SUSPENSION BRIDGE CABLES: 200 YEARS OF EMPIRICISM, ANALYSIS AND MANAGEMENT

Dr. Bojidar Yanev[1]

Abstract

Over the last 200 years suspension bridges have been at the forefront in all aspects of structural engineering, including empirical and theoretical studies, and construction and lifecycle management. Their spans have grown from 50 to 2000 m with designs for 3000 m under consideration. Key elements of modern suspension bridges are their cables. The evolution of suspension cables is examined. The most recent developments in their design, maintenance and inspection are described. Conclusions regarding suspension bridges and engineering structures in general are drawn.

Introduction: Empiricism and Theory

Engineering structures must succeed in three domains: the theoretical, the physical and the social. The operating tools of these domains are abstract analysis, empirical application and economics. Whether successful or not, the required synthesis is particularly spectacular in long-span bridges, among which the suspension ones are the undisputed champions. Suspension bridges using natural fiber ropes have existed since prehistoric times, but their modern history begins with iron chains. The first United States patent for a bridge suspended on iron chains was awarded to James Finley (1762 - 1839), a land-owning judge, in 1808. The patented bridges had a stiffened roadway, designed to carry pedestrians and horse carriages cost-competitively over spans up to 76 m. Judge Finley arrived at his modest but reliable bridges by experiments. His "empirico-inductive" understanding of the suspension structural scheme inspired him to forecast that "something further may be done in the art of bridge building than has yet been accomplished" (Karnakis, 1997, p. 53). Finley appears to have been fully aware of the need for a satisfactory analytical model.

Navier (1785 - 1836) formulated suspension bridge theory in his Memoire sur les Ponts Suspendus in 1823. The analysis, design and construction of suspension bridges in France is concisely and comprehensively reviewed in LCPC/SETRA, 1989. In 1827 Navier's extensively analyzed 170 m span at Pont des Invalides, barely completed, had to be dismantled, primarily due to the malfunctioning anchorages. Ever since, analysis and experiment have relentlessly challenged and stimulated the art and science of bridge building. Marc Seguin (1786 - 1875) complained that Navier treated "theoretical notions or mathematical solutions [as] *a priori* admissible" (Karnakis, 1997, p. 269). Nonetheless, further developments, including Seguin's, owe much to the theoretical backing of Navier's Memoire.

---

[1]Executive Director, Bridge Inspection & Management, New York City DOT, Adjunct Professor, Columbia University

In the American Railroad and Mechanics Magazine of April 1, 1841, No. 379, Vol. XII, J. Roebling (1806-1869) argued that "the successful introduction of cable bridges into the United States would require the combination of "scientific knowledge and practical judgment of the most eminent Engineers". Roebling proved singularly capable of providing both, along with the organizational abilities required for manufacturing the high strength galvanized cable wires and the procurement of the financial backing his monumental bridges needed. Thus his Brooklyn Bridge in New York City (main span 487 m), completed in 1883 by his son and daughter in law, Washington and Emily Roebling, is carrying more than 80,000 passengers daily in 2009. Billington (1983) concluded that the integration of theory and practice achieved by Roebling and Eiffel (1832 - 1923) elevated structural engineering into an art. One disadvantage of great artistic accomplishments, however is that they do not lend themselves to standardization and mass production. Engineering practice, in contrast, must deliver utility repeatedly, reliably and cost-competitively. This apparent contradiction is demonstrable in bridge design where spans shorter than 150 m are subject to detailed specifications by AASHTO (American Association of State Highway and Transportation Officials), but the longer ones are not. Neither are cable-supported bridges of any length. Thus in the domain of the foremost structural accomplishments, failures and successes jointly advance the state of the art.

Failure: the Ultimate Test

Despite the advances in abstract analysis and controlled testing, failures have the most conspicuous influence on bridge design, construction and management. No planned engineering experiment can match the social and professional impact of an unintended structural collapse. The shock value of the suddenly perceived ignorance and the usually considerable losses add up to a lesson easy to understand and remember by non-professionals. Bridge failures in the United States from 1966 to 2005 are attributed to various respective causes in Fig. 1. Natural hazards (and mostly hydraulic-related ones) dominate this listing as they do in most classifications.
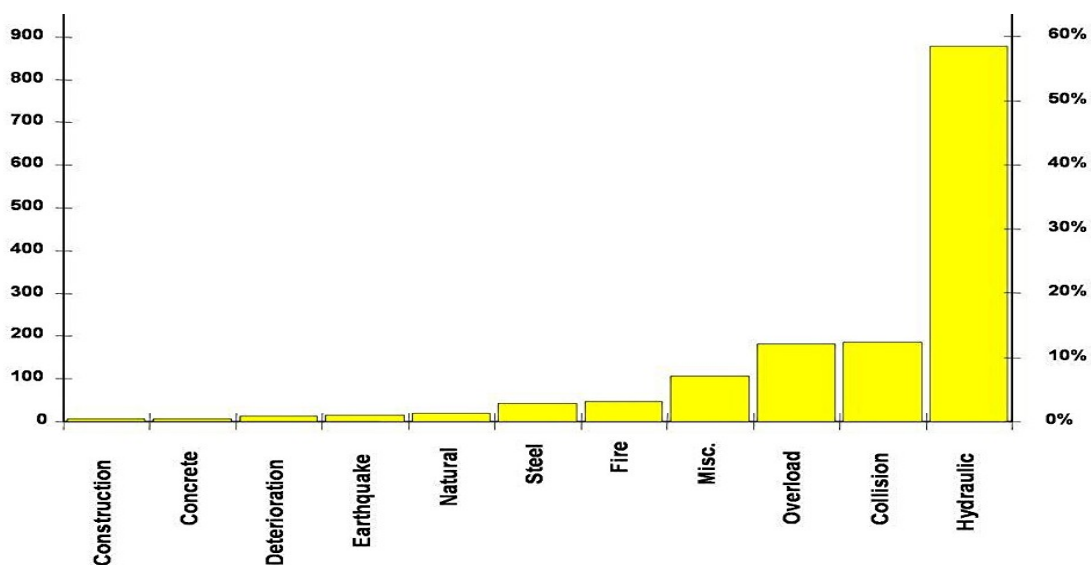

Figure 1. 1502 bridge failures in the USA, 1966 – 2005 (courtesy STV Engineers)

Suspension bridges represent a small portion of the total bridge population, however they are essential to transportation, and at the forefront of engineering innovation. Consequently their failures are critically important not only quantitatively but also qualitatively. H. Petroski (1993) regards failures are inherent in the creative process of bridge design and construction. Citing earlier work by Sibly and Walker in England, he argues that each innovative bridge form is developed by trial and error until its limits are surpassed and spectacular failure occurs. Only then does theory catch up with the practice and fully explains the structural behavior. By this estimate, the still evolving cable-stayed bridges should be viewed with particular concern. A few popular examples may add a historic perspective to this controversial subject.

19th Century cable-supported bridges

Nineteenth Century engineers experimented with both suspension and cable-stay bridges using both ropes and chains. The more complex dynamic behavior of these structures resulted in greater lapses in the designer's knowledge, and hence, the many failures under wind loads, as well as those caused by rushing crowds.

The cable-stayed bridge at Dryburgh collapsed in a gale storm in 1818, 6 months after it was completed. Samuel Brown's Brighton Chain Pier was destroyed by a storm in 1836. The landmark bridge crossing the Menai Straights in England with a main span of 177 m was designed by Thomas Telford (1757 - 1854) and completed in 1826. The bridge chain suspension system never failed, but high winds caused significant damage and repairs were necessary at least twice.

With a main span of 308 m, Charles Ellet's (1810 - 1862) Wheeling suspension bridge over the Ohio River was the world's longest. Ellet had to repair it after it was destroyed by wind in 1854. Roebling later replaced it with his hybrid suspension/stay system. That system created enduring dynamically stable structures without a rigorous theoretical backing. Theoretical advances in the 1920s enhanced suspension bridge analysis. Roebling's semi-empirical hybrid system was stripped of the diagonal stays and the stiffening trusses, ultimately producing the aerodynamically unstable Tacoma bridge.

Tacoma Bridge (1940)

The bridge, in Washington State, was designed by the early proponent of large displacement theory Leon Moiseiff (1872 – 1943), renowned for his contributions at Manhattan Bridge in New York and the Golden Gate in San Francisco. Distinguished by a relatively narrow cross section and longitudinal stiffening girders in lieu of the traditional trusses, it collapsed under wind loads after months in service. The failure is attributed to ignorance of the dynamic phenomenon later recognized as flutter. Yet flutter had already been identified by Von Karman (1940) in research on aircraft wing stability. Thus the ignorance could be perceived as design error. The new Tacoma suspension bridge has a deep stiffening truss. In 1937 Othmar Ammann (1879 – 1966) won an award for the design of the Whitestone Bridge in New York City. Following the collapse of the Tacoma, the very similar Whitestone was stabilized with stiffening

trusses, diagonal stays and a tuned mass damper mid-span. None of these enhancements proved conclusive. They have been removed or are reassessed during a current structural reconfiguration.

Silver Bridge (1967)

The bridge over the Ohio River at Point Pleasant collapsed due to a brittle fracture of an eye-bar of the suspension system. Poor quality control during construction, inability to inspect, ignorance of fatigue behavior and lack of redundancy all contributed to a degree that is still debated. The variety of contributing causes entailed similarly wide ranging responses. The non-redundant 2-eye-bar suspension chain was entirely discredited, leading to the controlled demolition of a similar structure over the Ohio River at St. Mary's, West Virginia. Impressed by the collapse, which took 46 lives, the United States Senate mandated biennial bridge inspections, clearly recognizing that the lack thereof is a management deficiency. The outcome was the National Bridge Inventory (NBI) which currently contains data on more than 600,000 vehicular bridges, 200,000 railroad bridges and more than 100,000 culverts. Fracture-critical details became an important part of the inventory and special provisions were designed for their inspections.

Partial failures

Partial failures, particularly when they appearas near misses are easily overlooked and their lessons are harder to learn. For example, a database developed at Cambridge University blames deterioration for only 3% of the considered bridge failures. Yet deterioration is recognized as widespread. The losses it has caused are vast but not quantifiable, because they rarely include fatalities. As a result, deterioration-related failures are on the rise. They can be traced to neglected maintenance, as well as to the design which did not anticipate such neglect. Since deterioration is a relatively slow process (compared to natural hazards, fractures and losses of stability for example), its effects have been intercepted by inspections and arrested by rehabilitations. Thus partial failures have become increasingly significant. Both Pont de Tancarville and Pont d'Aquitaine in France had their suspension cables replaced without incident following the discovery of breaks caused by corrosion. The original cables had no wrapping and consisted of helical non-galvanized strands. The 4 cables of the Williamsburg Bridge (1903, 488 m main span) in New York City were rehabilitated in the 1990s after a long debate over their viability. The cables, each consisting of 7696 non-galvanized parallel 5 mm wires grouped in 37 strands, were rapidly corroding. During the rehabilitation several strands were re-anchored, many broken wires were spliced, corrosion-inhibiting oil was added and water-proofing was supplied in the form of new wrapping.

Many suspenders and stay-cables have broken, however the effect of these failures has been localized by redundancy. In 1981 one of the diagonal stays of the Brooklyn Bridge broke due to corrosion and killed a pedestrian. All suspenders and stays were subsequently replaced. A suspender broke at the first Bosporus Bridge in 2004 and a stay cable was burned by lightning at the Rion – Antirion Bridge in 2006,

neither one causing significant traffic interruptions. Suspender and stay replacement is recognized as a periodic necessity. The suspenders of Manhattan Bridge have been replaced at least once and will be replaced again under a pending contract.

Overview of failures

Structural failures in general do not lend themselves to purely quantitative assessments. To the inability to quantify loss of life and other user costs, there are difficulties with evaluating the loss of public and professional confidence.   Forensic investigations are post-event efforts to eliminate failure causes. Once identified such causes are perceived as vulnerabilities requiring special treatment. The identification and elimination of vulnerabilities is a perpetual task of management depending on every specific circumstance, however some common characteristics have emerged (also discussed in Yanev, 2007). Failure causes have been classified according to a variety of criteria including the following:

- Event-based
The classifications of the Cambridge database and Fig. 1, cited earlier are of this type, however they also refer to the structural type and material.

- By the mode of material non-performance
This classification would identify metal fatigue and fracture, corrosion, corrosion fatigue, ductile failure, residual stress, yield, shear, concrete fatigue, chemical reactivity, temperature, and so on. Although helpful, this classification misses certain vulnerabilities, such as instability. Thus another classification becomes necessary, distinguishing between local and global behavior. Whereas a material failure is by definition local and can lead to global consequences, instability is a global failure which causes local material non-performance.

- "As designed" and "not as designed" modes (Thoft-Christensen and Baker, 1982)
The failure of the deck truss bridge carrying Rte. I-35-W in Minneapolis on Aug. 1, 2007, caused by overstress of poorly dimensioned gusset plates, is in the "as designed" mode. These are practical failures resulting from poor execution. The failure of the Tacoma bridge, caused by the hitherto unknown flutter is in the "not as designed" mode. Such failures point o theoretical deficiencies.

- According to the bridge life-cycle, e.g. design, construction, maintenance, and operation during which the failure occurs (Yanev, 2007)
This approach reveals that failures occur when each phase excessively relies on the others, whereas it should have assumed responsibility as if it were critical. For example after the collapse of the bridge over the Schoharie Creek (New York State, 1987) caused by scour, it was concluded that the bridge could have survived if underwater inspections had been more effective, however the collapse could have also been prevented by selecting a longer span with piers out of the channel.

- No failure classification can be entirely independent, and neither are the revealed vulnerabilities. A redundant approach to identifying vulnerabilities by various

partially overlapping criteria has a better chance of capturing all possibilities. New York State for example has established screening procedures for the following vulnerabilities: hydraulic, seismic, overload, steel details, concrete details, collision, sabotage.

- Significant failures are usually caused by combinations of two or more vulnerabilities. In all of the preceding examples, design, construction and maintenance might have intervened to avert the disaster, if they had seen their own role as the most critical. The investigation following the collapse of the Sungsu truss bridge in Seoul in 1994 concluded that the bridge had been "poorly designed, built, maintained and used".

- Significant failures invariably exhibit a discontinuity in the process, in the product or in both. A common one is between experience and theory. This can be expressed as an over-reliance of each on the other or as an ignorance of each about the other. For example, the rationale justifying the use of non-galvanized high strength cable wires on the Williamsburg Bridge was summarized as follows: "If the wires are maintained dry, they do not need zinc coating. If not, it will not save them in the long term." This reasoning may appear sound, yet it lacks both the practical knowledge of life-cycle cable performance and a theoretical model of wire deterioration. The opposite solution would have been correct: wires should have been locally protected by galvanization, whereas the cable should have been globally protected by superior wrapping. As in all discontinuities, the remedy is redundancy.

- Failures can be traced to deficiencies in both the engineered product and the managed process. Thus the gap between theory and practice is matched by an equally detrimental gap between the increasingly diverging professions of engineering and management. The current trend is to rely on engineering competence during the design and construction of the product (in this case the bridge) and to assign the process of its operation to a management, guided primarily by economic considerations. The latter typically minimize initial costs, while shortening the structural life-cycles and increasing the demand for future expenditures.

- Most simplistic and yet inevitable are the failure assessments according to the amount of damage and the responsibility. All structural failures are to some extent attributable to management and this is reflected in the way society reacts to them. The failure of the Silver Bridge was at the origin of the NBI and the biennial bridge inspection program, the partial failure of the Williamsburg bridge led to the re-establishment of the Bridge Division at the New York City Department of Transportation, the numerous failures caused by earthquakes in California and in Japan have influenced the design and construction of bridges worldwide, the collapse of the I-35-W bridge in Minneapolis has stimulated the use of non-destructive structural monitoring techniques. Appropriate as these measures are, they remain reactive, whereas the purpose of engineering management and design is to anticipate. The history of suspension bridge cables offers examples of such anticipation.

Suspension bridge cables

The temporary closure, in 1988, and subsequent rehabilitation of the Williamsburg Bridge attracted much attention to the condition of suspension cables. (That project is currently concluding at a total cost of approximately $US 1 billion.) The affected structures are among the oldest, largest and historically most significant nationwide. The Cincinnati - Covington Bridge was opened to traffic in 1867, the Brooklyn Bridge - in 1883, Williamsburg - in 1903 and Manhattan - in 1908. Ten major suspension bridges, including Ammann's George Washington (1931 and 1957, 1068 m main span), Verrazano (1964, 1300 m main span), Triborough, Whitestone, Throg's Neck and the East River bridges are in the New York Metropolitan area. During the 1990s the author co-sponsored a comparative survey of these structures along with all respective owners. The resulting report by Columbia University summarized cable conditions, design, construction and maintenance practices. Key findings were presented by Betti and Yanev (1999).

The National Cooperative Highway Research Program (NCHRP) at the Transportation Research Board (TRB) expanded the study to all parallel wire cable-supported bridges nationwide and published NCHRP Report 534 by Mayrbaurl and Camo (2004) of Weidlinger Assoc. The report addressed twenty nine suspension bridges, constructed in North America by the aerial spinning method up to the year 2000 and two (Newport, R.I. and William Preston Lane Jr., MA.) built with shop-fabricated parallel wire strands. The twenty one (shorter) spans supported by helical strand cables were not considered in this study. NCHRP Report 534 recommended further investigation of the behavior of cables under controlled and actual field conditions.

Wires: condition, deterioration and failure

The replacement of suspension chains and eye-bars with parallel wires or helical strands radically improved the *internal redundancy* of suspension cables. The wires in the investigated cables (Williamsburg excepted) are galvanized, have an approximately 5 mm diameter and their strength is around 1515 mPa. In a load-free state, they assume a curvature with roughly 1 m diameter, which indicates their state of bending under working conditions. The non-galvanized wires of the Williamsburg Bridge failed when corrosion reduced their cross-section (Fig. 2 - A and - B). This mode of wire failure is atypical, as well as relatively simple and has not attracted much interest. Failures of non-galvanized helical strands have been reported in detail by Virlogeux (1999) and Kretz et al. (2006), along with the ensuing cable replacements. In contrast, high-strength galvanized wires have proven susceptible to "flat and invert" breaks, and the rarer spiraling breaks (Mayrbaurl and Camo, 2004). Flat breaks are believed to develop as follows:

- The approximately 0.05 mm zinc coating oxidizes and fails over a small areas of the wire surface.
- The exposed steel begins to oxidize, causing surface irregularities, such as pitting and, consequently, stress concentration.

- Cracks develop transversely to the wire surface, further concentrating the stress.
- The cracks propagate at an angle towards the center of the wire until the area is critically reduced and the wire breaks normally to the axis. The sequence raises the following critical questions:

- *Quantifying and qualifying wire corrosion.*
      The strength evaluation of a cable is based on visual inspections which, in turn must estimate the state of corrosion. To facilitate these estimates, 4 stages of wire corrosion were described by Hopwood and Havens (1984) as follows:
Stage 1 - Spots of zinc oxidation on the wires;
Stage 2 - Zinc oxidation on the entire wire surface;
Stage 3 - Spots of brown rust covering up to 30% of the surface of a 3 to 6 inch
         (75 to 150 mm) length of wire;
Stage 4 - Brown rust covering more than 30% of the surface of the 3 to 6 inch length of wire.

Figure 2. Suspension cable wires, z-shaped wrapping wire, and 127 wire strand

      Despite the lack of phenomenological backing, this rating system endures, as do all visual inspections. NCHRP Report 534 proposed a model linking the visual findings, classified in the 4 - stage system to an estimate of the number of cracked and broken wires, and ultimately, the cable strength.

- *Does the oxidized zinc contribute the embrittlement of the steel?*
      Mayrbaurl and Camo (2004) point out that much depends on the further reactions of the resulting zinc oxide (ZnO). Those, in turn depend on the environment and, particularly on the type of humidity. Zinc carbonate ($ZnCO_3$) can form an effective protective film, whereas zinc hydroxide ($Zn(OH)_2$) easily dissolves, leading to the formation of carbonic acid (H_2CO_3), which becomes a source of embrittling hydrogen.

- Is there a threshold level of zinc depletion and steel corrosion beyond which cracks begin to occur spontaneously?

To some extent that would depend on the level of stress in the "as-built" cable. Modeling that stress however, besides relying on the uncertain state of the unstressed wires, also depends on their stress level, subject to diverse uncertainties.

Wire stresses

Suspension cable wires are not in uniaxial tension as is generally assumed in calculating safety factors. High-strength wires are manufactured by the cold-drawn method by extruding them through progressively smaller openings and thus modifying the molecular shape of the original mild steel. Eventually the wires of the desired diameter (roughly 5.1 mm) are dipped in molten zinc for protection against corrosion. The galvanized wires cool off in a permanently curved shape with a diameter of approximately 2 m. Consequently, the wires experience bending stresses in order to conform to the shape of the cable. Mayrbaurl and Camo (2004) estimate (based on X-ray diffraction tests) that the straightening produces bending stresses of up to $\pm$ 240 mPa (36 ksi). Eliminating a curvature with radius R induces in a wire with radius r_{wire} bending moment M as follows:

$$M = E\,I\,/\,R \tag{1}$$

The corresponding maximum bending stress σ is:

$$\sigma \;=\; M\,/\,S = E\,r_{wire}\,/\,R \tag{2}$$

where: $I = \pi\,r_{wire}{}^{4}\,/\,4$ is the section moment of inertia.

$S = I\,/\,r_{wire}$ is the section modulus.

For $r_{wire} = 5$ mm and $R = 2000$ mm in Eq. (2), $\sigma\ = 500$ mPa (75 ksi), matching the uniform working stress level for which many suspension cables are designed. Even after many years of service, suspension bridge wires extracted from a cable regain some of their original curvature, testifying to inelastic deformation. Wires invariably crack on the side where straightening has produced tension. Bending is extreme in the traditional anchorages which use sheaves of relatively small diameters. This has been a primary reason for the recent shift, lead by Japan, to prefabricated straight wire strands, originally developed under a U.S. patent (Fig. 2 – F).

Residual stresses

Residual stresses would be caused by the extrusion process and by the hot-dipping into zinc. The steel surface under the zinc is irregular. Therefore stress concentration and even cracks may exist in the wires before the zinc coating fails.

From wires to a cable: condition, deterioration, maintenance and repair

The knowledge gained from individual wires must be fully used in evaluating the condition of suspension cables. Whereas wire failures have provided understanding of the limits of the material, a cable failure model does not exist and allowing for one to develop spontaneously is not acceptable. Consequently,

physical and statistical, deterministic and probabilistic, analytic and empirical methods are comtinually developed and combined. All stages of the structural life are closely examined.

Design and construction
  Over the 20th Century parallel wire suspension cables (Fig. 3) evolved as follows:

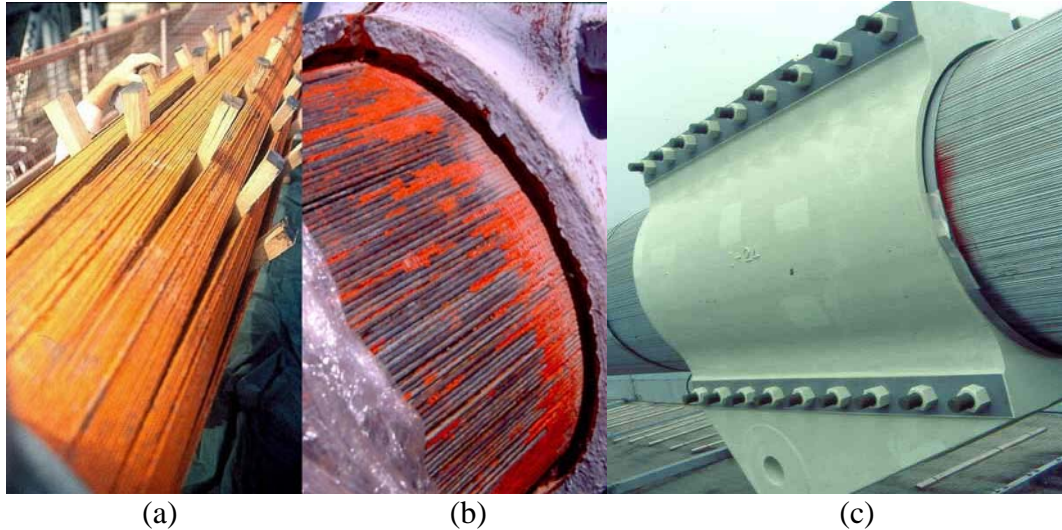| (a) | (b) | (c) |

Figure 3. Cables: (a) wedging; (b) cable band with red lead;   (c) Akashi – Kaikyo

        The number of suspension cables evolved from 2 to 4 and back to 2. Examples of the 4 cable configuration include the East River bridges, the George Washington and the Verrazano in New York City. Notable among the 2 cable bridges are the Ambassador, Macinac (1,158 m), Bay Bridge and Golden Gate (1,280 m). Four cables were originally contemplated at the Akashi-Kaikyo, however the 2 cable cross-section was selected because of its superior aerodynamic properties. The proposed 3,000 m span at the Messina Straights assumes 4 cables. The *global redundancy* of the 4 cable configuration may have saved some bridges from demolition. For his bridge at Oporto, D. Steinman anticipated a second pair of cables carrying added traffic. That modification was completed in the 1990s. At Pont de Tancarville the two original cables were replaced by two pairs, allowing for a future replacement of each pair by a single cable at the original location (Virlogeux, 1999). The two cables of the Storabaelt Bridge in Denmark (1996, main span 1,624 m) were built by the traditional air-spinning method with 18,648 wires of diameter 5.37 mm and minimum strength of 1,570 N/mm^2 (Fig. 2 - C). The Akashi – Kaikyo (1998, main span 1991 m) cables are built with shop-fabricated parallel wire strands, composed of 127 straight galvanized wires with yield up to 1800 N/mm^2 (Figs. 2 - D and F, Fig. 3 - c). Each of the two cables have 290 hexagonal strands (Figs. 2 – F and 3 - c).

Suspenders
         Once spaced at roughly 7 m, suspenders are now spaced at about 20 m. The increased distance between suspenders affects primarily their own behavior as well as that of the bridge superstructure. The cables, however are also influenced in at least two ways. The cable bands which improve the behavior of broken wires and the

compaction of the cable are reduced. Consequently they transmit greater concentrated loads to the wires. As the distance between the cable bands increases so does the bending moment introduced by them into the cable.

The traditional corrosion protection of suspenders has been galvanization and paint. Stays are protected in various encasements, primarily cement grout. The limitation of the latter method, however have prompted Japanese designers to use parallel wire suspenders and stays with rubber vulcanization, as at the Akashi - Kaikyo and Tatara Bridges.

Wind loads

The potential for aerodynamic instability demonstrated by Tacoma and other suspension bridges led to extensive testing of models of all major suspension and cable-stayed bridges in wind tunnels. For the Akashi – Kaikyo Bridge, a 1/100 scale model was tested in a wind tunnel built expressly for that purpose.

The inclined suspenders used on many bridges in Europe, including the record-breaking Humber in Wales (1981, 1,410 m main span) proved even more vulnerable to wind loads and fatigue. After relatively short useful lives, such suspenders were replaced at the Severn and Brotonne bridges while their sockets were modified. Water on the surface of stays was found to modify their aerodynamic response, causing the so-called galloping oscillations. The phenomenon is mitigated by surface obstructions to running water. Dampers are added on longer suspenders and stays.

Clamping length

 The overall stress in cables is estimated by cutting wires and measuring their retraction. That retraction, however is constrained by the friction with adjacent wires and is therefore limited to the so-called *clamping length* of the wires. The clamping length is very important in estimating the contribution of broken wires at longer distances from the fracture points. It is often assumed that tension in a broken wire is restored over the distance of 3 cable bands. Apart from the hypothetical nature of that assumption, it clearly depends on the distance between cable bands and on the level of compaction of the cable. Thus the local condition of the wires must be assessed jointly with the global condition of their totality as a cable.

Safety factor

This term is somewhat discredited, because of the uncertain assessments of both stress and resistance levels discussed in the preceding section. Nonetheless, the ratio between the ultimate load of the cables at yield and the maximum expected load during its service has declined from over 4 to 2.2. It is argued that 90% of all loads on long-span bridges consist of the relatively constant dead load. Furthermore, the effective stiffness of the cables increases proportionally to the cube of the stress divided by the square of the span length. The formula, attributed to Tischinger and to Ernst, can be written as in Eq. 3.

$$1/E_{ef} = 1/E + (\gamma L)^2 / (12\ \sigma^3) \tag{3}$$

where: E_{ef} - effective modulus of elasticity of the cable;
$\quad\quad\quad E$ - modulus of elasticity of the steel;
$\quad\quad\quad L$ - span length;
$\quad\quad\quad \gamma$ - specific weight of the cable;
$\quad\quad\quad \sigma$ - stress in the cable.

Thus longer spans achieve the desired stiffness at the expense of higher stresses in the wires, implying an increased likelihood for "stress-corrosion" and hence a lower tolerance for deterioration.

Cable protection

Cable protection once consisted of linseed oil introduced into the cable voids, a red lead paste coating over the wires, plastic wrapping over the lead, spiral wire wrapping on top and paint over the spiral wire (Fig. 3 – a and b). Because of environmental objections, lead is being substituted by zinc paste. The long-term effects of this modification are yet to be observed, because lead is passive whereas zinc oxidizes. The benefits from the spiral wire wrapping have been disputed and it has been replaced by polyethylene wrapping at some bridges.

On several record-breaking bridges in Japan, such as the Kurushima and Akashi-Kaikyo, managed by the Honshu-Shikoku Bridge Authority, pressurized dry air is injected under the cable wrapping. The system is now under consideration as a rehabilitation feature on older bridges. Z-shaped wires have been used for wrapping, as in Fig. 2 - E.

Compaction

Compaction is a form of cable protection, because it impedes the penetration of water and increases the clamping effect. Air-spinning achieves a relatively uneven compaction of 75%-80%. In a perfectly compact cable all wires except the external ones are in contact with 6 adjacent wires, forming a hexagon. For T perfectly compacted concentric layers, the net wire to gross cable ratio of areas can be computed as in Eq. 4.

$$2\pi\, r^2\, [3T(T + 1) + 1]\, /\, r^2\, [(2T + 1)^2\, 3^{3/2}]\, \approx\, 1.211\ x\ 3/4\, \approx\, 0.907 \quad\quad\quad (4)$$

In cables built by the traditional air-spinning method 80% compaction is realistic, but variations are quite broad.

Anchorages

The anchorages of air-spun cables (Fig. 4 - a) feature three critical transitions:
- From a compacted cable to splaying strands. In this area wires no longer have the benefit if clamping effects and are fully exposed to humidity. As a result entire strands have been lost to corrosion in anchorages, and re-anchorings have been necessary (Fig. 4 - a).
- From strands to eye-bars. The bending over the pins of the eye-bars can cause yield in the wires, but few breaks have been noted in these areas.

- From exposed eye-bars to concrete encasement. Eye-bars corroded significantly at that juncture and have been replaced by new anchoring systems, for example at Manhattan Bridge. Most anchorages (including old ones) are being equipped with de-humidification systems.

| (a) | (b) |

Figure 4. Anchorages (a) air-spun cable with re-anchoring; (b) prefabricated cable

Uncertainty

For engineering purposes uncertainty has been separated by ISO (1995) into randomness (for example of natural phenomena), vagueness (as in condition evaluation) and ignorance (of actual conditions). Ang and De Leon (2005) recognize aleatory and epistemic uncertainties, associated with random natural variables and deterministic risk-informed decisions, respectively. In either classification, all types of uncertainties are present in bridge cables. Report NCHRP 534 treats probabilistically the key parameters of the cable model as follows:

Wire condition

Acoustic emission has been used to detect wire breaks of cables in use. The labor-intensive and intrusive unwrapping and wedging (Fig. 3 - a) remains the only reliable source of information about the condition of the wires. NCHRP 534 proposes methods of projecting the number of wires in each of the four states, based on the limited findings of such inspections. The extrapolations depend on the sizes of the cables and the sample, and the observed conditions.

Cable strength

NCHRP 534 proposes three models as follows:

Ductile wire. Strain increases incrementally. A wire reaching yield carries the corresponding stress until the rest of the wires reach that point and they all fail simultaneously. The cable strength is the average strength of the wires multiplied by their number. This model has a limited application to cables (possibly such as those of the Williamsburg) where loss of section, rather than cracking is critical.

Brittle wire. Each wire fails at its tensile strength limit. A step-wise stress-strain diagram results, as the number of wires declines. The model tends to underestimate the cable strength by as much as 20%.

Limited ductility model. This more elaborate model recognizes that when the first wire breaks, the force in the remaining wires does not change because the change in the overall strain is negligible. As the ratio of broken to active wires grows, so does the *strain rate*. The process is dynamic.

Health monitoring

Wind tunnel testing has become standard practice on all suspension and cable-stay bridges, however scale models cannot fully simulate the actual structural response. Consequently, dynamic analysis must follow a redundant path, seeking acceptable convergence of analytic and experimental results, as has been the case since the origin of bridge building. The process does not stop with the completion of construction. Monitoring systems collect "real-time" data including dynamic response, stresses, temperatures, traffic weigh in motion, wind velocity.

Figure 5. Cable model at Columbia University

New suspension bridges are heavily instrumented with various devices, measuring acceleration, vehicular weight, wind speed, temperature and other environmental and structural parameters. Global positioning systems (GPS) can monitor the movements of large structures with a satisfactory accuracy. Recently Manhattan Bridge was scanned with interferometric radar by IDS Ingeneria of Pisa, Italy and with GPS by Columbia University with closely agreeing results. The time

has come to introduce state of the art instrumentation into the cables and to employ it for their continuous health monitoring. Under a current project sponsored by FHWA, Columbia University researchers are investigating the methods for non-invasive monitoring of cable condition. The research encompasses the testing of a cable model (Fig. 5) under controlled conditions and transferring the monitoring technology to a suspension bridge for field verification. The cable model is instrumented with 76 sensors of different types, placed along three diameters, inclined at 60° angle with respect to each other, so to have a dataset that represents the central conditions for the 6 sectors in which the cross-section is typically subdivided. These sensors measure temperature, relative humidity, pH and corrosion rate. The sensor allocation is shown in Fig. 6. The 16 temperature/relative humidity sensors were evenly spaced along the three diameters in order to have measurements for the temperature and humidity variations inside the entire cross-section. Corrosion rate sensors used different principles to measure corrosion rates: some use Linear Polarization Resistance, some others are Bi-metallic and some others, like the Couple Multiple Array Sensors, measure the number of electrons that flow from anodic to cathodic zones.
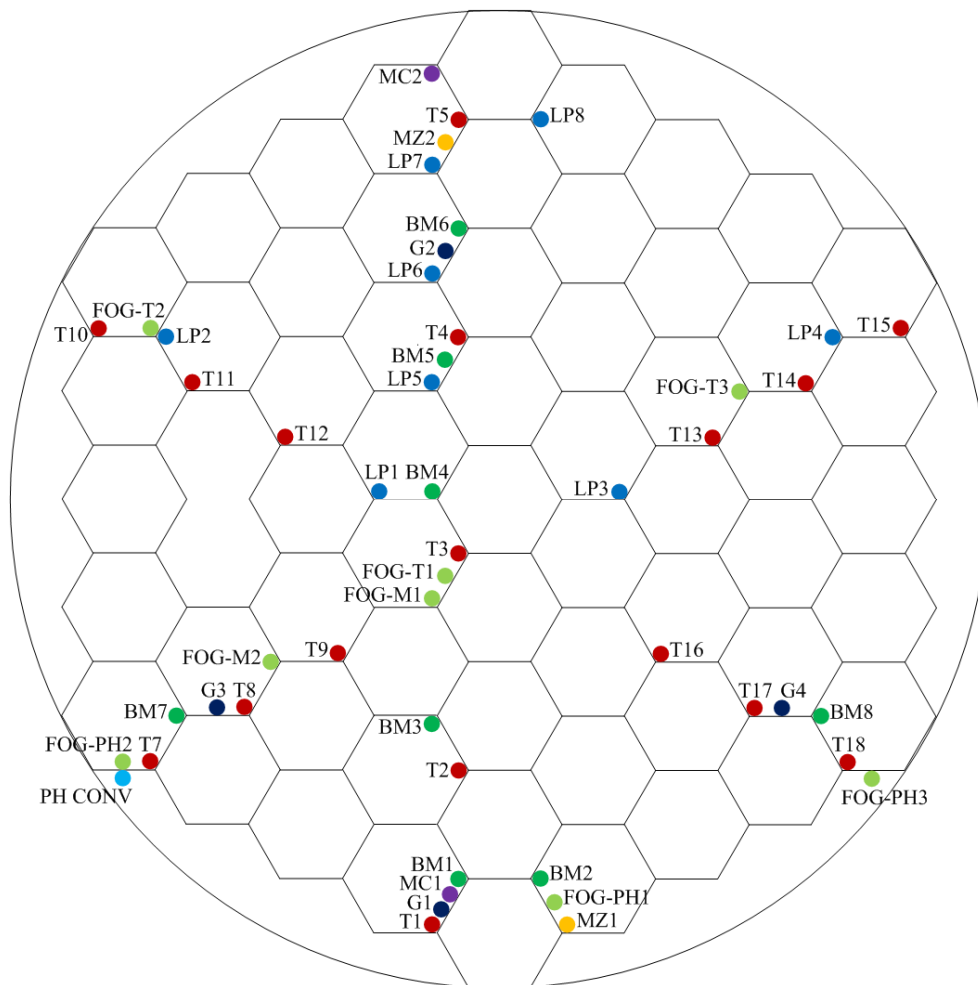

Figure 6. Sensor allocation in the cable model

The objective is to identify methods suitable for monitoring the condition of cables without the need to unwrap and wedge them. The instruments selected during the laboratory testing phase will be installed in one of the cables of Manhattan Bridge for field testing in 2010. Thus the design, construction and service of suspension bridges reaches for new levels of theory and application. Once again theory must catch up with practice.

Acknowledgment

References

Ang, A. H.-S. and D. De Leon (2005) Modeling and Analysis of Uncertainties for Risk-Informed Decisions in Infrastructure Engineering, *Structure and Infrastructure Engineering*, Vol. 1, No. 1, pp. 19 - 31.

Betti, R. and B. Yanev (1999) Condition of Suspension Bridge Cables, New York Case Study, Paper No. 99-0603, *Transportation Research Record 1654*, National Academy Press, Washington, D.C., pp. 105 - 112.

Betti, R. et al. (2009) Aging Cables in Suspension Bridges, Proceedings, Aging Infrastructure Workshop, Dept. of Homeland Security, New York.

Billington, D. P. (1983) **The Tower and the Bridge**, Basic Books, New York.

Hopwood, T. and J. H. Havens (1984) Corrosion of Cable Suspension Bridges, Kentucky Transportation Research Program, Univ. of Kentucky, Lexington.

ISO (1995) **Guide to the Expression of Uncertainty in Measurement**, 2nd Ed., International Organization of Standardization, Geneva.

Karnakis, E. (1997), **Constructing a Bridge**, MIT Press, Cambridge, Mass., USA.

LCPC/SETRA (1989) Les Ponts Suspendus en France, Laboratoire Central des Ponts et Chaussees, Paris / Service d'Etudes Techniques des Routes et Autoroutes, Bagneux, France.

Kretz, T. et al. (2006) Rehabilitation and Widening of the Aquitaine Bridge, Conference on Operation of Large Infrastructure Projects, Bridges and Tunnels, IABSE, Copenhagen.

Mayrbaurl, R and S. Camo (2004) Guidelines for Inspection and Strength Evaluation of Suspension Bridge Parallel-Wire Cables, NCHRP Report 534, TRB, Washington, D.C.

Petroski, H. (1993) Predicting Disaster, *American Scientist*, Vol. 81, Mar. – Apr., pp. 110 - 113.

Thoft-Christensen, P. and M. J. Baker (1982) **Structural Reliability Theory and Its Applications**, Springer - Verlag, Berlin.

Virlogeux, M. (1999) Replacement of the Suspension System of the Tancarville Bridge, *Transportation Research Record 1654*, National Academy Press, Washington, D.C., pp. 113 - 120.

Von Karman, T. and M. A. Biot (1940) **Mathematical Methods in Engineering**, McGraw-Hill, New York.

Yanev, B. (2007) **Bridge Management**, John Wiley & Sons, Inc, Hoboken, N.J.