Large Eyebar Suspension Bridge in South America

Span of 1,114 Feet Under Construction in Brazil Has Chain of Eyebars with New Form of Stiffening Truss

A COMBINED highway and railway bridge under construction at Florianopolis, in the province of Santa Catharina, Brazil, connecting the island on which the city is located with the mainland, represents the first application of modern eyebar-chain construction to suspension bridges. It is no less interesting because of its unusual stiffening truss construction, which, in connection with the use of eyebars for the chain and the unloaded backstays, results in a high degree of rigidity and economy. Because of its long main span, nearly 1,114 ft., this structure will take rank among the largest suspension bridges in existence. It was designed by Robinson and Steinman, of New York, as consulting engineers for Byington and Sundstrom, of São Paulo, Brazil, contractors for the bridge.

In the original design for the crossing, whose span is controlled very closely by the conditions of the site (see the location plan), a wire-cable suspension bridge was contemplated. This design had a parallel-chord stiffening truss with hangers at all panel points. When, however, the American Bridge Co. made a tender of heat-treated eyebars under a guarantee of 75,000 lb. per square inch minimum elastic limit, permitting a working stress of 50,000 lb. per square inch to be used in the chain, the structure was redesigned to take advantage of eyebar construction, with the result of showing a distinct economy in total cost and permitting the stiffness of the main span to be increased very greatly. The eyebar-chain design, shown in the drawings, is now being carried out.

Special Features—From the design details on p. 593 it will be seen that the towers are two-column steel bents with battered legs and with line bearing at the base, while the stiffening trusses have curved top chords so disposed as to give maximum depth at the quarter-points, the top chord in the middle section being formed by the suspension chain itself. This novel combination of chain and truss yields large savings in steel due to the elimination of the middle half of the top-chord and other members, and to the more economical truss outline in which the depth conforms to the variation in bending moments along the span. The traffic requirements are satisfied by a 28-ft. clear roadway width, which fixed the truss spacing at 33½ ft. Accordingly the cross-section involves no features beyond the ordinary. The backstays are unloaded, going down directly to masonry anchorages, one of which is on rock while the other is carried by a pile foundation including both vertical and batter piles. The approaches are steel viaducts with spans of 185 ft. directly adjacent to the towers.

Loading—The loadings assumed for the design of the various parts of the structure vary from 1,850 lb. per lineal foot of bridge for the chain and 2,000 lb. for the trusses (the latter with 10 per cent impact) to a 6-ton truck or 60 lb. per square foot (with 25 per cent impact) on the roadway stringers, and a 50-ton electric locomotive followed by 2,000 lb. per lineal foot (50 per cent impact) on the railway stringers. Wind was taken at 25 lb. per square foot on the suspension bridge and 30 lb. on the viaduct, and the temperature variation was taken at the moderate amount of 30 degrees rise or fall, in conformity with local climatic conditions. In comparison with these figures it may be noted that the total dead load is 4,370 lb. per lineal foot of main span, or about 4,000 lb. for structure and deck excluding the water main which extends along one side of the roadway.

With this loading the greatest allowable stresses were set at 50,000 lb. per square inch for the eyebars of the main chain, and 20,000 lb. per square inch in the stiffening trusses. Some margin below these amounts was maintained, however, and the final stresses in the chain do not exceed 46,500 lb. per square inch, and those in the trusses 18,500 lb. per square inch, as calculated by the ordinary, approximate method. In the case of the stiffening trusses, additional margin is represented by the difference between analyses of the structure by the ordinary method and by the more exact or deflection method; the latter showed considerably lower stresses, by about 25 to 30 per cent, but no reduction in section was made to suit these stresses. The reduction in the chain stress was made possible by the smaller weight of the combination stiffening truss of variable depth as compared with the parallel-chord truss. The floor framing has its bending stresses limited to 17,000. In all the analyses, the value of the modulus of elasticity of the truss steel was taken at 29,000,000 lb. per square inch and that of the eyebars was taken at 27,000,000 lb.

MAIN FEATURES OF EYEBAR-CHAIN SUSPENSION BRIDGE, FLORIANOPOLIS, BRAZIL.

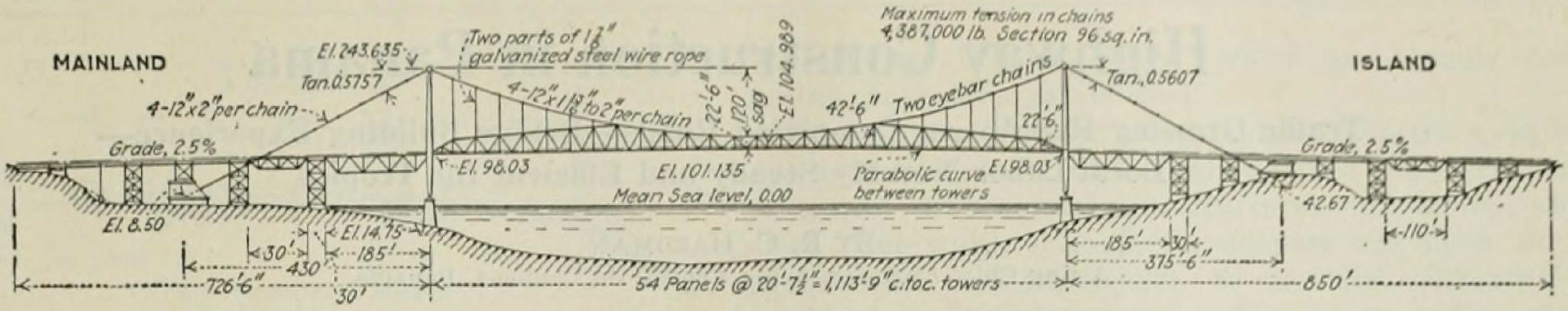
This eyebar-suspension bridge has a span of 1,113 ft. 9 in., which makes it one of the longest suspension bridges ever built. Note the unusual stiffening trusses with use of suspension chain in combination.

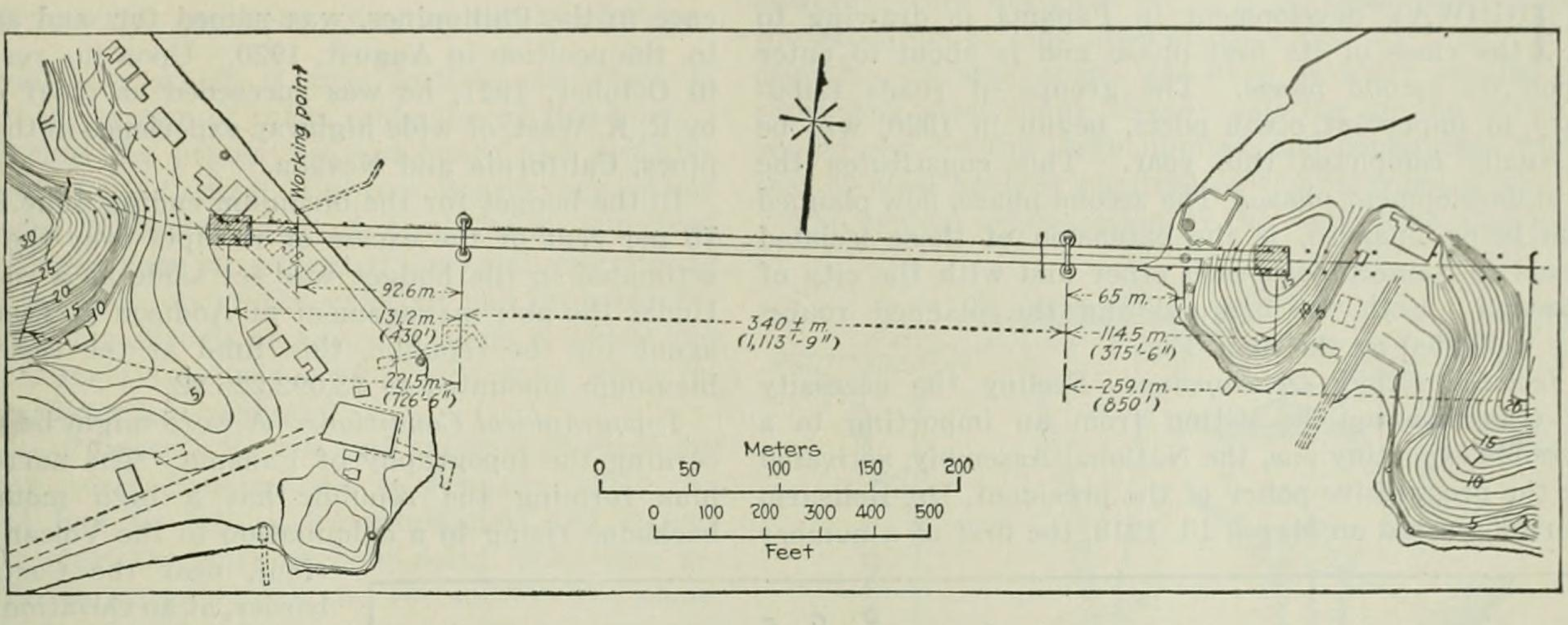
(*On page* 593)

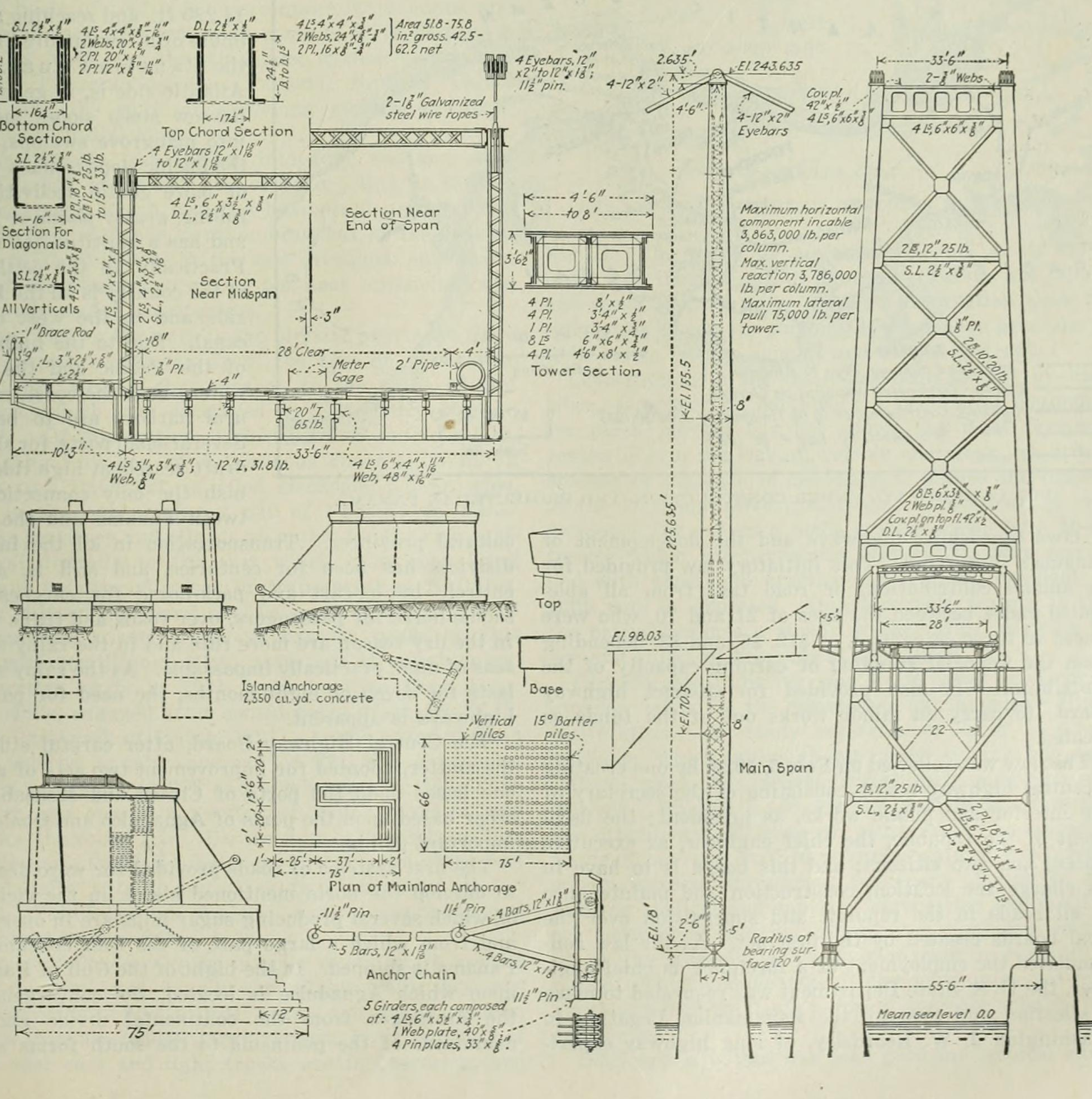
The two chains consist each of four eyebars 12 in. wide, from anchorage to anchorage. The eyebars of the backstays are 2 in. thick, while those of the main span range from 2 in. to 1 13/16 in., with chain section ranging from 96 to 87 square inches. Steel-rope suspenders are used, consisting of two parts of 1⅞ in. galvanized steel wire rope socketed to clevis attachments at the top chords of the trusses.

Calculations of the deflections of the structure show that under live-load on one-half the span, the quarter-point deflection is only about 13 in., or 1/1,000th of the span, while under full-span loading the center deflection is about 17 in. or about 1/800th of the span. These extraordinarily low values result from the following factors: the use of eyebars in the chains, with their greater section and resulting greater dead weight and reduced elongation; the unloaded backstays; the great depth of stiffening truss at the quarter-points, about double the depth of the originally-planned parallel-chord truss; the large chord-section of the stiffening trusses in the middle portion, where the chains serve as chords; and the novel combination of chain and truss whereby full live-load produces tension instead of compression in the middle half of the top-chord.

Construction and fabrication are now under way and the bridge is expected to be completed by the end of next year.


MAINLAND
ISLAND
El. 243.635
Two parts of 1 1/8" galvanized steel wire rope
Maximum tension in chains 4,367,000 lb. Section 96 sq.in.
Tan. 0.5757
Tan. 0.5607
4-12"x2" per chain
4-12"x1 1/8" to 2" per chain
Two eyebar chains
42'-6"
22'-6"
120' sag
El. 104.989
Grade, 2.5%
Grade, 2.5%
El. 98.03
El. 101.135
El. 98.03
Parabolic curve between towers
Mean Sea level, 0.00
El. 8.50
El. 14.75
30'
430'
185'
726'-6"
30'
54 Panels @ 20'-7 1/2" = 1,113'-9" c. to c. towers
42.67
110'
185'
30
375'-6"
850'


Working point
92.6 m.
131.2 m. (430')
221.5 m. (726'-6")
340 ± m. (1,113'-9")
65 m.
114.5 m. (375'-6")
259.1 m. (850')
Meters
0 50 100 150 200
0 100 200 300 400 500
Feet


Bottom Chord Section
Top Chord Section
Section For Diagonals
All Verticals
Section Near Midspan
Section Near End of Span
4 Eyebars, 12" x 2" to 12" x 1 1/8"; 11 1/2" pin.
2-1 1/8" Galvanized steel wire ropes
28' Clear
Meter Gage
2' Pipe
33'-6"
10'-3"
Tower Section
4 Pl. 8' x 1/2"
4 Pl. 3'-4" x 1/2"
1 Pl. 3'-4" x 3/4"
8 Ls 6" x 6" x 3/4"
4 Pl. 4'-6" x 8' x 1/2"
El. 243.635
4-12"x2" Eyebars
Maximum horizontal component in cable 3,863,000 lb. per column.
Max. vertical reaction 3,786,000 lb. per column.
Maximum lateral pull 75,000 lb. per tower.
El. 155.5
El. 98.03
El. 70.5
225.635'
Main Span
Top
Base
Radius of bearing surface 120"
33'-6"
28'
22'
55'-6"
Mean sea level 0.0
Island Anchorage 2,350 cu. yd. concrete
Plan of Mainland Anchorage
Vertical piles
15° Batter piles
75'
66'
Anchor Chain
11 1/2" Pin
5 Bars, 12" x 1 1/8"
4 Bars, 12" x 1 1/8"
5 Girders, each composed of: 4 Ls 6" x 3 1/2" x 3/4", 1 Web plate, 40" x 5/8", 4 Pinplates, 33" x 5/8".
12'
75'

Design of the Florianopolis Suspension Bridge

New Stiffening Truss Produces More Rigid Bridge With Less Material—Main Chains Made of Heat-treated Eyebars With Yield Point Above 75,000 Lb. per Sq.In.—Towers Designed to Rock on Base

By D. B. Steinman
Consulting Engineer, New York

THE FLORIANOPOLIS Bridge (main span 1,113 ft. 9 in.), now under construction in Santa Catharina province, Brazil, will be the longest-span bridge in South America and the longest eyebar suspension span in the world. In addition, the structure will be of interest to engineers as the first example of a new departure in suspension design, and as the first application of an important new structural material.

Holton D. Robinson and the writer prepared the design, as consulting engineers for Byington & Sundstrom, of São Paulo, Brazil, who had already secured the general contract from the State of Santa Catharina. Minimum cost was the outstanding requirement governing the design.

New Form of Stiffening Trusses—The form of stiffening construction adopted for the Florianopolis bridge departs rather radically from precedent. The following considerations led to its conception:

In the central portion of the conventional form of suspension construction, the cable, which sustains a heavy tensile stress, is in close proximity to the upper chord of the stiffening truss, where compression is the governing stress. Such juxtaposition of two principal elements carrying opposing stress represents a waste of material, or, rather, a neglected opportunity for increasing economy. By combining the two opposing structural elements a great saving can be effected: the result is a subtraction of stresses instead of an addition of sections. There is thus secured a partial neutralization of the maximum tensions in the middle portion of the cable, and the corresponding portion of the upper chord of the truss is dispensed with.

This utilization of the cable as the upper chord of the stiffening truss should be limited to the middle portion of the span. To extend this construction to the ends of the span would not be economical, because the saving of the top chord in the outer quarters would be offset by the increase in length of the web members in a region where the stiffening truss has its maximum shears. Moreover, beyond the quarter points there would be an addition instead of a subtraction of stresses, since the condition of loading that produces maximum tension in those top chords is one that produces nearly maximum tension in the cable.

Another neglected opportunity for increasing economy and efficiency in the conventional form of suspension construction is represented by the use of parallel-chord stiffening trusses. For maximum economy the truss should have a profile conforming to the variation of maximum bending moments along the span, a principle which receives recognition in the design of other structures, such as simple trusses, cantilevers, continuous trusses and arches. Since the economic depth at any section is a function of the governing bending moment, a truss should have its greatest depth at the points of greatest bending moment, and should be made shallow where the bending moments are comparatively small. In a suspension-bridge stiffening truss, the greatest bending moments occur near the quarter-points of the span; consequently the economic profile of a stiffening truss is one having maximum depth near the quarter-points and minimum depth at mid-span and at the ends. This conclusion is strengthened by the fact that the shears in a stiffening truss are a minimum near the quarter-points and attain maximum values at the middle and ends of the span, so that a truss-profile with maximum depth near the quarter-points also gives economy in web members since it provides the shallowest depth in the regions where the web stresses are greatest. Such a profile yields the additional advantage of greater uniformity of required chord-sections throughout the span. A wide range of variation in required sections generally involves a waste of material in those chord members where minimum sections are required.

Another consideration governing truss-depth is that of efficiency in reducing deflections. The most serious deflection of a stiffening truss, as measured by the resulting deflection gradients, is produced under the condition of live-load covering approximately one-half of the span. Half-span loading produces a downward deflection of the loaded segment and a somewhat smaller upward deflection of the unloaded segment, with a maximum deflection gradient at the loaded end. The magnitude of this deformation depends upon the truss-depth at and near the quarter-points. Calculations show that to limit the deflection gradient to 1 per cent a truss depth of 1/45 to 1/40 of the span is required. A parallel-chord stiffening truss of such depth (as illustrated by the Williamsburg bridge) would render the structure unsightly. In order to secure the requisite stiffness without resorting to a stiffening truss of clumsy proportions, we must abandon the parallel-chord type and adopt an outline providing the extreme depth only where it is needed, namely in the vicinity of the quarter-points of the span.

The foregoing considerations show that the logical directions for improvement of conventional suspension design for increased economy and efficiency are along two lines: (1) Utilization of the cable as the top chord of the stiffening truss (preferably limited to the middle half of the span); and (2) variation of truss profile to give maximum depth near the quarter-points of the span. The two requirements, fortunately, are congruous; complying with the first automatically facilitates compliance with the second. The result is the new form of suspension construction adopted for the Florianopolis bridge.

Comparative Economy—The bridge had already been designed along conventional lines, when the decision to substitute eyebar chains for wire cables prompted consideration of the revised truss design and facilitated its application. Fig. 1, comparing the adopted design with the layout which it superseded, shows the essential features of the new form of construction. The parallel-chord stiffening truss was 25 ft. deep throughout; the revised layout, utilizing part of the cable (or chain) as top chord, provides a truss with depth varying from 22.5 ft. to 42.5 ft.

In the first sketches for the new design, the upper chord in the outer quarters of the span was made curved so as to produce an effect of symmetry about the quarter-points. This yielded a most pleasing outline. Straight chords were substituted, however, in deference to the preference expressed by our client.

Comparative cost-estimates of the two designs shown in Fig. 1 demonstrate a material saving in favor of the new design. In addition to the major elements of economy outlined in the preceding general discussion, there are a number of incidental savings arising from the change in design. The new form of construction yields the following contributions to a reduction in the total cost of the structure:

(1) Saving the material represented by the middle half of the top chord of each stiffening truss; (2) a general saving in the remaining chord material resulting from the use of an economic truss-profile conforming to the variation of bending moments along the span, and in particular, a material reduction in the maximum chord-sections previously required in the vicinity of the quarter-points of the span; (3) a saving in details and in minimum sections resulting from the greater uniformity of required chord-sections throughout the span; (4) a saving in web material on account of the reduced truss-depth in the regions of maximum shear; (5) dispensing, in the middle half of the span, with the subverticals previously required to half-length the compression-chord members which are now replaced by tension members; (6) dispensing with the intermediate top laterals previously required to half-length the same compression-chord members in a horizontal plane; (7) as a result of these various savings, a reduction of about one-third in the total weight of the stiffening truss, and a consequent saving in all portions of the structure affected by the dead-load of the truss; (8) a saving in chain (or cable) sections resulting from the reduced dead-load of truss and the consequent reduced dead-load of chain; (9) saving of the suspenders in the middle half of the span and a reduction in length of the remaining suspenders; (10) a reduction (6.5 ft. in the case of the Florianopolis bridge) in the total height of the towers in consequence of the reduced distance between cable and lower chord; (11) a saving in the towers resulting from the reduction in height combined with the reduction in dead-load of truss and chain; (12) a material saving in the anchorages resulting from the reduced dead-load of truss and chain and the reduced elevation of the backstays.

In the case of the Florianopolis bridge, a large portion of the economy yielded by the change of design was not capitalized but was turned back into the structure in the form of reduced unit stresses to provide a greater margin of safety for future load-increase. Thus the design stress for the stiffening truss was reduced from 20,000 to 18,500 lb. per sq.in. (actually 14,000 as calculated by the exact method); and the unit stress in the chain was reduced from 50,000 lb. per sq.in. to 46,500.

Fig. 2, condensed from the general stress-sheet of the final design, presents the principal features of loading, stresses, dimensions and sections.

Rigidity of the Design—While the governing consideration in the change of design was that of economy, the change yielded greatly increased rigidity as an incidental advantage. The deflection graphs calculated for the adopted design are submitted in Fig. 3. According to these graphs, the maximum deflection under full-span loading is only 1.88 ft. or 1/592 of the span; and the maximum deflection under half-span loading is only 1.36 ft. or 1/820 of the span, with uplift in the unloaded half practically eliminated. The actual deflections will be about 25 per cent less than these calculated values, since the calculations were based on the "approximate method" and since no allowance was made for the stiffening effect of details. These deflections are approximately one-fourth of the corresponding values for the previous (conventional) design. Since the Florianopolis bridge is designed to carry a railway as its principal element of live-load, this reduction of the governing deflections is of practical significance.

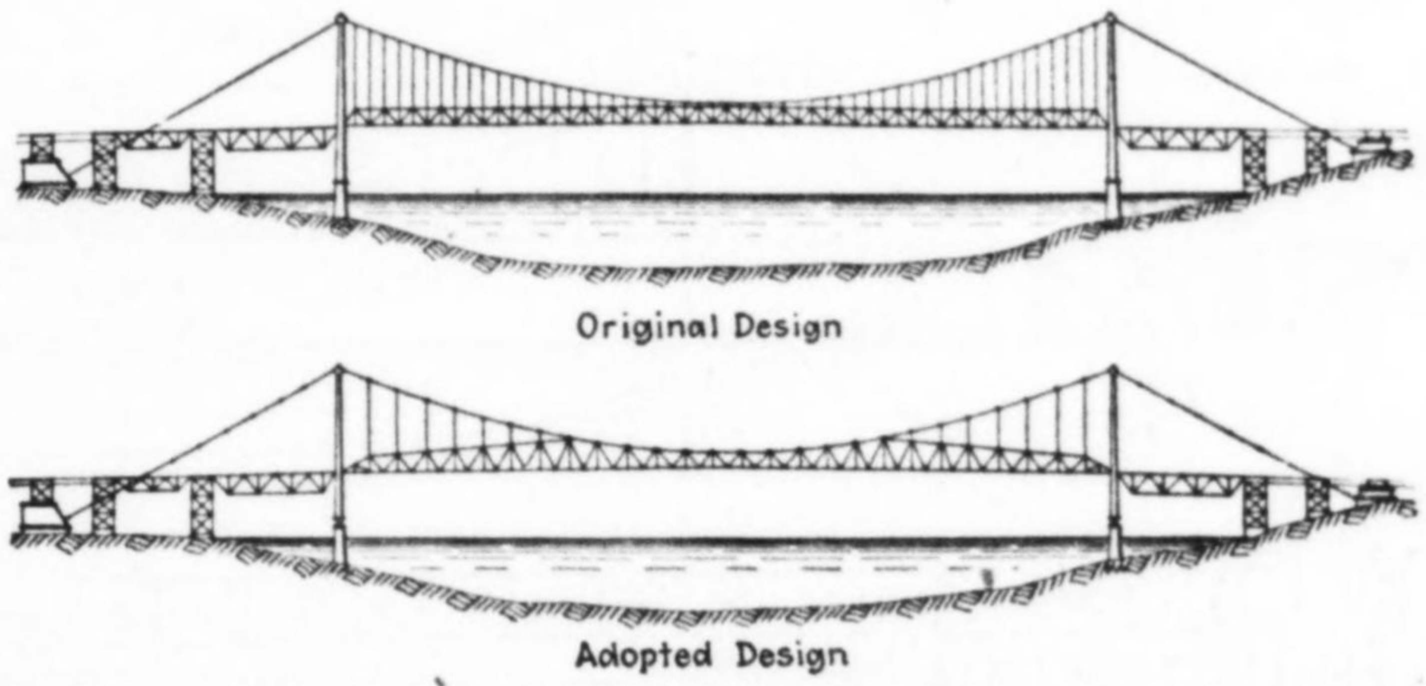


FIG. 1—ORIGINAL (PARALLEL-CHORD) AND ADOPTED (CURVED-CHORD) DESIGNS FOR FLORIANOPOLIS BRIDGE
Adopted design secures great increase of rigidity with less material than required for parallel-chord design

About 25 per cent increase in rigidity may be attributed to the substitution of eyebars for wire cables; the remainder of the 300 per cent increase is the direct consequence of the new form of stiffening construction.

The following elements of the new design contribute to this increase in rigidity: (1) The revised truss-profile is more efficient in resisting deflections, since it provides maximum depth in the regions of greatest bending moment; (2) the depth at the quarter-points has been made nearly twice as great as in the previous design, and the stiffness in the vicinity of the quarter-points is the principal factor in determining the rigidity of a suspension bridge under the critical condition of half-span loading; (3) the functioning of the full section of the cable as top chord of the stiffening truss in the middle-half of the span greatly increases the moment of inertia in that portion of the span; (4) the fact that live-load introduces tension in the middle half of the top chord (by virtue of its forming part of the cable) further reduces mid-span deflections.

As a result of these various factors, the change of design yields greater stiffness with less material in the structure. In approximate figures, the design is four times as rigid with only two-thirds as much material in the stiffening truss. Thus, greater efficiency has been secured through a more scientific design of the suspension stiffening system.

In addition to the marked increase in vertical rigidity yielded by the new design, the lateral stiffness is improved by the large cable (or chain) sections functioning as wind-chords in the middle-half of the span; and ideal longitudinal rigidity is secured by the direct connection of the truss to the cables. Longitudinal or braking forces are carried directly into the cable without producing bending moments in the towers.

The net result of the change in design as applied to the Florianopolis bridge is a reduction in cost (through actual saving in material), an increase in safety and longevity (through lower unit stresses), and an increase in efficiency (as measured by resistance to deflections).

Erection Considerations—Designs have been proposed, in the past, in which the cable (or chain) would be utilized as the top chord of an overhead bracing system. It is believed that the Florianopolis design is the first one in which the desired advantages are secured while retaining a stiffening truss at roadway level from tower to tower. The use of an overhead trussing system (departing from roadway level) necessitates separate wind-chords for lateral stiffening, and would incur a number of other disadvantages; the principal disadvantage would be the increased difficulty of erection. With the truss at roadway level, as in the Florianopolis design, erection can proceed exactly as in the conventional type of suspension structure, namely with the travelers advancing along the roadway deck.

Experts on bridge erection, to whom the two Florianopolis designs were submitted, agreed that the new design would not exceed the original design in cost of erection. In fact, on account of the smaller number of pieces to be handled, the opinion was expressed that the cost of erection should be reduced.

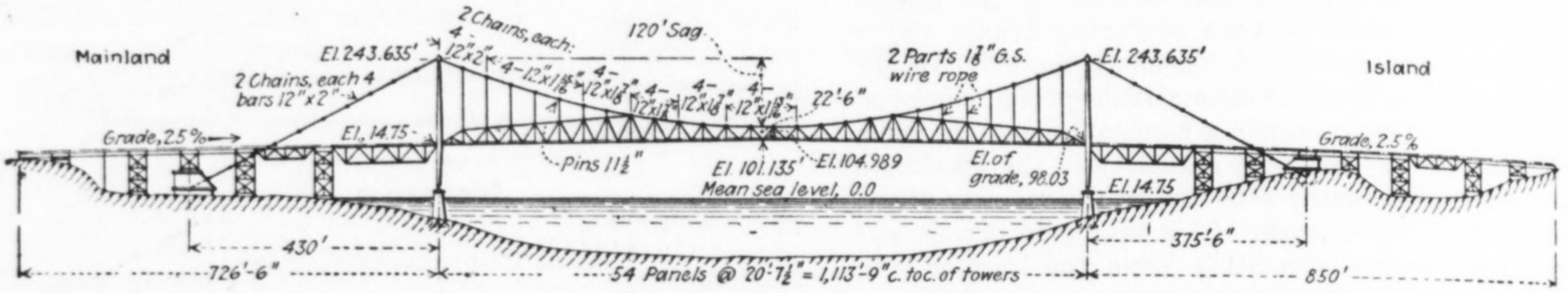
The only new difficulty in the erection arises from the absence of suspenders for adjustment in the middle half of the span. Adjustment of the remaining suspenders, however, suffices for the manipulation of the truss in order to permit connections to be made and in order to bring the truss to a final unstressed condition. Moreover, no difficulty has been experienced in the erection of suspension bridges in which no hanger adjustments were provided. With careful triangulation and fabrication, and with accurate knowledge of the elastic properties of the materials, provision for hanger adjustment can be dispensed with in modern suspension design.

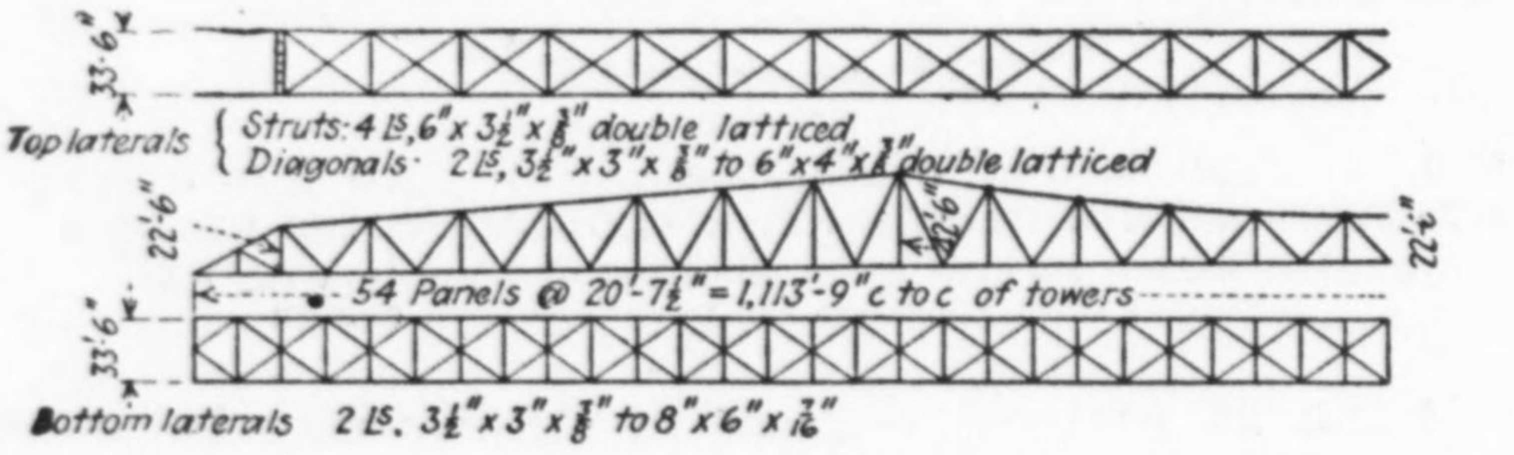
However, the erection contractor decided to leave some of the truss-members with blank connections to be drilled in the field as a safeguard against contingent difficulties. On this account (also to cover drafting-room expense for revision) a charge was made in the erection contract for the privilege of changing the design; but this extra charge was relatively small and amounted to only a minor fraction of the saving in cost effected by the adoption of the new design.

Heat-Treated Eyebars—In the Florianopolis bridge a new material, in the form of high-tension, heat-treated carbon-steel eyebars, finds its first application. This material, intended to be used with a working stress of 50,000 lb. per sq.in., has been developed through recent experimental research by the American Bridge Co. It is furnished under guarantee of minimum elastic limit of 75,000 and minimum ultimate strength of 105,000 lb. per sq.in., and minimum elongation of 5 per cent in 18 ft.

Cables of parallel wire, with long-lay steel rope strands as an alternative, were specified in the original design. After bids had been secured in this country and abroad for steel wire and for rope strands, the American Bridge Co., through the U. S. Steel Products Corp., submitted the suggestion of substituting chains of heat-treated eyebars. They offered to furnish the material and erect the chains at a price which equated the total cost to the estimate based on the lowest bid received for wire. This proposal was attractive to the general contractor, particularly as it relieved him of the cable-erection contract, and the new material was accepted.

When fabrication of the eyebars was commenced, we learned that our client had waived the privileges of inspection, and that the bridge company maintained a policy of secrecy covering the material and the processes of treatment. There was no way for the engineer to assure himself of the uniformity of the product. Full-size tests were, of course, provided; but even these were inconclusive since they were made on eyebars especially fabricated for these tests. Selections by the engineer from the finished lots of eyebars could not be used for the tests because, with few exceptions, the eyebars exceeded in length the capacity of the testing machine. Cutting and re-heading of selected bars exceeding the





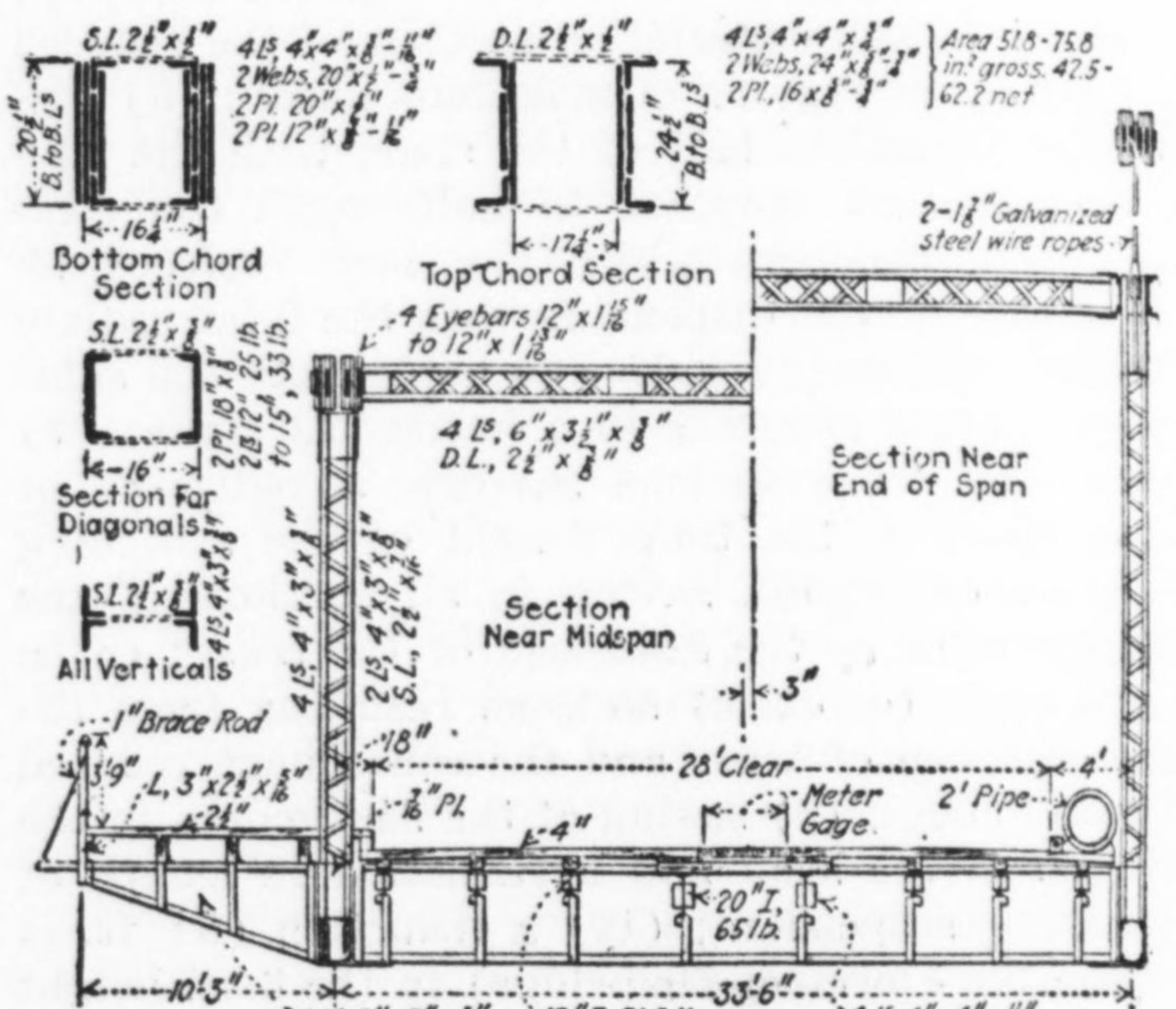


FIG. 2—STRESS SHEET AND CHARACTERISTIC FEATURES OF DESIGN OF FLORIANOPOLIS BRIDGE

40-ft. length-limit was precluded as the operations would disturb the qualities produced by the previous heat-treatment. To every suggestion advanced by the engineers toward securing a verification of the uniformity or reliability of the eyebars fabricated for the structure, some objection was advanced; and the bridge company maintained its position that the eyebars were being furnished under its contractual guarantee. Under these circumstances there was nothing left for the consulting engineers to do but to decline to assume any responsibility for the strength and safety of the eyebars furnished for the structure; and the matter was left with the understanding that the bridge company assumed sole responsibility for this material.

temperatures) on the basis of its chemical analysis and sectional dimensions.

A medium grade heat-treated steel (with 50,000 elastic limit) has been used in recent years for eyebars in several structures with satisfactory results.

The new material (high-tension, heat-treated steel with 75,000 elastic limit) offers attractive advantages to the structural engineer. For its more general adoption, however, the confidence of the engineering profession must be secured; and this awaits the satisfactory solution of the present problems of inspection restriction and test limitation.

A novel detail was adopted for the Florianopolis eyebars in the form of oval pin-holes. The hole is made

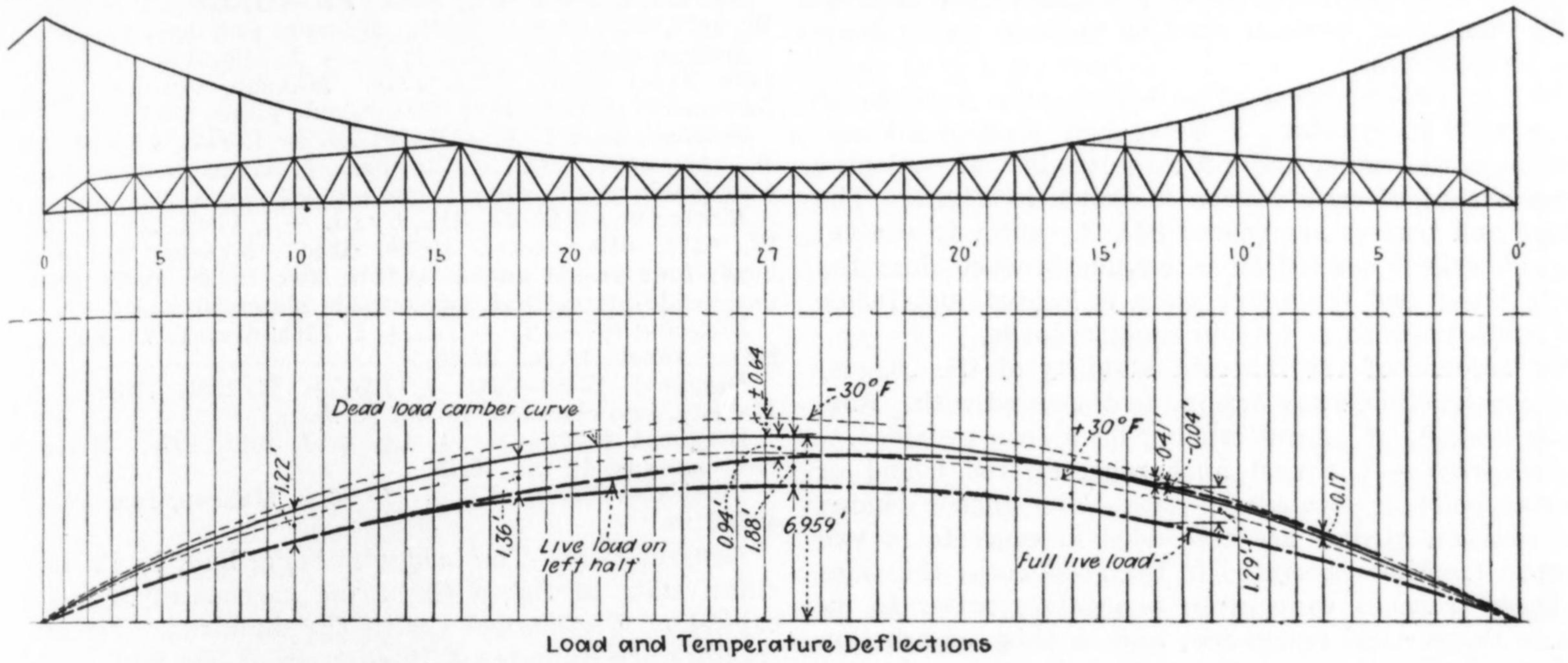


FIG. 3—DEFLECTIONS UNDER LIVE-LOAD AND TEMPERATURE

Apart from the unsatisfactory circumstances at present surrounding the furnishing of these eyebars in respect to limitations of inspection and testing, the material appears to be fairly satisfactory so far as can be judged from the reported tests. In nine full-size tests made during the period of experiment prior to the offering of the material for the Florianopolis bridge, the elastic limit ranged from 78,500 to 89,800 with an average value of 83,920; the ultimate strength ranged from 113,900 to 132,600 with an average value of 123,-200; the elongation in 10 ft. ranged from 6.1 to 9.2 per cent with an average value of 7.2; and the contraction of area ranged from 16.5 to 39.2 per cent with an average value of 24.6. In thirteen full-size tests made during the fabrication of the eyebars for Florianopolis, the elastic limit was 78,180 minimum, 96,830 maximum, 84,560 average; the ultimate strength was 114,660 minimum, 137,900 maximum, 122,990 average; the elongation in 18 ft. was 3.8 minimum, 8.1 maximum, 6.7 average; and the contraction of area was 7.6 minimum, 32.3 maximum, 23.9 average. Only one eyebar failed to meet the specified requirements, yielding an elongation of only 3.8 per cent; a new eyebar was made up for "re-test" and passed the requirements with 8.1 per cent elongation.

Since the previous experimental work had been confined to eyebars of 12x2-in. section, the eyebars for Florianopolis were made to conform to this section as nearly as possible, ranging from 12 by $1\frac{13}{16}$ to 12 by 2.

It is claimed that each eyebar requires individual adjustment of heat-treatment (with respect to time and

somewhat elongated (axially) and enlarged in back to facilitate entry of the pin, while retaining close fit in the segment of contact. The front or bearing semicircle exceeds the diameter of the pin (11½ in.) by only 5/1,000 in.; the rear semicircle is bored to the diameter of the pin plus $\frac{1}{32}$ in. and the two centers are separated $\frac{1}{8}$ in. along the axis of the eyebar. The unusually close fit thus secured along the bearing surface reduces the secondary stresses in the eyebar head. This detail is copyrighted by the American Bridge Co.

The 11½-in. pins for connecting the 12-in. eyebars are of special heat-treated steel, with a yield point ranging between 60,000 and 65,000, and a tensile strength ranging between 100,000 and 105,000. A few pins are of chrome-nickel steel, of same strength values.

Design of Towers—The Florianopolis bridge will be the first American suspension bridge built with rocker towers. The only large bridges previously built with this feature are the Elizabeth bridge at Budapest (1903) and the bridge over the Rhine at Cologne (1915).

The rocker type offers the most economical and scientific design for suspension bridge towers. It eliminates the bending stresses from unbalanced cable pull, thereby yielding a saving in tower material, and it eliminates the difficulties of the necessary erection operation of pulling back the tops of the towers prior to stringing the cables or chains.

In the case of the Florianopolis bridge, the change was made from fixed-base towers after comparative estimates showed a net saving of about 20 per cent in the cost of the towers in favor of the rocker type.

There is a material reduction in the main sections by the elimination of the bending stresses.

A general drawing of the towers was given in a previous article (*Engineering News-Record*, Oct. 11, 1923, p. 592). The pedestal casting is finished to a plane top surface 27x45 in. The rocker casting is finished on its lower or bearing surface to a radius of 12 ft. The line of contact is 45 in. long. For security against creeping displacement, four screw-dowels of 3 in. diameter are provided. The rocking of the upper casting on the lower was tested in the shop with the dowels temporarily in place. The bottom face of the pedestal casting is cast with two full-length diagonal lugs which engage corresponding grooves in the masonry to prevent any possible sliding of the casting.

The maximum vertical reaction on each rocker-bearing is 2,000 tons.

Bridge Cross-Section—The bridge was required to carry a 28-ft. roadway, a meter-gage electric railway, a 24-in. water main, and a 9-ft. sidewalk. The adopted arrangement of cross-section is shown in Fig. 2. The cables and trusses are spaced 33$\frac{1}{2}$ ft. center to center; the sidewalk is carried on an outside bracket along the north truss, and the water main is located just inside the south truss so as to help equalize loads.

On account of the inherent stability of the suspension construction, sway-bracing is dispensed with. Adequate systems of lateral bracing have been provided.

Anchorages—The east anchorage, on the island of Florianopolis, is located on rock. The anchor chains and reaction girders are embedded in concrete in two stepped trenches excavated in the solid rock; the sides of these trenches were given a negative batter to increase the vertical resistance; and on this construction is superimposed the buttressed concrete anchorage structure. The west anchorage is situated on lower ground on the mainland. The excavation did not reveal rock where anticipated, and a pile foundation had to be used; about 25 per cent of the piles (located under the forward portion of the anchorage) are battered in the direction of the resultant pressure. Both anchorages are U-form in plan, for maximum efficiency.

Each anchor-chain, consisting of heat-treated eyebars, divides into two branches for connection to the anchor girders. Each anchor girder, 16 ft. long, consists of five built-up girders connected together by tie-plates and reinforced with pin-plates. At the Y-point of each anchor chain, a built-up pin-seat was provided to hold the pin in accurate position during the placing of the concrete around the anchor girder and the connecting eyebars. Above these points, the anchor chains were boxed-in during the completion of the concreting in order to prevent adhesion of the concrete until full dead-load strain is in the chains.

Loads and Stresses—The dead-load used in the design of the main span totaled 4,370 lb. per lin. ft. (chains 770 lb., and suspenders, trusses and bracing 1,530 lb., floor 1,670 lb., water pipe 400 lb.). The live-load was taken at 2,000 (plus 10 per cent) and 1,850 lb. per lin. ft. for the design of trusses and chains respectively. The floor was proportioned for a 50-ton electric locomotive followed by 2,000 lb. per lin. ft., plus 50 per cent impact, a 6-ton motor truck (or 60 lb. per sq.ft.) with 25 per cent impact, and a sidewalk load of 60 lb. per sq.ft. Wind was taken at 25 lb. per sq.ft. (30 lb. on viaduct), and temperature variation at ±30 deg. F. With these loads the floor steel was designed for a stress of 17,000 lb. per sq.in., the towers and trusses for 18,500 (increased 25 per cent for temperature and wind addition), the suspenders for 55,000, and the chains for 46,500.

The following typical stresses and sections of stiffening truss members are quoted from the stress sheet (panel-point being at the tower, 16 at the junction of top chord with chain, and 27 at midspan). Stresses are given in thousand-pound units per truss.

Top chord 6-8—DL, 0; LL + I, 372t, 776c; T, 57; W, 155t, 223c; total, 584t, 1056c. Makeup, two webs 24x$\frac{5}{8}$, four outside angles 4x4x$\frac{3}{4}$, two outside plates 16x$\frac{3}{4}$ in.

Top chord 14-16—DL, 0; LL + I, 244t, 755c; T, 71; W, 385t, 425c; total, 700t, 1251c. Makeup,two webs 24x$\frac{5}{8}$, four outside angles 4x4x$\frac{3}{4}$, two outside plates 16x$\frac{3}{4}$ in.

Top chord 26-26—DL, 2823t; LL + I, 318t, 332c; T, 196; W, 527t, 527c; total, 3864t. Makeup, four bars 12x1$\frac{13}{16}$ in.

Bottom chord 5-7—DL, 0; LL + I, 730t, 364c; T, 52; W, 244t, 311c; total, 1026t, 727c. Makeup, two webs 20x$\frac{3}{4}$, four inside angles 4x4x$\frac{5}{8}$, two outside plates 20x$\frac{1}{2}$ in.

Bottom chord 15-17—DL, 0; LL + I, 717t, 220c; T, 71; W, 514t, 548c; total, 1302t, 839c. Makeup, two webs 20x$\frac{3}{4}$, four inside angles 4x4x$\frac{11}{16}$, two outside plates 20x$\frac{1}{2}$ in.

Bottom chord 25-27—DL, 0; LL + I, 1180t, 33c; T, 165; W, 615t, 615c; total, 1960t, 813c. Makeup, two webs 20x$\frac{3}{4}$, four inside angles 4x4x$\frac{11}{16}$, two inside plates 12x$\frac{11}{16}$, two inside plates 18x$\frac{5}{8}$, two outside plates 20x$\frac{1}{2}$ in.

Diagonal 6-7—DL + LL + I, 126t, 61c; T, 7. Makeup, two channels 12 in., 25 lb.

Diagonal 15-16—DL + LL + I, 198t, 186c; T, 1. Makeup, two channels 15 in., 33 lb.

Diagonal 26-27—DL + LL + I, 218t, 192c. Makeup, two channels 12 in., 25 lb.

Verticals—DL + LL + I, 93t. Makeup, four angles 4x3x$\frac{5}{8}$ in.

General Data—The bridge is being built by the Brazilian state of Santa Catharina to connect its island capital of Florianopolis with the mainland. The contractors are Byington & Sundstrom of São Paulo. The United States Steel Products Corp. has the subcontract for furnishing and erecting the steelwork. The substructure, including the foundations, piers, anchorages and abutments, was completed several months ago by the principal contractors. The steelwork has been fabricated and shipped, and erection is in progress. It is expected that the bridge will be completed in the summer of 1925. Robinson & Steinman, of New York, are the consulting and designing engineers. L. N. Gross has been associated with them as consulting engineer on the construction.

A preliminary article on the design of the Florianopolis bridge was published in *Engineering News-Record*, Oct. 11, 1923, p. 592, and an illustrated account of the construction of the piers and anchorages appeared in the issue of July 10, 1924, p. 51.

The approximate weight of steel in the bridge (including approaches) is 4,400 tons, made up as follows:

	Tons
Chains—Eyebars and pins	780
Main Span—	
Trusses and bracing	840
Floor system	400
Main Towers—	
Columns and bracing	850
Castings	90
Anchorages—Eyebars and girders	130
Approaches—	
Spans (incl. floor and bracing)	940
Towers and bracing	290
Miscellaneous—Railings, etc.	80
Total	4,400

The approximate total quantity of concrete in the island anchorages is 2,350 cu.yd., in the contingent anchorage 5,250 cu.yd., and in the piers and abutments 5,000 cu.yd., a total of 12,600 cu.yd.