Cable-Stayed Bridges

WALTER PODOLNY, JR.

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HISTORICAL BACKGROUND

THE IDEA OF supporting a bridge deck by employing one or more inclined stays is not a new concept. In the early 1600's, a Venetian engineer, Verantius, built a bridge with several diagonal chain stays.[7,8] It was proposed in 1784 by C. J. Löcher, a German carpenter in Fribourg, who conceived it as a timber bridge consisting of timber stays attached to a timber tower (Fig. 1). In 1817, two British engineers, Redpath and Brown, built the King's Meadow Footbridge in England, which had a span of approximately 110 ft, using sloping wire cable stay suspension members attached to a cast iron tower. In 1821, the French architect, Poyet, suggested the use of bar stays suspended from tall towers (Fig. 2), while still another version of the sloping stay configuration is the Gischlard-Arnodin type bridge (Fig. 3). In 1840, an Englishman, Hatley, suggested an arrangement of chain stays in a parallel configuration (Fig. 4).[5,13,18]

With this brief historical data on the origins of the cable-stayed bridge concept, the question naturally arises as to why this system met with very little success until a rebirth occurred within the last two decades. The reason for this obscurity no doubt lies in the collapse of some of the very early attempts at cable-stayed bridges. In 1818, a pedestrian bridge crossing the Tweed near Dryburgh Abbey, England, with a span of 259 ft, collapsed when the chain stays broke at the joints as a result of wind excited oscillations.[10] A 256-ft span bridge constructed over the River Salle near Neinburgh in 1824 collapsed the following year as a result of overloading by a crowd of people.[5]

The famous French engineer, Navier, reported on these failures and his comments condemned the stayed beam bridge to obscurity until recent times. Navier's statements led to the preference of the suspension-bridge type by bridge engineers. In a few early suspension bridges by John Roebling, such as the one near Niagara

Walter Podolny, Jr. is Structural Engineer, Bridge Division, Office of Engineering, Federal Highway Administration, U. S. Department of Transportation, Washington, D. C.

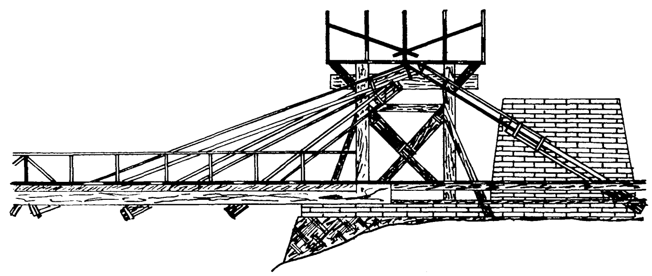

Fig. 1. Löscher type timber bridge, 1784 (Courtesy of The British Constructional Steelwork Association, Ltd.)

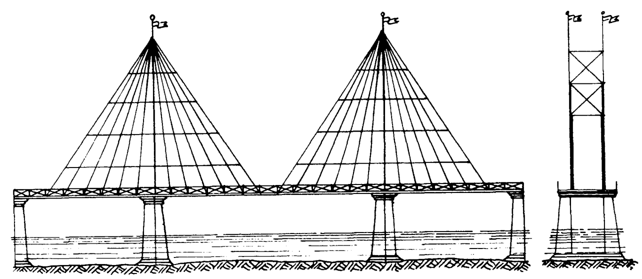

Fig. 2. Poyet type bridge, 1821 (Courtesy of The British Constructional Steelwork Association, Ltd.)

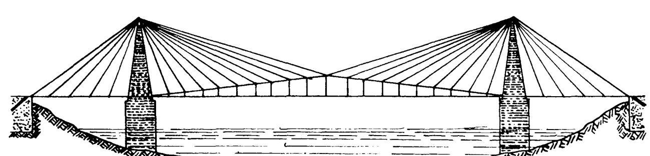

Fig. 3. Gischlard-Arnodin type sloping cable bridge (Courtesy of The British Constructional Steelwork Association, Ltd.)

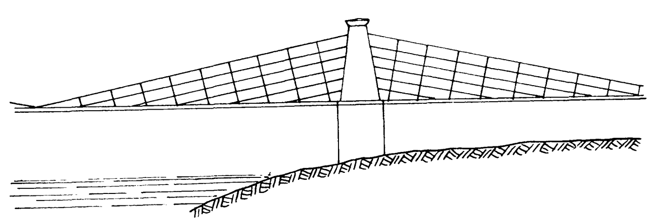

Fig. 4. Hatley type chain bridge, 1840 (Courtesy of the British Constructional Steelwork Association, Ltd.)

Falls (Fig. 5) and the old St. Clair Bridge in Pittsburgh (Fig. 6), stay ropes were used in addition to the vertical hangers. They are still in evidence on the Brooklyn Bridge and the Wheeling, W. Va., Bridge.

Unfortunately, during the birth of the cable-stayed system, the engineers of that period were not able to calculate the forces in the stays or to understand or control the equilibrium of these highly indeterminate systems. Unsuitable materials such as timber, round bars, or chains were used for stays. The stays constructed in this manner were, therefore, of low strength material and could not be fully tensioned. In a slack condition they required substantial deformations of the deck before they could participate in taking the tensile load for which they were intended.[13,16]

A German engineer, F. Dischinger, rediscovered the cable-stayed bridge in 1938 while attempting to design a 2,460-ft span suspension bridge to cross the River Elbe near Hamburg, that would accommodate full railroad loading on two tracks. Dischinger found that excessive deflection under heavy railroad loading could be considerably reduced by the incorporation of stay cables[2] (Fig. 7). This concept is exemplified in the Salazar Bridge across the Tagus River in Portugal (Fig. 8). At the top is the present structure, a conventional suspension bridge. At the bottom is the future structure, when cable stays will be added to accommodate additional rail traffic.

Modern implementation began in 1955 with the completion of the Strömsund Bridge in Sweden, of steel construction. In the period since 1955, about 50 bridges of this type have been built and more are in the design or construction stage.

In the United States, at the present time, only two cable-stayed bridges have been built, Sitka Harbor Bridge in Alaska[6] and the Menomonee Falls, Wisconsin, pedestrian bridge.[20] Two cable-stayed structures have been proposed for the I-410 New Orleans by-pass crossing the Mississippi River,[9] a structure in the State of Washington between Pasco and Kennewick, crossing the Columbia River, and a city bridge in Seattle.

GEOMETRIC CONFIGURATIONS

A wide variety of configurations have been employed in the construction of cable-stayed bridge systems. However, in a direction transverse to the longitudinal axis of the bridge, there appear to be basically two available arrangements: those where the cables are arranged in two planes and those in a single plane (Fig. 9).

The single-plane system is normally confined to a divided highway system, whereby the cable plane is vertical and is placed in the median strip (Fig. 10). An adaptation of this system is where the vertical plane of the cables is positioned laterally from the longitudinal center line of the deck.

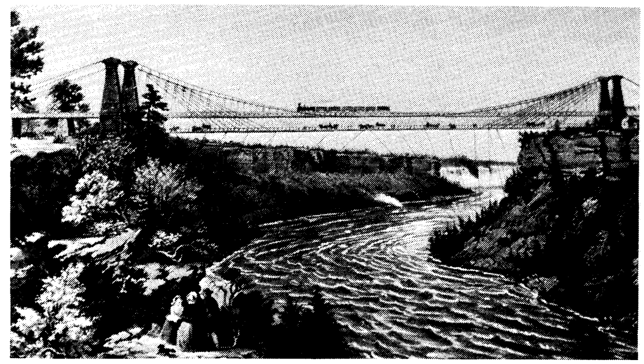

Fig. 5. Niagara Falls Bridge

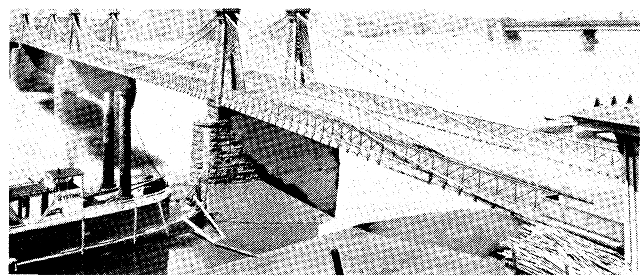

Fig. 6. Old St. Clair Bridge, Pittsburgh, Pa.

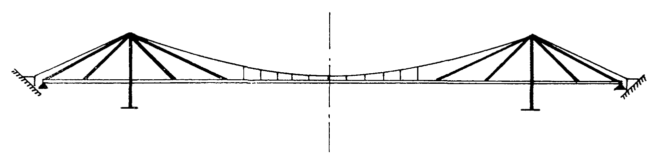

Fig. 7. Bridge system proposed by Dischinger

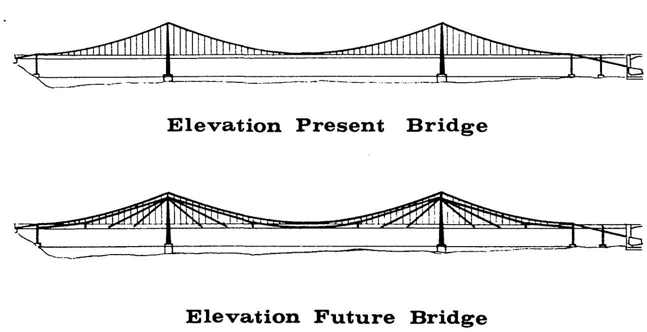

Fig. 8. The Salazar Bridge

The double-plane system has two possible arrangements: two vertical planes located outside the roadway (Fig. 11) or two planes sloping toward each other and intersecting on the longitudinal center line of the bridge, usually at some point on an A-frame tower (Fig. 12).

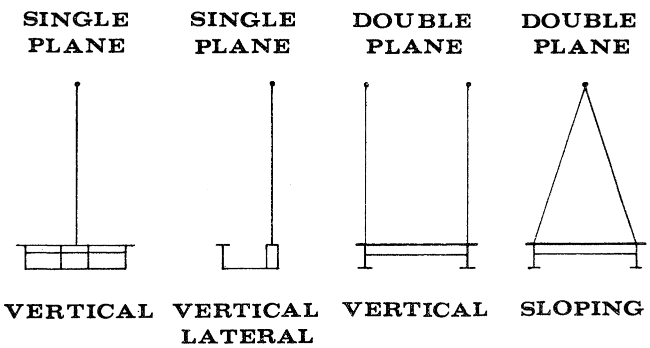

Fig. 9. Transverse cable arrangement

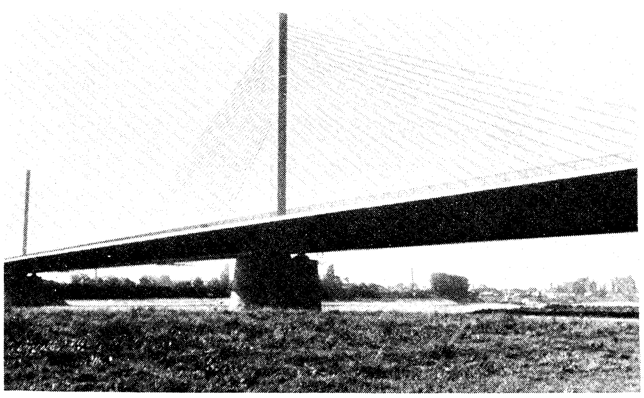

Fig. 10. Friedrich-Ebert Bonn Nord Bridge (Courtesy of Beratungsstelle Für Stahlverwendung, Dr.-Ing. H. Odenhousen)

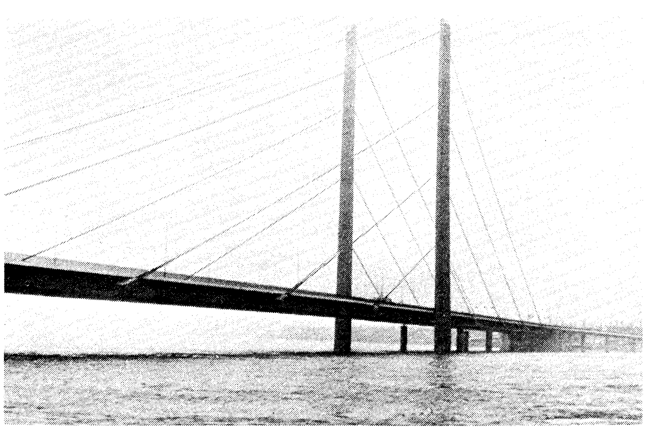

Fig. 11. Kniebrücke (Courtesy of Beratungsstelle Für Stahlverwendung, Dr.-Ing. H. Odenhousen)

In elevation, there are four basic cable configurations that are amenable to the single- and double-plane systems (Fig. 13). First, there is the radiating or converging type, where all the cables are taken to the top of the tower (see Fig. 12). Second, the harp type, where the cables are parallel to each other, being equally spaced along the girder and the tower (see Fig. 11). Third, the fan type, where the cables are equally spaced along the girder and the tower but are not parallel to each other, a hybrid of the first two types (see Fig. 10).

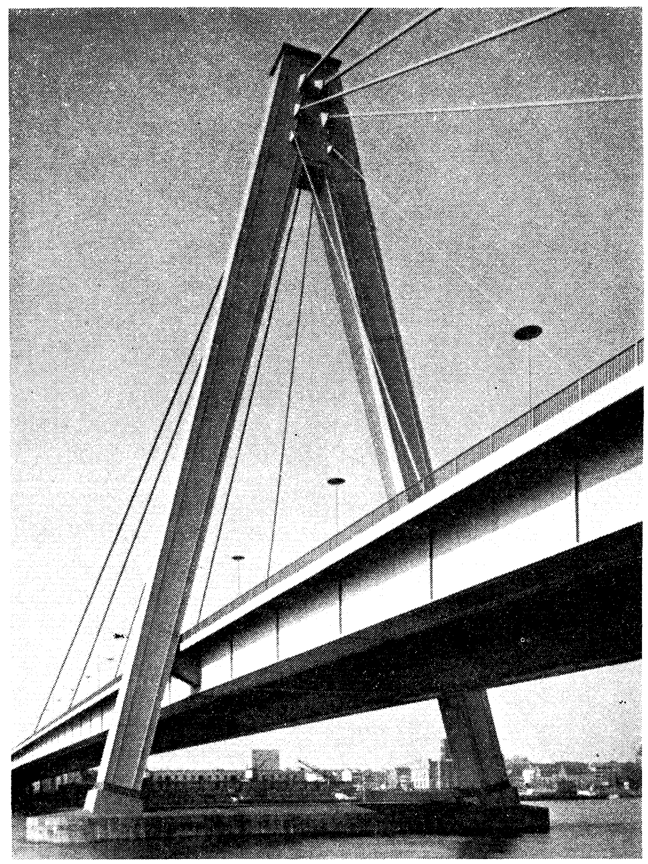

Fig. 12. Severin Bridge, Cologne, Germany

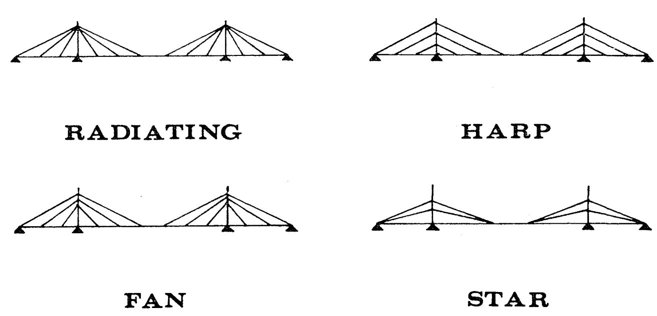

Fig. 13. Longitudinal cable configuration

The fourth and last basic type is the star configuration, where the cables are spaced along the tower and converge to a common point on the girder (Fig. 14).

There is the possibility of a wide variation of combinations of the four longitudinal configurations. It is obvious that numerous combinations are available to fit any conceivable design and aesthetic requirements. In elevation there is the possibility of the combination of the four basic types along with the single-plane, double-plane, vertical or sloping, along with a symmetric or an unsymmetric pylon arrangement.[13]

TOWERS

Cable-stayed bridge towers are of various types to suit site and design conditions and cable geometry. They may be A-frame and fixed to the pier, as is the Severin Bridge in Cologne, Germany (Fig. 12), which has a sloping, double-plane cable geometry, or an A-frame that is pinned in the longitudinal direction and fixed in the transverse direction for earthquake stability as the bridge over the Yodo River in Osaka, Japan, which has a single vertical cable plane (Fig. 15).

The Galipeault Bridge near Montreal, Canada, is of the standard portal frame type (Fig. 16). The Kniebrücke at Düsseldorf (Fig. 11) has the cross-member of the portal removed. Single towers for a single-plane cable geometry is illustrated by the Friedrich Ebert Bridge north of Bonn (Fig. 10).

The case of a lateral tower fixed to the pier has thus far only been employed on the footbridge at the 1958 German Pavilion at Brussels (Fig. 17), later dismantled and re-erected over a roadway near Duisburg.[13]

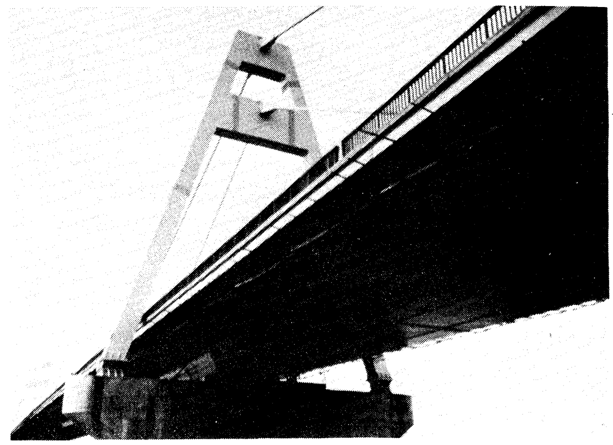

Fig. 15. Yodo River Bridge, Osaka, Japan

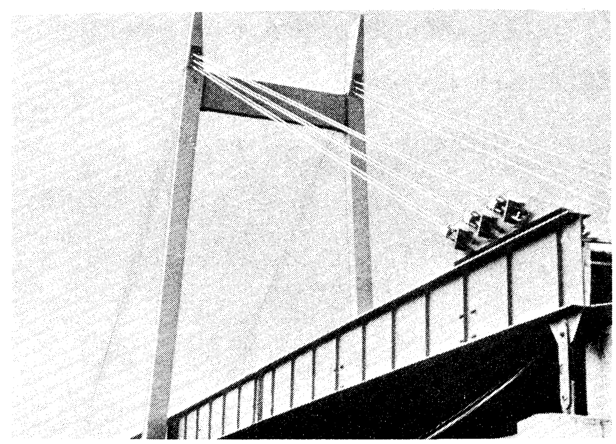

Fig. 16. Galipeault Bridge, Montreal, Canada

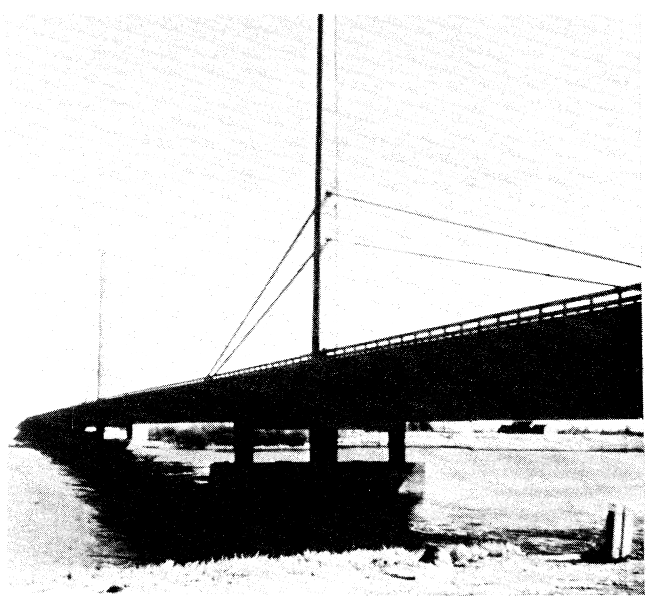

Fig. 14. Norderelbe (Courtesy of Beratungsstelle Für Stahlverwendung, Dr.-Ing. H. Odenhousen)

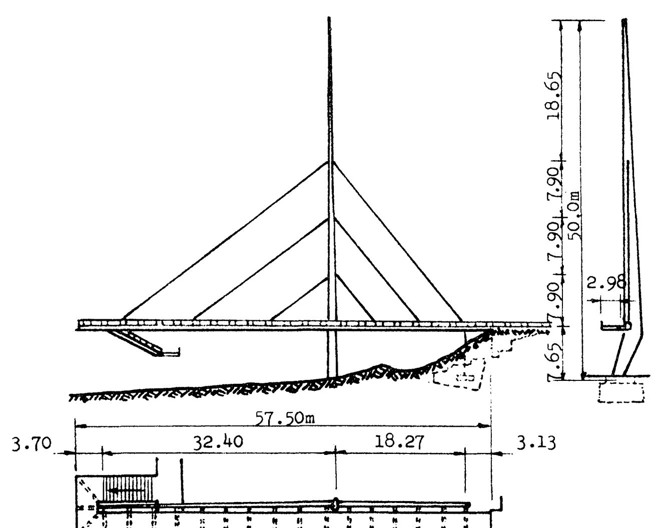

Fig. 17. Footbridge, German Pavilion, Brussels, 1958 (Courtesy of The British Constructional Steelwork Association, Ltd.)

DECK STRUCTURES

Various types of bridge decks have been employed in cable-stayed bridges; a few have been of prestressed concrete, but for the most part they have been steel bridge decks. The steel decks used in cable-stayed construction may be categorized as composite concrete deck or orthotropic steel plate deck; further, they may be comprised of two or more I-shaped girders or one or more box sections. The current trend in deck types appears to favor the orthotropic deck box-girder, especially the trapezoidal box-girder with side cantilevers, as a superior section for aerodynamic stability. It is felt that box-girder design is to be preferred for cable-stayed bridges, especially for a single, vertical plane system of cable geometry, because of its better torsional stiffness.[13]

WIND EFFECTS

Because of their flexibility, cable supporting systems may be subjected to potentially large dynamic motions as induced by wind forces. Wind oscillations have caused severe damage or destruction to at least 10 cable supported structures, including the Tacoma Narrows Bridge, over the last 15 decades. A popular misconception is that wind forces are only significant for long span bridges. This is obviously incorrect, as can be seen from Table 1; bridges have been destroyed or damaged in a span range from 245 ft to 2,800 ft and some have undesirable oscillations outside of this range.[4]

A few structures which have been observed to have undesirable oscillations as a result of wind forces are listed in Table 2.[4]

We have experienced, in the relatively recent past, the evolution and development of many innovations and concepts in structural analysis, materials, fabrication and erection procedures. These include the use of high-strength steels, welding, orthotropic decks, box girders, computer technology, cable-stays, mono-cable suspension, longitudinal as well as transverse inclined hangers, cantilever construction, parallel wire cables, along with other technological advances.

As a result, recent bridge structures have increased dimensions and flexibility and decreased dead weight and damping characteristics. Reduction of dead weight produces a magnification of wind effects relative to the inertia of the structure. Increased flexibility decreases the natural frequency of vibration. Modern fabrication techniques have decreased the structure's ability to absorb energy by sliding friction between component parts and thus less energy is required to initiate and maintain vibration.[17]

Therefore, recent structures are becoming more sensitive to not only static wind effects, but dynamic ones as well. Some existing and relatively recent structures have been so affected by wind oscillations that they required reinforcement. As a consequence, wind forces have taken on an increased importance and significance and can be a major problem in cable-supported bridge systems; serious consideration of these forces is required by the designer. It is also to be noted that consideration of wind effects must be considered on individual elements of the structure (suspended or deck structure, cables, towers, hangers, etc.) as well as on the cable-suspended system as a whole.[15]

If a steady wind blows against a cylinder or other obstruction, the wake is made up of what is termed the von Kármán Vortex Street or, for short, Vortex Shedding (Fig. 18). Vortices are formed as a result of interference of air flow by a cross-section, separating the flow from the boundary of that section. The shedding of these

Table 1. Bridges Severely Damaged or Destroyed by Wind

Bridge	Location	Designer	Span (ft)	Failure Date
Dryburgh Abbey	Scotland	John and William Smith	260	1818
Union	England	Sir Samuel Brown	449	1821
Nassau	Germany	Lossen and Wolf	245	1834
Brighton Chain Pier	England	Sir Samuel Brown	255	1836
Montrose	Scotland	Sir Samuel Brown	432	1838
Menai Straits	Wales	Thomas Telford	580	1839
Roche Bernard	France	Le Blanc	641	1852
Wheeling	U.S.A.	Charles Ellet	1010	1854
Niagara Lewiston	U.S.A.	Edward Serrell	1041	1864
Niagara Clifton	U.S.A.	Samuel Keefer	1260	1889
Tacoma Narrows	U.S.A.	Leon Moisseiff	2800	1940

Table 2. Modern Bridges Which Have Oscillated in Wind

Bridge	Year Built	Span (ft)	Type of Stiffening
Fyksesund (Norway)	1937	750	Rolled I-beam
Golden Gate	1937	4200	Truss
Thousand Island	1938	800	Plate Girder
Deer Isle	1939	1080	Plate Girder
Bronx Whitestone	1939	2300	Plate Girder

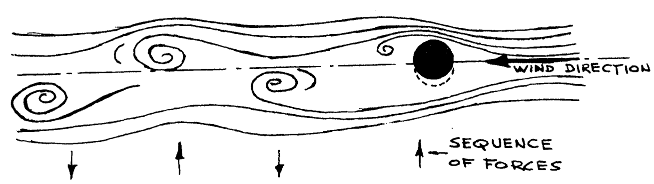

Fig. 18. Von Karman vortex street

vortices on the leeward side causes forces to act on the cross-section at right angles to the direction of the wind in an alternating and periodic fashion.

The periodic nature of the wake causes an oscillatory vertical force upon the cross-section unless it is very shallow with respect to its width or has a streamlined shape that reduces the tendency of the air flow to separate from the section. Analytical solutions for the elimination of vortex excitations are not available; however, wind tunnel tests can indicate the types of cross-section whereby the excitation is a minimum. Obviously desirable cross-sections are those which allow a near laminar flow around them.

Let us now investigate and compare the aerodynamic capabilities of a cable-stay system with that of a conventional suspension system. The modes of vibration in a suspension system are the symmetric mode and the anti-symmetric mode. In the symmetric mode the towers are deflecting toward each other, causing the center span to deflect downward while the end spans are deflecting upward. In the anti-symmetric mode the towers are deflecting in the same direction, resulting in the anti-symmetry of deformation of the deck with respect to mid-span.[1]

In the suspension bridge the most dangerous mode of oscillation is the anti-symmetric flutter mode (Fig. 19a). This type of mode caused the destruction of the Tacoma Narrows Bridge. This mode is easily developed by the pitching moment of wind forces, as a result of the two main cables moving against one another, one going up while the other goes down in half the span. The cable system gives no resistance to the induced torsional deformation of the deck. Therefore, a relatively high torsional and bending restraint deck system is required.[10,12]

In a cable-stayed structure with many stays, it is relatively more difficult to provide and maintain a resonant oscillation with attendant large amplitudes. The different length cables with different individual frequencies tend to disturb the formation of the first or second mode of oscillation by interfering with smaller wave lengths of higher order. Thus, the inherent system damping produces relatively smaller amplitudes as compared with the suspension system. The differences in deflection of the girders in the two cable planes results primarily from the different deflections of the pylons in each plane (Fig. 19b). If an A-frame pylon is utilized, then the differential deflection of the pylons in each cable plane is negated and the resistance of the cable-stay system to torsional oscillations of the roadway deck is further enhanced (Fig. 19c).[10,12]

As a result of the inherent system stiffness and damping, the cable-stayed bridge is not as sensitive to wind oscillations, relative to the suspension bridge. Therefore, the cable-stayed bridge requires less torsional stiffness in its deck system. However, this conclusion is only qualitative, since the difference in response of the two systems has yet to be demonstrated by wind tunnel tests with the present sectional model tests.

Numerous wind tunnel tests by investigators in several countries have indicated that bluff cross-sections have characteristics that produce intense von Kármán vortex shedding and large fluctuating vertical forces, resulting in vertical bending coupled with a torsional response. These studies have led to the development of what is considered to be a favorable aerodynamic deck cross-section. Aerodynamic stability of suspension or cable-stay bridges can be achieved by shaping the cross-section such that:[10,12]

1. The wind eddies that produce the von Kármán vortex shedding effect will be diminished or eliminated.
2. A minimum of lift and pitching moment will be produced and thus minimize bending and torsional oscillation.

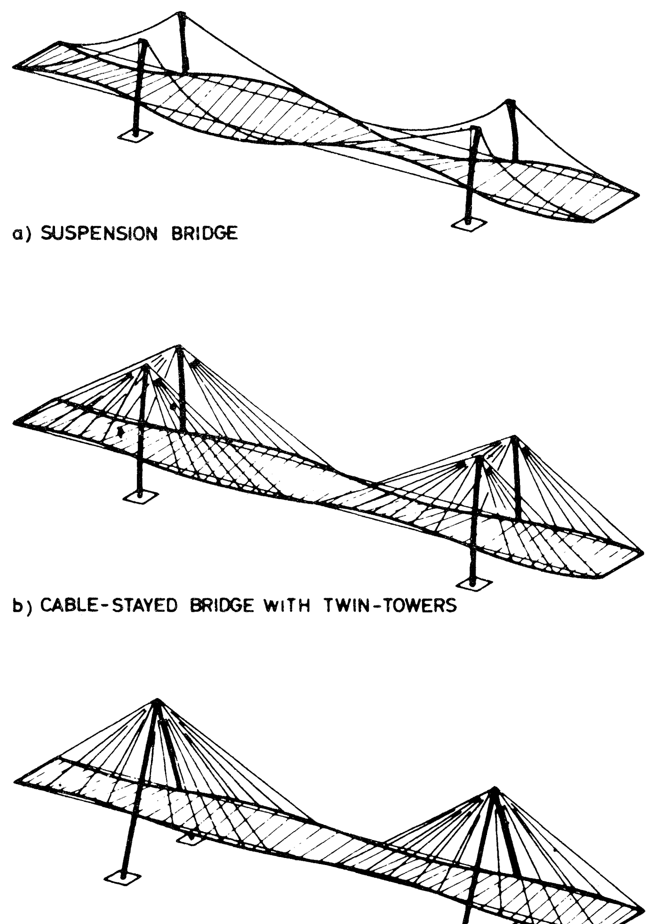

Fig. 19. Torsional mode of oscillation of suspension and cable-stayed bridge (from Leonhardt and Zellner by permission of the Canadian Steel Industries Construction Council)

Additional studies and tests have tended to validate this principle. It can therefore be stated that the conventional stiffening truss as used in the Verrazano Narrows suspension bridge (Fig. 20) is designed for increased flexural and torsional stiffness to resist the *effect* of wind forces, whereas the aerodynamically "streamlined" cross-section as used on the Severn suspension bridge in England (Fig. 21) is designed to minimize the excitation force and motion which is the *cause* of aerodynamic instability. Therefore, the streamlined cross-section seeks to eliminate the *cause* rather than totally resist the *effect.* Also, an obvious secondary improvement is the aesthetics of the structure.

It therefore appears that with a sufficiently wide aerodynamically shaped girder that is continuous at the pylons, aerodynamic stability can be attained even for extremely long spans. Although, the previous discussion had indicated a relatively favorable response for cable-stayed structures as compared to the conventional suspension structure, disturbing aerodynamic oscillations can and do occur in cable-stayed structures.

The usual section model wind tunnel tests for the Kniebrücke Bridge in Düsseldorf indicated instability at high wind speeds due to its sensitivity to torsional oscillation, as a result of the omission of the bottom plate that would have formed a closed box-section. The omission of this bottom plate was intentional, as an economy measure (Fig. 22a). Further studies were performed to give an indication of what remedial measures might be undertaken should undesirable wind oscillations develop. The results of these studies indicated that an additional lining outside of the main girders would be sufficient to produce stability. As a result, Leonhardt,[10,12] has proposed on subsequent structures that a cross-sectional configuration with triangular boxes at the edges and an open bottom deck be utilized.

The Long's Creek Bridge in New Brunswick, Canada, is a structure that has a center span of 713 ft with two vertical cable planes in cross-section and in elevation only one pair of radiating stays from each pylon (Fig. 23a). At speeds of 30 mph the structure was observed to have a vibration frequency of 0.6 cps and an amplitude up to 4 in., or 8 in. when snow fills the railing open-

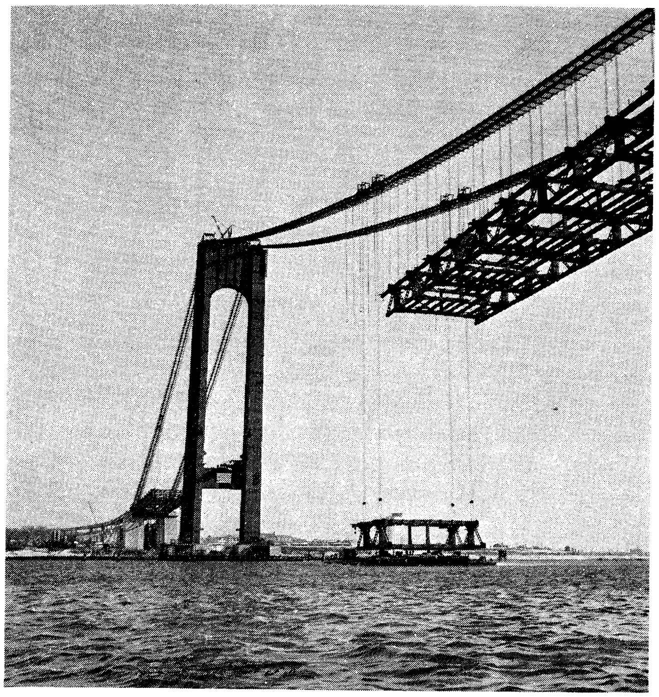

Fig. 20. Verrazano Narrows Bridge, New York

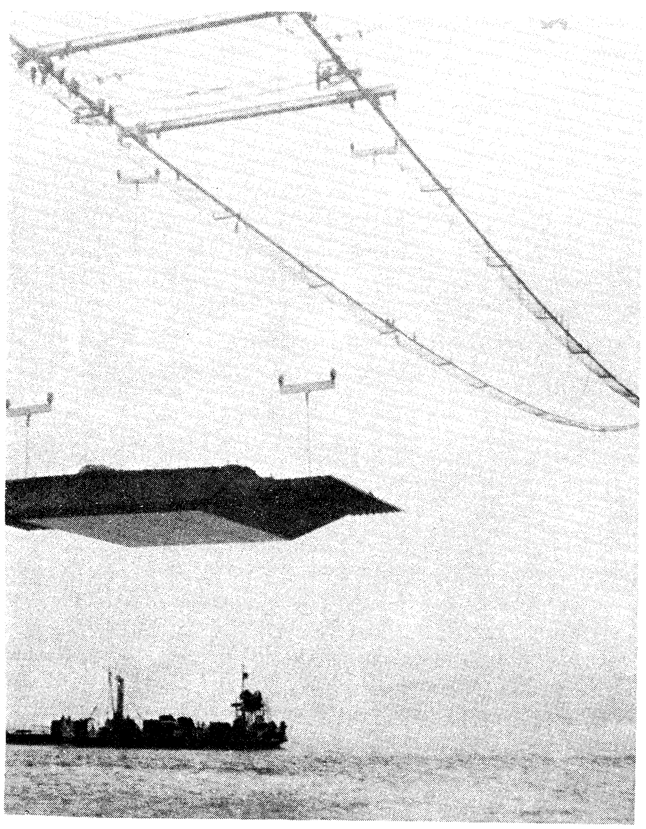

Fig. 21. Severn Bridge, England

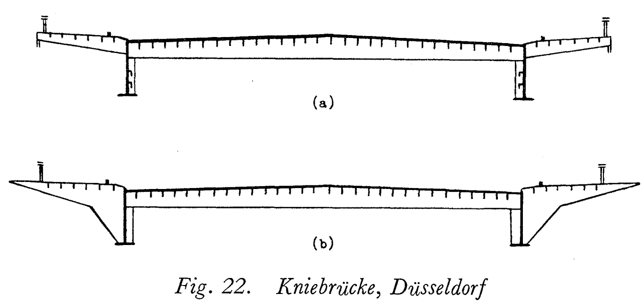

Fig. 22. Kniebrücke, Düsseldorf

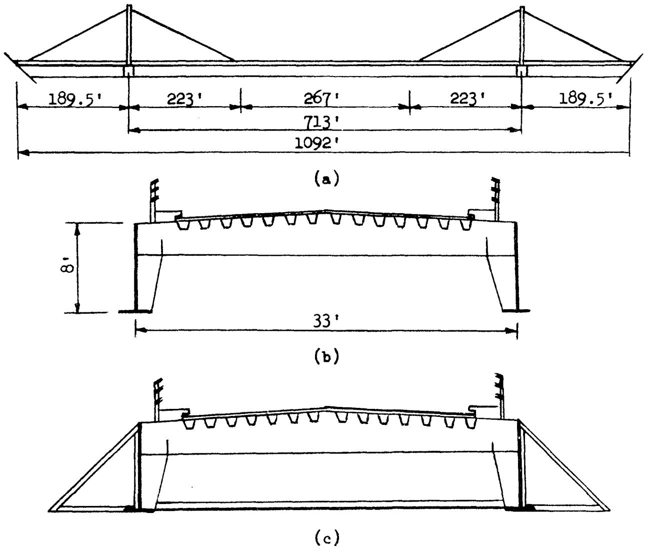

Fig. 23. Long's Creek Bridge, New Brunswick

ing.[1,19] The bluff cross-section produces an unfavorable aerodynamic section (Fig. 23b). As a result of wind tunnel tests, a soffit or bottom plate was installed to produce a box-section, thus increasing torsional stiffness (Fig. 23c). Further streamlining the cross-section was accomplished by the addition of 10-ft wide trianglular edge-fairings. These remedial measures decreased the amplitude of motion by as much as 40 percent.[1,19]

Wind tunnel tests were conducted at the National Physical Laboratory in Ottawa on a cross-section proposed by Leonhardt for a 2,460-ft span structure in Vancouver, B. C., to determine the necessity for the large torsional stiffness requirements for extremely wide bridges with a streamlined cross-section. These tests indicated that aerodynamic stability could be achieved on the deck cross-section even without the enclosing bottom plate (Fig. 24) and without consideration of the favorable system damping provided by the cable-stays. The section does incorporate a triangular box-section at each edge as well as a down sloping wind nose at the top outside edge.

ECONOMIC COMPARISONS

Obviously, one of the first questions that has to be considered in the selection of the bridge type is economics. Cost data and comparisons in this country are limited. Therefore, of necessity, data has had to be accumulated from published information in other countries and is accordingly referenced in the following discussion.

During the late 1950's and 1960's the competitive design situation that exists in Germany indicated that cable-stayed bridges were an economic solution, in Germany, for bridges having main spans ranging from about 500 to 1200 ft.

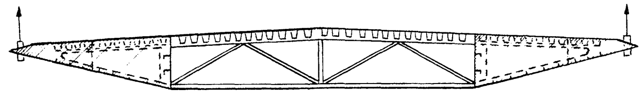

Fig. 24. A proposed cross-section, New Burrard Inlet Crossing, Vancouver, B. C., Canada

Based on bridges in Germany, Thul[18] has compared the center span length to the total length for three-span continuous girder bridges, cable-stayed bridges, and suspension bridges (Fig. 25). This plot indicates that the three-span continuous girder bridge has an upper limit of about 700 ft with a ratio of center span total length of not more than 50 percent. Suspension bridges apparently have a lower economic limit of about 1,000 ft with a ratio of center span to total span of approximately 65 percent. The cable-stayed bridge fills the void between the girder and suspension bridge with a ratio of center span to total span of approximately 55 percent.

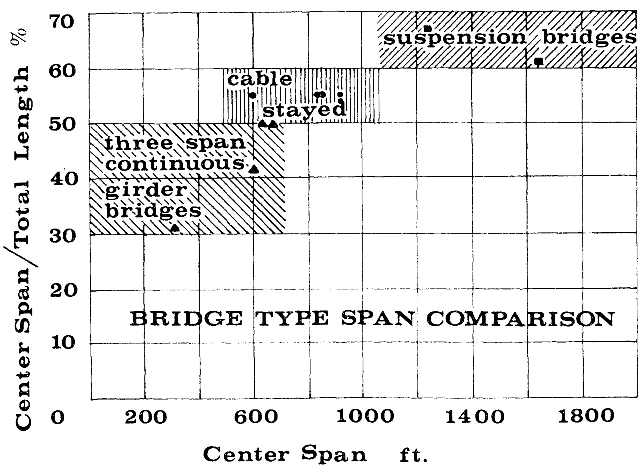

Fig. 25. Bridge type span comparison (Courtesy of The British Constructional Steelwork Association, Ltd.)

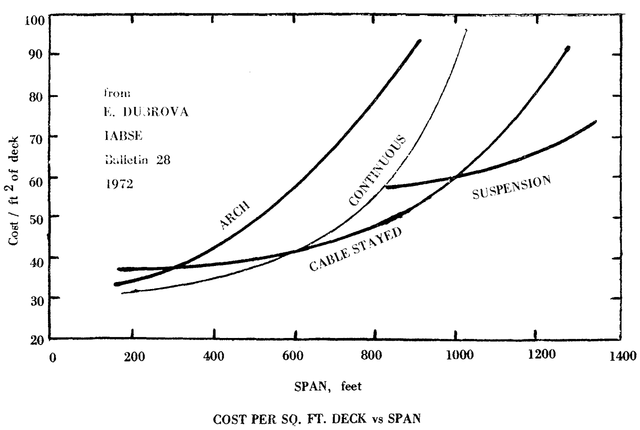

Fig. 26. Cost per sq ft deck vs span (from E. Dubrova, IABSE Bulletin 28, 1972)

Some interesting data has been presented by Dubrova[3] on the economics of various type bridge structures in the U.S.S.R. Dubrova's data for the steel arch, continuous girder, cable-stayed, and classical suspension bridges are plotted in Fig. 26. Taking note that economic comparisons are quite different in the United States or Canada than in the U.S.S.R., this data indicates that cable-stay structures fit in the span range of about 600 to 1,000 ft.[14]

Another economic comparison, weight of structural steel in lbs per sq ft of deck against center span in feet for orthotropic steel bridges, has been presented by Taylor[17] (Fig. 27). This study also indicates that cable-stayed bridges fill the void between continuous girder bridges and suspension bridges. Economic considerations in Europe are quite different than in North America, especially in regard to the ratio of material to labor costs. Taylor reports, however, that a study in Canada for highway bridges in the span range of 700 to 800 ft indicates that cable-stayed bridges are the most economical by a margin of 5 to 10 percent.

In Fig. 27, for cable-stayed bridges, the point plotted at an approximate main span of 1,050 ft and 115 psf is apparently based on data for the Kniebrücke Bridge at Düsseldorf (Fig. 11). However, this bridge is a single tower unsymmetrical bridge. If it were to be considered as one-half of a symmetrical two-tower structure with a center span of approximately 2,000 ft, then a different interpretation could be drawn from Taylor's curves[14] (Fig. 28).

There appears to be a reasonable optimism that cable-stayed bridges may, in the near future, penetrate into the range of spans dominated by suspension bridges. If the example of the "symmetric" Kniebrücke just discussed is considered, then it can be seen that a 2,000-ft main span may be feasible for cable-stayed bridges, and in fact has been considered in the United States. Leonhardt[10,11,12] has concluded that cable-stayed bridges are particularly suitable for spans in excess of 2,000 ft and may be employed for spans over 5,000 ft in the future.

The Louisiana Department of Highways, with the assistance of the Federal Highway Administration, proposes to construct a new segment of the Interstate Highway System known as the New Orleans By-Pass, I-410, around the New Orleans metropolitan area along an alignment which will necessitate two new Mississippi River crossings. These crossings are expected to be in the vicinity of Chalmette and Luling. Modjeski and Masters[9,21] in their feasibility study compared pure suspension, cable-stayed, and steel cantilever structures. The report was favorable toward cable-stayed and contemplated either 1,600-ft or 2,100-ft main spans.

Figure 29 is taken from Modjeski and Masters' feasibility study and indicates the total project cost. These curves permit a comparison of the three main bridge types, conventional suspension, steel cantilever, and cable-stayed for six-lane and four-lane bridges for this particular site.

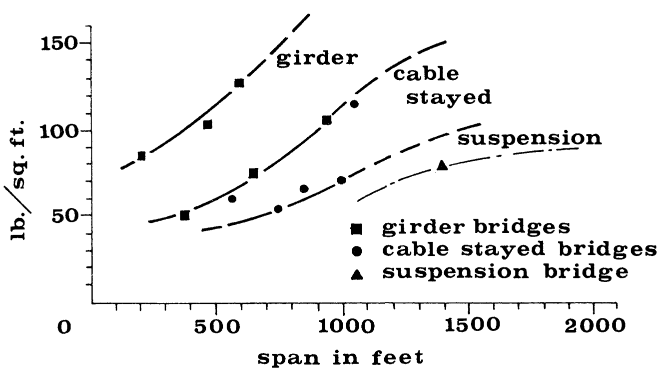

Fig. 27. Weight of structural steel in lbs/sq ft of deck for orthotropic steel bridges (Taylor)

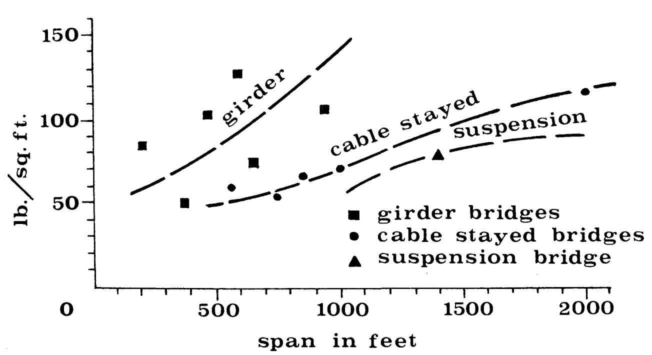

Fig. 28. Weight of structural steel in lbs/sq ft of deck for orthotropic steel bridges (Taylor, adjusted)

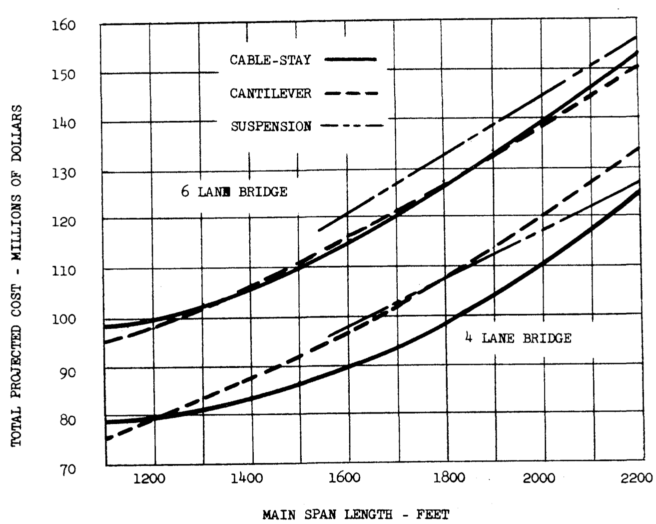

Fig. 29. Comparative construction cost estimates, I-410

It is to be noted that the total project cost for the six-lane suspension bridge is significantly higher than the cantilever or the cable-stayed. The total project cost is essentially the same for the six-lane cantilever and cable-stayed bridge. For the four-lane bridges, the cable-stayed bridge has a significant cost advantage compared to either the suspension bridge or the cantilever bridge.

The high cost of the suspension bridge as compared to the cantilever and cable-stayed is primarily due to substructure cost. These costs are much higher in the suspension system, due to the high costs for the cable anchorages and foundations, which are so much affected by the relatively poor foundation condition found in the Mississippi River Delta.

It is, therefore, impossible to arrive at any generalized conclusion relative to the comparison of the cantilever and cable-stayed bridges to the suspension bridge. However, because the transmission of load to the foundation for the cantilever and the cable-stayed are essentially equal in character, one can use this data as a generalization for a relative cost comparison for these two types of bridges.

SUMMARY

In recent years, effects of wind action and oscillation have been investigated very thoroughly for suspension bridges; such analyses are also required for cable-stayed bridges. Purely mathematical analysis of this problem is impractical at this time, so that model testing is used instead. It has been indicated that because of inherent system stiffness and damping, the cable-stay system is not as sensitive to wind oscillations as a conventional suspension system. Further, it has been indicated that vortex shedding and bending and torsional oscillation can be minimized with streamlined deck cross-sections. However, these conclusions are generalizations based upon data available from tests conducted on relatively few structures and it is felt that wind tunnel tests are essential for any given major cable-stayed structure.

As has been indicated, the cable-stayed bridge type is one that is certainly competitive with other type structures up to 2,000 ft spans and possibly larger spans. There is certainly the potential for this type of structure to become economically favorable in the long span ranges dominated by the suspension bridge. In the author's opinion, it is a foregone conclusion that as more designers and State Highway Departments become aware of the advantages and potential this type of structure has to offer, a greater number of cable-stayed bridges will appear on the American bridge construction scene.

The development of cable-stayed bridges in the United States has been hampered by the lack of familiarity on the part of designers and fabricators and by the lack of a code or specification to lay down guidelines. Therefore, there is a need for a tentative criteria, specification, or code which will present information, data, or guidelines as related to the present state of the art. Further, such a document must be able to be amended easily to accommodate the knowledge and information that will be forthcoming as more interest is generated in this topic.

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