pressure of 6 atm. with a normal type trass-cement grout.

The longitudinal prestress was applied in the same way. The 54 wires of 6 mm diameter were replaced by three wires which were also anchored and wedged in the manner indicated above. Because of the larger forces involved, tensioning in this case was effected by means of a converted single-wire FREYSSINET jack, a sectional diagram of which is shown in Fig. 17.

The whole of the transverse and part (about half) of the longitudinal prestress were applied while the girder was supported over its entire length, as will also be the case in reality. At this stage there were, therefore, no stresses in it due to dead weight. The girder was then placed in position on the "piers". To avoid the occurrence of inadmissible tensile stresses in the concrete, the rest of the longitudinal cables were not allowed to be tensioned until after the compensating forces for producing the dead weight stresses had been applied. This stage corresponds to the actual condition in which the girder is jacked up at the piers and thus becomes detached from the supporting structure (see Fig. 13), after which the rest of the cables are tensioned.

Fig. 9. View of an end diaphragm almost ready for concreting.



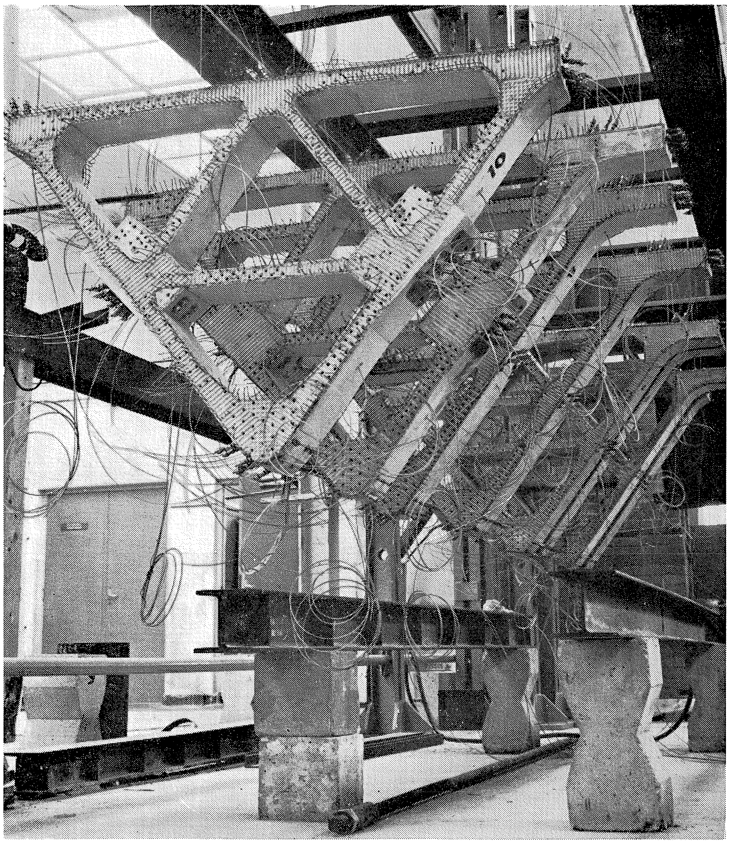


Fig. 10.
Precast units for the model girder.

Fig. 12.
Precast units for the actual girder.

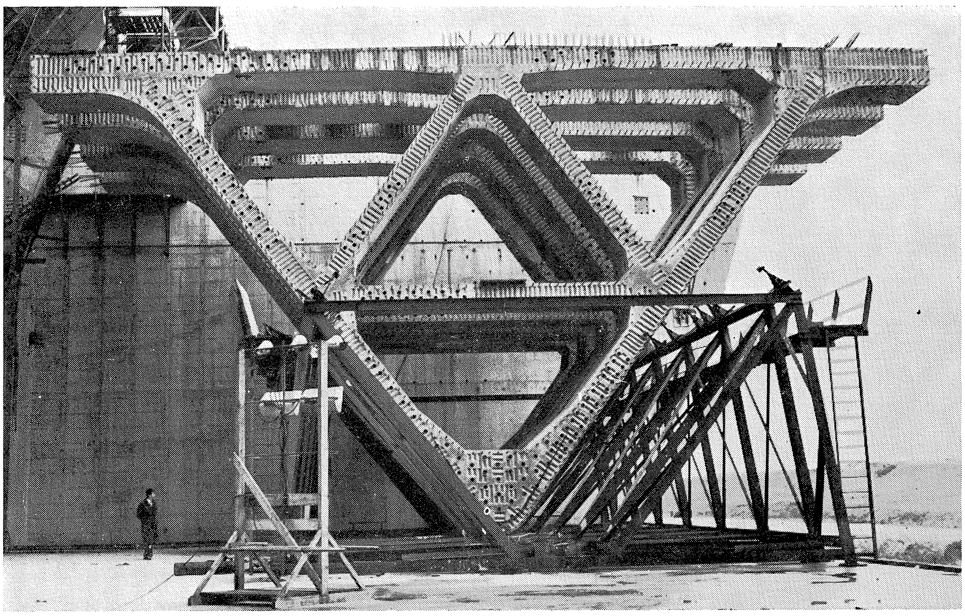


Fig. 11.
An end diaphragm
of the model girder.

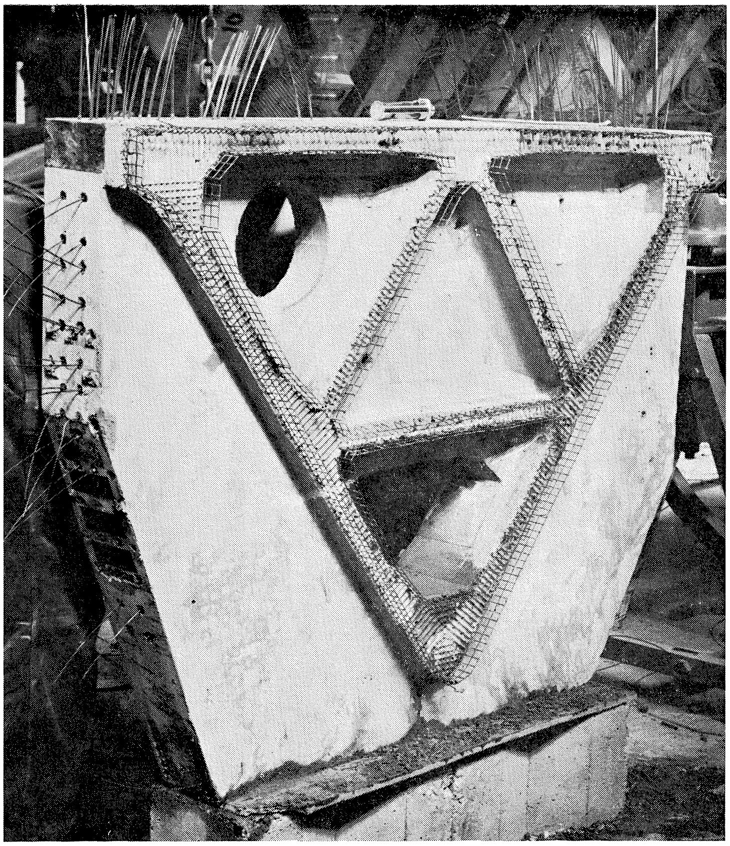
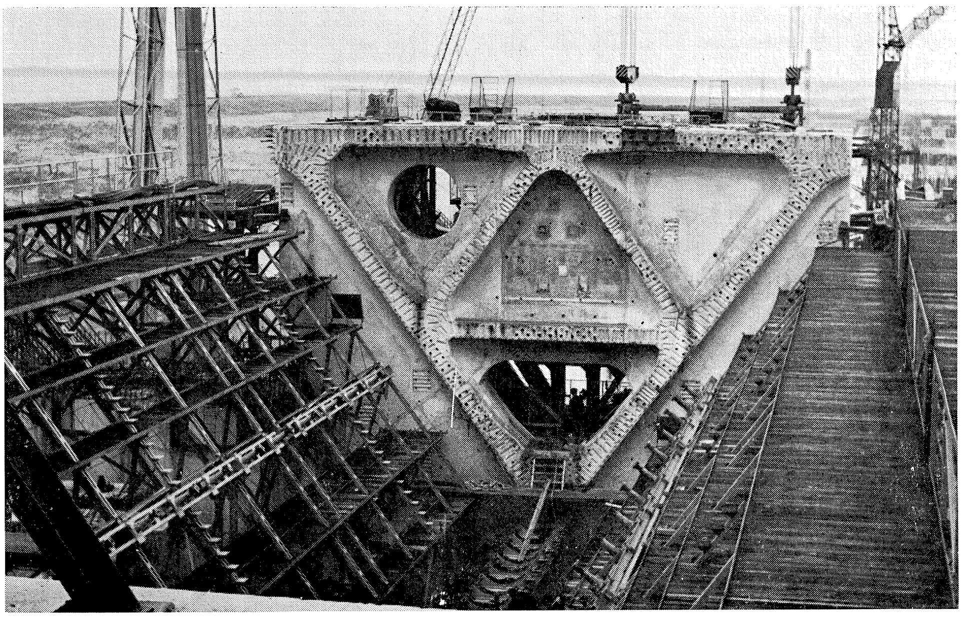


Fig. 13.
An end diaphragm
of the actual girder.



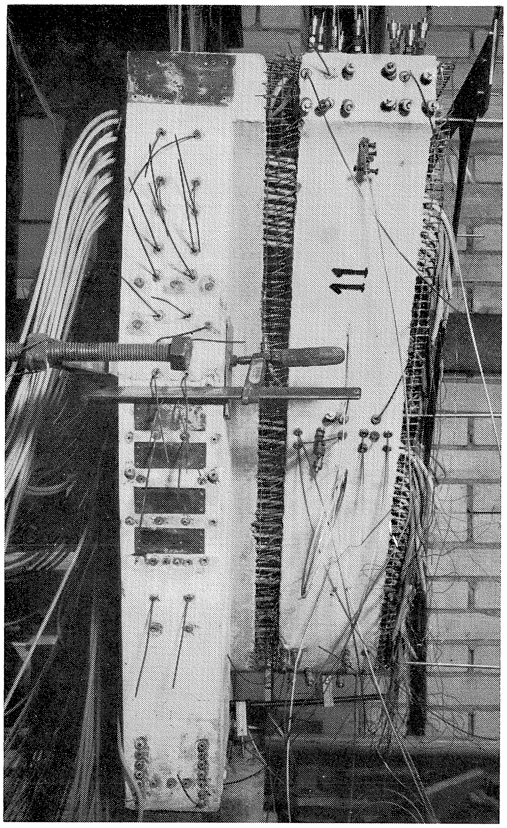


Fig. 14. The method of forming the ducts for the longitudinal wires at the joints. Some anchorages of transverse wires are also visible in the photograph.

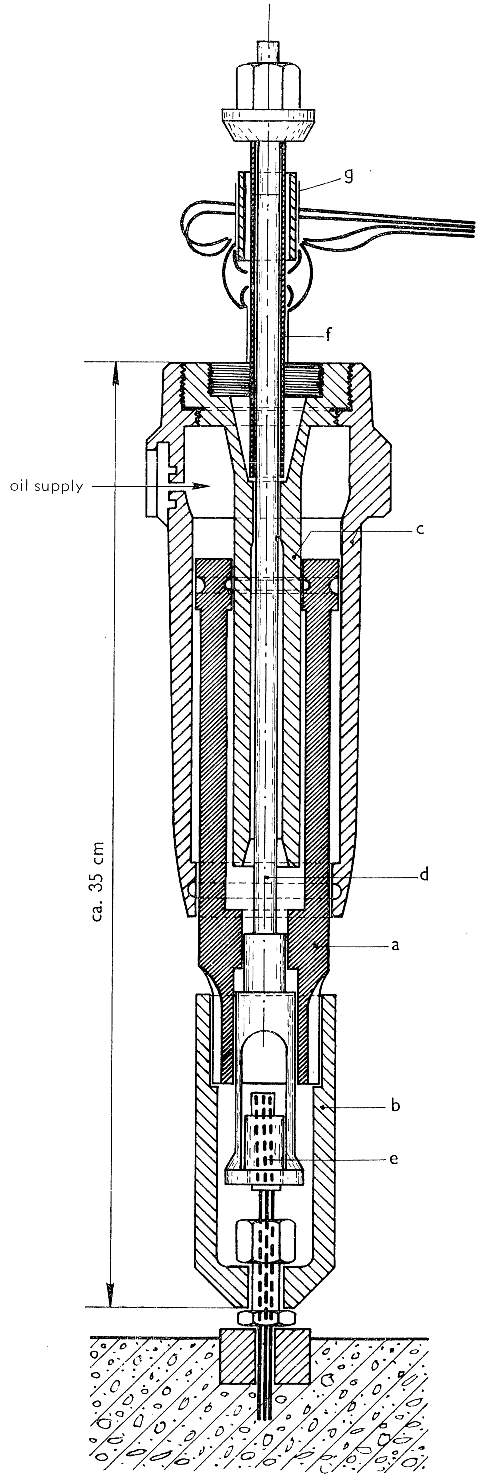
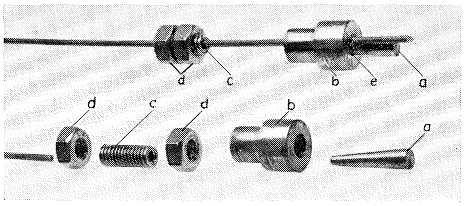


Fig. 17. The jack for tensioning the longitudinal wires.

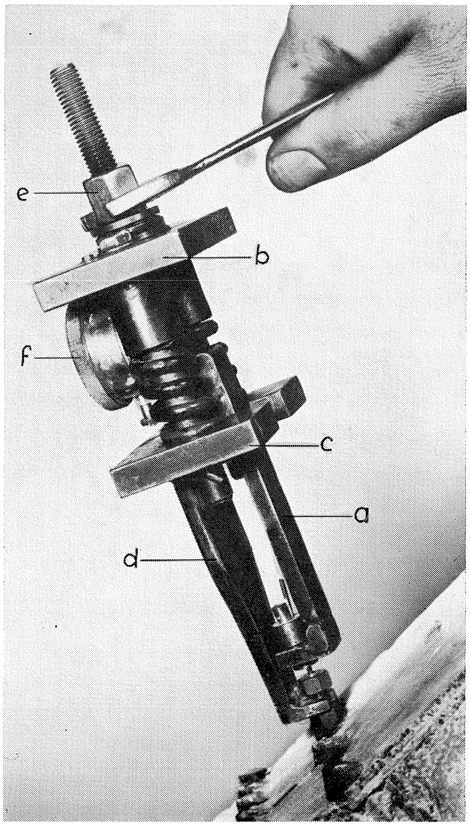
The portion (a) of the jack was strutted against the surface of the concrete by means of an auxiliary component (b). The portion (c) of the jack, which was movable in relation to this strutting component by pumping oil into it, was connected to the prestressing wires through the agency of the rod (d). This rod (d) was, for this purpose, provided with a slotted foot which fitted round the thin portion of the barrel (e). At the upper end of the jack the rod (d) passed through a tube (f) to which strain gauges (g) for measuring the magnitude of the prestressing force were affixed. Pumping oil into the jack therefore caused the rod (d) and the prestressing wires to be loaded in tension, while the measuring element was loaded in compression.

Fig. 15. Anchoring and wedging device for the transverse wires.



After the wire had been threaded through the preformed duct in the concrete, it was anchored by means of wedge (a) in a barrel (b) provided with a conically tapered hole. Wedging the tensioned wire was effected with the aid of an externally threaded sleeve (c). The nuts (d) were screwed an appropriate distance apart so that the force was transferred from the prestressing jack to the wedging device. Grouting was done through the passage (e).

Fig. 16. The jack for tensioning the transverse wires.



The rod (a) was provided with a slotted foot which fitted round the thin portion of the barrel. The upper end of the rod was threaded and it projected through a hole in the plate (b) which bore against the plate (c) through the agency of two compression springs. The latter plate was secured to the rod (d) provided with a foot. Tightening the nut (e) caused the prestressing wire to be tensioned and the springs to be compressed, with the result that the distance between the plates (b) and (c) decreased. The required prestressing force was determined from this decrease, which was measured by means of a dial gauge (f).

4 Loading equipment

The pier construction employed with the model is shown in Fig. 18. For reasons associated with the auxiliary structures necessary for the application of the various loads, the shape of these piers was very different from that in the actual structure. The method of applying the loads will be described with reference to the accompanying diagrams showing the general arrangement of the model girder and auxiliary structures (see Fig. 19 and 20). The arrangement, as viewed from the seaward side, is illustrated in Fig. 21.

4.1 Compensating forces for dead weight

The requisite compensating forces for the deck (top slab) were obtained by means of levers (a) mounted over the girder. By loading these levers with weights at points located beyond the piers a uniformly distributed loading was applied to the deck through the rockers (b) and the distributing structures (consisting of short joists on two supports and boards provided with foam rubber). The compensating forces for the rest of the girder were obtained as follows: at 13 points in each precast unit steel wires (c) were embedded, which were connected through tension springs (with spring constants having different values) to a rolled steel joist placed longitudinally under the girder. With the aid of tension rods (d) which were attached to a frame fixed to the concrete floor this joist was screwed downwards to such an extent as to produce compensating forces of the required value.

4.2 Water load on the seaward side

The compression load acting on the seaward side was applied by means of two jacks (e). The reaction forces for the jacks were provided by a steel beam (f) which was secured to the "piers" by means of eight prestressing rods. The magnitude of the forces was measured with dynamometers (g). The tensile load was applied by tightening the nuts (h) on four tension rods. The force was measured with the aid of dynamometers (j). In addition to varying the magnitude of the compressive and tensile loads, it was necessary also to vary the direction. In the model the change in direction was produced by applying a vertical force at each gate pivot by means of a rod (k). This rod was connected to a lever (l) pivoting about the point (m), and a tensile or a compressive force was applied to the other end of this lever with the aid of two small jacks.

4.3 Water load on the landward side

These forces were applied in the same manner as those acting on the seaward side, though with the difference that jacks were used for producing the tensile

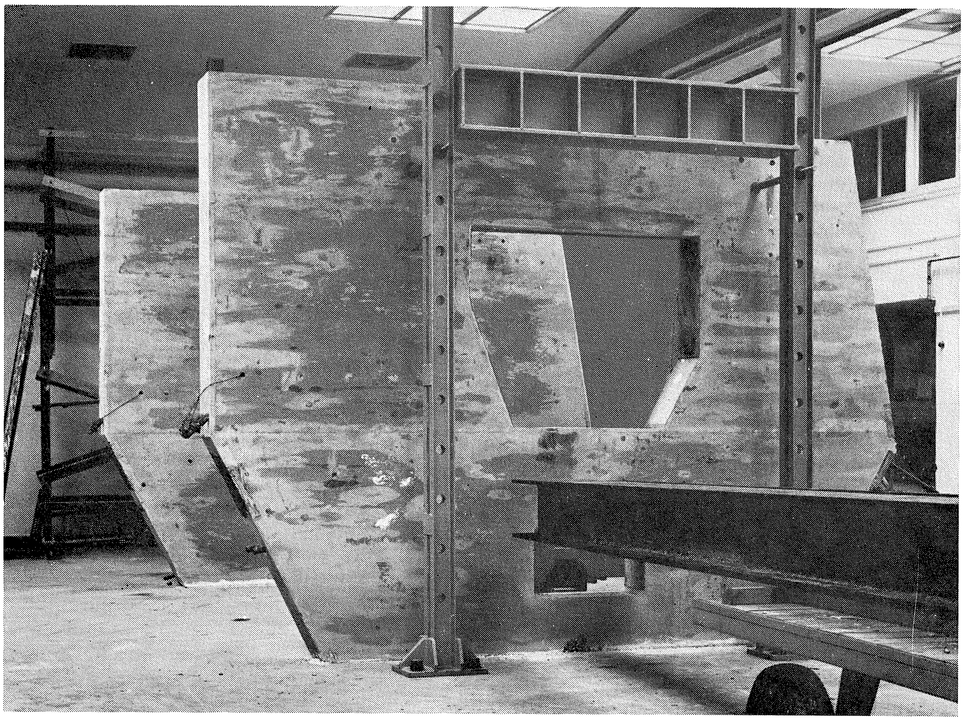


Fig. 18. Pier structure employed with the model.

as well as the compressive forces. The vertical forces at the pivots were obtained in the same way as on the seaward side. For this purpose the equipment illustrated in Fig. 20 was dismantled and was reassembled in similar fashion on the other side of the girder.

4.4 *Live load*

The uniformly distributed live load (due to traffic) was reproduced by using the same equipment that was required for applying the compensating forces for the dead weight of the deck. By this means it was possible to load the whole or part of the deck surface.

4.5 *Rotation of the piers*

In the design the torsional moments occurring in the girder in consequence of possible rotation of the supporting piers were taken into account. To enable this loading condition to be simulated, one of the piers was so constructed as to be capable of rotation, in its own plane, about an axis extending in the longitudinal direction of the girder. The lower aperture in one of the piers was used for this purpose (see Fig. 18).

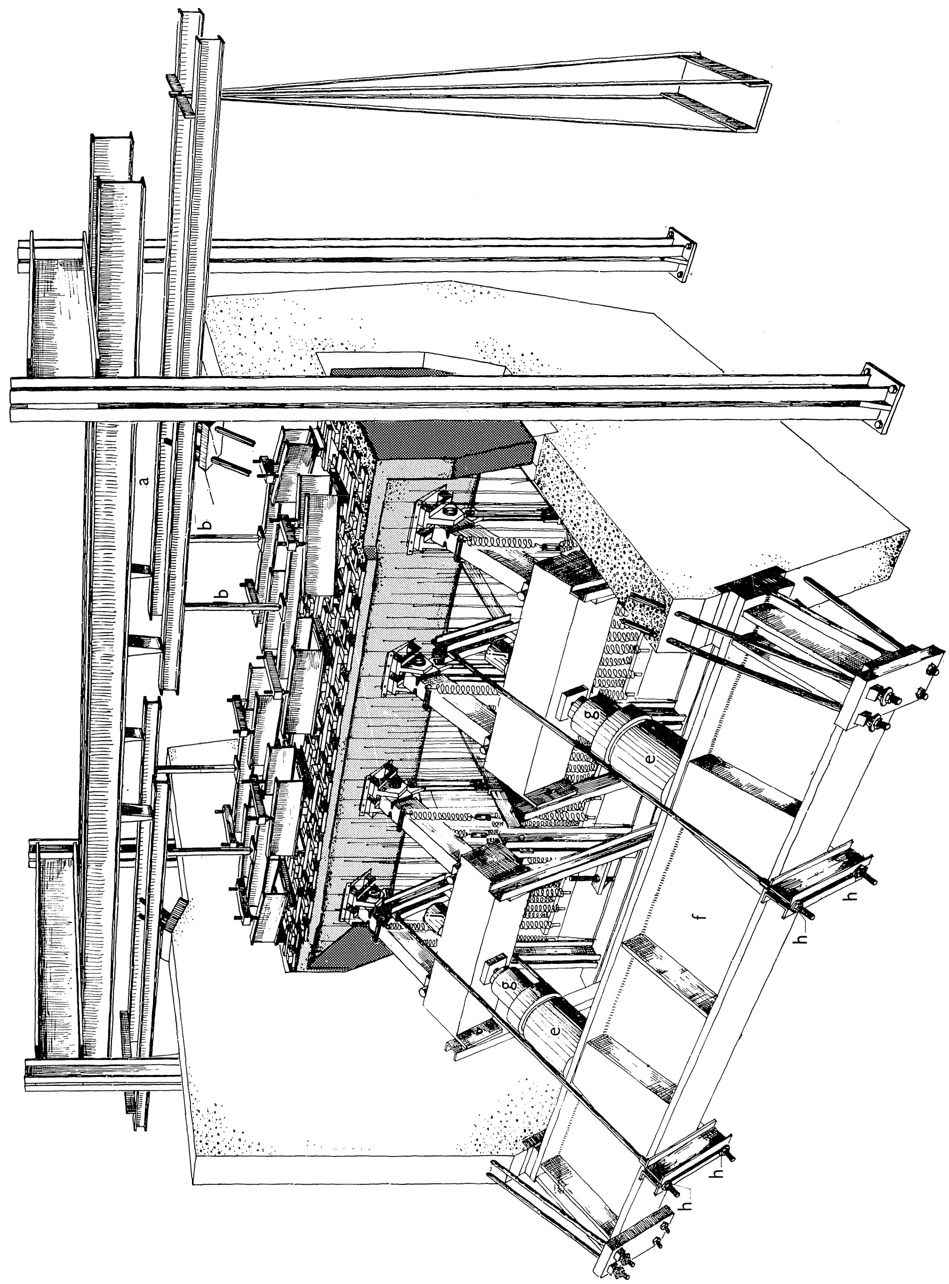


Fig. 19. Perspective view of the test arrangement as seen from the "seaward" side.

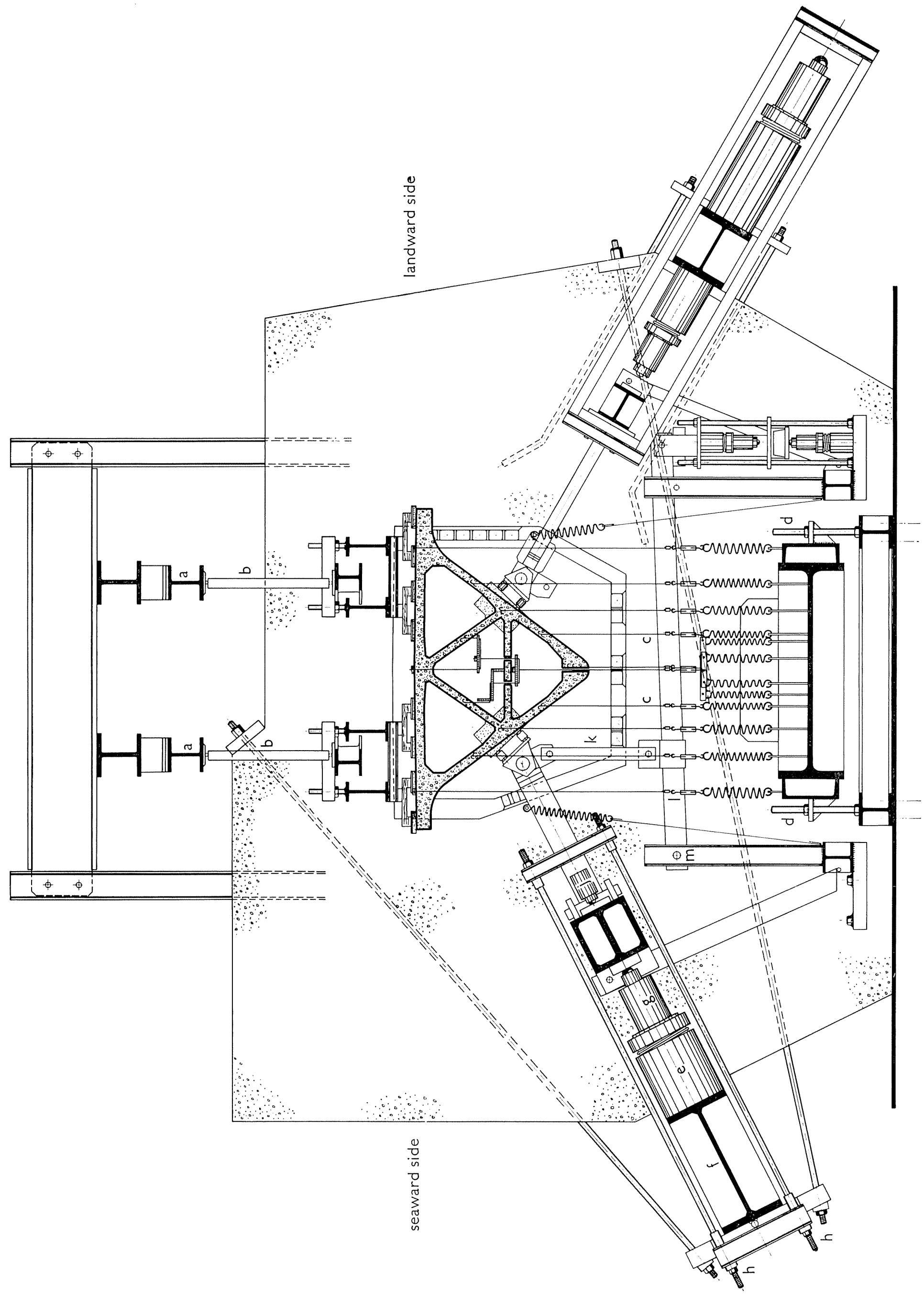


Fig. 20. Cross-section through the test arrangement.

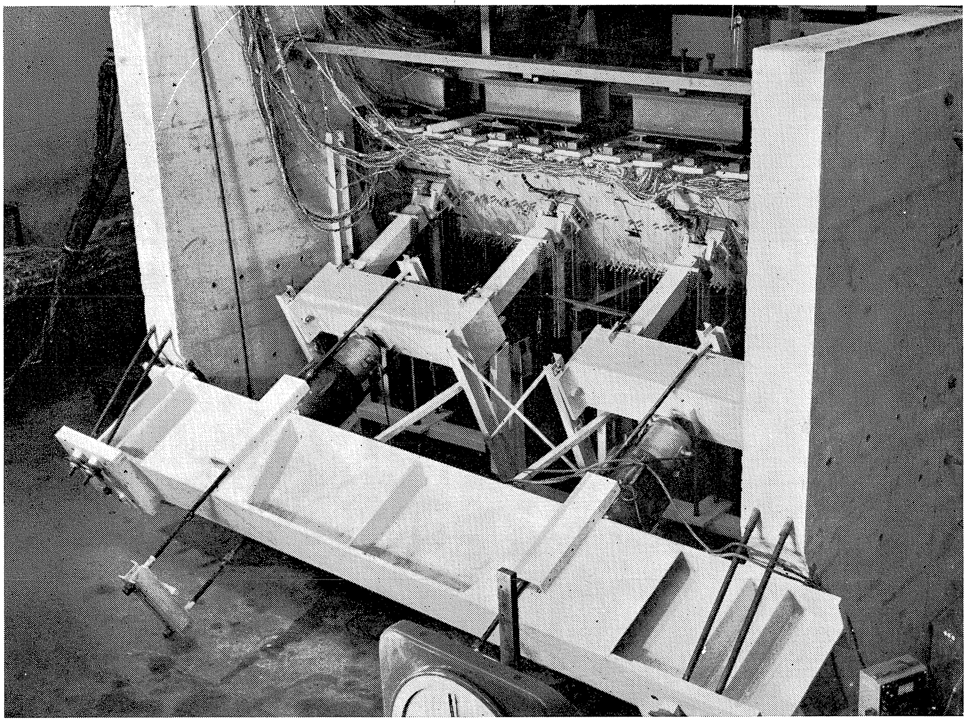


Fig. 21. The test arrangement as seen from the "seaward" side.

5 Testing

To begin with, the behaviour of the model was observed at loads corresponding to the design loads. Six load combinations were investigated. For four cases (see Fig. 4), the load was thereupon increased by 20–30% whereby the condition was obtained in which the first small cracks might be expected to occur. After the girder had, for each of these cases, been subjected to further loading corresponding to approximately three times the design load, the compressive force acting on the seaward side was increased to 3.9 times the design value. The test had to be discontinued at this stage, as the loading equipment had been designed for only three times the design load (permissible steel stress = 2000 kg/cm^2). It was not possible to reach the ultimate load of the girder.

6 Measurements and observations

In order to gain an insight into the behaviour of the structure, the strains occurring in it were measured at about 600 points during the application of the longitudinal prestress, the compensation of the dead weight, and the loading tests. For this purpose electrical resistance strain gauges were chiefly employed, which were fixed both in the interior and on the external surface of the model. The following strains were measured:

a. in the longitudinal direction at mid-span (region of maximum bending moment);
b. in the extreme fibres at the joints, so as to enable detection of incipient cracking;
c. in three directions at the end sections (region of maximum shear force);
d. in the parts of the structure directly behind the gate pivots, in order to inform about the spread of the concentrated loads.

In addition, measurements were made with mechanical strain gauges at the readily accessible points of the model. The horizontal and vertical displacements of the mid-span section were moreover measured by means of dial gauges. In order also to be able to inspect the interior of the girder during the test, three periscopes were constructed, which could be introduced into the girder through apertures in the end diaphragms.

7 Results

In general it can be stated that in the elastic range the longitudinal stresses determined from the strain measurements are in good agreement with the calculated values. By way of illustration the stresses at the mid-span section for one loading condition are indicated in Fig. 22.

The first crack occurred at mid-span, at the interface between a precast unit and a joint in the cantilevered edge of the deck slab on the landward

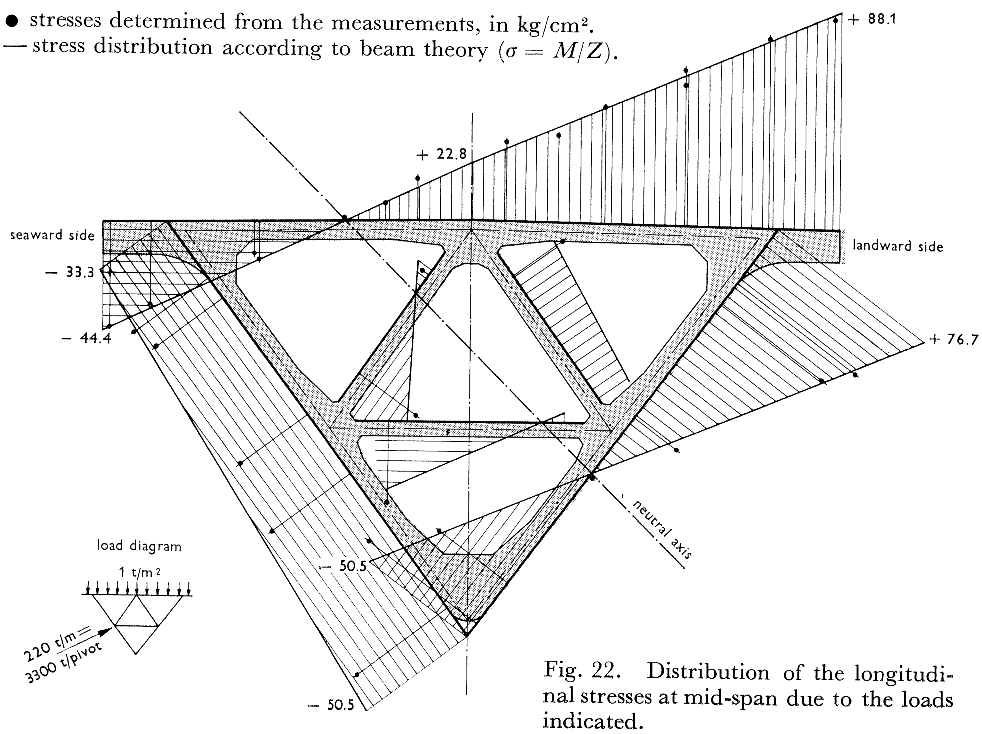


Fig. 22. Distribution of the longitudinal stresses at mid-span due to the loads indicated.

side, the load then being approximately 1.2 times the (compressive) design load acting on the seaward side of the girder.

As has already been mentioned, the test was stopped at a load corresponding to 3.9 times the (compressive) design load on the seaward side. At this stage there were cracks at a number of the interfaces between precast units and joints in the mid-span region. In addition, incipient crushing of the concrete at a joint was observed at mid-span on the cantilevered edge of the deck slab on the seaward side. In the other parts of the girder no cracks, either on the outside or in the interior, were detected by observation with the naked eye. In Fig. 23 the strains at the mid-span section are indicated for a loading condition consisting of the said load, dead weight and the prestress. These strains are based on the strain measurements performed in the compression zone, on the assumption that plane sections remain plane. For this state of strain, in conjunction with the known properties of the steel and the concrete, the moment transmitted by the section was calculated. This moment was found to be 0.93 times the moment determined from the applied loads. The discrepancy may have been caused by the occurrence of some degree of restraint of the girder at the piers.

Summarising, it can be stated that the model investigation has shown the structure to be capable of completely fulfilling the requirements applicable to it.

An additional aspect of the model investigation was that, as a result of the difficulties encountered in constructing the model, it was possible to give useful guidance in the construction of the actual structure.

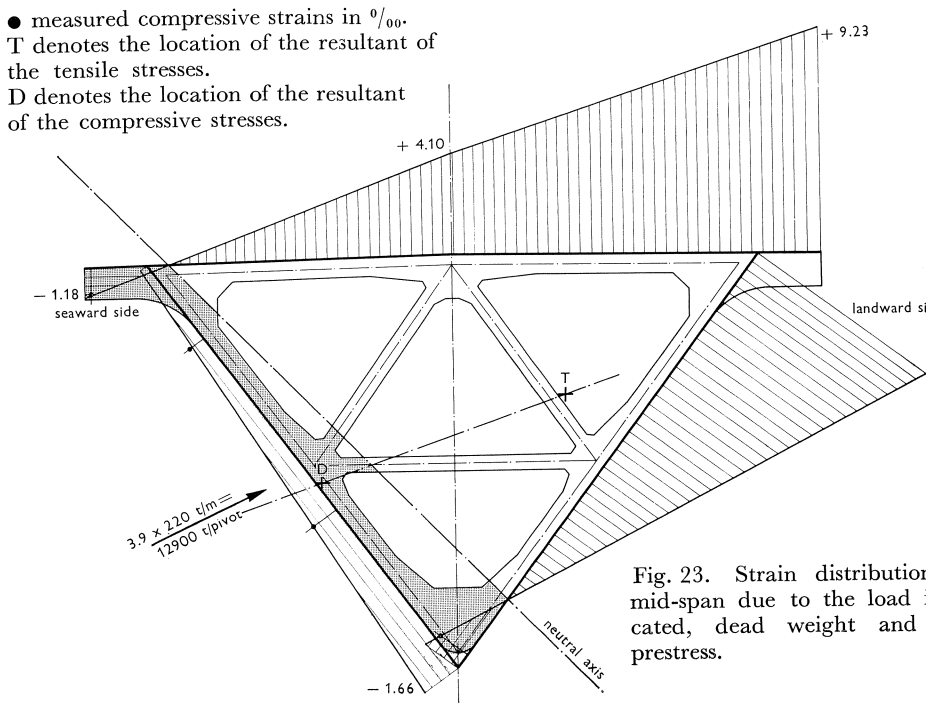


Fig. 23. Strain distribution at mid-span due to the load indicated, dead weight and the prestress.