Design of the Rio-Niteroi Bridge

H. J. Graham, Howard, Needles, Tammen and Bergendoff, Jacksonville, Florida

A brief overview is presented of the design of the highway bridge linking Rio de Janeiro and Niteroi, which was completed and opened to traffic in 1974. The main navigation spans contain a record girder span of 300 m in length. Inasmuch as the design of long span bridges cannot be separated from their erection, a brief description of the bridge erection is also included.

The cities of Rio de Janeiro and Niteroi, on the eastern coast of Brazil, are separated by the Guanabara Bay (Figure 1). The bay, bordered by mountains rising precipitously from the beaches, forms both an excellent harbor and a formidable barrier to transbay commerce and the expansion of Rio. Before the bridge crossing was completed in 1974, motorists had to choose either a lengthy wait for a vehicular ferry or a 90-km drive skirting the bay.

Serious planning for a crossing of Guanabara Bay was started in 1963. In 1967-68 a feasibility study sponsored by the U.S. Agency for International Development was carried out by a consortium of Brazilian and American engineers. This work included studies of location, tunnel crossings, alternative bridge types, traffic and revenue projections, and cost estimates. A toll project was recommended.

The crossing adopted was a six-lane high-level 13.3-km-long bridge (Figures 2 and 3). Of this length, 8.8 km is over water, and, at the location of the main spans, it is 22 m deep.

Opened to traffic in March 1974, the bridge forms an important link in Brazil's National Highway BR101 that will connect its principal coastal cities and ultimately will extend from Osorio to Natal. Traffic and revenues have thus far substantially exceeded projections.

OVER-WATER APPROACHES

The over-water approach spans consist of twin precast posttensioned concrete box girders (Figure 4). A constant span length of 80 m is used for the continuous spans. Expansion joints are provided 20 m from the piers in every fifth or sixth span. The structure depth is 4.7 m.

The piers for the approach structure are of cellular reinforced concrete and rest on footing blocks near the water surface and on 2-m-diameter reinforced concrete piles reaching to competent founding strata below water.

Precast concrete segments were cast in a yard, barged to the site, and lifted to the final position. Three basic types of precast elements were used: support sections over the piers, normal sections, and hinge sections at the expansion joints.

Segments were erected in pairs by traveling gantries and posttensioned in place; the sequence was repeated until the span projected 40 m on each side of a pier axis. The gantries then moved to the next span and the process was repeated. Concrete was added at the midspan joints; positive moment tendons were inserted, stressed, and grouted; and the span was made continuous. Four traveling gantries were used in the erection of the approach spans.

MAIN NAVIGATION SPANS

Site Constraints

Sited in 22 m of water, the main spans were required to provide 60 m of vertical clearance for navigation without encroaching into the aviation space for aircraft approaching the international and domestic airports at Galleao and Santos Dumont. Because of the 72.4-m maximum height of the fixed structure above sea level and the necessary vertical clearance, only a girder type of bridge was considered.

The selected structure is a three-span continuous steel box girder with spans of 200, 300, and 200 m (Figures 5 and 6). The 300-m main span is the longest unstayed girder span in the world, surpassing the length of its nearest rival by 39 m. The 30-m end cantilevers and 44-m suspended spans complete the 848-m-long steel structure, which makes a smooth transition to the concrete approach spans. The depth of the structure varies from 4.75 m at the juncture with the concrete structure to 13 m at the main piers to 7.5 m over the navigation

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channel. For erection, the superstructure was divided into pieces of 44, 292, 176, 292, and 44 m.

Main Piers

The main channel piers (Figure 7) are supported on forty 1.8-m-diameter reinforced concrete piles that are drilled into bedrock 50 to 60 m below sea level. Under each end pier, 32 piles are used.

Figure 1. Location plan.

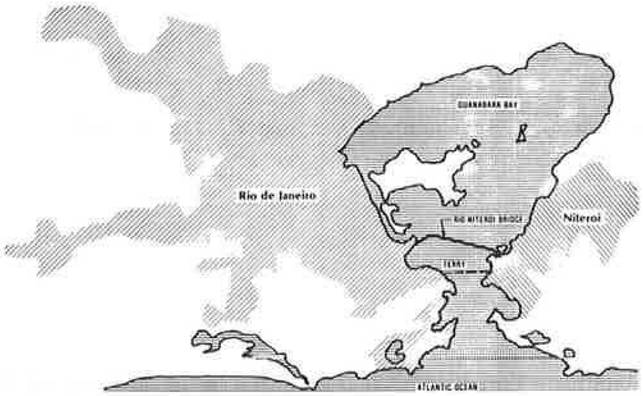


Figure 2. Northwest view of Rio-Niteroi bridge.

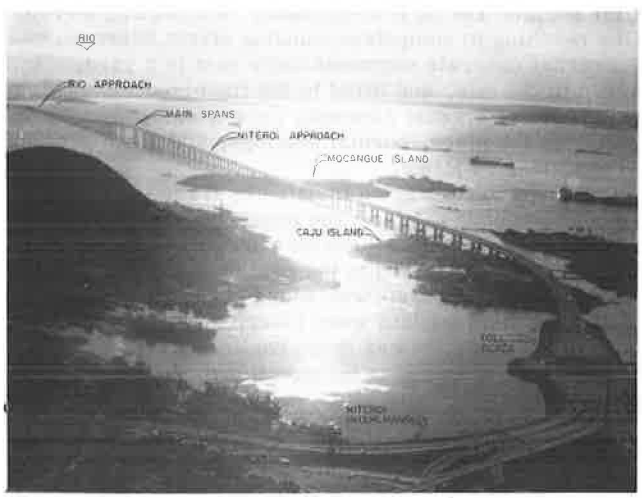
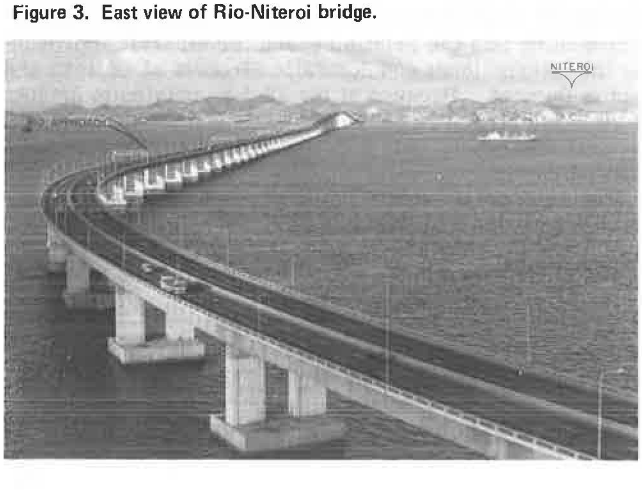


Figure 3. East view of Rio-Niteroi bridge.



Design loads for the piles were 650 Mg per pile for dead plus live load and dead load plus 118 Mg of horizontal load at ultimate (4700 Mg per main pier for ship impact).

Massive reinforced concrete footing blocks, extending from 2.5 m below to 2.5 m above sea level, rest on the large piles and support twin tapered hollow box pier shafts. The top of the end pier shafts is 59.96 m above sea level while the top of the main channel piers is at an elevation of 56.44 m.

The twin hollow pier shafts are each a constant 6.86 m wide (transverse to the bridge) and have variable thickness (parallel with the bridge). The walls, which are uniformly 65 cm thick, were constructed with slip forms. The piers are capped with solid blocks 3 m deep.

The depth of the bay at the main piers varies between 21 and 22 m. The 1.8-m-diameter piles were constructed by using large jack-up islands incorporating floats manufactured in Brazil, legs manufactured in Holland, tubes and drills manufactured in Germany, and large-capacity cranes manufactured in the United States. Each island

Figure 4. Main spans.

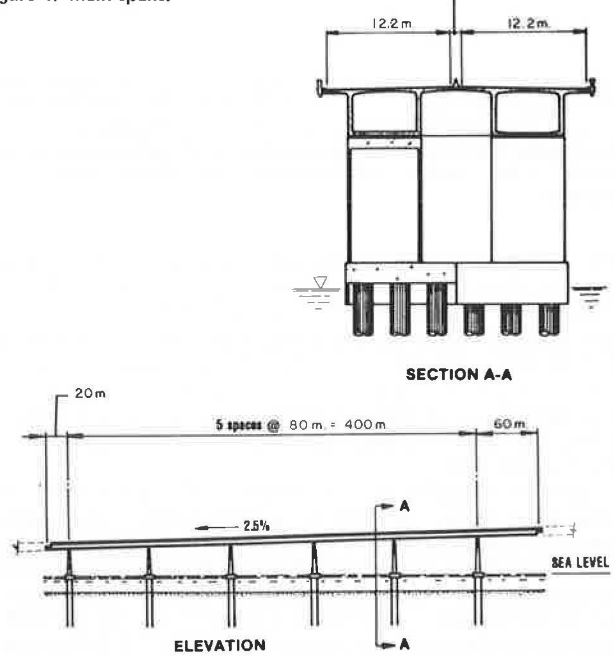


Figure 5. Prestressed box girder approach spans.

was equipped with two drills and ancillary equipment so that two piles could be constructed simultaneously. The procedure for constructing a pile is described below.

1. The 2.2-m inside diameter oscillating tube and 2.0-m outside diameter drill were advanced into the sea bottom until the cutting teeth of the tube reached refusal in residual soil or decomposed rock. Cuttings were removed by reverse circulation of water through the drill stem.
2. Drilling was continued until it penetrated about 1 m into solid rock.
3. The hole was cleaned out by reverse circulation of water through the drill stem, and then it was inspected by a diver.
4. A 10-mm thick, 1.8-cm inside diameter casing was lowered through the tube until its lower edge was about 60 cm above the bottom of the hole. The casing was held in this position from above.
5. Reinforcing cages were lowered into the casing, supported from above, and the casing was filled with high-quality tremie concrete. Concrete was added to well above the bottom of footing.
6. To remove it, the tube was oscillated while sand was placed in the space between the outside of the casing and the inside of the 2.3-m hole formed by the tube.

When all the piles were complete for a pier, concrete boxes made of cast-in-place bottom slabs and precast walls were lowered over the piles, sealed, and pumped out. The casings were removed to an elevation -2.4 m, concrete in the piles was removed to sound concrete, reinforcing was placed, and concrete was added to footing blocks in the dry (generally in three lifts). The pile reinforcing extended 4.8 m into the footing so that the pile heads would be rigidly fixed in the footings and the pier would have added stability.

Figure 6. Elevation of main spans.

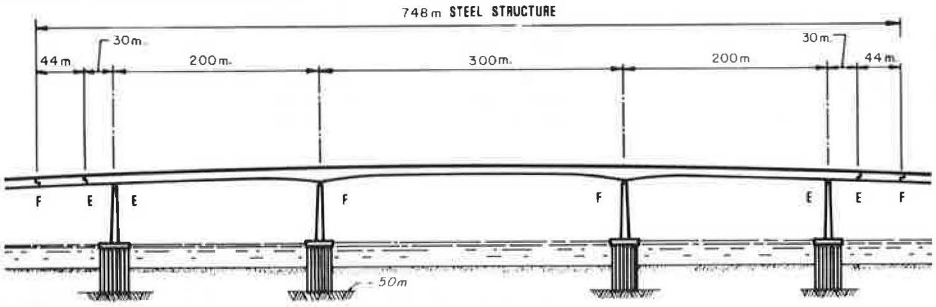


Figure 7. Main channel piers.

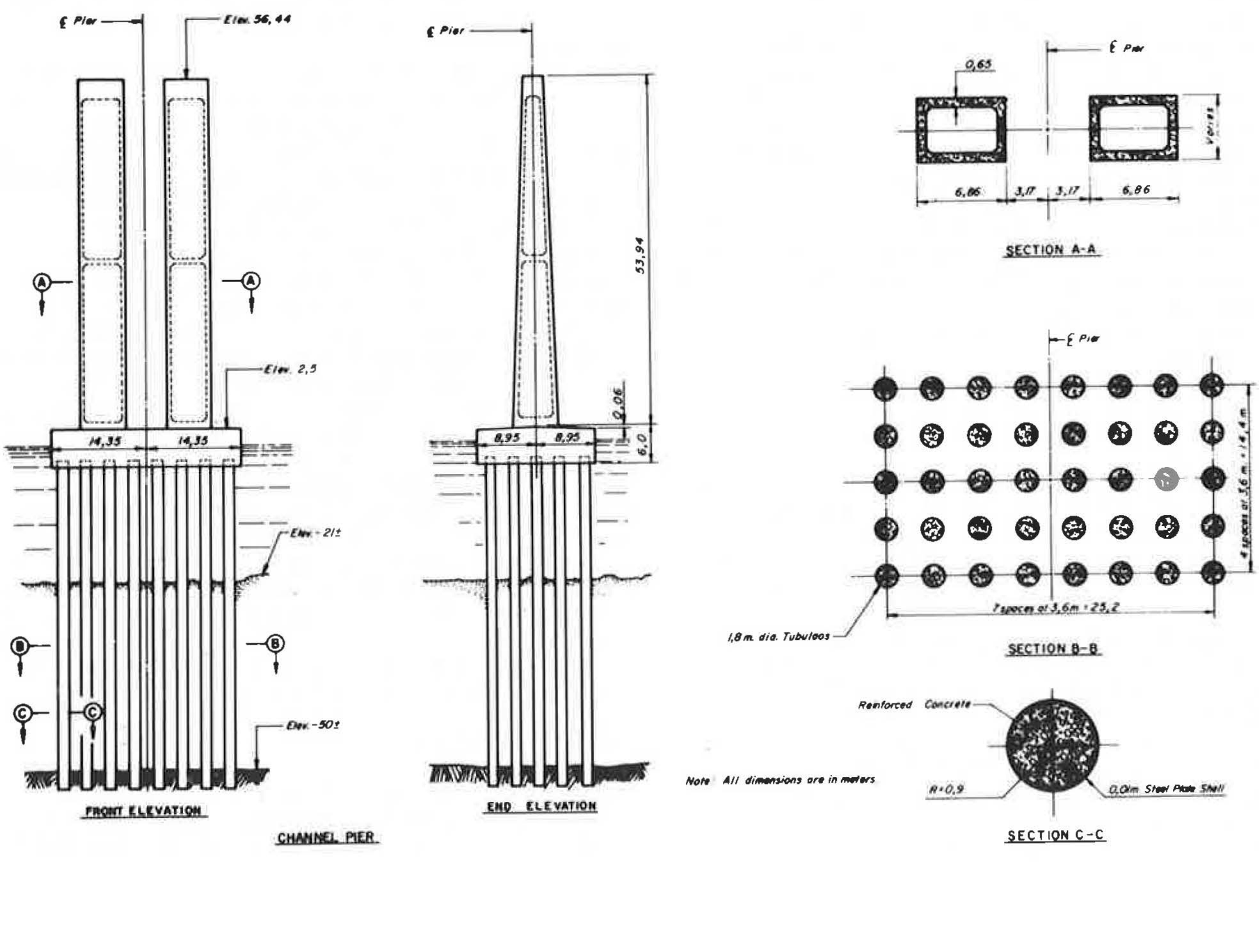
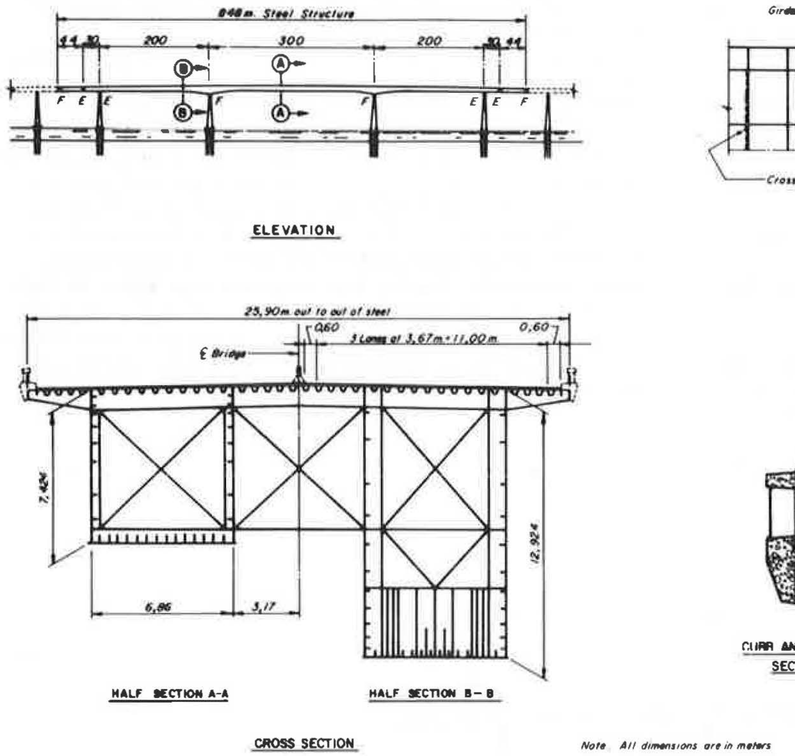


Figure 8. Navigation spans: cross section and typical details.



Main Superstructure

The two 12.2-m-wide roadways are supported on a pair of steel boxes centered 13.2 m apart (Figure 8). A central barrier, curbs, railings, and an epoxy asphalt wearing surface complete the traffic roadways.

The orthotropic steel deck is comprised of plates varying between 10 and 25 mm thick and is stiffened with 25-cm-deep trapezoidal ribs 8 to 12 mm thick spaced on about 60-cm centers.

Lower flanges varying from 10 to 45 mm thick are 6.89 m wide and are stiffened by steel plates or bulb flats on 46-cm centers. Stiffener size varies as required by stress or buckling stability or both.

Vertical webs vary between 12 and 18 mm thick and are stiffened with 20-cm-deep bulb flats whose number and spacing are as required for buckling stability.

Transverse floor beams and vertical stiffeners are spaced at 5-m intervals. Cross frames, internal and between boxes, are provided at 30-m spacings.

Three grades of weldable steels, conforming to British Standard 4360 (1968), were used. Yield strengths were 250, 350, and 440 N/mm^2 for grades 43A, 50, and 55 respectively.

The steel superstructure was designed to accommodate the contractor's fabrication and erection scheme. The steel was fabricated in England in 15 by 3.5-m modules, shipped to Brazil, and assembled into full-size boxes in a zero stress cambered shape on cribbing on land.

The two 172.6-m boxes composing the center (pontoon) unit were fabricated side by side, connected together, skidded sideways over storage jetties, and ultimately lowered into the water to form a transport barge for the side span pieces. These units were provided with several watertight bulkheads for safety against sinking and for control of water ballasting during subsequent erection uses.

Next, the two Niteroi side span boxes, each 293.71 m long, were assembled side by side on cribbing, and the centerline connections were fitted. They were jacked up off the cribbing and skidded one box at a time out on the storage jetties. This operation was repeated for the identical Rio side span pieces.

The 44-m spans were assembled on a slipway, fitted with end bulkheads, and launched into the water as pontoons.

Temporary jacking columns were erected alongside the pier shafts, and large temporary ring girders were assembled on top of the pier footings. The side span pieces were floated out (one box at a time) on the pontoon unit and set on the ring girders. They were then jacked to the top of the piers by using twelve 450-Mg jacks, skidded laterally on Teflon skids on the ring girders over the tops of the piers, and placed on the shoes. The total weight lifted, including erection gear, was about 5280 Mg. The jacking operation required $3\frac{1}{2}$ days for the Niteroi side spans and 3 days for the Rio side spans.

The center span was hoisted by using 8 of the 12 jacks and four of the six jacking columns. For this operation the columns were in tension whereas the side spans were in compression. The weight of the center piece was about 3200 Mg (3600 Mg including erection gear).

The 44-m end spans weighing 225 Mg/box were floated to the site and lifted one box at a time by using jacking frames and wire rope tackle.

Precast curb and parapet and median sections of lightweight concrete match the dimensions of those on the approach spans. These are securely anchored to the steel work.

The 6-cm-thick surfacing for the steel deck was of epoxy-asphaltic concrete and was placed in two lifts.

DESIGN CRITERIA

The concrete elements of the bridge were designed to conform to the applicable Brazilian norms. Because no specifications existed for the design of orthotropic steel deck girders, special criteria were developed. In general, these criteria resulted from a combination of the recommendations of the American Institute of Steel Construction (1) and European practice. The specified theoretical margins of safety against yielding and buckling were slightly higher than those contained in Brazilian and European steel design norms (in accordance with AASHTO specifications).

Design traffic live load was class 36 as given in Brazilian Norm NB-6. For the main steel spans, this loading of 4.26 Mg/m/box resulted in main girder live load moments about 1.8 times as large as those from AASHTO HS-20 loading.

Because no applicable Brazilian norms existed for designing for fatigue in steel highway bridges, AASHTO specifications and loadings were used.

DESIGN METHODS

Concrete Elements

Ultimate design methods were used in the design of the reinforced and prestressed concrete elements of the bridge.

Steel Structure

For the main steel superstructure a working stress design was performed, although some elements, e.g., girder webs and vertical stiffeners, were checked for ultimate strength.

Local traffic stresses in the stiffened deck plate, longitudinal ribs, and transverse floor beams were computed by methods given by Troitsky (2), Sievers (3), and Pelikan and Esslinger (4).

In the design of stiffened plates subjected to compressive or shearing stresses, the buckling stability was investigated for each element in the assembly as well as for the complete assembly. In general, the computations of plate stability were based on the classical elastic plate buckling theories as specified by German Industrial Norm 4114.

When the computed buckling stress of a given element or assembly exceeded the elastic limit of the material, the critical stress was reduced so as not to exceed the yield stress.

In the evaluation of the elastic buckling stresses of the multiple stiffened plate panels of the main girder flanges and webs, extensive use was made of the charts and tables given by Kloeppel and coauthors (5, 6).

Stresses in the intermediate cross frames due to asymmetrical transverse loads were calculated by Homberg's method and checked by the Guyon-Massonet-Sattler method as described by Kuzmanovic (7).

Checks were made for variations in temperature of 15°C below and 30°C above a normal temperature of 24°C. Further, the difference in temperature between a structural member exposed to sunlight and another in shade was assumed as 15°C.

Erection Forces

The contractor's planned erection methods were known at the time of the actual final design. Thus, the moments and shears in each major piece were computed for each stage of its erection, and the structural stability was investigated for the critical stages.

For the 172.58-m-long center span section, the stages investigated were

1. When supported on swan necks at the site fabrication yard thus spanning 192 m while loaded with its own dead load and distributed erection gear;
2. When serving as a floating pontoon and loaded with its own dead load, the weight of erection gear, ballast water, and the weight of a 293.7-m-long side span piece; and
3. While being lifted into final position.

The 293.7-m-long side pieces were investigated for the following stages:

1. While spanning 192 m between the jetties on Caju Island and loaded with dead load plus erection gear,
2. While spanning 110 m between the trestle supports on the pontoon unit,
3. While spanning 200 m between piers, and
4. While spanning 200 m between piers and supporting the weight of the pontoon unit and erection gear during the lifting of the center span.

In addition, the stability of the pontoon-side span combination was investigated for wind and waves when the pieces were floated out.

Although, in general, the dimensioning of the structure was controlled by in-service stresses, additional stiffening for erection was provided at all temporary support points, and additional cross frames were provided between the pontoon boxes to distribute the concentrated reaction of a side span piece to both boxes of the pontoon.

Camber

The computed deflection of the center span under the full design live load (4.26 Mg/m/box) is 0.82 m while that of a 200-m side span is 0.56 m downward and 0.47 m upward. The computed maximum live load deflection at the joint between the 44-m-long suspended span and the 30-m-long cantilever is 0.30 m downward and 0.32 m upward.

The main steel girders were fully cambered throughout their length for dead load deflections. During erection, a substantial weight of erection gear was acting at the end of the 63.7-m temporary cantilevers in the 300-m spans. So that the end slopes of each piece would match at the bolted splices, the structure was cambered in space so that these slopes would match when the girders were resting on the piers and erection gear was in place. By this means excessive temporary raising or lowering of girder ends during closure of the bolted splices was precluded and built-in angle breaks were avoided at the splices.

ACKNOWLEDGMENTS

The Rio-Niteroi bridge, now called Ponte Presidente Costa e Silva in honor of a former Brazilian president, is a project of the Brazilian Ministry of Transport administered through the National Highway Department (DNER) and S. A. Empresa de Construção e Exploração da Ponte Presidente Costa e Silva (ECEX). Mario Andreazza was minister of transport; Eliseu Resende was director of DNER; and Joao Carlos Guedes was director-president of ECEX.

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