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19

Cable-Stayed Bridges

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19.1 Introduction

Since the completion of the Stromsund Bridge in Sweden in 1955, the cable-stayed bridge has evolved into the most popular bridge type for long-span bridges. The variety of forms and shapes of cable-stayed bridges intrigues even the most-demanding architects as well as common citizens. Engineers found them technically innovative and challenging. For spans up to about 1000 m, cable-stayed bridges are more economical.

The concept of a cable-stayed bridge is simple. A bridge carries mainly vertical loads acting on the girder, Figure 19.1. The stay cables provide intermediate supports for the girder so that it can span a long distance. The basic structural form of a cable-stayed bridge is a series of overlapping triangles comprising the pylon, or the tower, the cables, and the girder. All these members are under predominantly axial forces, with the cables under tension and both the pylon and the girder under compression. Axially loaded members are generally more efficient than flexural members. This contributes to the economy of a cable-stayed bridge.

At the last count, there are about 600 cable-stayed bridges in the world and the number is increasing rapidly. The span length has also increased significantly [2,7].

Some milestones: the Stromsund Bridge in Sweden, completed in 1955 with a main span of 183 m is usually recognized as the world's first major cable-stayed bridge; the Knie Bridge (320 m) and Neuenkamp Bridge (350 m) in Germany, Figure 19.2, were the longest spans in the early 1970s, until the Annacis Island–Alex Fraser Bridge (465 m) was completed in the mid 1980s. The 602-m-span Yangpu Bridge was a large step forward in 1994 but was surpassed within about half a year by the Normandie Bridge (856 m), Figure 19.3. The Tatara Bridge, with a center span of 890 m, is the world record today. Several spans in the range of 600 m are under construction. Longer spans are being planned.

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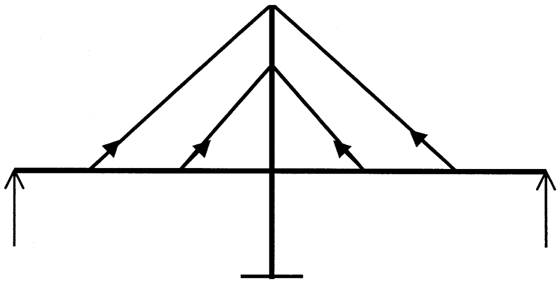

FIGURE 19.1 Concept of a cable-stayed bridge.

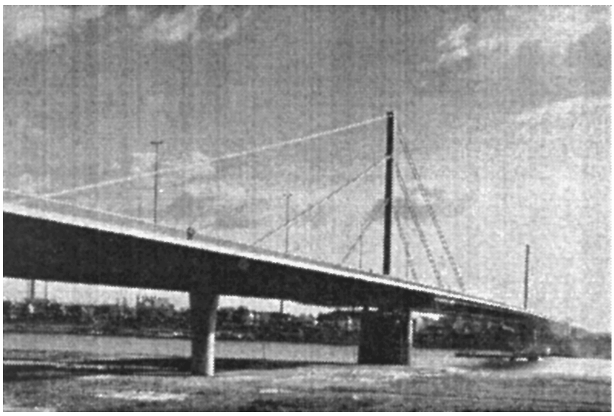

FIGURE 19.2 Neuenkamp Bridge.

FIGURE 19.3 Normandie Bridge.

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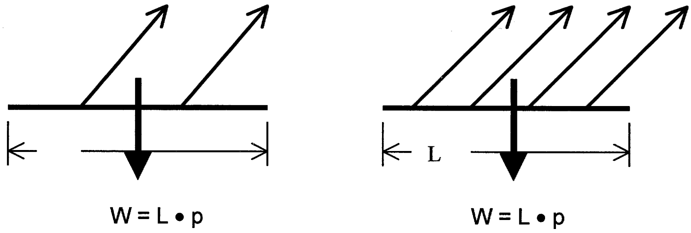

FIGURE 19.4 Cable forces in relation to load on girder.

19.2 Configuration

19.2.1 General Layout

At the early stage, the idea of a cable-stayed bridge was to use cable suspension to replace the piers as intermediate supports for the girder so that it could span a longer distance. Therefore, early cable-stayed bridges placed cables far apart from each other based on the maximum strength of the girder. This resulted in rather stiff girders that had to span the large spacing between cables, in addition to resisting the global forces.

The behavior of a cable-stayed girder can be approximately simulated by an elastically supported girder. The bending moment in the girder under a specific load can be thought of as consisting of a local component and a global component. The local bending moment between the cables is proportional to the square of the spacing. The global bending moment of an elastically supported girder is approximately [5]

$$M = a * p * \sqrt{(I/k)} \tag{19.1}$$

where a is a coefficient depending on the type of load p, I is the moment of inertia of the girder, and k is the elastic support constant derived from the cable stiffness. The global moment decreases as the stiffness of girder, I, decreases.

Considering that the function of the cables is to carry the loads on the bridge girder, which remains the same, the total quantity of cables required for a bridge is practically the same independent of the number of cables, or cable spacing, Figure 19.4. But if the cable spacing is smaller, the local bending moment of the girder between the cables is also smaller. A reduction of the local bending moment allows the girder to be more flexible. A more flexible girder attracts in turn less global moment. Consequently, a very flexible girder can be used with closely spaced cables in many modern cable-stayed bridges. The Talmadge Bridge, Savannah, Figure 19.5, is 1.45 m deep for a 335 m span, The ALRT Skytrain Bridge, Vancouver, Figure 19.6, is 1.1 m deep for a 340 m span and the design of the Portsmouth Bridge had a 84-cm-deep girder for a span of 286 m.

Because the girder is very flexible, questions concerning buckling stability occasionally arose at the beginning. However, as formulated by Tang [4], Eq. (19.2), using the energy method,

$$P(cr) = \left\{ \int EIw''^2 ds + \sum EC * Ac * Lc \right\} \Big/ \left[\int (Ps/Pc) w'^2 ds \right] \tag{19.2}$$

where E is modulus of elasticity, I is moment of inertia, A is area, L is length, w is deflection, and $(\ )'$ is derivative with respect to length s. The buckling load depends more on the stiffness of the cables than on the stiffness of the girder. Theoretically, even if the stiffness of the girder is neglected, a cable-stayed bridge can still be stable in most cases. Experience shows that even for the most flexible girder, the critical load against elastic buckling is well over 400% of the actual loads of the bridge.

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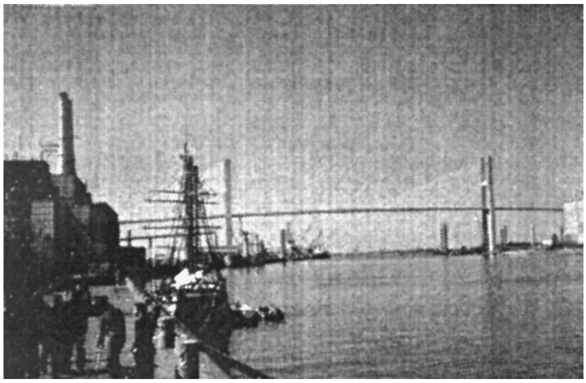

FIGURE 19.5 Talmadge Bridge.

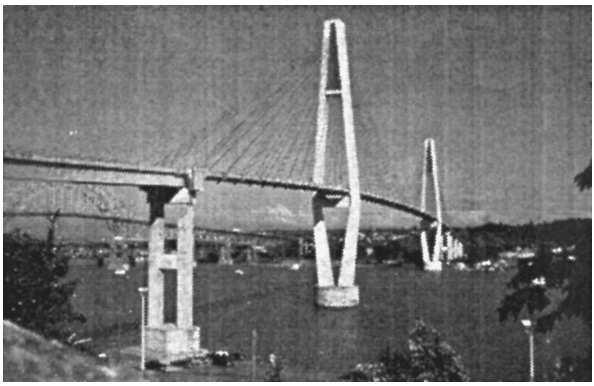

FIGURE 19.6 ALRT Skytrain Bridge.

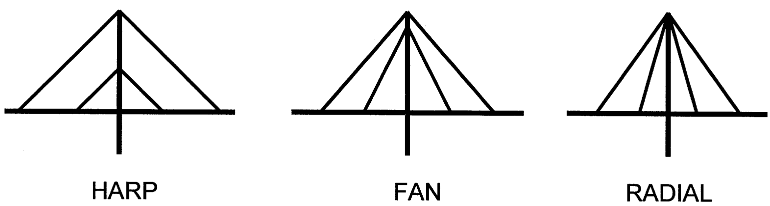

FIGURE 19.7 Cable arrangements.

The recently adopted design requirement that all cables be individually replaceable makes closely spaced cables more desirable. It is usually required that one cable can be detensioned, dismantled, and replaced under reduced traffic loading. The additional bending moment in the girder will not increase excessively if the cable spacing is small.

Availability of ever more powerful computers also helps. The complexity of the analysis increases as the number of cables increases. The computer offers engineers the best tool to deal with this problem.

Harp, radial, fan, Figure 19.7, or other cable configurations have all been used. However, except in very long span structures, cable configuration does not have a major effect on the behavior of the bridge.

A harp-type cable arrangement offers a very clean and delicate appearance because an array of parallel cables will always appear parallel irrespective of the viewing angle. It also allows an earlier

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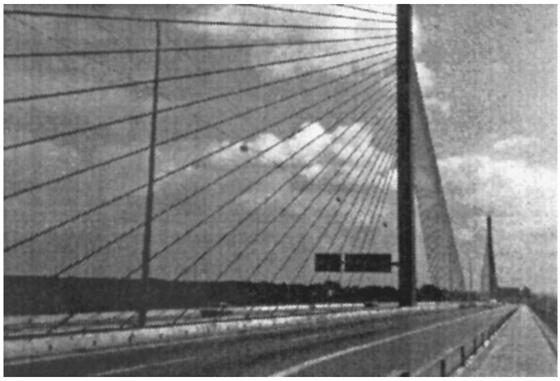

FIGURE 19.8 Nord Bridge.

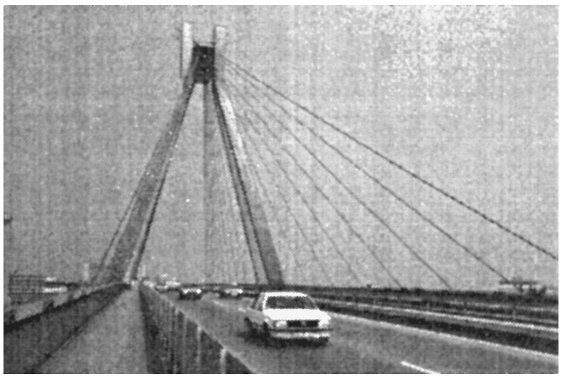

FIGURE 19.9 Ludwighafen Bridge.

start of girder construction because the cable anchorages in the tower begin at a lower elevation. The Hoechst Bridge and the Dames Point Bridge are examples that fully utilized this advantage.

A fan-type cable arrangement can also be very attractive, especially for a single-plane cable system. Because the cable slopes are steeper, the axial force in the girder, which is an accumulation of all horizontal components of cable forces, is smaller. This feature is advantageous for longer-span bridges where compression in the girder may control the design. The Nord Bridge, Bonn, Figure 19.8, is one of the first of this type.

A radial arrangement of cables with all cables anchored at a common point at the tower is quite efficient. However, a good detail is difficult to achieve. Unless it is well treated, it may look clumsy. The Ludwighafen Bridge, Germany, Figure 19.9, is a successful example. The Yelcho Bridge, Chile, with all cables anchored in a horizontal plane in the tower top, is an excellent solution, both technically and aesthetically.

When the Stromsund Bridge was designed, long-span bridges were the domain of steel construction. Therefore, most early cable-stayed bridges were steel structures. They retained noticeable features from other types of long-span steel bridges.

In the 1960s, Morandi designed and built several relatively long span concrete cable-stayed bridges. His designs usually had few cables in a span with additional strut supports at the towers for the girder. They did not fully utilize the advantages of a cable-stayed system. The concrete cable-stayed bridge in its modern form started with the Hoechst Bridge in Germany, followed by the Brotone Bridge in France and the Dames Point Bridge in the United States, each representing a significant advance in the state of the art.

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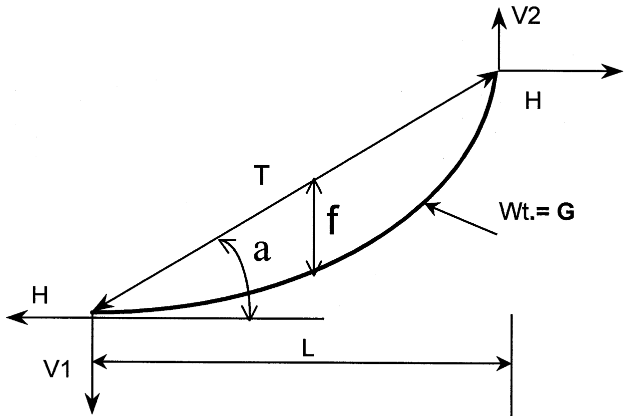

FIGURE 19.10 An inclined cable.

19.2.2 Cables

Cables are the most important elements of a cable-stayed bridge. They carry the load of the girder and transfer it to the tower and the back-stay cable anchorage.

The cables in a cable-stayed bridge are all inclined, Figure 19.10. The actual stiffness of an inclined cable varies with the inclination angle, *a*, the total cable weight, *G*, and the cable tension force, *T* [3]:

$$EA(\text{eff}) = EA \Big/ \left\{1 + G^2\ EA \cos^2 a \Big/ \left(12\ T^3\right)\right\} \tag{19.3}$$

where *E* and *A* are Young's modulus and the cross-sectional area of the cable. And if the cable tension *T* changes from *T*1 to *T*2, the equivalent cable stiffness will be

$$EA(\text{eff}) = EA \Big/ \left\{1 + G^2\ EA \cos^2 a\,(T1 + T2) \Big/ \left(24\ T1^2\ T2^2\right)\right\} \tag{19.4}$$

In most cases, the cables are tensioned to about 40% of their ultimate strength under permanent load condition. Under this kind of tension, the effective cable stiffness approaches the actual values, except for very long cables. However, the tension in the cables may be quite low during some construction stages so that their effectiveness must be properly considered.

A safety factor of 2.2 is usually recommended for cables. This results in an allowable stress of 45% of the guaranteed ultimate tensile strength (GUTS) under dead and live loads [9]. It is prudent to note that the allowable stress of a cable must consider many factors, the most important being the strength of the anchorage assemblage that is the weakest point in a cable with respect to capacity and fatigue behavior.

There have been significant developments in the stay cable system. Early cables were mainly lock-coil strands. At that time, the lock-coil strand was the only cable system available that could meet the more stringent requirements of cable-stayed bridges.

Over the years, many new cable systems have been successfully used. Parallel wire cables with Hi-Am sockets were first employed in 1969 on the Schumacher Bridge in Mannheim, Germany. Since then, the fabrication technique has been improved and this type cable is still one of the best cables commercially available today. A Hi-Am socket has a conical steel shell. The wires are parallel for the entire length of the cable. Each wire is anchored to a plate at the end of the socket by a button head. The space in the socket is then filled with epoxy mixed with zinc and small steel balls.

The Hi-Am parallel wire cables are prefabricated to exact length in the yard and transported to the site in coils. Because the wires are parallel and therefore all of equal length, the cable may

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sometimes experience difficulty in coiling. This difficulty can be overcome by twisting the cable during the coiling process. To avoid this problem altogether, the cables can be fabricated with a long lay. However, the long lay may cause a very short cable to twist during stressing.

Threadbar tendons were used for some stay cables. The first one was for the Hoechst Bridge over the Main River in Germany. The Penang Bridge and the Dames Point Bridge also have bar cables. They all have a steel pipe with cement grout as corrosion protection. Their performance has been excellent.

The most popular type of cable nowadays uses seven-wire strands. These strands, originally developed for prestressed concrete applications, offer good workability and economy. They can either be shop-fabricated or site-fabricated. In most cases, corrosion protection is provided by a high-density polyethylene pipe filled with cement grout. The technique of installation has progressed to a point where a pair of cables can be erected at the site in 1 day.

In search of better corrosion protection, especially during the construction stage before the cables have been grouted, various alternatives, such as epoxy coating, galvanization, wax and grease have all been proposed and used. Proper coating of strands must completely fill the voids between the wires with corrosion inhibitor. This requires the wires to be loosened before the coating process takes place and then retwisted into the strand configuration.

In addition to epoxy, grease, or galvanization, the strands may be individually sheathed. A sheathed galvanized strand may have wax or grease inside the sheathing. All three types of additional protection appear to be acceptable and should perform well. However, a long-term performance record is not yet available.

The most important element in a cable is the anchorage. In this respect, the Hi-Am socket has an excellent performance record. Strand cables with bonded sockets, similar to the Hi-Am socket, have also performed very well. In a recently introduced unbonded anchorage, all strands are being held in place only by wedges. Tests have confirmed that these anchorages meet the design requirements. But unbonded strand wedges are delicate structural elements and are susceptible to construction deviations. Care must be exercised in the design, fabrication, and installation if such an anchorage is to be used in a cable-stayed bridge. The advantage of an unbonded cable system is that the cable, or individual strands, can be replaced relatively easily.

Cable anchorage tests have shown that, in a bonded anchor, less than half of the cyclic stress is transferred to the wedges. The rest is dissipated through the filling and into the anchorage directly by bond. This is advantageous with respect to fatigue and overloading.

The Post Tensioning Institute's "Recommendations for Stay Cable Design and Testing," [9] was published in 1986. This is the first uniformly recognized criteria for the design of cables. In conjunction with the American Society of Civil Engineers' "Guidelines for the Design of Cable-Stayed Bridges" [10], they give engineers a much-needed base to start their design.

There have been various suggestions for using composite materials such as carbon fiber, etc. as stays and small prototypes have been built. However, actual commercial application still requires further research and development.

19.2.3 Girder

Although the Stromsund Bridge has a concrete deck, most other early cable-stayed bridges have an orthotropic deck. This is because both cable-stayed bridge and orthotropic deck were introduced to the construction industry at about the same time. Their marriage was logical. The fact that almost all long-span bridges were built by steel companies at that time made such a choice more understandable.

A properly designed and fabricated orthotropic deck is a good solution for a cable-stayed bridge. However, with increasing labor costs, the orthotropic deck becomes less commercially attractive except for very long spans.

Many concrete cable-stayed bridges have been completed. In general, there have been two major developments: cast-in-place construction and precast construction.

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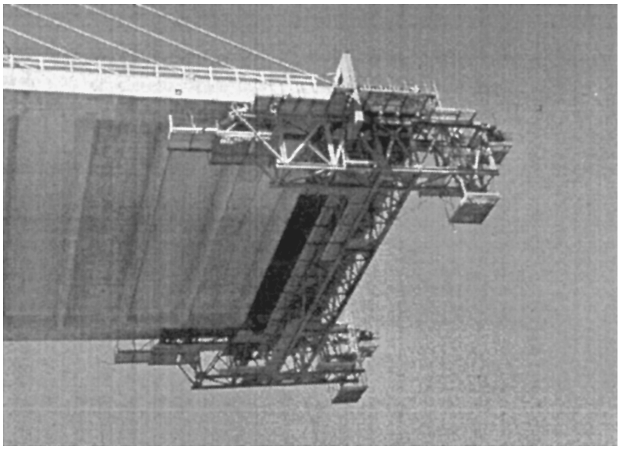

FIGURE 19.11 Form traveler of the Dames Point Bridge.

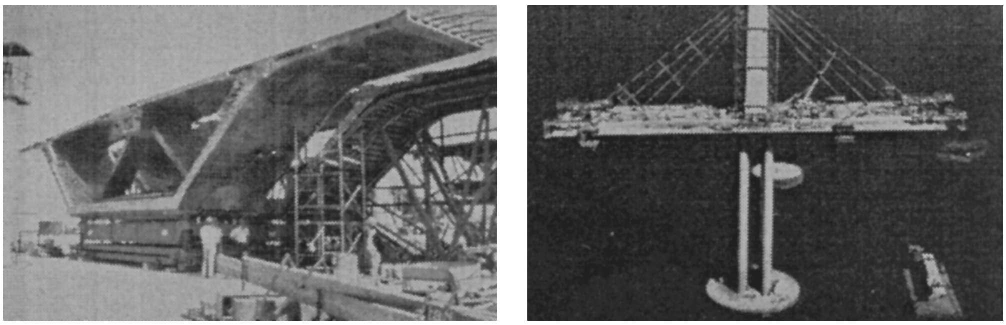

FIGURE 19.12 Sunshine Skyway Bridge – precast box and erection.

Cast-in-place construction of cable-stayed bridges is a further development of the free cantilever construction method of box-girder bridges. The typical construction is by means of a form traveler. The box girder is a popular shape for the girder in early structures, such as the Hoechst Bridge, the Barrios de Luna Bridge, and the Waal Bridge. But simpler cross sections have proved to be attractive: the beam and slab arrangement in the Penang Bridge, the Dames Point Bridge, and the Talmadge Bridge, or the solid slab cross section as in the Yelcho Bridge, the Portsmouth Bridge, and the Diepoldsau Bridge are both technically sound and economical. Use of the newly developed cable-supported form traveler, Figure 19.11, makes this type girder much more economical to build [8].

Precast construction can afford a slightly more complicated cross section because precasting is done in the yard. The segments, however, should all be similar to avoid adjustment in the precasting forms. The weight of the segment is limited by the transportation capability of the equipment used. Box is the preferred cross section because it is stiffer and easier to erect. The Brotonne Bridge, the Sunshine Skyway Bridge, Figure 19.12, and the Chesapeake and Delaware Canal Bridge are good examples. However, several flexible girder cable-stayed bridges have been completed successfully, notably the Pasco–Kennewick Bridge, the East Huntington Bridge, and the ALRT Skytrain Bridge.

As concrete technology advances, today's cable-stayed bridges may consider using high-strength lightweight concrete for the girder, especially in high seismic areas.

Although the steel orthotropic deck is too expensive for construction in most countries at this time, the composite deck with a concrete slab on a steel frame can be very competitive. In the Stromsund Bridge, the concrete deck was not made composite with the steel girder. Such a construction is not economical because the axial compressive force in the girder must be taken entirely by

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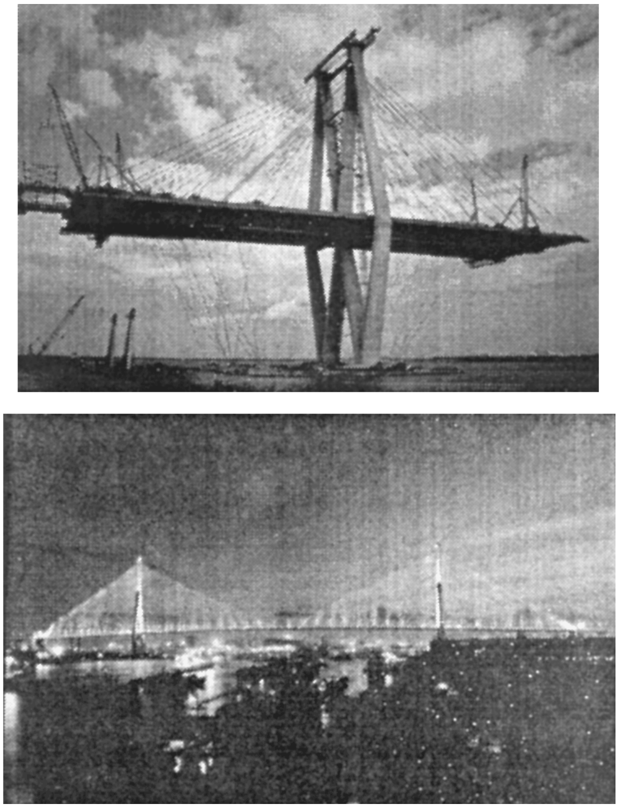

FIGURE 19.13 Baytown–LaPorte Bridge and Yang Pu Bridge.

the steel girder. Making the deck composite with the steel girder by shear studs reduces the steel quantity of the girder significantly. The compressive stress in the concrete deck also improves the performance of the deck slab. The Hootley Bridge was the first major composite bridge designed, but the Annacis Island Bridge was completed first. The Yang Pu Bridge is the longest span today. The Baytown–LaPorte Bridge, Figure 19.13, has the largest deck area.

Precast slab panels are usually used for composite bridges. Requiring the precast panels to be stored for a period of time, say, 90 days, before erection reduces the effect of creep and shrinkage significantly. The precast panels are supported by floor beams and the edge girders during erection. The gaps between the panels are filled with nonshrinking concrete. The detail for these closure joints must be carefully executed to avoid cracking due to shrinkage and other stresses.

Most portions of the girder are under high compression, which is good for concrete members. However, tensile stresses may occur in the middle portion of the center span and at both ends of the end spans. Post-tensioning is usually used in these areas to keep the concrete under compression.

Several hybrid structures, with concrete side spans and steel main span, such as the Flehe Bridge, Germany and the Normandie Bridge, have been completed. There are two main reasons for the hybrid combination: to have heavier, shorter side spans to balance the longer main span or to build the side spans the same way as the connecting approaches. The transition, however, must be carefully detailed to avoid problems.

19.2.4 Tower

The towers are the most visible elements of a cable-stayed bridge. Therefore, aesthetic considerations in tower design is very important. Generally speaking, because of the enormous size of the structure,

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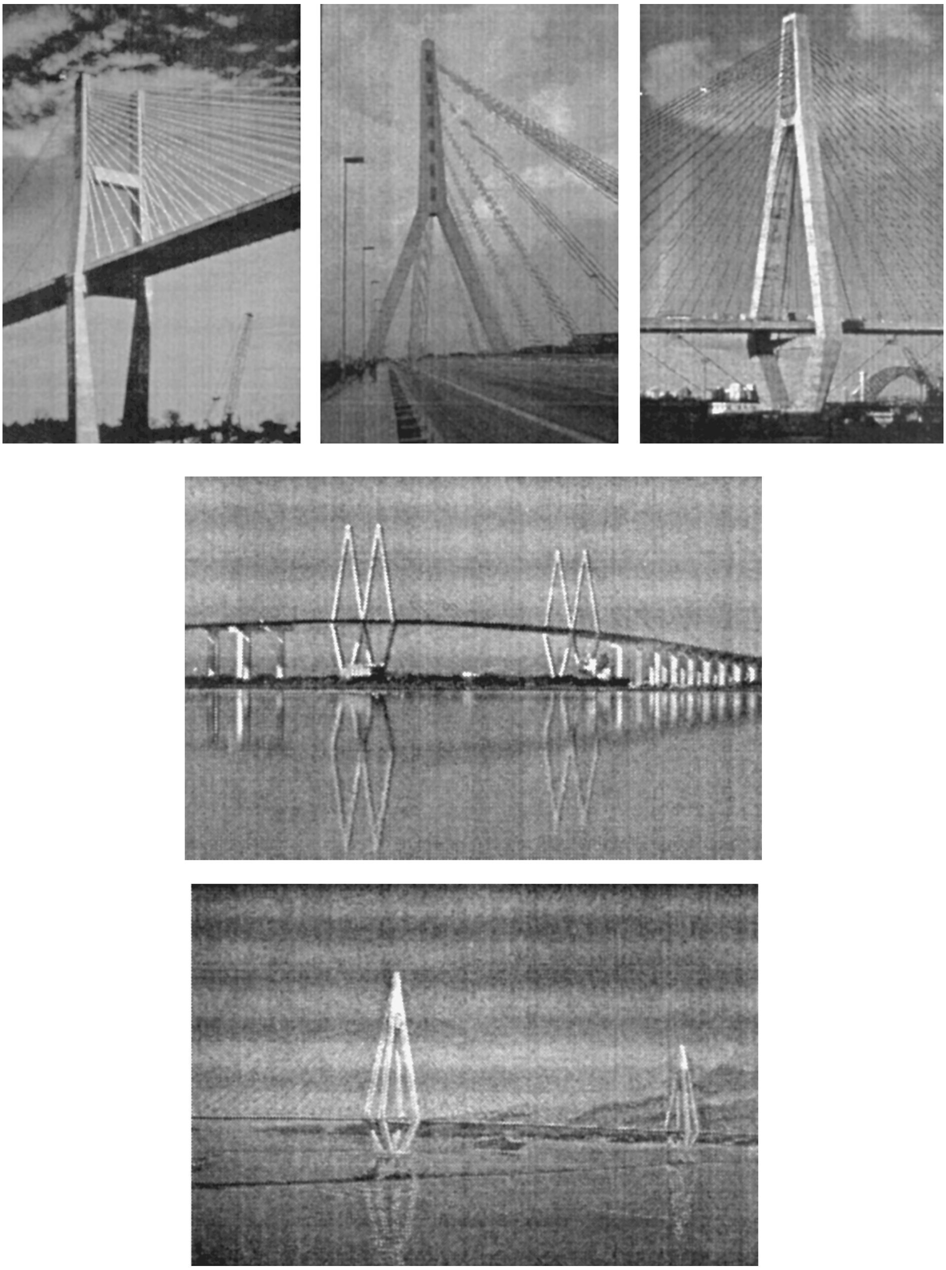

FIGURE 19.14 Tower shapes — H (Talmadge), inverted Y (Flehe), diamond (Glebe), double diamond (Baytown), and special configuration (Dan Chiang).

a clean and simple configuration is preferable. The free-standing towers of the Nord Bridge and the Knie Bridge look very elegant. The H towers of the Annacis Bridge, the Talmadge Bridge, and the Nan Pu Bridge are the most logical shape structurally for a two-plane cable-stayed bridge, Figure 19.14. The A shape (as in the East Hungtington Bridge), the inverted Y (as in the Flehe Bridge), and the diamond shape (as in the Baytown–LaPorte and Yang Pu Bridges) are excellent choices for long-span cable-stayed bridges with very flexible decks. Other variations in tower shape are possible as long as they are economically feasible. Under special circumstances, the towers can also serve as tourist attractions such as the one proposed for the Dan Chiang Bridge in Taiwan.

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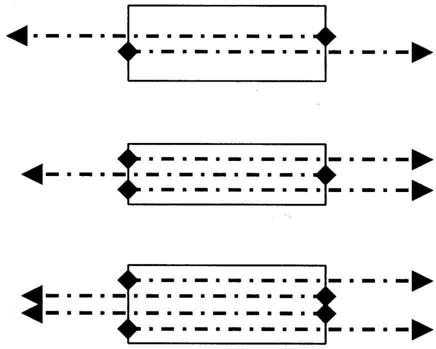

FIGURE 19.15 Crisscrossing cables at tower column.

Although early cable-stayed bridges all have steel towers, most recent constructions have concrete towers. Because the tower is a compression member, concrete is the logical choice except under special conditions such as in high earthquake areas.

Cables are anchored at the upper part of the tower. There are generally three concepts for cable anchorages at the tower: crisscrossing, dead-ended, and saddle.

Early cable-stayed bridges took their anchorage details from suspension bridges that have saddles. Those saddles were of the roller, fixed, or sector types. Roller-type saddles have a roller at the base similar to a bridge bearing. Fixed-type saddles are similar except the base is fixed instead of rolling. Sector-type saddles rotate around a pin. Each of these saddles satisfies a different set boundary condition in the structural system. Cable strands, basically lock-coil strands, were placed in the saddle trough as in a suspension bridge. To assure that the strands do not slide under unequal tension, a cover plate is usually clamped to the saddle trough to increase friction. This transverse pressure increases friction but reduces the strength of the cable. This type of saddle is very expensive and has not been used for recent cable-stayed bridges.

A different type of saddle was used in the Brotonne Bridge and the Sunshine Skyway Bridge. Here the seven-wire strands were bundled into a cable and then pulled through a steel saddle pipe which was fixed to the tower. Grouting the cable fixed the strands to the saddle pipe. However, the very high contact pressure between the strand wires must be carefully considered and dealt with in the design. Because the outer wires of a strand are helically wound around a straight, center king wire, the strands in a curved saddle rest on each other with point contacts. The contact pressure created by the radial force of the curved strands can well exceed the yield strength of the steel wires. This can reduce the fatigue strength of the cable significantly.

Crisscrossing the cables at the tower is a good idea in a technical sense. It is safe, simple, and economical. The difficulty is in the geometry. To avoid creating torsional moment in the tower column, the cables from the main span and the side span should be anchored in the tower in the same plane, Figure 19.15. This, however, is physically impossible if they crisscross each other. One solution is to use double cables so that they can pass each other in a symmetrical pattern as in the case of the Hoechst Bridge. If A-shaped or inverted-Y-shaped towers are used, the two planes of cables can also be arranged in a symmetrical pattern.

If the tower cross section is a box, the cables can be anchored at the front and back wall of the tower, Figure 19.16. Post-tensioning tendons are used to prestress the walls to transfer the anchoring forces from one end wall to the other. The tendons can be loop tendons that wrap around three side walls at a time or simple straight tendons in each side wall independently. The Talmadge, Baytown–LaPorte, and Yang Pu Bridges all employ such an anchoring detail. During the design of the Yang Pu Bridge, a full-scale model was made to confirm the performance of such a detail.

As an alternative, some bridges have the cables anchored to a steel member that connects the cables from both sides of the tower. The steel member may be a beam or a box. It must be connected

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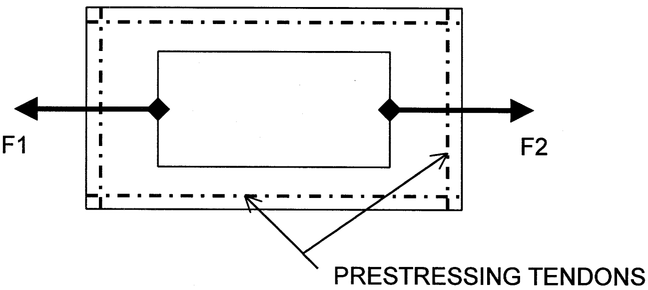

FIGURE 19.16 Tendon layout at the anchorage area.

to the concrete tower by shear studs or other means. This anchorage detail simulates the function of the saddle. However, the cables at the opposite sides are independent cables. The design must therefore consider the loading condition when only one cable exists.

19.3 Design

19.3.1 Permanent Load Condition

A cable-stayed bridge is a highly redundant, or statically indeterminate structure. In the design of such a structure, the treatment of the permanent load condition is very important. This load condition includes all structural dead load and superimposed dead load acting on the structure, all prestressing effects as well as all secondary moments and forces. It is the load condition when all permanent loads act on the structure.

Because the designer has the liberty to assign a desired value to every unknown in a statically indeterminate structure, the bending moments and forces under permanent load condition can be determined solely by the requirements of equilibrium, $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$. The stiffness of the structure has no effect in this calculation. There are an infinite number of possible combinations of permanent load conditions for any cable-stayed bridge. The designer can select the one that is most advantageous for the design when other loads are considered.

Once the permanent load condition is established by the designer, the construction has to reproduce this final condition. Construction stage analysis, which checks the stresses and stability of the structure in every construction stage, starts from this selected final condition backwards. However, if the structure is of concrete or composite, creep and shrinkage effect must be calculated in a forward calculation starting from the beginning of the construction. In such cases, the calculation is a combination of forward and backward operations.

The construction stage analysis also provides the required camber of the structure during construction.

19.3.2 Live Load

Live-load stresses are mostly determined by evaluation of influence lines. However, the stress at a given location in a cable-stayed bridge is usually a combination of several force components. The stress, *f*, of a point at the bottom flange, for example, can be expressed as:

$$f = (1/A) * P + (y/I) * M + c * K \tag{19.5}$$

where A is the cross-sectional area, I is the moment of inertia, y is the distance from the neutral axis, and c is a stress influence coefficient due to the cable force K anchored at the vicinity. P is the axial force and M is the bending moment. The above equation can be rewritten as

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$$f = a1 * P + a2 * M + a3 * K \quad (19.6)$$

where the constants $a1$, $a2$, and $a3$ depend on the effective width, location of the point, and other global and local geometric configurations. Under live load, the terms P, M, and K are individual influence lines. Thus, f is a combined influence line obtained by adding up the three terms multiplied by the corresponding constants $a1$, $a2$, and $a3$, respectively.

In lieu of the combined influence lines, some designs substitute P, M, and K with extreme values, i.e., maximum and minimum of each. Such a calculation is usually conservative but fails to present the actual picture of the stress distribution in the structure.

19.3.3 Thermal Loads

Differential temperature between various members of the structure, especially that between the cables and the rest of the bridge, must be considered in the design. Black cables tend to be heated up and cooled down much faster than the towers and the girder, thus creating a significant temperature difference. Light-colored cables, therefore, are usually preferred.

Orientation of the bridge toward the sun is another factor to consider. One face of the towers and some group of cables facing the sun may be warmed up while the other side is in the shadow, causing a temperature gradient across the tower columns and differential temperature among the cable groups.

19.3.4 Dynamic Loads

19.3.4.1 Structural Behavior

The girder of a cable-stayed bridge is usually supported at the towers and the end piers. Depending on the type of bearing or supports used, the dynamic behavior of the structure can be quite different. If very soft supports are used, the girder acts like a pendulum. Its fundamental frequency will be very low. Stiffening up the supports and bearings can increase the frequency significantly.

Seismic and aerodynamics are the two major dynamic loads to be considered in the design of cable-stayed bridges. However, they often have contradictory demands on the structure. For aerodynamic stability a stiffer structure is preferred. But for seismic design, except if the bridge is founded on very soft soil, a more flexible bridge will have less response. Some compromise between these two demands is required.

Because the way these two dynamic loads excite the structure is different, special mechanical devices can be used to assist the structure to adjust to both load conditions. Aerodynamic responses build up slowly. For this type of load the forces in the connections required to minimize the vibration buildup are relatively small. Earthquakes happen suddenly. The response will be especially sudden if the seismic motion also contains large flings. Consequently, a device that connects the girder and the tower, which can break at a certain predetermined force will help in both events. Under aerodynamic actions, it will suppress the onset of the vibrations as the connection makes the structure stiffer. Under seismic load, the connection breaks at the predetermined load and the structure becomes more flexible. This reduces the fundamental frequency of the bridge.

19.3.4.2 Seismic Design

Most cable-stayed bridges are relatively flexible with long fundamental periods in the range of 3.0 s or longer. Their seismic responses are usually not very significant in the longitudinal direction. In the transverse direction, the towers are similar to a high-rise building. Their responses are also manageable.

Experience shows that, except in extremely high seismic areas, earthquake load seldom controls the design. On the other hand, because most cable-stayed bridges are categorized as major structures, they are usually required to be designed for more severe earthquake loads than regular structures.

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Various measures have been used to reduce the seismic response of cable-stayed bridges [1]. They range from a simple shock damper with a hydraulic cylinder that freezes at fast motion, to different types of friction dampers. Letting the bridge girder swing for a certain distance like a pendulum is another efficient way of reducing the seismic response. This is especially effective for out-of-phase motions. Because many cable-stayed spans are in the range of the half wavelengths of seismic motions, the out-of-phase motion can create very large reactions in the structure. A partially floating girder is often found to be very advantageous.

19.3.4.3 Aerodynamics

Aerodynamic stability of cable-stayed bridges was a major concern for many bridge engineers in the early years. This was probably because cable-stayed bridges are extremely slender. Lessons learned from aerodynamic problems in suspension bridges lead engineers to worry about cable-stayed bridges.

In reality, cable-stayed bridges, especially concrete cable-stayed bridges, have been found to be surprisingly stable aerodynamically. Although the prediction that cable-stayed bridges could not seriously vibrate under wind due to interference of the different unrelated modes that exist in such a structure was not correct, extremely few cable-stayed bridges were found to be susceptible to wind action after construction. The superior aerodynamic behavior of cable-stayed bridges is one reason for this. But the lessons learned from suspension bridge have educated many engineers so that they are aware of aerodynamic problems and can identify the preferred cross sections against wind actions. The wider deck width of most modern cable-stayed bridges also makes the structure more stable.

Several bridges did require special treatment against aerodynamic action. The Annacis Island Bridge added wind fairing to the main span; the Quincy Bridge added vertical plates to the girder in addition to horizontal fairings. Longs Creek Bridge has a tapered wind nose at each side of the girder.

During the design of the Knie Bridge in the early 1960s, a wind tunnel study was performed to search for a good solution to increase the aerodynamic stability of the bridge in case its responses were found to be unacceptable. Among various alternatives, the tapered nose was the most efficient option. This same idea has been used in many cable-stayed bridges and suspension bridges since then.

Although a cable-stayed bridge is mostly stable in its final condition, it is often vulnerable during construction stages. During the construction of the Knie Bridge, bottom bracing was added to provide the required torsional stiffness in the girder to eliminate flutter possibility. Most high-level cable-stayed bridges in high-wind areas have required wind tie-downs to stabilize the structure against buffeting.

Back in the 1960s, it was thought that a structure would not vibrate in a turbulent flow. However, it was found that by having a certain intensity and frequency content in a turbulent wind, buffeting may occur. In many cases, the buffeting responses of a cable-stayed bridge during construction can be quite severe unless specific measures are taken to stabilize the structure.

The most efficient way to stabilize the structure against buffeting is to increase its fundamental frequency by tie-downs, Figure 19.17. Most tie-downs are simple cables of seven-wire strands anchored to pile foundations, dead weights, or soil anchors such as in the Annacis and Baytown–LaPorte Bridges. Stabilizing the tower by front and back staying cables can also have the same effect as in the ALRT Fraser River and East Huntington Bridges.

Use of tie-downs can also help reduce the unbalanced bending moment in the tower during construction which is inherent in a cantilever method.

The amount of damping can have a decisive effect on the aerodynamic behavior of a cable-stayed bridge, especially on the critical flutter wind speed. Consequently, the assumption of a proper damping ratio is very important in the design. Table 19.1 shows the calculated critical flutter wind speed of the Sidney Lanier Bridge, Georgia, based on sectional model test results.

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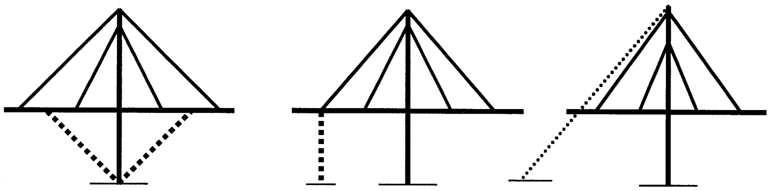

FIGURE 19.17 Temporary stabilization by tie-down cables.

TABLE 19.1 Critical Flutter Wind Speed — Sidney Lanier Bridge

Damping ratio (% of critical)	0.5%	1.0%	2.0%	3.0%	4.0%
Critical flutter wind sped, km/h	160	180	230	340	450

Practically all field measurements of damping ratio of cable-stayed bridges were performed with small amplitudes. Generally, it was found to be between 0.5 and 1.0%. Such measured values correspond to the behavior of low-amplitude votex-shedding responses, which usually happen under relatively low wind speed. However, flutter is a phenomenon represented by large amplitudes; the actual damping coefficient is much higher.

Flutter is considered an extreme natural event that may happen once every 1000 to 2000 years. There will also be no people on the bridge under such high winds. Therefore, very large amplitude oscillations with cracking of concrete and partial yielding of steel are considered acceptable. Under such conditions, the damping increases significantly. A cable-stayed bridge, being a very redundant structure that may allow many plastic hinges to form, offers decisive advantages over regular girder bridges.

A damping ratio of 5% is usually assumed in seismic analysis, which is a similar extreme natural event. A value of 2 to 4% may be conservatively assumed for a concrete cable-stayed bridge. Higher damping can also be achieved by installation of artificial dampers.

The knowledge of cable vibrations has also progressed extensively. The first stay cable vibration problem appeared in the Neuenkamp Bridge. This was a wake vibration of two parallel and horizontally located cables. The problem was new at that time. It was identified, and further vibration was suppressed by connecting the pair of cables together with a damper. This concept was used for several other subsequently built bridges.

Severe cable vibrations were observed in the Brotone Bridge. Dampers were installed and they were successful in suppressing the vibrations. The same concept was used for the Sunshine Skyway.

Rain–wind vibrations were discovered in a few bridges. This phenomenon appears only during light rain in combination with light wind. This problem was found to be caused by the change of shape of rain water mantle on the cable. Increasing the damping of cables can suppress this vibration. Tying cables together by wires, Figure 19.18, and draining the water away from the cable before it accumulates are the more common and effective methods to combat this problem. Adding dimples or spiral-wound ridges on the cable surface have also been found to be effective.

19.4 Superlong Spans

Conceptually, cable-stayed bridges can be used for very long spans. Because a cable-stayed bridge is a closed structural system, similar to a self-anchored suspension bridge, but can be built without temporary supports, it is especially advantageous in areas where the soil condition is not good and anchoring the main suspension cables becomes prohibitively expensive. Span length of over 2000 m is entirely feasible.

Three major details must be properly attended to in the design of superlong-span cable-stayed bridges: the effectiveness of very long cables, the compression in the deck, and the torsional stiffness of the girder for wind stability. When a cable is too long, it becomes ineffective. This can be resolved

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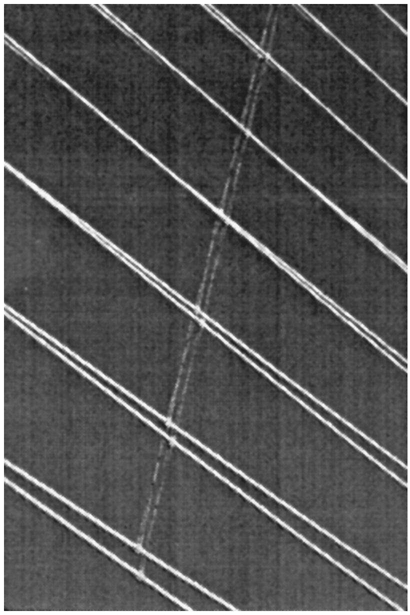

FIGURE 19.18 Tie cables against rain–wind vibrations — Dames Point Bridge.

by providing intermediate supports for the cable to reduce its sag. The compression stress in the deck increases proportionally to the span length. Depending on the type of material used for the deck girder, the maximum span length of a uniform girder is limited by its allowable stresses. This problem can best be solved by using a girder with variable cross section. By increasing the girder section gradually toward the towers, where compression is the highest, the compression stress can be reduced to an acceptable level. The torsional stiffness of the girder can be supplemented by having sufficiently wide towers so that the cables are inclined in the transverse direction.

If the deck is too narrow for the very long span, horizontal staying cables in the transverse direction at the deck level can provide stiffness in that direction.

19.5 Multispan Cable-Stayed Bridges

Most cable-stayed bridges have either three or two cable-stayed spans. The back-staying cables and the anchor pier play an important role in stabilizing the tower. When a bridge has more than three spans, the bending moment in the towers will be very large. One solution is to design the towers to carry the large bending moment [6,11]. This is usually not the most economical solution.

Several methods are available to strengthen the towers of a multispan cable-stayed bridge [6], as shown in Figure 19.19:

1. Tying the tower tops together with horizontal cables;
2. Tying the tower tops to the girder and tower intersection point at the adjacent towers;
3. Adding additional tie-down piers at span centers; and
4. Adding crossing cables at midspans.

19.6 Aesthetic Lighting

Aesthetic lighting is now part of the design of most cable-stayed bridges. Lighting enhances the beauty and visibility of the bridge at night. There are various schemes of lighting a cable-stayed bridge as shown in Figure 19.20.

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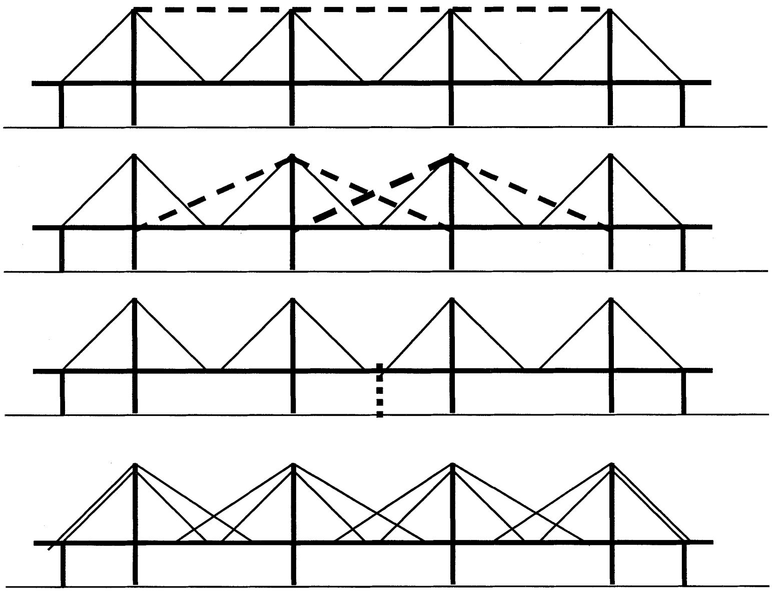

FIGURE 19.19 Multispan cable-stayed bridge system.

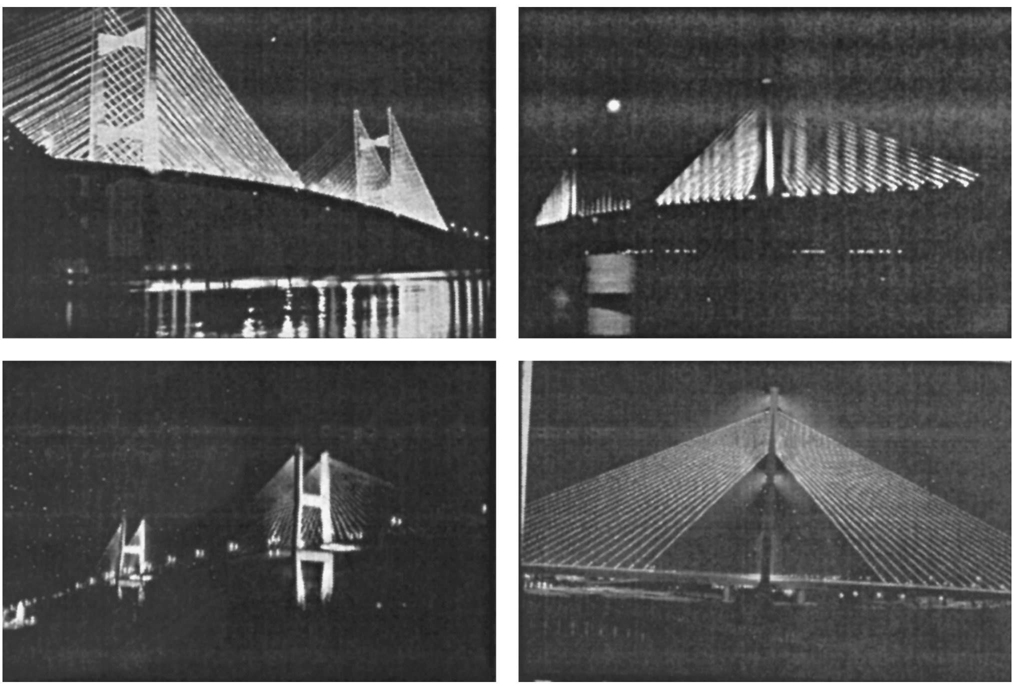

FIGURE 19.20 Aesthetic lighting of cable-stayed bridges.

19.7 Summary

Cable-stayed bridges are beautiful structures. Their popular appeal to engineers and nonengineers alike has been universal. In a pure technical sense this bridge type fills the gap of efficient span range between conventional girder bridges and the very long span suspension bridges.

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TABLE 19.2 Milestone Cable-Stayed Bridges (as of April 1998)

Features	Bridge Name	Country	Span (m)	Year of Completion
First successfully built cable-stayed bridge	Stromsunde	Sweden	183	1955
First bridge with closely spaced cables	Bonn–Nord[a]	Germany	280	1967
All-steel open-deck plate girder	Knie[a]	Germany	320	1969
Center spine, single plane, all steel	Neuenkamp[a]	Germany	350	1971
First major concrete span, box spine girder	Hoescht	Germany	148	1972
First major precast box girder	Sunshine Skyway	U.S.	366	1986
First solid concrete flat-plate girder designed	Portsmouth	U.S.	286	Not built
First major composite girder completed	Annacis Island[a]	Canada	365	1986
First flat-plate girder completed	ALRT Skytrain	Canada	340	1988
Flexible girder built by cable-supported traveler	Dame Point	U.S.	396	1988
First major composite girder designed	Second Hoogly	India	457	1992
Longest composite girder	Yang Pu[a]	China	602	1993
Hybrid steel main span + concrete side spans	Normandy[a]	France	856	1994
Steel girder, conc. towers, longest span to date	Tatara[a]	Japan	890	1998

[a] World record span at time of completion.

Table 19.2 is a list of milestone cable-stayed bridges. It illustrates the general evolution of modern cable-stayed bridges.

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