Design and Construction of Hudson Hope Bridge

by J. Dudra*

INTRODUCTION

The Hudson Hope Bridge, which crosses the Peace River in northern British Columbia, is located between the town of Hudson Hope and the giant Peace River Power project. The site is approximately 600 miles north of the Canada-United States border and is located in the Peace River Valley, east of the Rocky Mountains. The crossing is on the newly constructed Chetwynd-Hudson Hope highway which provides a shorter link for traffic moving from central British Columbia to the Hydro Electric project. Eventually, when additional highway construction is completed, the above will provide a shorter route connecting to the Alaska Highway.

In 1961, the British Columbia Department of Highways appointed Phillips, Barratt and Partners, consulting engineers, to conduct site surveys for a feasible crossing of the Peace River upstream from Hudson Hope, and to proceed rapidly with the final design of a bridge structure which would facilitate the passage of heavy equipment and material to the Peace River Hydro Project.

It was at this time that Col. H. H. Minshall, a well-known bridge erector in British Columbia, conceived the idea of a precast concrete suspension bridge. This concept appeared to be particularly suitable to the Peace River site which was rich in local concrete aggregates, far removed from railhead and deprived of good highway access. Consequently, the consultants were requested to give consideration to the Minshall concept if the design proved workable and if the construction appeared economically feasible.

The initial surveys disclosed several potentially feasible crossing sites, the most economically favorable of which was in a constricted area of the river where both banks were nearly vertical. Design studies of a precast concrete suspension bridge to span the river at this site confirmed both workability and economy and this concept was therefore adopted for the final design of the crossing.

Eleven tenders were received for the construction of this bridge and the contract was awarded to the low bidder, Hans Mordhorst Ltd., in September 1963. The structure was completed and opened to traffic one year later in September 1964 (Fig. 1).

SITE AND SUBSURFACE CONDITIONS

The site is in a portion of the river which is considered part of the Lower Peace River Canyon. The river occupies a rock walled valley which

*Partner—Phillips, Barratt and Partners, Consulting Engineers, Vancouver, B.C., Canada.

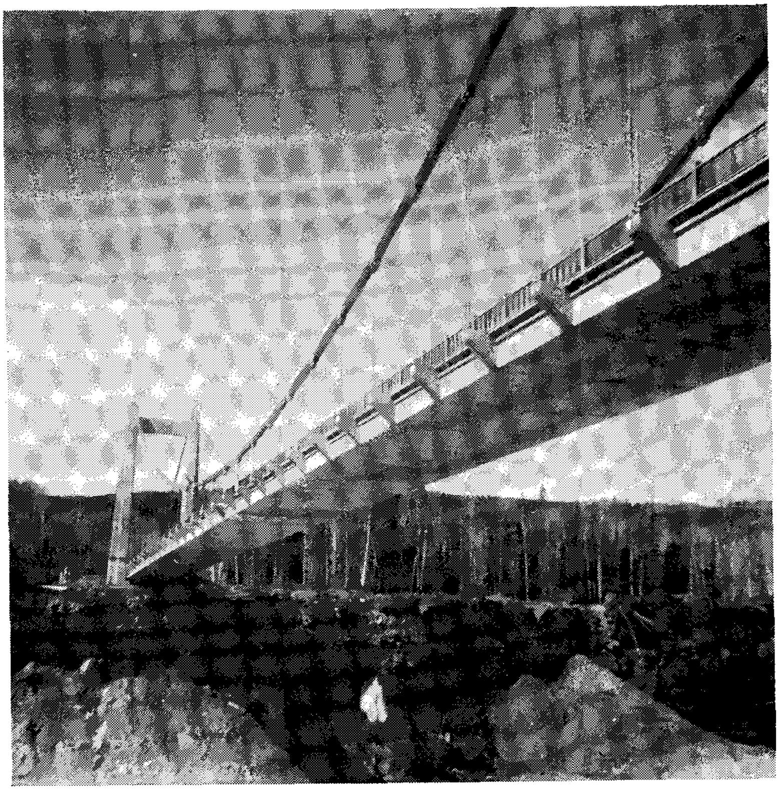

Fig. 1—Completed Bridge

has several constrictions between the dam site and a point about 3.3 miles upstream of Hudson Hope. The chosen site is the second constriction upstream of Hudson Hope.

Close to the river, the overburden is fairly shallow. The underlying strata consists of varying grades of sedimentary rock layers, which are composed of sandstones and sandy shales at the surface, and shales in the deeper portions.

These sedimentary rocks are members of the Fort St. John group of the Lower Cretaceous Age and as a result have been "prestressed" and fairly well cemented. However, they do exhibit a development of shaly weathering when exposed to air. At some locations, where the river flow impinges on the banks, erosion or scour, chiefly of the shale, takes place. In colder weather, frost action loosens the outside layers of rock. This action, quite commonly produces a 3 to 5 ft. projection or overhang of the uppermost layers of rock. For the above reasons, any foundations above the canyon walls had to be located more than 40 ft. away from the upper edge of the walls.

The Peace River, one of the major rivers in British Columbia, is presently being prepared for the Portage Mountain Dam, some 8 miles upstream of the crossing. Until completion of the dam, when discharge will be stabilized and kept fairly uniform, the river discharge varies considerably between the minimum and maximum. At the crossing site the low water elevation of 1503 ft. provides a channel depth of 40 ft.; whereas, at the high water elevation of 1527 ft., the channel depth becomes 64 ft. Winter freeze-up conditions have been known to produce an 18 ft. thick ice layer on the surface in some locations. Spring breakup of the ice causes a severe build up in the constricted channel portions of the river.

ALTERNATIVE CROSSINGS

Site surveys indicated two possible locations for a crossing over the river. Site *A*, the first constriction upstream of Hudson Hope, had a distance of 670 ft. face to face of banks. Site *B*, ¾ of a mile further upstream had a distance 565 ft. between the near vertical banks. The Chetwynd-Hudson Hope highway *desire* line passed east of both crossings. Therefore, Site *B*, when evaluated against Site *A*, had to be charged with an additional one and a half miles of highway. Four alternative crossings were considered:

Scheme 1—Fixed steel deck arch at Site *B*, 565′ c/c of skewbacks

Scheme 2—Continuous steel deck truss at Site *B*, 160′-340′-160′ main spans

Scheme 3—Concrete box girder suspension span at Site *B*, 675′ c/c of towers

Scheme 4—Continuous steel deck truss at Site *A*, 150′-340′-270′ main spans

Scheme 3 was a modified scheme of a segmental concrete box girder suspension bridge conceived by Col. H. H. Minshall.

Schemes 2 and 4 had their main piers located within the channel constrictions.

The fixed steel deck arch and the concrete deck suspension bridge were not actively considered at Site *A* because of less favorable river bank elevations and significantly longer span requirements.

In summary, the results of the economic investigations revealed the following. Costs shown were only relative, and did not include bridge details, approaches and other portions common to all schemes.

Site B	
Scheme 1 (Steel arch)	\$1,321,000
Scheme 2 (Steel truss)	\$1,357,000
Scheme 3 (Concrete suspension)	\$1,275,000
Site *A*	
Scheme 4 (Steel truss less credit of \$206,000 for shorter highway)	\$1,343,000

From the foregoing cost data, the suspension scheme offered a slight saving over the other schemes considered. Additional considerations were as follows:

1. The desirability of keeping piers and thrust blocks out of a river which is being developed for hydro electric power so close upstream.

2. The problem of potential long term scour around piers or thrust blocks which are located in a narrow river constriction.

3. The indeterminacy of ice build-up and ice loads on river piers in the narrow channel.

4. The relatively short construction period during which river pier or arch thrust block work could safely be carried out in this area. It may be noted that the river peak flow occurs during the period when ideal construction time prevails.

On the basis of the above conclusions, Scheme 3 was adopted as the choice for the crossing. The choice was a sound one, as the concrete box girder suspension bridge utilized native materials to a very large extent. The area contained good aggregates for the concrete deck, towers, and anchor housings. The sedimentary rock strata on each bank provided excellent support for the tower footings and relatively low cost anchorages for the main cables.

The successful completion of this project at a cost slightly below the consultant's estimates, served to confirm the preliminary studies on this project. Fig. 2 shows the elevation of the crossings.

BASIS OF DESIGN

The requirements for the deck and its supports as stipulated by the Department of Highways were as follows:

1. Roadway width—curb to curb 28′-0″ for two 14′-0″ lanes.
2. No sidewalks—curb to handrail face-10 in.
3. Design live load:
   a. H25-S20 for slab, cross girders, suspenders and immediate connections.
   b. H20-S16 for stiffening girder and main cable.
4. Design Specifications—"Specifications for Highway Bridges" by British Columbia Department of Highways and "Standard Specifications for Highway Bridges"—1961 edition by the American Association of State Highway Officials.

ANALYSIS OF STIFFENING GIRDER

The deck cross-section was given considerable study. The heavy wheel loads (with possible overloads of construction equipment) limited the practical span of the slab if the dead load of the deck was to be kept minimal. After several trial sections were evaluated, together with their added function as a longitudinal stiffening element, the section as shown in Fig. 3 was chosen as the one best fulfilling all the requirements.

The analysis of the longitudinal stiffening girder was based on the Deflection Theory as developed by Dr. D. B. Steinman.[1] For preliminary design of the structural components an A.S.C.E. paper by Hardesty and Wessman[2] was utilized.

The decision to use the deflection rather than the elastic theory for the analysis of this structure was made primarily because the former takes into account the effect of the deadload of the suspended deck components in partially overcoming the bending moments due to live load. Admittedly, the elastic theory, based on superposition of combined loading on the structure and the utilization of influence lines involves a simpler analytical process; and, where the deck dead load is a relatively small increment of the total

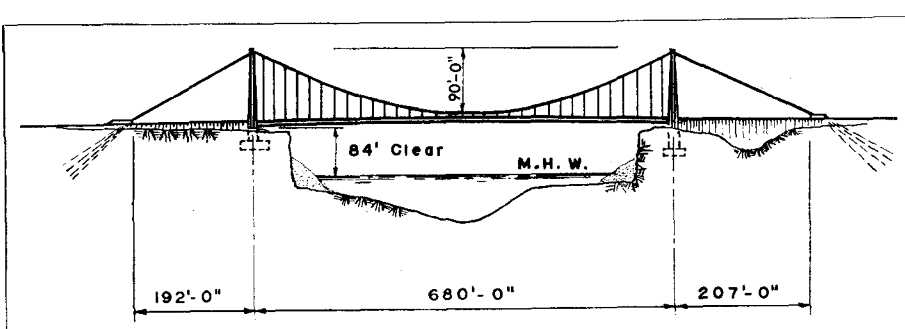

Fig. 2—Elevation

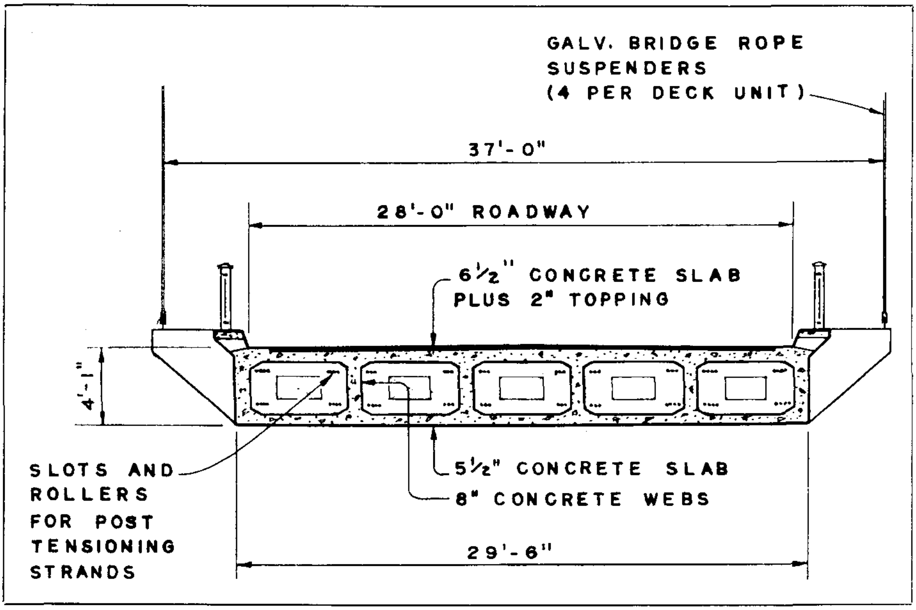

Fig. 3—Typical Section—Suspended Precast Deck Unit

load, this approximate theory is understandably adequate. However, in the case of a suspended concrete structure, the dead weight is a significant load factor which, if ignored in the live load analysis, would make the design of a concrete suspension structure completely impractical.

A further differentiation from conventional steel suspension bridge design should also be noted.

In conventional suspension bridges, the stiffening element has been the truss or girder, which is located directly below each cable. The trusses span longitudinally and are supported elastically by the suspenders. The floor system only distributes the load outward to the trusses and is ineffective as a longitudinal stiffening element.

The box girder, by its two-way load distributing action of the longitudinal webs, flanges and cross-girders restrains the loads in a more efficient and safer manner. Because of this, the concrete box girder deck can economically and functionally compete against the conventional steel stiffening truss for spans below 800 feet and possibly longer if further improvements are made.

Fig. 4 shows the geometrics of the behavior and functions of a stiffening girder under the action of a particular live load. Observing the figure, and remembering the law of virtual work, it is evident as to the part the dead load performs in contributing to the restraint of the live load moments.

In general, the properties of the stiffening girder were arrived at from the following requirements:

1. Transverse bending set the top slab thickness of 6½ in.

2. Bottom slab thickness of 5½ in. was set by the Code.

3. Web thickness of 8 in. was the minimum that was felt to be practical for "on site" construction.

4. The depth, *d*, was a variable which required several trial analyses

before it could be established. A large d produced high unit stresses in the flanges. A small d produced a very flexible stiffening girder which approached the unstiffened cable condition. In addition, a small d prevented the creation of cell openings in the box girder through which construction and maintenance personnel could pass. Various trial designs finally confirmed that the most functional depth for this span was 4'-0" combined with a cable sag of 90 ft. (It is interesting to note that the economic and functional ratio of cable sag to span, n, for steel suspension structures is between 0.09 and 0.11, whereas the feasible ratio for this structure was 0.13.)

Fig. 5 shows the live load and its position, which produced the maximum moments along the girder.

The behavior and analysis of the concrete box section as a girder to resist the live load bending moments was directly related to the stiffness of the girder. This stiffness is not only a function of the moment of inertia, I, of the girder but also of the modulus of elasticity, E, of the concrete. In conventional steel suspension bridges, the E of steel regardless of grade is fairly definitive and constant. However, this is not the case with concrete, whose variable modulus of elasticity is affected by creep and flow and whose deflections therefore, are dependent on whether the loading is short or long term.

A study was therefore necessary to establish the relationship between the variations in the concrete modulus and the bending moments in the concrete stiffening girder. This relationship is shown in Fig. 6. An examination of the results of this study revealed that the variations in the girder moment were not directly proportional to E but still great enough to introduce significant errors into the analysis of the girder. Obviously, overstress could only occur if the in-situ modulus E, was greater than the design modulus. As a safeguard against the possible overstress, a layer of reinforced neoprene was placed between each segmental concrete deck unit. This material, with its very low modulus of elasticity, served to achieve the following:

1. Reduced the modulus of elasticity of the composite girder

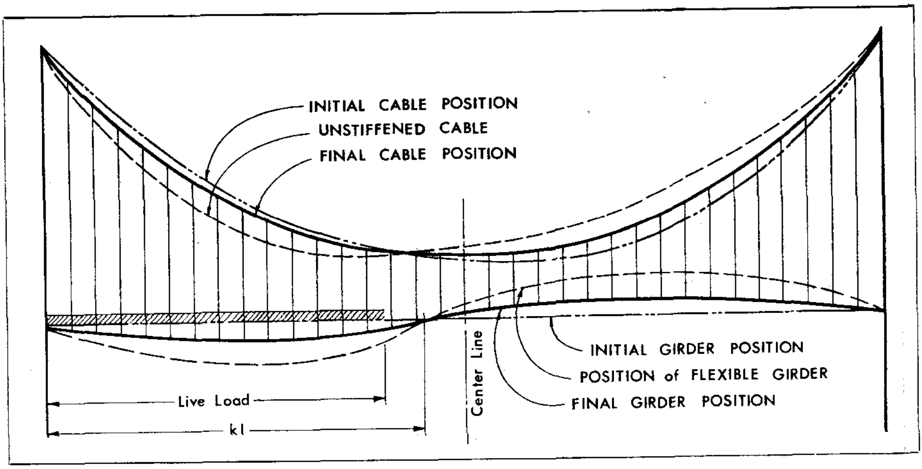

Fig. 4—Deflection of cable and stiffening girder under partial live load

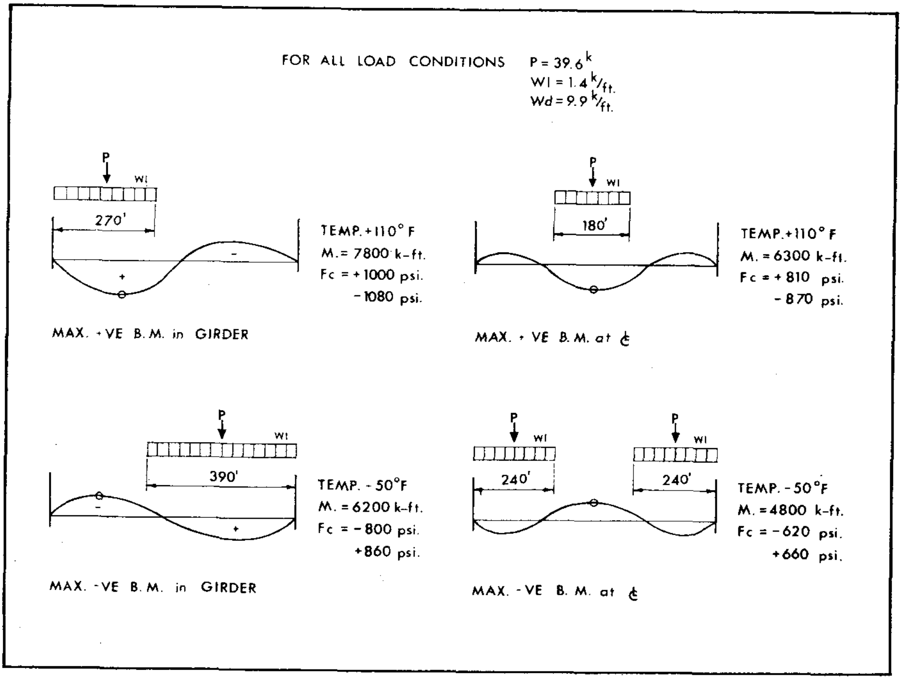

Fig. 5—Live Load Positions for Maximum Moments

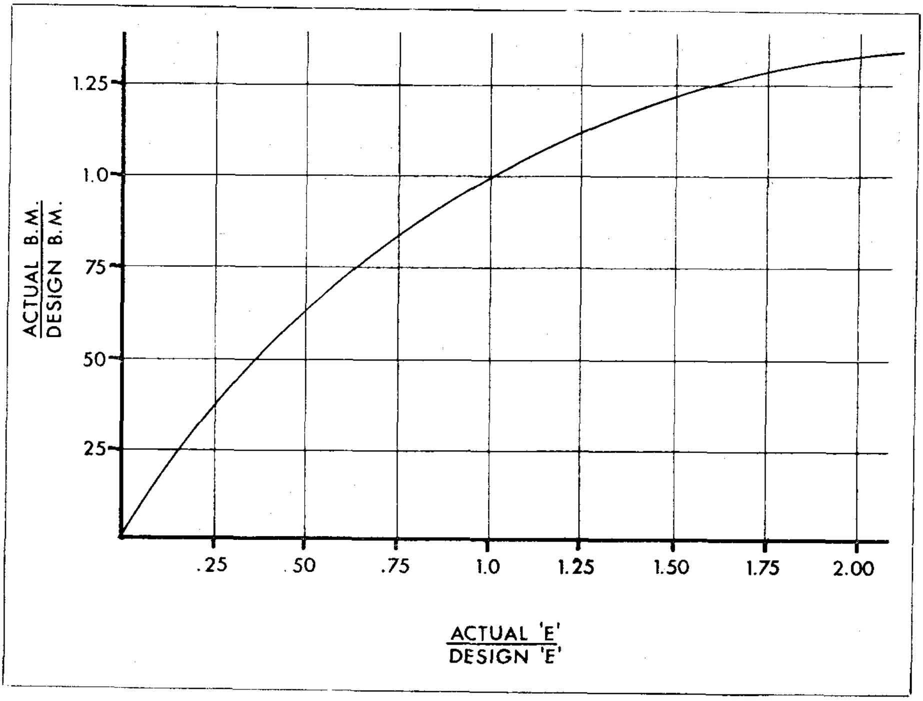

Fig. 6—Relationship between Girder Modulus and Bending Moment

as a safeguard for short term live load conditions.

2. Functioned as a "spring" or plastic hinge in the event of heavy load concentrations on the bridge.
3. Protected the abutting faces of the deck units during erection.

The design thickness of the reinforced neoprene, ¼ in., was established by analysis of the required composite modulus which, conservatively, was assumed to be a combination of the E for the neoprene and a value of the concrete modulus 50% greater than normal.

The stiffening girder was divided into 34–20 ft. long segmental units. Each unit weighed 90 tons. A cross girder located at each end of the unit served as the connection to the suspenders of the main cable system. By this system, as shown in Figs. 3 and 8, each segmental unit was supported by 4 suspenders. The units were interlocked not only by the longitudinal prestress force, but also by 6 curved keys, one in each web. The latter provision also facilitated proper alignment of the units during and after erection.

An operation with 34 precast segmental units could result in near disaster if the joints were not matched exactly. The use of grouted joints on a suspension span which experiences considerable vertical and lateral movement was considered unsatisfactory. The solution therefore was to cast the units on a bed set to a predetermined profile and with their respective faces abutting. Detailing prevented the adjacent cross girders from bearing on each other. The reinforced neoprene, cemented to one abutting face, served as a form for its mate. Sixteen units were cast on each bank (Figs. 7 and 8) and after erection, the two end units, which contained the end

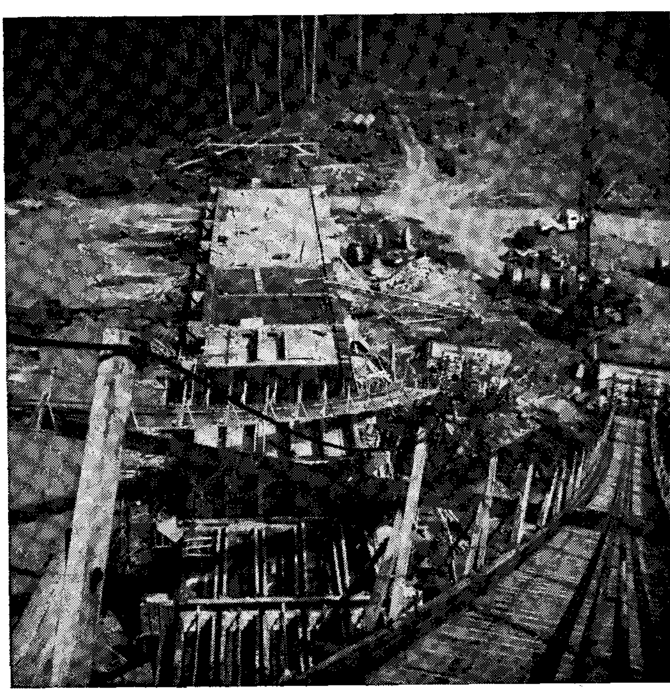

Fig. 7—Segmental Units in Casting Bed

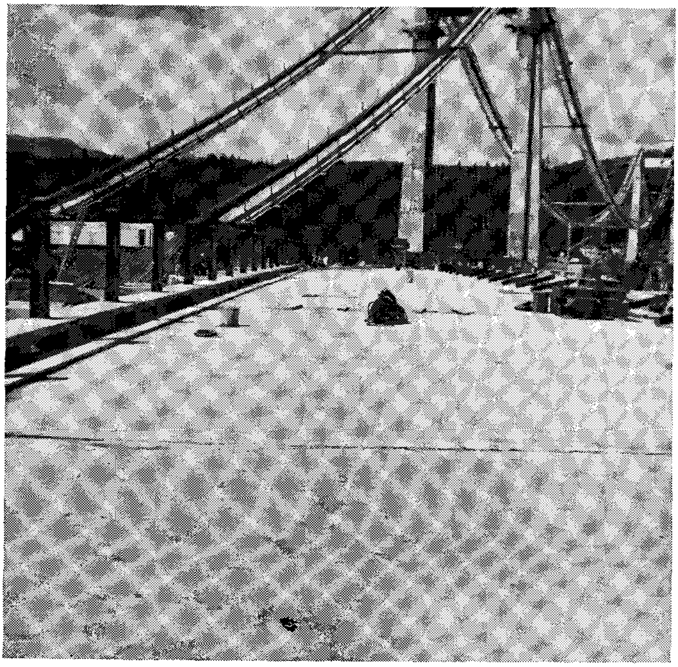

Fig. 8—Segmental Units in Casting Bed

blocks and spiral anchorages were cast in place.

After erection and post-tensioning was completed, the deck slab was protected by a 2 in. thick layer of epoxy bonded concrete topping. A transverse joint was placed over the girder joint to prevent structural action of the topping with the stiffening girder.

AERODYNAMIC STABILITY

The areodynamic stability analysis for this bridge was based on an A.S.C.E. Proceedings paper by D. B. Steinman.[3]

Findings indicated that the structure was stable for both vertical and torsional oscillations—firstly, because of its relatively high dead load and secondly, due to the shape of the deck. The coefficient of rigidity, the stiffness ratio and the aerodynamic stability constant, all had values which exceeded the suggested ones by an adequate margin. Diagonals between the stiffening girder and the main cables at the center of the span prevent relative longitudinal movement and thus help to inhibit torsional motion. Site observations since completion of the bridge have confirmed that the vibrations and oscillations of the structure are negligible under live load and high wind conditions.

WIND ANALYSIS

The structure was designed for 100 m.p.h. wind velocity and the forces were applied in accordance with A.A.S.H.O. Specifications. The main cables were assumed to be completely encased by ice at the time of maximum wind occurrence. The lateral flexibility of the main cable and suspenders caused a redistribution of some load from the main cables to the stiffening girder. The analysis, based on deflection, revealed that the lateral deflection at the center of span was 11½ in. for the cables and 6¼ in. for the stiffening girder.

The small deflection value for the stiffening girder was due to its shallow depth and large lateral rigidity. Longitudinal wind forces were transferred from the deck to the main cables by tie rod connections at the center. The lateral wind load on the deck was transferred to the lower cross beam of the tower by an articulation joint. This joint allows the end of the stiffening girder to move longitudinally and also to rotate in a horizontal and vertical plane about its center.

The end bearings of the structure (roller type) were centered under the webs of the end unit. Should shrinkage and creep of the deck exceed the assumed value, longitudinal adjustment of the bearing is possible.

POST-TENSIONING

The live load analysis as previously outlined revealed flexural stresses which are shown on Fig. 5 for the various loading and temperature conditions. The girder was subject to stress reversals with the maximum ordinates occurring near the quarter point of the span. The stress reversals, though not of equal magnitude, required girder post-tensioning forces such that the combined stress for any condition of loading would remain within the following limits:

1. For Group I loading:
   +0 psi to +1800 psi
2. For Group III loading:
   +0 psi to +2250 psi

A 28 day ultimate concrete strength of 4500 psi was sufficient for the above stresses.

Several prestressing methods were investigated including consideration of grouted or ungrouted tendons. At the outset, the investigation was influenced by the fact that this structure was the first of its kind and therefore warranted close observation over a sustained period of time after its completion. It was also apparent that the bridge could function, in limited fashion, without any longitudinal deck prestressing. The decision was therefore made to use strands which were continuous from one end of the structure to the other, ungrouted and so located that they could be inspected, adjusted and even replaced if the need ever arose.

The use of ungrouted tendons is normally discouraged because of corrosion and lower ultimate load capacity. For this installation the galvanized unbonded system was justified for the following reasons:

1. The crossing is in a dry belt and the girder detailing allowed for the circulation of air around the strands. Moisture is practically excluded from the strands.
2. The galvanizing, even though sacrificial, is considered sufficient for protection. Strands can be removed and replaced if ever required.
3. The post-tensioning system, is not the primary load carrying system. Its failure would not cause collapse of the structure, only limit its live load capacity.

The absence of bond between the prestressing strands and the concrete girder forced consideration of the relative movements between the strands and the girder. The maximum movement, which occurred near the point of inflection of the girder, was calculated to be 2¾ in. Intermediate fixed or sliding supports for the strands were considered prohibitive because wearing forces would soon cause the strands to fail and high friction would cause most of the prestressing force to concentrate near the girder ends where the force was least required by design. It was therefore decided to adopt simple roller supports for each of the strands, which supports, though expensive, resolved all the aforementioned problems.

The 19 wire, 1⅛ in. dia. galvanized strand, CCL system, was chosen for the following reasons:

1. 34 strands in the top layer and 36 strands in the lower layer gave the desired distribution of the prestress force.
2. The end anchorage system proved relatively compact and simple.
3. Adjustment of tension or replacement of strands was possible if ever required.

The end blocks for the anchorages were designed and reinforced for stresses as outlined by Guyon.[4]

Stressing of the strands was performed by calibrated center hole jacks, one at each end of the strand. In this fashion, the value of friction could be determined. Because of the roller support system, the friction was found to be less than 3%.

MAIN CABLES

The maximum tension in the main cables occurred at the tower saddles with the span fully loaded and a temperature of $-50^{\circ}F$. This loading produced a tension of 4200 kips in each main cable. In addition to the direct tension, bending stresses also occurred in the cable when it approached the curvature of the saddle or suspender clamps.

The above factors and the North American practice of allocating axial unit stresses of 70 to 85 ksi for suspension cables, led to the selection

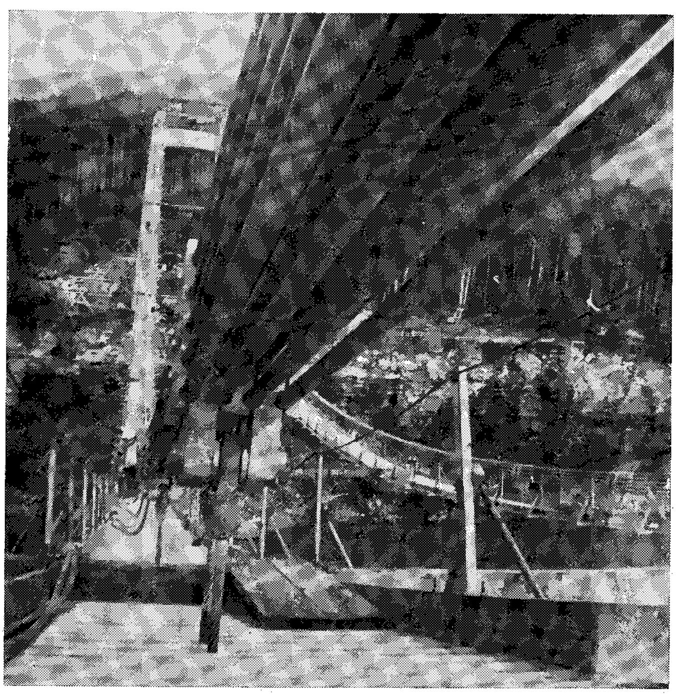

Fig. 9—Main Cable and Suspenders

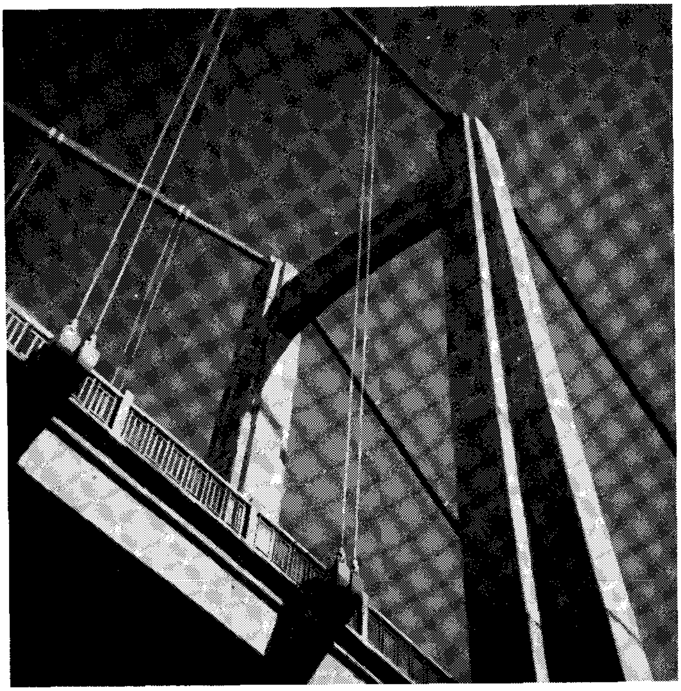

Fig. 10—Main Cable and Suspenders

of 20—2⅛ in. dia. galvanized bridge strands for each main cable. A maximum axial tension of 210 kips was therefore imposed on each strand whose ultimate load carrying capacity was greater than 554 kips. The ratio of working to ultimate load was therefore substantially less than the permissible prestressed concrete standard of 0.6.

It is interesting to note that for the Merelbeke Concrete Suspension Bridge in Belgium,[5] the main cables were allocated an axial unit stress of 112 ksi. The latter was more in line with the allowable stress designations by prestressed concrete codes.

Each 2⅛ in. dia. strand consisted of galvanized wires and all wires were full length with no splices. All strands were prestressed to one half the specified ultimate load and held at this load for 4 hours. The load was then released to the average dead load tension occurring at 30°F. At this tension, the strand was measured and marked for the locations of sockets, saddles, suspenders, etc. The modulus of elasticity for each strand was obtained and the average value of 24.2 ksi was above the minimum of 23.5 ksi specified.

The 20 strands which made up each cable (Figs. 9 and 10) were left separated and unwrapped, unlike the wrapped cable system employed on most other suspension bridges. The considerable economy achieved by this open system appeared justified because this Peace River area is noted for its dry climate and absence of industrial fumes. Sufficient space has been left between the strands to permit the future application of a corrosion inhibitor if ever required.

ROCK ANCHORAGES

The rock in the area of the anchorages proved to be suitable for drilled rock anchors. The following program was performed to test its suitability.

1. Removal of overburden and washing the surface so that a visual inspection for cracks and fissures could be made.
2. Drilling core logs for the purpose of locating fissures or finding strata that was weak in shear.
3. Performing pull-out tests on both grouted and mechanical anchors. The flexibility of the rock was obtained by the use of dial indicators.

Evaluation of the above data by the Department of Highways' Geologists and the Consulting Engineers established the location of the anchorages, the minimum depth below the surface where the anchor force had to be located, and confirmed the feasibility of either mechanical anchors or grouted anchors. For either alternative, the design was such that the anchor force was located near the base of the drilled hole before final grouting, with the anchor acting in a manner similar to the end anchorages of a post tensioning strand. Figs. 11 & 12 show the basic layout of the anchorage housing, anchor plates and the mechanical anchor. The Contractor, Hans Mordhorst Ltd., selected the mechanical anchor alternative and modified the anchor shoe arrangement.

Each anchor had to carry the load of one cable strand, equal to a maximum tension of 210 kips. The specified minimum ultimate load for the anchor was 580 kips. The anchor was stressed to a load of 290 kips by the use of a center hole jack. This load was later relaxed to 230 kips and the top of the anchor locked to the anchor plate. After all the anchors were tensioned, they were grouted for corrosion protection. The above system, in essence, "prestressed" the band of rock between the base of the hole and the anchor plates in the housing. The maximum cable tension would re-

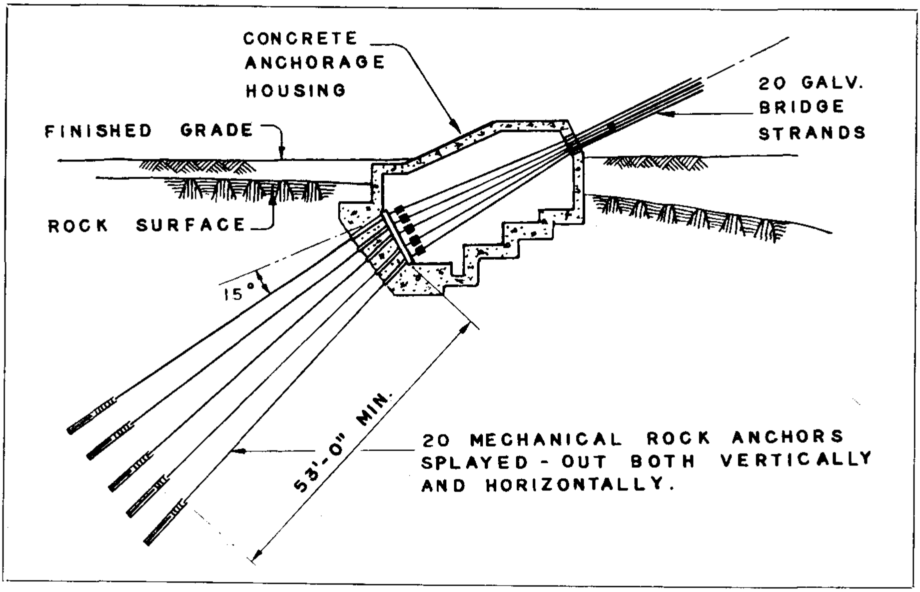

Fig. 11—Anchorage Housing—Typical Section

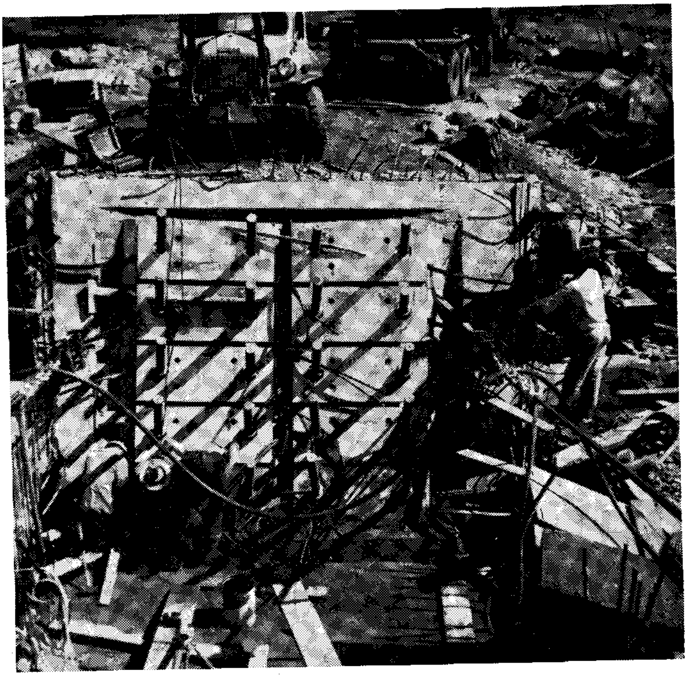

Fig. 12—Rock Anchorages

move most of the prestress, but a minimum of about 10%-15% of the anchor load would always remain. This system offered the following advantages:

1. No anchor load would be transferred to the surface rock which would normally be fissured or water bearing.
2. The anchor plates would remain virtually motionless under all load conditions. (This is important since it is desirable to keep the point of zero movement as close to the tower as possible.)
3. Fracturing of the surface rock by earthquake, erosion or explosives would not endanger the resistance of the anchor block.

TOWERS

The use of concrete for the project was also extended to the tower construction. Fig. 13 illustrates the geometry and cross section of each tower.

One of the most significant factors which affected the tower design was the saddle movement caused by temperature variation (+110°F to—50°F) and cable stretch under live load. The total movement for these conditions was 4¼ in. at the North Tower and 4½ in. at the South Tower. The placement of rollers under each saddle was considered as a means of reducing the bending effect on the towers caused by these movements. However there was no special advantage indicated in this detail since the tower section was adequate in resisting the bending stresses. Furthermore, with rollers under the saddles, the free condition at the top created a longer *effective* column length for the tower, which had disadvantages. Consequently, the saddles were fixed to each tower

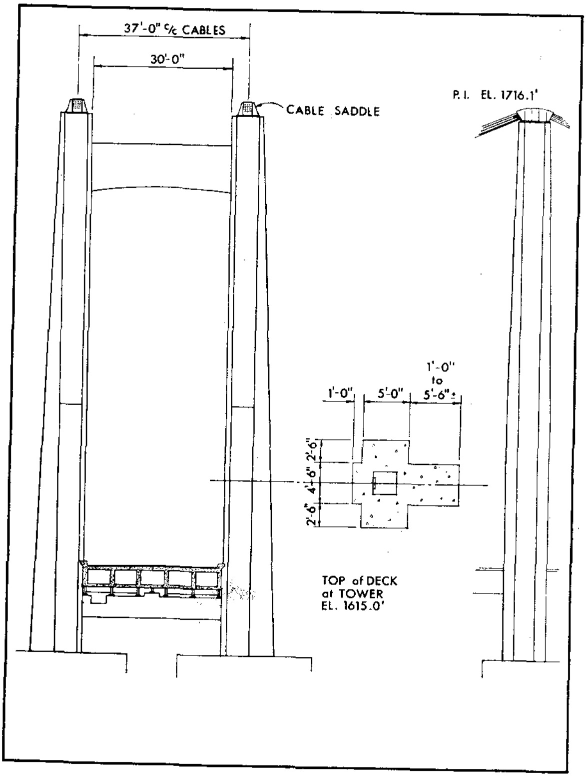

Fig. 13—Tower Details

leg.

The idea of using a pinned tower base in the longitudinal direction was intriguing, but closer examination from the viewpoint of tower construction and the erection of the deck units, made this type of detail impractical. However, to keep the stresses from the longitudinal movements as low as possible the longitudinal depth of the tower from mid height to the base was made constant.

The tower was assumed to have the following properties for the design analysis:

1. Transverse Direction—Rigid frame fixed at the base. Shafts and cross girder having a varying moment of inertia.
2. Longitudinal Direction—Column fixed at the base, pinned at top but subject to partial longitudinal movement. Varying moment of inertia from base to top.

The critical design stresses in the towers occurred from the axial load, longitudinal bending from the temperature movements and transverse bending from lateral wind. The above, when summated and applied to a rectangular cross section, produced high governing stresses at one corner of the tower leg section. By using the cruciform cross section for the tower shafts, a more efficient distribution of the combined stresses was achieved. The cruciform shape was not only more structurally efficient, but also more aesthetically attractive. A 28 day ultimate concrete strength of 3000 psi was used for all the tower concrete.

Before the erection of the deck units commenced, the saddles were offset from the tower center lines. These offsets, 16½ in. for the north tower and 18 in. on the south tower, were required for the effects of the removal of sag and stretch of the backstays. The tower tops could only be deflected 2½ in., otherwise overstress would have occurred at the base due to longitudinal bending. A saddle jacking arrangement which consisted of hydraulic jacks, and a fixed greased plate under each saddle resolved the above problem. As erection of the units proceeded, the tower top offset was kept within the specified limits by this jacking system. After the full dead load was in place, each saddle was permanently fastened to the tower by ten 1⅜ in. dia. anchor rods grouted into preformed holes.

ERECTION OF SEGMENTAL UNITS

The erection of the segmental concrete deck units (each weighing 90 tons) posed several problems not common to conventional steel suspension bridges. The most significant problem was the imposition of the very heavy deck unit load on a cable system before the cables were substantially tensioned, thereby introducing the possibilities of local bending or "kinking" of the cables in the region of the suspender clamps.

Initially, it was felt that some preloading of the cables would have to be resorted to for the purpose of keeping the angle change at the suspender clamps within tolerable limits. The contractor devised two alternate systems of erection, each of which would have partially preloaded the cable prior to the erection of the deck units. Finally, the

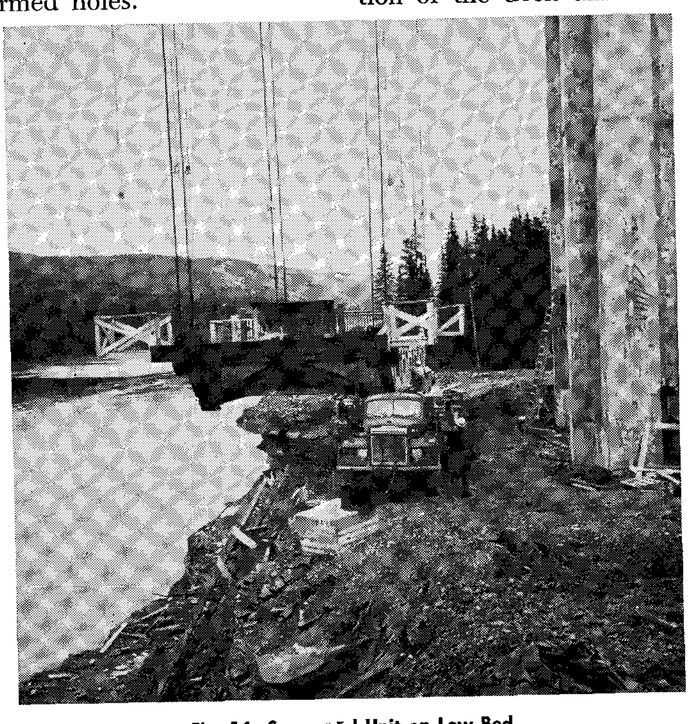

Fig. 14—Segmental Unit on Low Bed

contractor commissioned Dr. R. F. Hooley, Professor of Civil Engineering, University of British Columbia, to conduct tests on a portion of the manufactured 2⅛ in. dia. strand in order to determine the safe angle change the cable could withstand. Results of the tests indicated that *no* preloading was necessary and values for the critical angle changes were established for various cable tensions. Professor Hooley developed a revised erection system which utilized the findings to advantage. The system of a cable traveller, sling carriage and winching for erecting the units was thus developed. A model was constructed to ensure that the erection sequence adhered to the limits that were established in the cable testing program.

Each precast unit was first raised in the casting bed, transferred to a low bed trailer and trucked to the river side of the tower (Fig. 14). From this position, a traveller, sling and two winches took over and swung the units to their final position over the river (Figs. 15 and 16).

The traveller, a specially fabricated steel frame on four rubber-covered wheels, was mounted on the permanent cable system of the bridge. Winch lines from each shore of the river controlled the traveller movements along the cable. A steel sling, contoured to fit under the deck units, was hung from the traveller.

The typical procedure for erecting each precast unit started with the unit being lifted from the low bed at the shore line of the river. Vertical dogs, bolted to the traveller and bearing against the main cable clamps, prevented the traveller from slipping down the cable during the lifting operation. Two pairs of suspenders immediately in advance of the unit were then pulled over and connected to the sling. The unit was

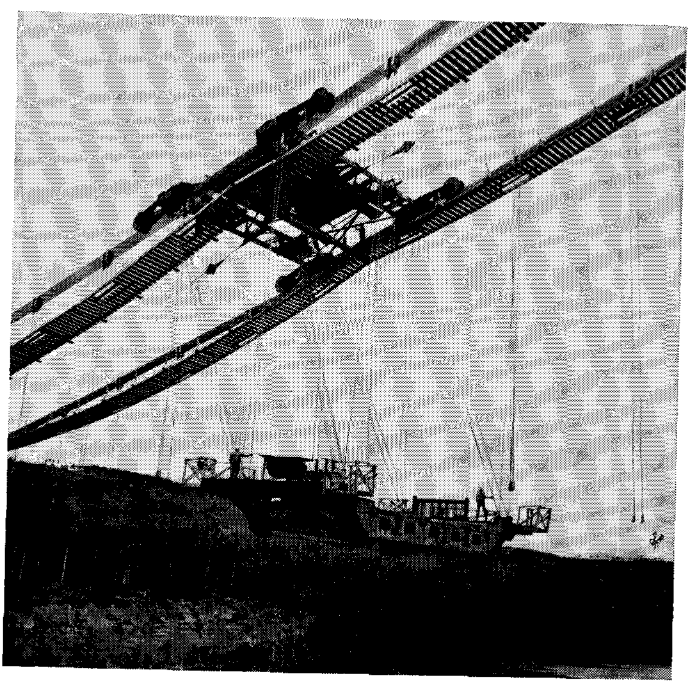

Fig. 15—Erection of Segmental Units

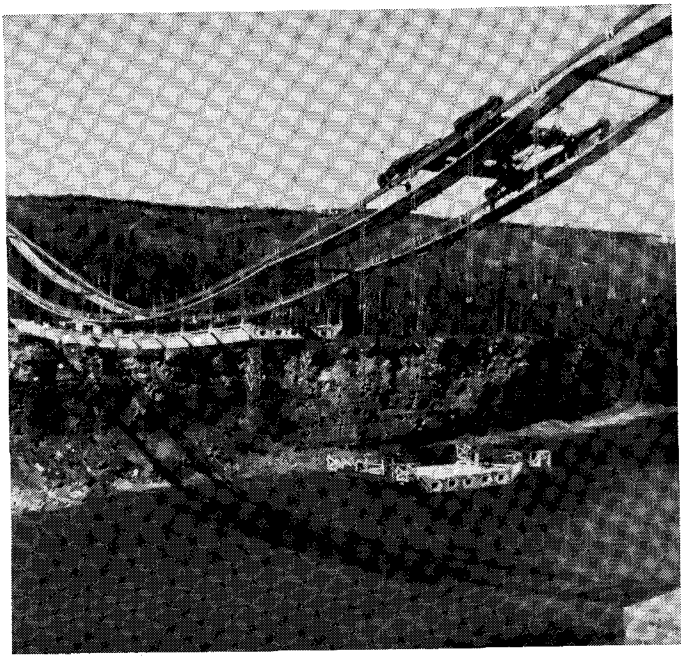

Fig. 16—Erection of Segmental Units

then attached to both the sling lines and two slack pairs of suspenders. With the traveller in a stationary position, the sling lines were lowered, the unit swung over like a pendulum, and its load was thus transferred to the suspenders. With the traveller lines now unloaded but still connected, the traveller dogs were retracted (by four hydraulic jacks mounted on the frame) and then the traveller was moved down the bridge cable to repeat the pendulum-like swinging operation which eventually enabled the concrete unit to be transferred to its final suspender position on the bridge. In the center half of the span, where the permanent bridge suspenders became too short for the swinging operation, temporary suspender extensions were attached.

In this relatively simple and inexpensive fashion, six precast units were erected from the North bank of the river, eleven from the South bank, then ten from the North and the final five from the South. This sequence was necessary to keep within the permissible angular distortion of the main cables. The closing units, one at each end of the bridge, were then cast in place.

It is interesting to note that the 32 precast deck units were placed in about 10 working days.

TEST PROGRAM

The actual behavior of this structure, possessing a concrete stiffening girder which is unique was a matter of interest. A study program was therefore undertaken during the summer of 1965, some 9 months after the bridge was opened to traffic, for the purpose of obtaining data on the behavior of this structure.

One aspect of the program was the comparison of the actual and theoretical behavior of the stiffening girder when subject to the passage of a 94 ton live load (two 47 ton trucks side by side) over the span.

The two trucks were positioned at 6 locations along the span. Deflection ordinates at the panel points for each position were recorded. The structure was then analyzed for two of the above locations.

Case 1—Live load at center line of span.

Case 2—Live load at S8—Probable position for maximum positive moment in girder.

The same span constants which were adopted for the design were also used for the above analysis.

Fig. 17 shows the theoretical and actual deflection curves for Case 2 above. The elastic theory curve is also shown as a matter of interest.

Examination of the results revealed that the girder stiffness assumed for the design of the structure is reasonably close to the actual in situ stiffness. The results for Case 1 (not shown) revealed an even closer agreement between the actual and theoretical deflections.

The stiffening girder moments for the live load position of Case 2 are indicated on Fig. 18. Again, the elastic theory moments are shown for comparison. The moment curves clearly indicate the large difference in moments when this structure is analyzed by either the deflection or elastic theory.

The maximum moment under the 94 ton live load is about 70% of the design *live load* moment for the stiffening girder.

Another analysis has been performed for the 146 ton live load spread over a length of 80 feet. This load will occur from a low bed

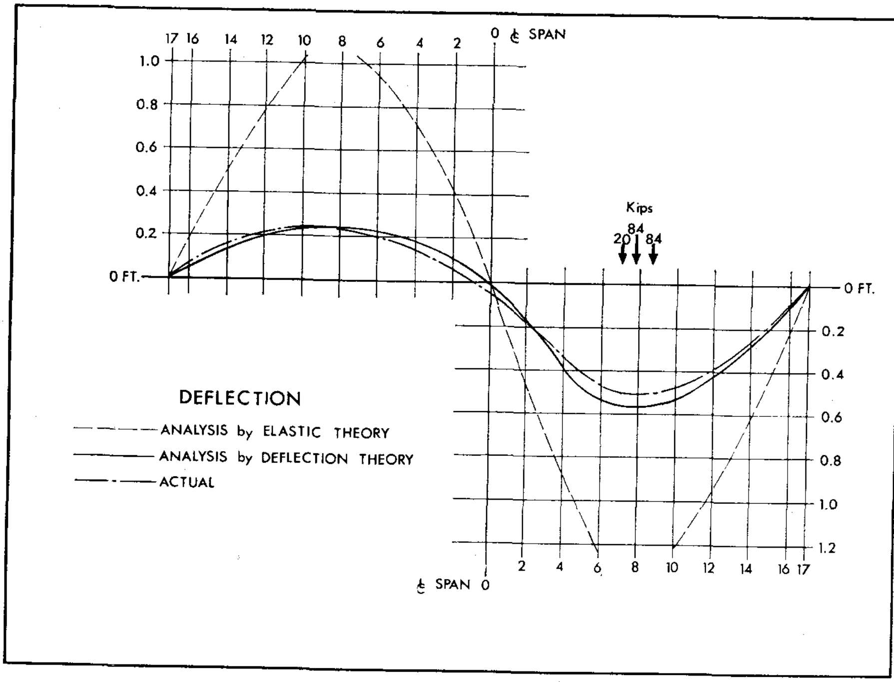

Fig. 17—94 Ton Test Load—Stiffening Girder Deflections

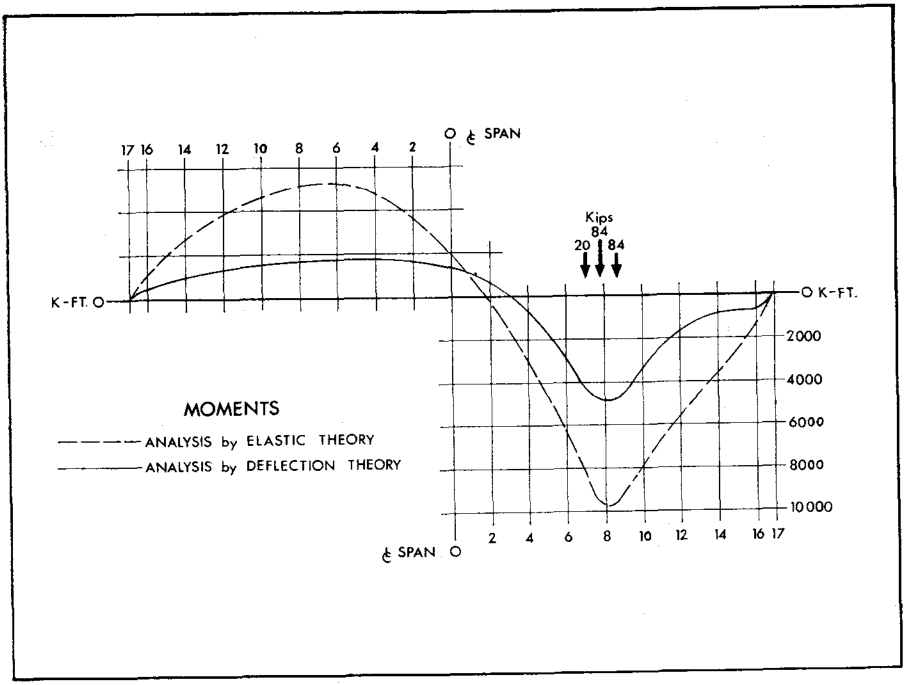

Fig. 18—94 Ton Test Load—Stiffening Girder Moments

trailer carrying generators to the Portage Mountain Dam. The results of the analysis indicated that this bridge can safely carry this load, whereas other two lane structures, designed for H25-S20 loading, would be subject to overstress from the 146 ton load.

CONCLUSIONS

The concrete box girder suspension bridge provided an economical and aesthetically pleasing solution to the problem of crossing the Peace River in the remote northern British Columbia location. The relatively short construction time of 10 months was further proof of the feasibility of the structure. The use of reinforced and prestressed concrete in a span of 680 feet confirms that concrete is able to compete in the relatively long span field which, to date, has been monopolized by structural steel.

ACKNOWLEDGEMENT

The author wishes to express his appreciation to the following personnel who took part in the development and completion of this project.

Messrs. H. J. Barratt and L. Osipov, partners in the firm, provided guidance and assistance during the course of the project.

Col. H. H. Minshall first projected the concept of a segmental unit concrete suspension bridge. Col. Minshall currently holds a Canadian patent on this concept.

Hon. P. A. Gaglardi, Minister of Highways of British Columbia, and Mr. J. Alton, Bridge Engineer for the Department of Highways, gave this project every encouragement.

Mr. Hans Mordhorst, the General Contractor, constructed the entire bridge.

Dr. R. F. Hooley, Professor of Civil Engineering, University of

British Columbia, assisted the contractor on all design problems pertaining to the erection system.

Mr. A. Hicks was the Resident Engineer for Phillips, Barratt and Partners during construction.

REFERENCES

1. Steinman, D. B., "Deflection Theory for Suspension Bridges", *Transactions A.S.C.B.* Volume 100, 1935.
2. Hardesty, S. and Wessman, H. E., "Preliminary Design of Suspension Bridges", *Transactions A.S.C.E.*, Volume 104, 1939.
3. Steinman, D. B., "Rigidity and Aerodynamic Stability of Suspension Bridges", *Transactions A.S.C.E.*, Volume 110, 1945.
4. Guyon, Y., *Prestressed Concrete*, Contractors Record Ltd., London.
5. Van De Pitte, Daniel, "Prestressed Concrete Suspension Bridges", *Proceedings on World Conference of Prestressed Concrete*, 1957.

Presented at the Eleventh Annual Convention of the Prestressed Concrete Institute, Miami Beach, Florida, December 1965.

Discussion of this paper is invited. Please forward your discussion to PCI Headquarters before June 1 to permit publication in the October 1966 issue of the PCI JOURNAL.