Composite Cable-stayed Bridges
The Concept with the Competitive Edge

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Recent tenders for three major cable-stayed bridges, Annacis Bridge in Vancouver at 1,525-ft span, Houston Ship Channel Bridge in Texas at 1,250-ft span and the Dartford Crossing in London at 1,476-ft span, have shown that in this span range composite construction can have a significant price advantage over other alternatives. Typically, these bridges (Figs. 1, 2) comprise shallow steel plate-girder cross sections with steel floor beams and a composite precast deck.

This paper describes the evolution of the composite cable-stayed bridge, examines the reasons which currently make it economical and looks forward to developments anticipated in the near future.

ITS EVOLUTION

Although the first modern cable-stayed bridge was built at Stromsund in Sweden, the initial evolution occurred almost exclusively in Germany. These early bridges had widely spaced cables which produced hard supports and resulted in bending moment diagrams similar to those for continuous beams on discrete piers. The first bridges of this type had two cable planes and plate girders or twin box girders. But as spans increased and aerodynamic stability became a concern, a single-cable plane on centerline with a torsion box became popular. At this stage of development, girder depths were typically about span/100, or more.

Closely spaced cables were introduced in the Bonn Bridge (1966). This system provides support to the girder analogous to a beam on elastic foundation, and the bending moment peaks of the discrete cable system are eliminated. The implications of this reduction in bending moments were not realized at first and girder depths continued to be proportioned at about span/100 until quite recently. Figure 3 compares the dead-load bending moments, plotted to the same scale, for a steel box girder with orthotropic deck supported by discrete cables and a composite girder supported by a distributed wall of cables. Note the reduction in girder depth to span ratio.

The steel industry is a dominant force in Germany and the vast majority of cable-stayed bridges built there are constructed entirely from steel. Orthotropic steel decks and steel towers are used almost invariably. The demand for new cable-stayed bridges in Germany decreased in the 1970s, but the need elsewhere in the world permitted designers in other countries to contribute to the evolution of this type of bridge. Japan, which also has a dominant steel

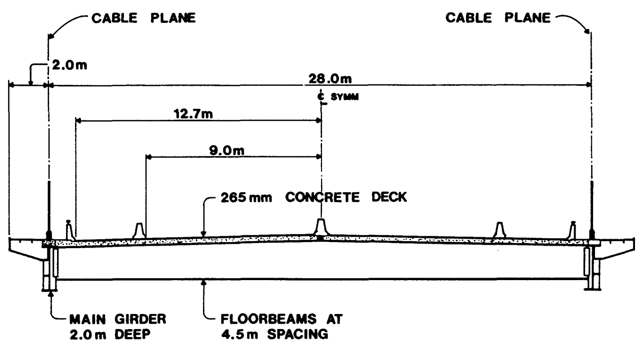

Fig. 1. Cross section—Annacis Bridge

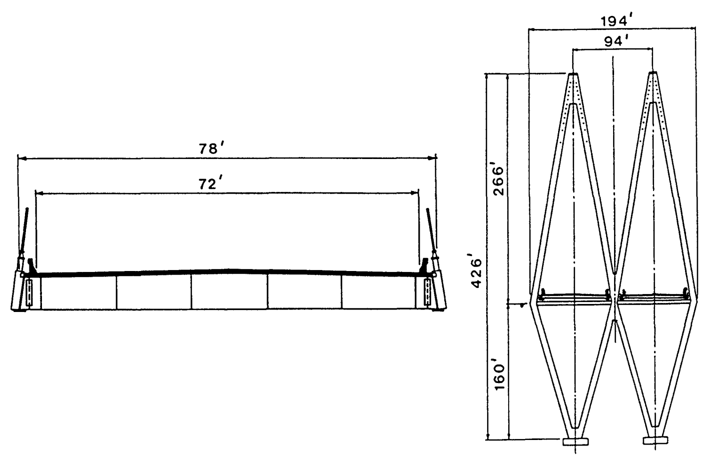

Fig. 2. Cross section—Houston Ship Channel Bridge

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industry, has embraced the cable-stayed concept with enthusiasm, has built, and is continuing to build a large number of them with spans up to 1,500 ft with, in some cases, two-tier deck systems. As in Germany, these bridges in Japan are almost exclusively constructed in steel.

St. Nazaire Bridge in France continued the German tradition of steel boxes with orthotropic decks. But Brotonne Bridge, in the same country, established an entirely new concrete box configuration with sloping webs and supported by a closely spaced wall of cables on bridge centerline. This concept was transplanted successfully to the Sunshine Skyway in Florida. Together with Dame Point and Pasco-Kennewick Bridge it established an all concrete precedent for major North American cable-stayed spans. This trend was reversed dramatically with the tender opening for Annacis Bridge, where alternative designs had been prepared for an all-concrete design and a steel-concrete composite design for the longest span (1,520 ft) cable-stayed bridge in the world. Not only was the bid price for the composite design 18% lower than for the all-concrete design, but also at US$32 million the bid translated into a unit price of US$119/sq. ft, exceptionally low for a bridge of this span in 1984.

The competitive economy of the composite system was confirmed recently in bidding for the 1,476-ft span Dartford Crossing in London and the 1,250-ft span Houston Ship Channel Bridge in Texas where, once again, competition was an all-concrete alternative.

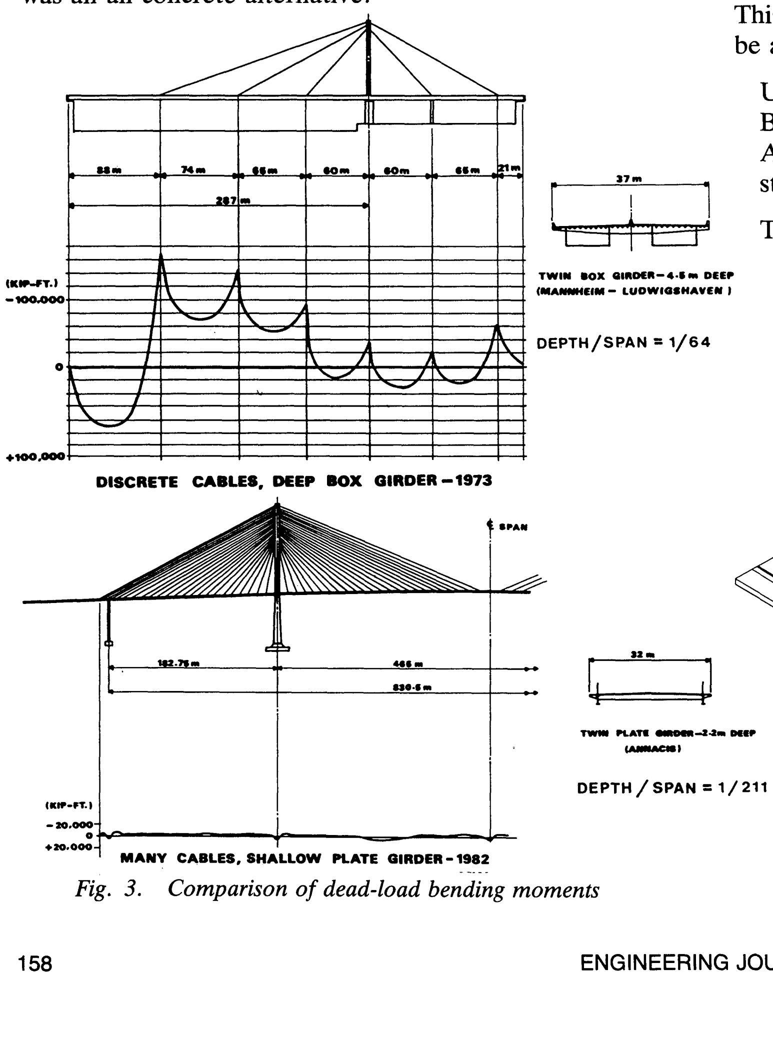

Fig. 3. Comparison of dead-load bending moments

REASONS FOR COMPOSITE ECONOMY

The reasons for the economy of composite alternatives lie in their ease of construction. The cross section in each case is a simple assemblage of high-strength modular elements, of appropriate material, all performing multiple functions—yet connected without difficult tolerance or match-casting demands. The floor-beam connections are simple web-plate splices. Integral composite action of the deck, girders and floor beams is achieved by pouring concrete around lapping rebar and shear connectors in the strips between the precast panels and above the structural steel. Figure 4 is a layered view of a typical composite deck structure, comprising high-strength precast deck slabs topped with a concrete overlay and acting compositely with steel girders and floor beams. Long production runs of repetitive modular elements ensure economical subcontractor prices for components manufactured off site and ease of assembly ensures erection speed and less contingency for the general contractor on site.

DESIGN CONCEPTS

To utilize the economies possible in a simple composite deck system, there are certain broad overall design concepts which must be addressed first. As mentioned earlier, close spacing of the cables is necessary if girder bending moments are reduced to permit a shallow girder. A shallow girder is also the key to the critical aerodynamic stability. This is a complex topic, but for a cable-stayed bridge it can be achieved by some or all of the following:

Use of torsionally stiff box girders
By using A-frame towers, or
Aerodynamic shaping of the profile of the suspended structure.

These alternatives are listed in descending order of cost.

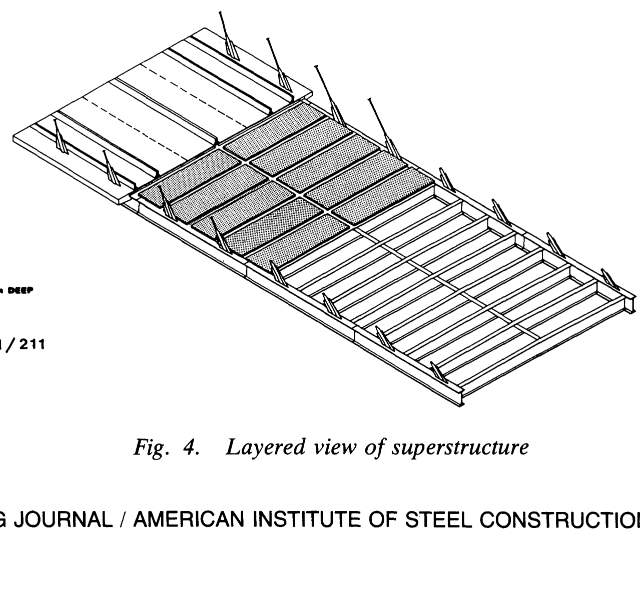

Fig. 4. Layered view of superstructure

The ideal situation is to avoid expensive box girders and expensive A-frame towers and to achieve aerodynamic stability with the most efficient structural cross section. This is not always possible.

Other design concept aspects which affect the overall economy of a slim, composite-deck system include the bearing configuration chosen at the towers and the backstay layout along the deck in the region of tie-down piers. The objective in both of these areas should be to avoid unnecessary bending moment peaks which will overload the slender girder.

DESIGN DETAILS

Cables and Cable Connections

The configuration of the cables and their connections to the deck and towers are significant factors in the economy of cable-stayed bridges. We have seen earlier that a closely spaced wall of cables leads to a reduction in girder bending moments and therefore economies in girder material. But this is negated if there is a large premium attached to the multiplicity of cable connections. These obviously must be kept as simple as possible and as direct as possible.

A typical cable connection for Houston Ship Channel Bridge is shown in Fig. 5, where the cable is anchored to a weldment on the side of the web. The slight eccentricity with respect to girder centerline in order to bypass the top flange of the girder introduces some secondary bending moments. Also installation of the cable's lower end requires some access platforms during bridge erection.

The simpler and more direct lower cable connection used at Annacis Bridge is shown in Fig. 6. This arrangement has easy access and does not introduce secondary moments. But welding the cable gusset to the top flange does introduce a biaxial stress state in that zone of the top flange. This is discussed in the section on fabrication.

For simplicity of geometry it is preferable to keep the cables in a vertical plane, while allowing the girders to pass through the tower. A slightly cranked tower leg (Fig. 7) achieves this without any significant premium. For the cables themselves, a number of systems are available which possess the required strength, fatigue resistance and corrosion resistance to perform adequately. Experience indicates it is desirable to be able to inspect both ends of the cables and to replace any one easily.

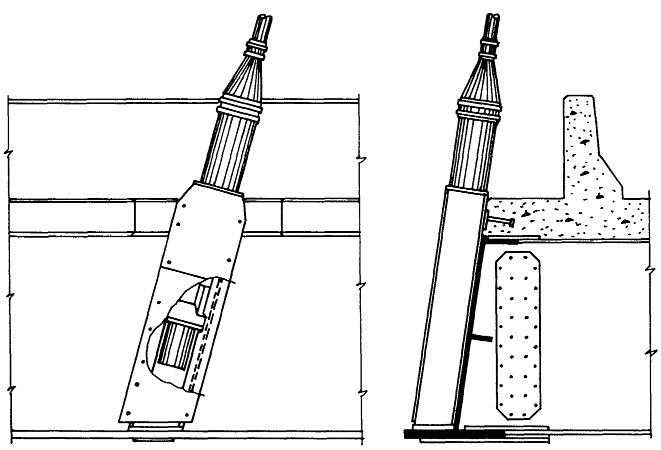

Fig. 5. Typical stay connection, Houston Ship Channel Bridge

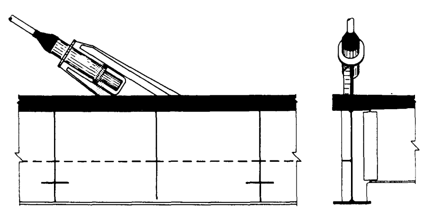

Fig. 6. Lower cable connection—Annacis Bridge

Girders and Floor Beams

The composite concrete deck provides a top flange for both the floor beams and girders. The steel top flanges primarily provide stability during erection, shear connection and support for the cast-in-place infill strips and as such are sized on a *b/t* ratio rather than by stress. Plate for the bottom flanges is much thicker to provide girder bending stiffness and strength. However, the variations in girder bottom flange thickness are not large because bending moment peaks are eliminated by the support from close spaced cables. Regardless of dead- and live-load bending moments, a minimum girder stiffness is necessary to prevent girder buckling under partial live load.

Girder design is not well covered by most current codes for conditions of combined axial load, bending and shear. The British Code BS5400 appears to be the only bridge code which addresses this situation fully for composite girders. Because girder axial stresses are high, particularly in the regions close to the towers, the effect of girder initial imperfections such as out-of-flat web panels requires consideration in both design and fabrication.

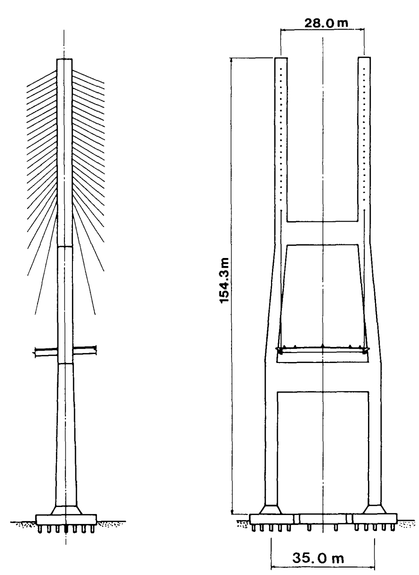

Fig. 7. Tower elevation

Provided the local environment is suitable, weathering steel would be an appropriate material for the structural steel elements. Critical components such as cable attachments should receive a high-grade protective coating.

Concrete Deck

The precast deck panels and poured-in-place infill strips together form a wide, stiff deck diaphragm which distributes lateral and axial loads well and participates in girder and floor beam bending. The deck is effectively post-tensioned in most areas by the horizontal component of the stay-cable loads, therefore deck cracking is not a problem after the initial erection phase. However, creep and shrinkage effects are of concern in the long term. These were analyzed carefully by a time history program during the design of Annacis Bridge, with these conclusions:

1. Shrinkage effects could be minimized by storing the deck panels for 60 days prior to erection.
2. Creep calculations should be based on long term tests of actual job mixes. These tests should be started as early as possible.
3. Creep effects must be included in bridge erection geometry calculations if good agreement with design geometry is to be achieved.
4. Variations of actual creep coefficient between 50% and 200% of assumed can be accommodated without significant axial load redistribution between the steel girder top flange and composite deck.

Careful detailing is necessary at the composite deck-structural steel interface (Fig. 8) to accommodate the necessary overlap of protruding rebar from the deck between shear studs on the steel flange. The steel bearing surface supporting the edge of the precast panel can be

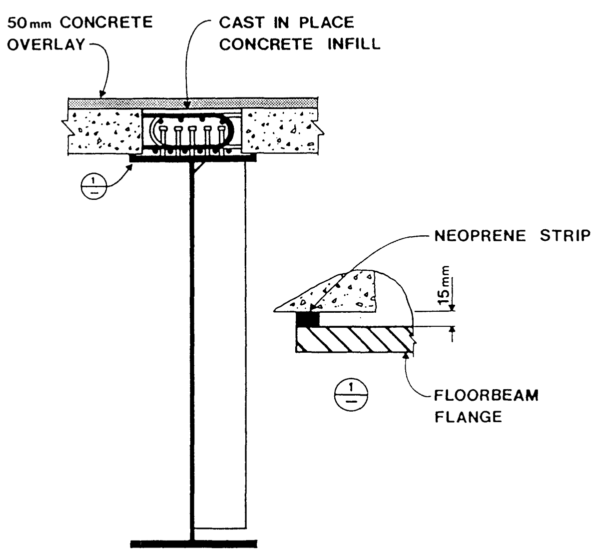

Fig. 8. Floor beam—precast deck joint

sealed against concrete leaks by a resilient strip. Epoxy-coated rebar should be used to prolong the deck life.

Shear lag in the deck can be critical in areas of rapidly changing bending moment and axial load such as the tie-down zone and inboard of erection equipment making heavy lifts.

FABRICATION

The basic structural steel elements of a composite cable-stayed bridge are three plate girder sections and three plate floor beams. These elements are among the simplest shapes to fabricate. If the principles of modular design have been followed rigorously, then long production runs of identical elements should result.

The fabrication of these elements is routine except for two aspects, overall geometric control and quality control in the cable attachment weldments, where particular care is necessary. Geometry control must be pursued in all elements of the bridge, but particular emphasis has to be paid to the main girder geometry because of the dominant role it plays in the final geometry and cable loadings in the suspended structure.

Girders may be detailed without camber and with a square cut at one end. Design vertical geometry can then be achieved by bevel trimming the other end of the girder and by drilling one side of the web and top flange splices from the solid, using the splice plates as templates, in a running, three-girder shop assembly set to the required vertical curve. With allowance for weld shrinkage, girder flanges may be drilled prior to assembly.

Running longitudinal dimensions in the girder assemblies may be maintained within tolerance by adjusting the length of specific girders to compensate for cumulative growth or shrinkage. The tighter tolerance for the longitudinal location of cable connection work points may be achieved by delaying the layout of the attachments until the girders are located in the trial assembly.

Because of the close spacing of the floor beams, extreme camber variations in successive floor beams could give rise to an unacceptable vehicle ride on the deck. Instead of specifying an onerous camber tolerance, floor beams may be categorized, by measurement after fabrication, into three camber categories, "plus," "nominal" and "minus." The erector is then required to ensure that adjacent floor beams in the bridge are of the same or adjacent camber categories.

The second aspect of fabrication requiring particular care is quality control in fabrication of the cable attachment weldments. Of particular concern would be details similar to Fig. 6, where there is a biaxial tension zone in the top flange immediately beneath the cable attachment gusset. Although this type of connection is not common in bridges, it is used extensively in gusset-to-can connections in offshore oil rigs, which are subject to severe fatigue and environmental loads. However, it is very important that the detail be sized so fatigue axial and bending stresses are very low and appropriate quality control effort is invested in the material and welding. The weld is not difficult to make, and provided the flange plate is free of laminations and is ductile in the through thickness direction, then careful welding supervision and N.D.T. inspection assure a defect-free detail. ASTM A770 covers the basic requirements for supply and testing of this class of plate, although a higher level of ultrasonic inspection in the weld zone may be desirable.

SUPERSTRUCTURE ERECTION

The design concepts and details for constructability mentioned earlier prove their value in the erection phase of a composite cable-stayed bridge. The system brings together the best of both concrete and structural steelwork. The latter provides a light, stiff, easily erected structural spine which supports the precast elements of the deck. With careful detailing—for example, see Fig. 8 showing the deck to floor beam connection—the two elements can be bonded together into composite action with poured-in-place infill strips requiring no formwork. The floor beams act compositely in bending under axle loads and the girders act compositely under lane loading and environmental loads. The high-strength, composite deck provides a shear stiff diaphragm for lateral loads and a large capacity for axial thrust from the cables. In fact, a basic precast deck of about 8-in. thickness and 8,000-psi strength can carry the axial thrust from a cable-stayed span up to about 1,500 ft without additional material.

The shop-assembled structural steel girders, together with carefully measured cables, automatically provide overall geometry control for the bridge; while the relatively forgiving splices of the precast deck elements permit fast infill behind the structural steel. If careful geometry control in all elements is followed through girder and cable fabrication and tower construction, then the bridge may be erected to geometry entirely and cable retuning eliminated.

Balanced cantilever is usually the simplest erection method for a major cable-stayed bridge, particularly in a high level crossing. Careful planning and coordination is necessary to maintain stability in such a sensitive structure (Fig. 11). External restraints against vertical movements of one cantilever and against "weathervaning" of the deck about the tower are usually necessary, for example, see Fig. 9. Aerodynamic stability in the temporary condition also should not be overlooked. The partly erected bridge has changing dynamic properties as the cantilevers are extended. In general, the structure is much less stiff than in its completed state, and violent instabilities can occur at low windspeeds.

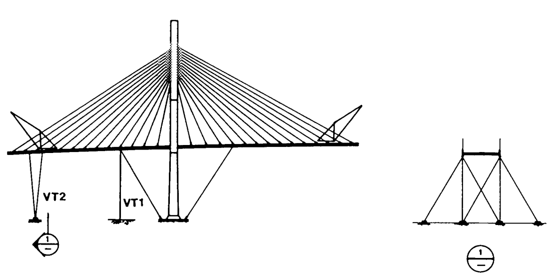

Fig. 9. Temporary restraints during erection

MONITORING OF ERECTION STABILITY

Due to the sensitivity of the structure to out-of-balance loads during construction, special efforts were made by the site engineering staff at Annacis Bridge to monitor independently bridge stability at every stage. Stability was monitored both by on-site computer analysis of the current erection situation and by observing load cell readings in the deck restraint at the tower and in the tie-down cables. A computer analysis was carried out routinely at the end of each full superstructure panel erection cycle prior to moving the derrick forward. The following data was collected:

1. A definition of the bridge structure
2. An inventory of deck superimposed loads and their locations
3. A survey of deck elevations and tower displacements
4. Temperatures on sun and shade faces of the tower
5. Load cell readings in restraining members.

The data collection took about one hour. The bridge structural elements and no-load geometry were already stored in the computer library, so it was only necessary to input the deck load magnitudes and their distance from a reference line. The on-site computer then plotted the deflected shape of the superstructure and towers, the tower and girder bending moments and the forces in the restraining members (Fig. 10).

Examination of the girder and tower bending moments vs. section capacities gave a measure of available load factor in the permanent structural elements at any stage. And comparison of calculated and measured restraining forces gave a check on load factors available in the temporary restraints. Comparison of calculated and measured geometry gave early indication of any deviations from theoretical geometry.

This on-site monitoring by computer was a very powerful check on the changing effects of the contractor's erection scheme on the permanent structure. Also, it permitted a good assessment of overall safety at each stage, including any deviations from planned procedures, while giving invaluable feedback on final geometry tolerance in this complicated structure.

FUTURE DEVELOPMENTS

We can anticipate gradual extension of the competitive span range for cable-stayed bridges up to 2,000 ft and higher, because the cost vs. span relationship does not show a steep increase in that range. Cable-stayed bridges will be particularly advantageous where the soil conditions are poor and suspension bridge anchorages are difficult and costly. High level bridges also give cable-stayed bridges advantages over suspension bridges because the latter must carry their main cables to ground level.

A conventional 8-in. thick reinforced or precast concrete deck of 8,000 psi strength can carry the axial dead- and live-load thrusts of a composite cable-stayed bridge up to spans of about 1,500 ft. Thus, there is no premium material required to resist axial load up to that span. The use of silica fume additives makes concrete strengths of 12,000 psi and higher attainable and will permit much larger composite cable-stayed spans than 1,500 ft with zero material premium to resist axial load.

Recent aeroelastic wind tunnel tests of a number of bridge superstructure cross sections have given us a better understanding of the parameters governing aeroelastic stability.[1] It is now evident the benefits of increased torsional stiffness are not large above a W_t/W_v ratio of about 1.6. Formerly, efforts were made to increase this ratio to 2.5 or greater, requiring large and costly torsion boxes in the superstructure. Modern wind tunnel technology permits us to use the facility as a design tool to achieve the necessary low speed aerodynamic stability and high-speed critical velocity at the minimum structural premium. Great economies can be achieved by this approach.

Another future development concerns a much more serious evaluation of the damping existing in the structure and ways this can be augmented externally. Damping is very significant in reducing amplitudes of aerodynamic and other sources of vibration and in raising the aerodynamic critical velocity. However, structural damping values in a typical composite cable-stayed bridge are very low, about 0.3% to 0.4% of critical. System damping cannot be relied on to increase these values and discrete cable and superstructure dampers are sometimes necessary for aerodynamic stability.

While it is evident improved technology can extend the economical span range of cable-stayed bridges, what are the consequent problems? The major problems are associated with the erection of such large spans and concern the static and aerodynamic stability of the partly erected bridge.

To extract the maximum economy from the design, it is

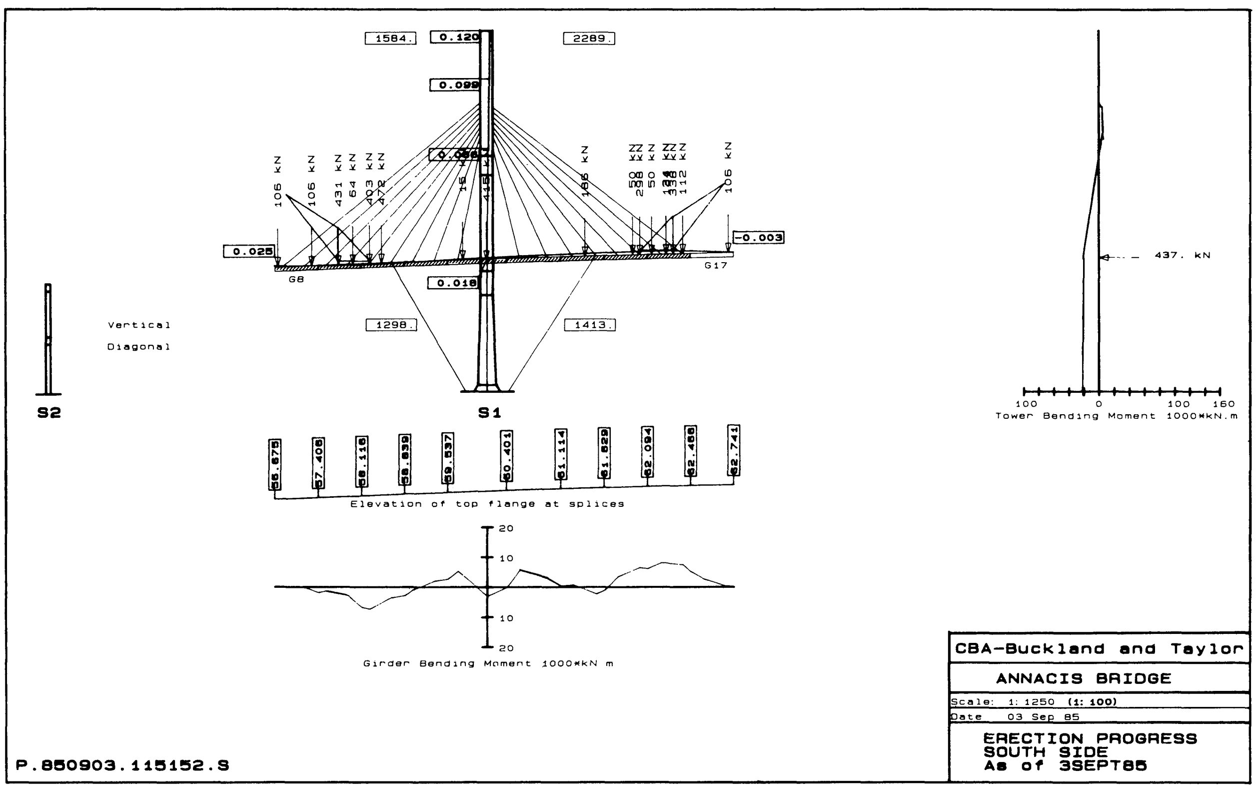

Fig. 10. Computer plot of balance analysis

usually necessary to erect a major cable-stayed bridge by the balanced cantilever method. As spans get longer, this method becomes sensitive to out-of-balance dead load and, more particularly, to out-of-balance wind load, both laterally and vertically. From the results of boundary layer, aeroelastic wind tunnel tests which have been conducted on models of partially erected cable-stayed bridges, it may be concluded that gust pressures in the side span and main span can produce dynamic responses equivalent to eccentric static vertical and horizontal pressures of the order of 20 lb/sq ft and 40 lb/sq ft respectively acting on the superstructure *on one side of the tower only*.

Because of the large projected areas of superstructure surface, the bending moments and torques resulting from these effective pressures are very large indeed. Recognizing the magnitude of these forces and designing effective temporary restraints to resist them is one of the problems of efficient long-span cable-stayed bridge erection.

The aerodynamic stability of the partly erected bridge is also of concern. This aspect of construction is frequently neglected, but as spans get longer it becomes more critical, particularly for narrow bridges. A thorough study of the stability of the section, the dynamic properties of the various erection stages and the wind climate at the site is necessary to ensure aerodynamic safety at all stages of construction.

CONCLUSIONS

The economical advantage currently enjoyed by composite cable-stayed bridges depends upon their ease of construction using simple, high-strength modular components which exploit the respective advantages of structural steel and precast concrete in the girder and deck. Careful attention in design to these factors will maintain the current economic advantages. Material developments in high-strength precast concrete promise to extend the economical span range of composite cable-stayed bridges.

Static and aerodynamic limitations during balanced cantilever erection require careful consideration as spans increase.

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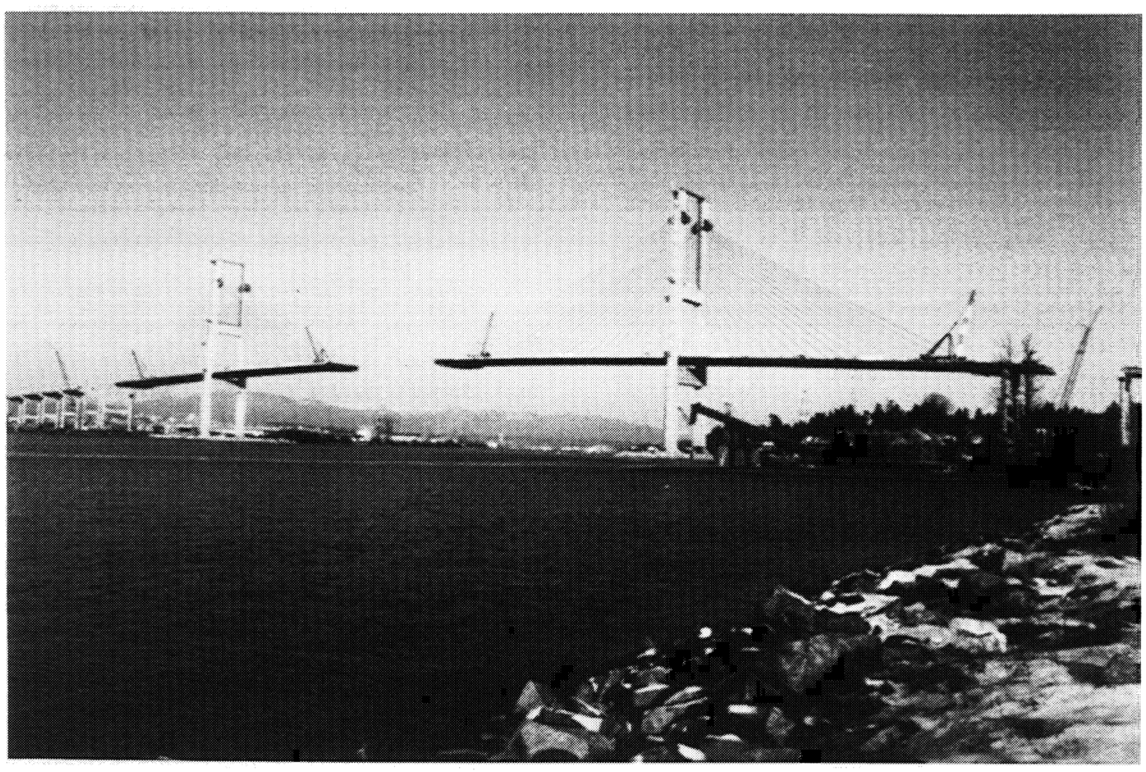

Fig. 11. Annacis Bridge during construction