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A Concrete Beam for Longer Spans

Poutre en béton pour plus grandes portées

Betonträger für größere Spannweite

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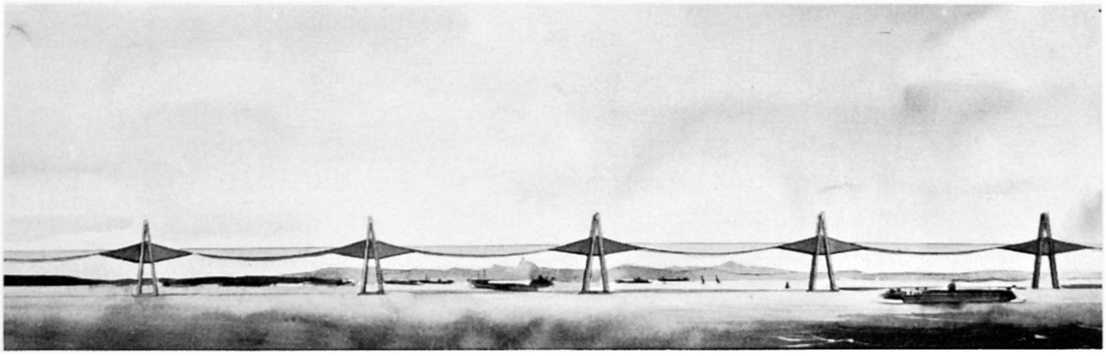
1. INTRODUCTION

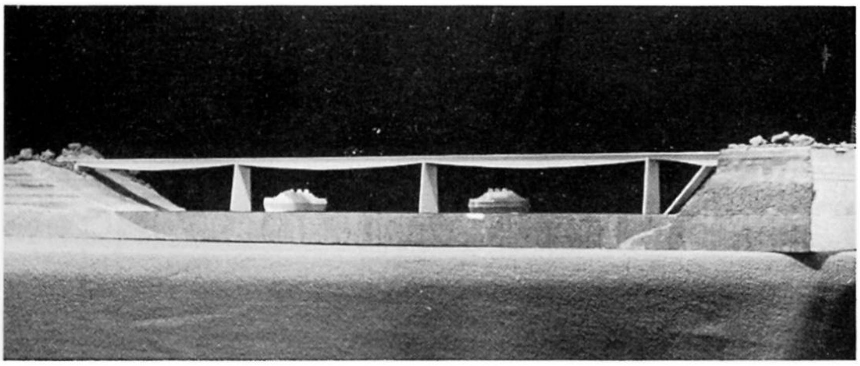
A new type of concrete structure has been in the beginning of



.Figure 1

1964 presented to a group created by the National Council of Transportation of Brazil for studies of the Guanabara bay crossing connecting the cities of Rio de Janeiro and Niterói. Later on as part of the Feasibility Studies a preliminary bridge design was made adopting this type of structure (Figure 1). In this paper some peculiarities of the new type are presented.

As it is known a prestressed concrete bridge-beam begins to become more expensive than a steel beam when the span of the bridge is somewhat larger than 200 m. A bridge built with the new type of structure presented here can lead to a successful bidding when the span of the bridge varies between 150 m and 450m, providing this way a wider field of use for the concrete beam.



.Figure 2

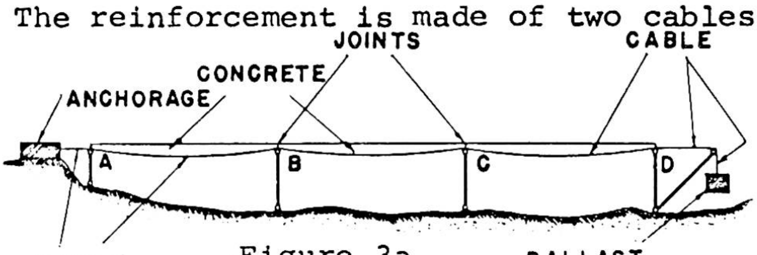
2. DEFINITION OF THE STRUCTURE

An example of the use of this new type of structure in a bridge design can be seen in figure 2. The structural behavior of the fish belly beams can be understood with the help of figure 3 where an idealized structure and a simplified cross-section of the bridge are shown.

The structure is basically composed by:

(a) The reinforcement

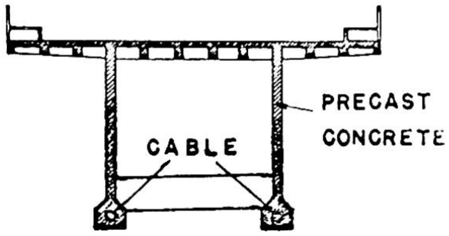
The reinforcement is made of two cables which have the same section along the bridge and are in contact with the concrete and anchored on the bridge.



.Figure 3a

(b) The anchorages at the bridge ends

On the left side the two main cables are anchored in an immovable anchorage and on the right side a ballast of a determined weight is always hanging on the cables, so that at the ends and over the supports A B C D the force of the cables is about constant during the entire life of the structure.



.Figure 3b

(c) The concrete section

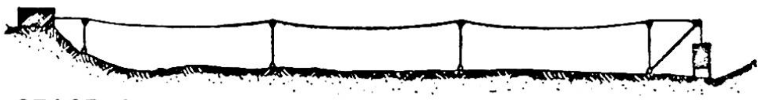
The concrete section of the bridge is basically composed of the upper flange, which is also the roadway, and the webs. They are precast and erected in a way which is explained later.

(c) The supports

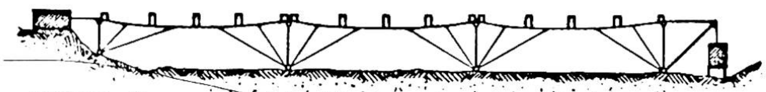
The supports are schematically designed as double hinged columns, but in the real case the supports are piers fixed at the foundations and they have at the top a special device to support the beams. Nevertheless in the real case they dont oppose any considerable resistance to a horizontal force applied at the top of the pier.

3. ERECTION OF THE BRIDGE

This structure can be successfully employed if the erection procedure is that indicated here and represented in figure 4.



STAGE A



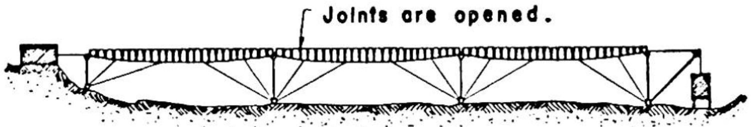
STAGE B

.Figures 4a, 4b

(a) The two cables are erected and fixed at the anchorages. The ballast on the right side is fixed to the soil during the construction.

(b) The precast concrete elements, which are segments of the whole section of the bridge, are placed over the cable in a convenient order.

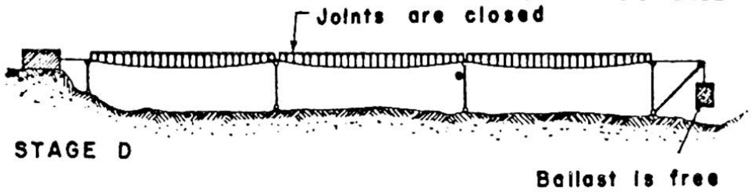
(c) All precast concrete elements are erected. Nevertheless, they are still separated from each other because the joints between these elements are left empty. The ballast could be set free because it is calculated to resist the horizontal force on the cable for mentioned dead load, but for stability convenience it is still left fixed to the soil.



.Figure 4c

At this stage of construction the dead load g_1, composed of the weight of the precast units and cables, is supported by the cable action only.

(d) Concrete is placed in the joints between the precast elements and a good bond is provided between concrete and reinforcement. The ballast is set free and will apply a fairly constant force upon the cables during the whole life of the structure. Later the dead load g_2 of the pavement and of the sidewalks is added. The dead load g_2 and the live load p act now over a reinforced beam which is subject to a tensile force applied by means of the ballast. For a variation of the temperature the beam is free to move and the ballast will go up and down.



4. ADVANTAGES OF THE STRUCTURE

The advantages of this type of structure can be seen immediately if the following points are considered:

(a) Since the concrete is stressed only by a small amount os dead load (g_2 = weight of pavement and side walks) and live load (p), the required concrete section is small, therefore resulting in a small dead load g_1 (weight of the precast units).

(b) The steel section of the cables must resist the forces caused by dead and live loads in two different ways:

1 - As a funicular cable for the dead load g_1.
2 - As reinforcement of a reinforced concrete beam for the load g_2 and live load.

Since the dead load g_1, as explained above, is small, the steel section of the cables is also comparatively small.

(c) The increase of the concrete beam deflections due to the creep of the compressed concrete is small since only small compressive stresses due to dead load g_2 are permanently acting.

(d) For the ultimate design of the structure a theory considering a deformed geometry of the beam must be used. That means that the conditions of equilibrium must be considered for the deformed position of the beam (for elastic structures this is known as second order theory). This is of great importance since the horizontal force of the cable applied by means of the ballast is always acting during the process of increasing the load until the total rupture.

(e) The period of time between the fabrication of the precast units and the placing of concrete in the joints between the precast units is of several months. Due to this the concrete shrinkage does not cause any major problem in the structure behavior.

(f) Before the beam is entirely formed the cable steel is under stress due to the force of the cables which appears during the erection of the precast units. For this reason - as in the case of prestressed concrete design - it is advantageous to use high strength steels, since their high capacities will be reached.

(g) The stress variation in the steel due to live load is small since for a large span the dead load is large in relation to the live load. The small variation of stress due to live load causes no fatigue of the steel or cracking problems in the concrete.

(h) It has been shown that the supports A B C D present no resistance to deformation of the beam in the horizontal direction , so that no force due to temperature is introduced in the structure if the simplified static system is considered (see figure 3). However, in the real structure, forces due to temperature variation appear but are of minor importance.

5. DESIGN OF THE STRUCTURE FOR BENDING

Only a few indications will be given here about the design of the structure.

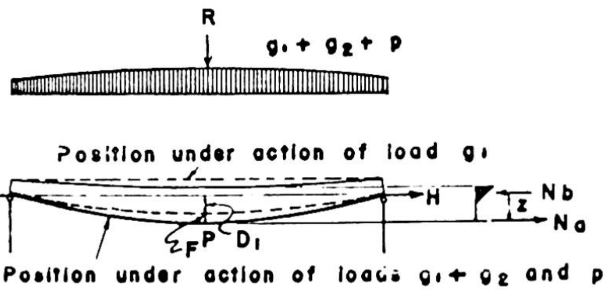
Careful calculations must be made regarding the height of the two cables in the center line of the span. The cable must be erected so that the top of the precast units is at the prescribed grade under full dead load. During this stage of the construction, all loads will be carried by cable action only.

If M_{g1} is the moment at the center line of the span of a simply supported beam with the loading g_1, the force H is

$$(1) \qquad H = \frac{M_{g_1}}{D_1}$$

where D_1 is the distance indicated in figure 5.

After the beam is formed the load g_2 (weight of pavement and sidewalks) and the live loads p are added.



.Figure 5

At this stage of the design the deflection,- the cracking - and fatigue-problems must be studied. For that it shall be considered the second order bending theory because a longitudinal tensile force acts. If some assumptions about the equivalent value of EI_b for the cracked concrete section are made, the differential equation of the bending theory of beams subject to tensile forces can be applied. The solution of the differential equation can be found numerically.

It is also possible to assume a determined position for the beam deflections in order to obtain the real beam deflections for a determined case of loading by use of iteration methods.

The equilibrium condition for moments with respect to point P (over the cable) gives us (see figure 5):

$$M_{g_1} + M_{g_2} + M_p = H\ (D_1 + F) + N'_b z$$

and since

$$M_{g_1} = H\ D_1 \qquad (2)$$

we obtain

$$M_{g_2} + M_p = H\ F + N'_b z$$

The equilibrium condition for the horizontal projections of the forces for point P section is:

$$N_a = H + N'_b \qquad (3)$$

where
z = lever arm of the cracked beam internal forces
N'_b = compression force in concrete
N_a = tensile force in cable
F = deflection of the beam at midspan

For the section-dimensioning the ultimate design will be considered as decisive. The CEB safety principles as well as notation will be used here.

The characteristic strength of the concrete and steel will be designated as σ_{bk} and σ_{ak} (only 5% of the test results can be lower than strength values).

The design strength of concrete and steel are defined by

$$\sigma'_b = \frac{\sigma_{bk}}{\gamma_b} \qquad\qquad \sigma_a = \frac{\sigma_{ak}}{\gamma_a}$$

where γ_b and γ_a are the reduction factors for strength.

The design dead and live loadings are designated as

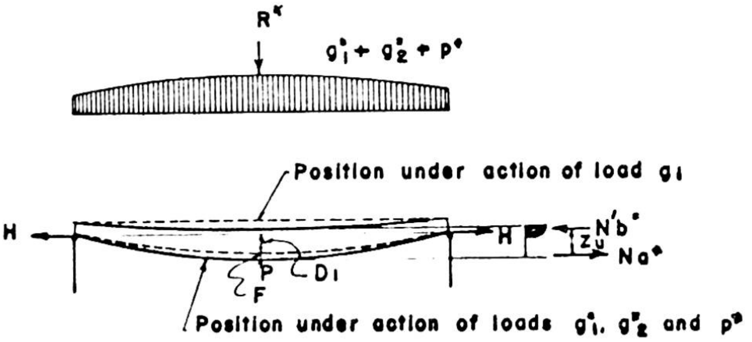
$$g_1^* = \gamma_g\ g_1 \qquad g_2^* = \gamma_g\ g_2 \qquad p^* = \gamma_p\ p$$

where γ_g and γ_p are the enhancement factors for the loads.

In the ultimate state the equilibrium conditions must be considered, in order to determine the internal forces in the deformed beam. This is of major importance to obtain a safe and economical design.

The equilibrium condition for moments with respect to point P (over the cable at midspan) yields (see figure 6):

$$\gamma g . M_{g_1} + \gamma g . M_{g_2} + \gamma p . M_p = H(D_1 + F_u) + N_b'^{*} . z_u \tag{4}$$



.Figure 6

where

F_u = deflection at point P

$N'_b{}^{*}$= compression force

$N_a{}^{*}$ = tensile force

z_u = lever arm

As it has been seen

$$M_{g_1} = H.D_1$$

Thus we obtain (5)

$$(\gamma g - 1)\, M_{g_1} + \gamma g . M_{g_2} + \gamma p . M_p = H.F_u + N'_b{}^{*} . z_u$$

The horizontal force equilibrium conditions for point P section yields

$$N_a{}^{*} = H + N_b{}'^{*} \tag{6}$$

or

$$S_a . \sigma_a{}^{*} = \frac{M_{g_1}}{D_1} + \frac{(\gamma_g - 1) M_{g_1} + \gamma_g M_{g_2} + \gamma_p M_p - H F_u}{z_u} \tag{7}$$

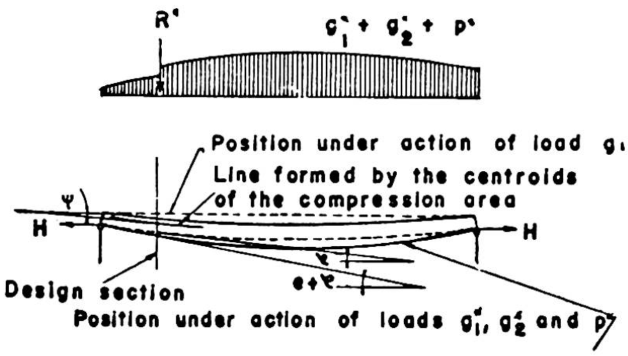
where S_a is the steel section of the cables.

It can be proved that the moment $H.F_u$ is of great importance for the dimensioning of the concrete and steel sections, because as it can be seen in expressions (5) and (7) the moment $H.F_u$ reduces the moment absorbed by the beam only $N'_b{}^{*}.z_u$, resulting in smaller concrete and steel sections.

For the determination of $H.F_u$ the value of the moments for loadings and the concrete and steel sections are needed. But for the dimensioning of the concrete and steel sections the value of $H.F_u$ is needed. In this case the method of trial and error can be applied to design the structure. However it is arbitrarily possible to determine the fraction of the applied moment that is absorbed by moment $H.F_u$.

6. DESIGN OF THE STRUCTURE FOR SHEAR

For the shear investigation the ultimate design must also be used.



.Figure 7

Here the CEB recommendations will also be applied. Here the equilibrium condition in the vertical direction will also be considered in the deformed position of the beam (see figure 7)

$$\gamma_g\ T_{g_1} + \gamma_g\ T_{g_2} + \gamma_p\ T_p = T_b^* + T_a^* + N_a^* \sin(\beta+\theta) - N'_b{}^* \sin\Psi \qquad (8)$$

where

Ψ = slope of the line formed by the centroids of the compression area

β = slope of the cables at the considered point before deformation

θ = rotation of the section under the design loads for the ultimate design

T_{g_1}, T_{g_2}, T_p = shear forces

T_b^*= shear absorbed in the concrete compression zone

T_a^*= shear absorbed by transverse reinforcement

N_a^*= tensile force in the reinforcement (in this case stresses are not always at rupture)

$N'_b{}^*$= compression force in concrete (in this case stresses are not always at rupture)

Relation (8) shows that the shear resisted by the beam itself ($T_b^* + T_a^*$) is reduced due to the influence of the expression

$$N_a^* \sin(\beta + \theta) - N'_b{}^* \sin\Psi$$

In this last expression three effects are included:

(a) All loads carried by the hanging structure during construction
(b) The vertical component of the tensile force of the cables
(c) The convenient shape of the cable after the deformation of the beam increasing the slope of the cable tensile force.

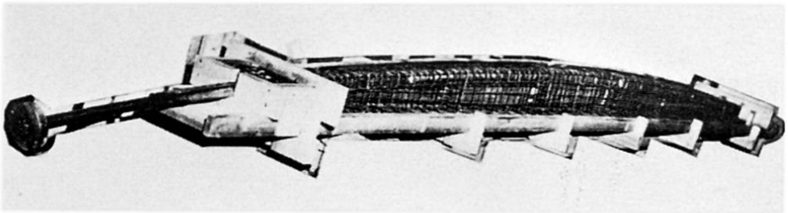
7. EXPERIMENTS

Experimental work is being carried out at COPPE, Federal University of Rio de Janeiro.

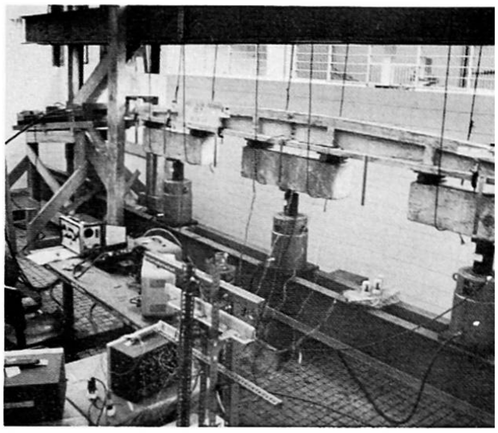
The reinforcement of the model of a simply supported T beam is shown in figure 8.

Figure 9 shows model being loaded.

First results indicated slightly higher strength to deformation ratios and higher ultimate strength than those predicted by theory. Discussions on the experimental results should wait until additional tests now programmed are completed. It is more appropriate to give details of the experimental studies at that future occasion.



.Figure 8



.Figure 9

8. CONCLUSION

It has been shown that a new type of concrete structure can be conveniently used for long span bridges. Advantages of this new type of concrete structure have been discussed, as well as some design peculiarities. Some variations of the structure on figure 2 can also be imagined. Only the simply supported beam was here discussed. Prestressed concrete, steel or lightweight concrete structures such as cantilever beams and continuous beams, as in the case of the design of the Rio-Niterói Bridge in Brazil, are also feasible.

SUMMARY

Concrete beams having longer spans than usual prestressed beams can be obtained as explained in this paper.

Basically the beam consists on the main reinforcement, the anchorages of the main reinforcement at the ends of the beam, and the concrete parts.

The following erection procedure is recommended: the reinforcement is erected and fixed at the end anchorages, the concrete parts are placed, and the parts are joined together and bond is provided between concrete and main reinforcement.

Advantages of this beam are mentioned in this paper.

Indications for the ultimate design of this structure are given.

RÉSUMÉ

Poutres en béton ayant portées plus grandes que celles des poutres précontraintes usuelles peuvent être obtenues comme il est expliqué en ce mémoire.

Fondamentalement la poutre est composée par les armatures principales, les ancrages des armatures principales aux extremités de la poutre et les parties en béton.

On recommande le procès d'assemblage suivant: les armatures sont placées et fixées aux extremités; les parties en béton sont mises en place; toutes ces parties sont mises ensembles et on fait la liaison entre le béton et l'armature principale.

En ce mémoire on présente des avantages de cette poutre et des indications pour le calcul plastique de la structure.

ZUSAMMENFASSUNG

Wie in diesem Aufsatz erläutert wird, können Betonträger mit grösserer Spannweite als übliche Spannbetonträger erzielt werden.

Der Träger besteht hauptsächlich aus der Hauptarmierung, den Hauptarmierungsverankerungen an den Enden der Träger und den Betonteilen.

Folgender Bauvorgang wird empfohlen: die Armierung wird eingebaut und an den Endverankerungen festgehalten; die Betonteile des Trägers werden montiert und miteinander verbunden; und zwischen Beton und Hauptarmierung wird ein Verbund hergestellt.

Die Vorteile dieser Trägerart werden erläutert.

Hinweise für die Bemessung im Bruchzustand werden angegeben.

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