Chapter 1

Suspension Bridges

An Overview

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1.1 FORM AND MATERIAL

All viable designs of human intelligence have natural analogs. In the fourth century B.C., Aristotle, learning from nature, identified the necessary ingredients in the life cycle of a successful structure as material, plan, execution, and service. Suspension bridges stretch all of these ingredients to their allowable limits. Appropriately, they were to take firm hold among the greatest structural achievements several millennia after their monumental predecessors made of timber, stone, and other naturally found materials.

Waddell (1916) illustrated the first volume of his *Bridge Engineering* with a timeless monkey bridge. The first suspension structures were natural fiber links. Nevertheless, suspension bridges could be engineered to Aristotle's specifications only under the new equilibrium of production and commerce introduced by the Industrial Revolution (Billington, 1983). Since that time, suspension bridges and their creators have consistently extended each other's physical reach and mental grasp. Functioning at the outer boundaries of performance and intelligence, the two must integrate seamlessly process and product, form and material, design and construction, theory and empiricism.

In his remarks on the comparative merits of cable and chain bridges (1841), John Roebling (1806–1869) cautions that "to ensure the successful introduction of cable bridges into the United States, their erection, and especially the construction of the first specimen, should not be left to mere mechanics." He seeks to integrate "the practical judgment" of "the most eminent Engineers" and the "rich store of scientific knowledge." Over the ensuing 172 years these two ingredients of the constructive process cooperated and competed in a "creative tension" (Billington, 1983, p. 52), producing the form and material of the longest tensile structures.

1.2 ART AND SCIENCE

Billington (1981) views the engineering of suspension bridges as a synthesis of art and science where form occasionally guided function. John Roebling and the Brooklyn Bridge serve as his primary examples. Kranakis (1997) analyzes the development of suspension bridges as a two-prong theoretical and empirical advance. Claude-Louis Navier (1785–1836) and his *Memoire sur les ponts suspendus* (1823) prominently represent the theoretical school. The bridges of John Finley (1762–1828), Thomas Telford (1757–1834), and Isambard Kingdom Brunel (1806–1859) exemplify the empirical approach.

The Maryland-born landowner and judge John Finlay obtained the first patent for a chain suspension bridge in 1808. Kranakis (1997, p. 43) describes his method as empirico-inductive, in the spirit of his contemporaneous thinkers of the Common Sense school. Thomas Paine (1737–1809) championed the new design. Finley modestly anticipated that "our puny canoe, with a little cultivation of genius, will soon spring into a formidable ship" (Kranakis, 1997, p. 53). The forecast proved prophetic. The chain suspension bridges designed by Telford and Brunel spanned 176 m at the Menai Straits (1826) and 214 m at Clifton (1864), respectively. In 1867, John Roebling commenced the construction of the Brooklyn Bridge between Manhattan and Brooklyn with spans of 284/487/284 m, supported by four parallel wire cables.

Metallurgy improved the material capable of reliably resisting high tension, from forged iron to high-strength steel. Design maximized the performance of this material by refining the suspension, the cable-stayed and various hybrid structural forms. Technological supply and transportation demand reshaped the main suspension elements from chains and ropes to helical strands, air-spun parallel wires, and prefabricated parallel wire strands. Construction grew capable of hoisting into position 4073 m long strands of 127 parallel wires and sinking 15,300-ton steel caissons 60 m below sea level. Theory advanced from empirical trial and error to nonlinear dynamic modeling, executable only by digital computers. By the end of the twentieth century, suspension spans reached the length of 1991 m. Another "puny canoe" in Finlay's creative stream was to launch the growing fleet of cable-stayed bridges.

Table 1.1 contains a partial list of noteworthy suspension bridges constructed since 1808 worldwide. Understanding the present and designing the future suspension bridges benefits from a review of several unique elements in their structure (i.e., the product) and stages in their life cycle (i.e., the process), as they have evolved over the encompassed period.

1.3 PROCESS AND PRODUCT

Engineering meets the physical demand of the applied loads with a supply of structural strength. Economy generates the financing for bridge

Table 1.1 Partial List of Noteworthy Suspension Bridges

Year	*Bridge*	*Span Length (m)*	*Suspension*	*Stiffening*	*Towers*
1808	Essex-Merrimack	40	2 IC	2 trusses	Masonry
1823	Saint Antoine	2×40	6 PIW	2 trusses	Masonry
1826	Menai Straits	120/176/120	EBC	2 trusses	Masonry
1834	Fribourg	273	4 PW	2 trusses	Masonry
1849	Wheeling (1872)	308	4 PW/SR	2 trusses	Masonry
1864	Clifton	214	EBC	2 trusses	Masonry
1866	Cincinnati-Covington	322	4 AS/SR	2 trusses	Masonry (Figure 1.1)
1883	Brooklyn	284/487/284	4 AS/SR	6 trusses	Masonry (Figure 1.2)
1903	Williamsburg	488	4 AS[b]	2 trusses	Steel (Figure 1.2)
1909	Manhattan	222/449/222	4 AS	4 trusses	Steel (Figure 1.2)
1926	Bear Mountain	688	2 AS	2 trusses	Steel
1928	Point Pleasant	213.5	2 EBC	2 trusses	Steel
1931	George Washington	186/1067/198	4 AS		
1962	Lower Leaf	186/1067/198	4 AS	Space truss	Steel (Figure 1.3)
1937	Golden Gate	343/1280/343	2 AS	Space truss	Steel (Figure 1.3)
1939	Bronx-Whitestone	224/701/224	2 AS	2 girders	Steel
1940	Tacoma Narrows	396/792/396	2 AS	2 girders	Steel
1957	Mackinac	549/1158/549	2 AS	Space truss	Steel
1964	Verrazano	371/1300/371	4 AS	Space truss	Steel (Figure 1.3)

(continued)

Table 1.1 (continued) Partial List of Noteworthy Suspension Bridges

Year	*Bridge*	*Span Length (m)*	*Suspension*	*Stiffening*	*Towers*
1981	Humber	280/1410/530	2 AS	Box girder	R/C concrete
1988	Kita Bisan-Seto	274/990/274	2 PWS	Space truss	Steel
	Minami Bisan-Seto	274/1100/274	2 PWS	Space truss	Steel
1995	Askøy	850	2 × 21 LCS	Box girder	Concrete
1996	Jiangyn	1385	2 AS	Box girder	Concrete
1996	Akashi Kaikyo	960/1991/960	2 PWS	Space truss	Steel (Figure 1.4)
1997	Tsing Ma	356/1377	2 AS	Box girder	Concrete (Figure 1.5)
1998	Great Belt	535/1624/535	2 AS	Box girder	Concrete (Figure 1.5)
1999	Kurushima I	149/600/170	2 PPWS	Box girder	Steel (Figure 1.6)
	Kurushima II	1020/250	2 PPWS	Box girder	
	Kurushima III	1030	2 PPWS	Box girder	
2012	Taizhou	1080/1080	2	Box girder	Central—steel 2 sides—concrete
2013	East Bay Crossing	385	1 PPWS[a]	Box girder	Steel (Figure 1.7)
2016	Izmit Bay	566/1550/566	2	Box girder	Steel
	Stretto di Messina	183/3300/183	4	3 linked box girders	

Note: AS, aerially spun parallel wires; EBC, eyebar chains; HWS, helical wire strands; IC, iron chains; LCS, locked-coil strands; PIW, parallel iron wires; PPWS, parallel wire strand; SR, steel rope stays.

[a] Self-anchored.

[b] Nongalvanized.

Figure 1.1 The Cincinnati-Covington Bridge (1866).

Figure 1.2 The Brooklyn (1883), Williamsburg (1903), and Manhattan (1909) Bridges, New York City.

construction and operation from the current and anticipated demand for transportation. During the reviewed period, that demand evolved from horse-drawn carriages to trains, then predominantly to internal combustion vehicles, and may be moving on to new forms of rail transport. In 1855, Roebling spanned 251 m at Niagara Falls with a "double decker," carrying trains on top and carriages below. The most ambitious spans of the early twentieth century emphasized rail transport and readily accommodated vehicular traffic. The Williamsburg Bridge (1903) carries eight

Figure 1.3 The George Washington (1931), Verrazano (1964), New York, and Golden Gate (1937), San Francisco, Bridges.

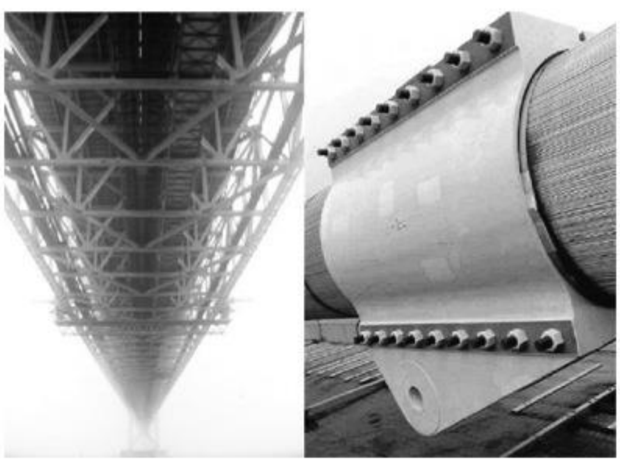

Figure 1.4 The Akashi Kaikyo (1996) Bridge.

vehicular lanes and two rail tracks. Manhattan Bridge (1909) carries seven traffic lanes and four rail tracks. By midcentury, vehicular traffic volume dictated that the George Washington Bridge would carry 14 (1962) and the Verrazano Bridge 12 traffic lanes (1964). The relatively lighter, purely vehicular traffic encouraged the growth of the longest spans. Mixed-mode service has not entirely disappeared, however. The Bisan-Seto and Tsing Ma Bridges carry six traffic lanes and two rail tracks. The Third Bosphorus Bridge is built for both vehicles and trains. Mixed-mode use is proposed for the Messina Straights Bridge.

Figure 1.5 The Tsing Ma (1997), Hong Kong, and Great Belt (1998), Denmark, Bridges.

Figure 1.6 The West Bay (1937), San Francisco–Oakland, and Kurushima (1999), Japan, Bridges.

The engineering product evolving under these dynamic social and physical constraints must maximize the efficient use of the strongest, lightest, and toughest structural materials. The resulting structure is the suspension bridge, with its unique cables, suspenders, towers, anchorages, saddles, decks, and stiffening systems. Equally unique is the associated process, including the analysis, design, construction, maintenance, repair, and replacement of these elements.

Figure 1.7 The East Bay San Francisco–Oakland Bridge during construction (2011).

1.4 SUSPENSION

1.4.1 Wires versus Chains

Chains of one kind or another supported most of the iconic early suspension bridges. The chain material evolved from forged wrought-iron links (1796) to wrought-iron eyebars (1818) and steel eyebars (1828). After 1915, nickel and heat-treated carbon steel alloys increased the ultimate strength of the eyebars to 105 ksi (75.6 kg/mm^2, 741 MPa). By the year 2000, electric arc furnaces had replaced the traditional open-hearth Siemens–Martin process.

Under the title "Wire Cables vs. Eyebar Chains" (1949, p. 74), David Steinman (1887–1960) allows that for span lengths of up to 214 m (700 ft), as in the bridges over the Ohio River at Point Pleasant and St. Mary's, heat-treated eyebars may be cost-competitive. Their principal advantages are the superior stiffness, speedier construction, and simpler global structural behavior—and hence analysis. The ultimate choice is reduced to the following reasoning: "Where two designs are of equal cost, the heavier bridge is to be preferred as giving a more rigid structure" (Steinman, 1949, p. 756). By the first writing of this text in 1922, John Alexander Low Waddell (1854–1938) was still championing trusses as the best long-span bridges. His foremost detractor, Gustav Lindenthal (1850–1935), was trying to graft eyebar trusses onto suspension spans of record length.

Against eyebars, Steinman cites high secondary and unequally distributed stresses, difficulty of inspection and painting, and "untried problems in the erection." The collapse of the bridge at Point Pleasant, 100 years after the commencement of the Brooklyn Bridge construction, settled the debate. The twin bridge at St. Mary's was promptly decommissioned.

Multiple eyebar chains have remained in service without incident, for example, on the cantilever trusses of the Queensboro Bridge over East River (Figure 1.8b) in New York City and the three self-anchored suspension bridges over the Allegheny River (Figure 1.8c) in Pittsburgh. The eyebar chains of historic suspension bridges, such as the Széchenyi Bridge over the Danube in Budapest (Figure 1.8a) and Telford's bridge over the Menai Straits, have been reconstructed reproducing as much as possible the original form, if not the material.

The steel wires of modern cables are strengthened by successive drawing (as opposed to extrusion) through dies with decreasing diameters. Cold drawing modifies the molecular configuration of mild steel, increasing considerably its yield and ultimate strength. The elastic modulus of the steel is retained; however, the ductility is reduced. Upon attaining their final diameter (4.8–7.0 mm), the wires are "hot dipped" in molten zinc. A zinc coating with a thickness of 20–40 μm forms over the steel surface, and provides galvanic protection against corrosion. Figure 1.9 shows different wires used in suspension bridge parallel wire cables. Figure 1.10 shows the "wire certificate" for the air-spun cables of the Great Belt Bridge.

In a noteworthy exception, the four cables of the Williamsburg Bridge consist of 7696 nongalvanized high-strength parallel wires. The cables were extensively rehabilitated in the 1990s. Also nongalvanized were the original helical strand cables of the Pont de Tancarville (176/608/176 m) over the Seine and Pont d'Aquitaine (143/394/143 m) over the Garonne in France (Figure 1.11). The cables at both bridges were replaced, in 1998 and 2002, respectively.

Heat treatment and refinements in the chemical composition of the alloy consistently increased the tensile strength of cable wires. A wire strength of 1600 MPa (155–160 kg/mm^2) was prevalent throughout most of the twentieth century, as reported by Gimsing (1997) and presented in Table 1.2.

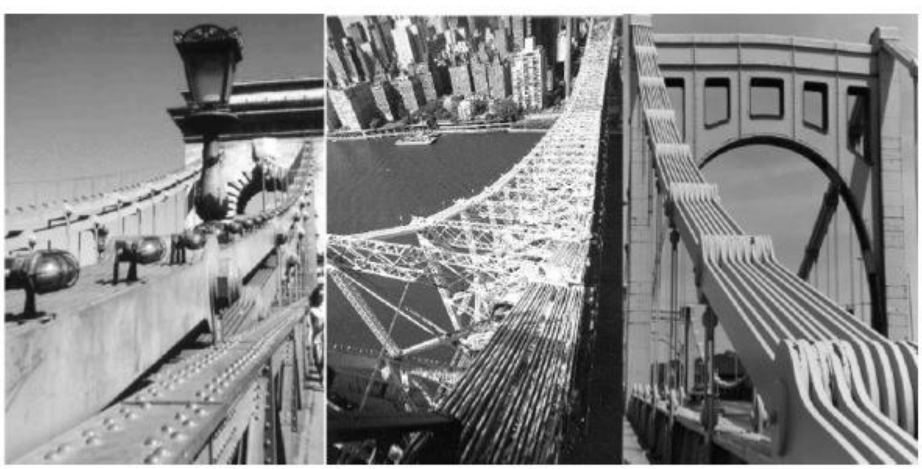

Figure 1.8 Széchenyi (1849, 1949), Budapest; Queensboro (1909), New York; and 6th St. Bridge (1928), Pittsburgh.

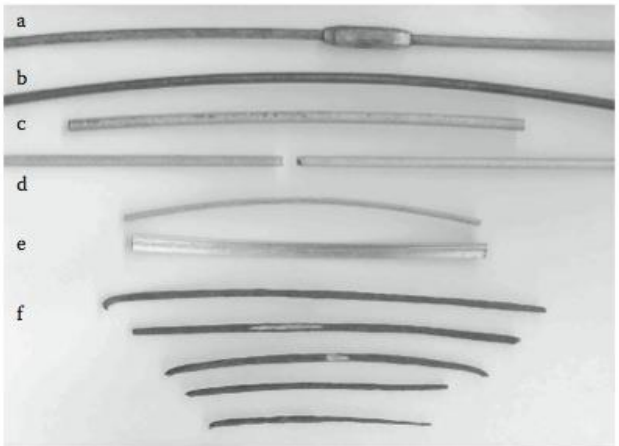

Figure 1.9 High-strength wires used in suspension bridge cables: (a) 4.8 mm diameter, galvanized, with splicing ferrule, Manhattan Bridge, 1909; (b) 4.8 mm diameter, nongalvanized, Williamsburg Bridge, 1903; (c) 5.37 mm diameter, galvanized, Great Belt Bridge, 1996; (d) 4.8 mm diameter, galvanized, heat straightened, ductile break; (e) Z-shaped galvanized wrapping wire, Kurushima Bridge, 1999; and (f) nongalvanized, corroded wires, Williamsburg Bridge, 1988.

Mayrbaurl (2006) reported strengths from 1644 to 1695 MPa for wires from three suspension bridges identified as W, X, and Z. According to Nishino et al. (1994), the record span of the Akashi Kaikyo Bridge required the higher strength of 1975 MPa (180 kg/mm^2), as shown in Figure 1.12.

1.4.2 Strands: Parallel Air-Spun, Prefabricated, and Helical

Table 1.2 contains a comparison of the mechanical properties (Gimsing, 1997) and some performance attributes of the different strands. The following three methods for assembling strands, and ultimately cables, from high-strength steel wires have evolved into current coexistence:

- Parallel wires by air spinning (AS) (Figure 1.13a)
- Prefabricated parallel wire strands (PPWSs) (Figures 1.13b, 1.14, and 1.15)
- Helical wire strands (HWSs) (Figure 1.14)

Mayrbaurl and Camo (2004) reported 52 suspension bridges in North America, of which 29 have aerially spun parallel wire cables (AS), 21 have HWSs, and 2 have PPWSs.

John Roebling introduced the air-spinning method of cable construction around 1845 as an application of his high-strength wires. The Roebling Wire Company fabricated high-strength galvanized wires and steel ropes

WIRE CERTIFICATE

This certificate documents that the enclosed piece of wire, is a part of the wire which was produced for the main cables of :

The East Bridge

Wire properties	
Material:	Steel
Diameter:	5.37 mm.
Mass:	180 g/m.
Tensile strength:	Min. 1,570 n/mm^2
Breaking load:	3.6 t/wire
Thickness of the galvanisation:	40 µm (~ 40/1,000 mm)
Properties of the main cable	
Number of wires in each main cable	18,648
Total length of wire used:	110,600 km, equal to almost three times around the world.
Main cable diameter:	827 mm.
Weight of the main cable:	3.3 t/m.
Total weight of the main cable:	9,850 t/cable
Breaking load of the main cable:	66,000 t/cable
The calculated max. load on the main cable:	33,700 t/cable
The wire was manufactured by:	Redaelli (Italy) Ryland Whitecross (England)

Some facts about the cable spinning

Cable spinning took place between the 6th July and the 19th of November 1996.
Each spinning wheel placed 8 wires on each round trip from the anchor block on the Halsskov side to the anchor block on the Sprogø side. A total of 4,662 trips were made in order to complete the spinning of the two main cables.
An average 7.69 ton of wire were placed each hour, which is equivalent to placing one wire, 2,965 m long, every 4 min.

The spinning operation on Storebælt has set new world standards for output and efficiency, never have so much wire been placed in such short time.

The spinning operation was carried out by:
Coinfra S.p.A. (Italy) main contractor for the East Bridge superstructure.
G.E.C. Alsthom - SDEM (France) subcontractor to Coinfra.
Comag (France) subcontractor to SDEM
Hordaland Mekaniske Værksted (Norway) manufacturer of the spinning equipment

H. Pedersen
Erection Supervisor
A/S Storebælt

Figure 1.10 Wire certificate for the Great Belt Bridge.

Figure 1.11 Pont de Tancarville (1998) and Pont d'Aquitaine (1996), France.

for the cables and suspenders of all the record-breaking bridges built over the ensuing century.

The AS method consists of unreeling several wires from one anchorage, over the tower saddles, to the opposite anchorage. At the anchorages the wires go over a strand shoe and reverse direction. At their ends, wires are spliced with crimped ferrules, as in Figure 1.9a, or with threaded collars (considered detrimental to the strength of the connection). Steinman (1949) reports wire lengths of up to 1000 m (later exceeded).

Air-spun strands typically comprise 200 to 300 wires bundled and anchored together. Circular cylinders with the same diameter can be compacted around a central one in layers containing increasing multiples of 6, as in $7 = 1 + 6$, $19 = 1 + 6 + 12$, $37 = 1 + 6 + 12 + 18$, $61 = 1 + 6 + 12 + 18 + 24$, and so on.

Hexagonal configurations of a 7-wire rope and a prefabricated strand (PPWS) consisting of 127 wires are shown in Figure 1.14.

The air spinning of the two parallel wire cables at the Golden Gate Bridge introduced several innovations (Strauss, 1938). In order to reduce the lateral load on the strands during compaction over the saddles, the strands were aligned vertically, along the $X'OY'$ axes in Figure 1.16, rather than horizontally, along the traditional XOY axes. To better approximate a circular cross section (Figure 1.13), the 27,572 wires in each of the two cables were bundled in 122 strands of unequal size. The circular cross section of the Akashi Kaikyo cables was also achieved by adding wires to the polygonal shape obtained by compacting the 127-wire PPWSs.

While Roebling's AS cables remained prevalent on the American continent, European engineers retained a preference for steel wire ropes. With the decisive influence of Eugene Freyssinet (1879–1962), the practice evolved to prestressing strands (HWS). Multiwire helical strands are built of layers spun in alternating directions. Gimsing (1997) reports a nominal elastic

Table 1.2 Properties and Attributes of Modern High-Strength Strands with the Following Wire Properties: Diameter = 4.8–5.37 mm, Elastic Modulus E = 205 GPa, σ_{ult} = 1570 MPa, σ_y = 1180 MPa

	7-Wire	*Parallel (AS)*	*Helical (HWS)*	*Locked-Coil*	*Prefabricated (PPWS)*
Elastic modulus E (GPa)	190	205	170	180	200
Ultimate strength σ_{ult} (MPa)	1770–1860 (>1570)	1570	0.9×1570	1370–1570 (of individual Z-shaped wires)	1570
Positive		Familiar construction, accessible for inspection	Compaction, accelerated construction, maximized clamping effect	Compaction, accelerated construction, maximized clamping effect	Compaction, accelerated construction, improved clamping effect, reduced bending
Negative		Slower construction, reduced compaction, bending stresses	Limited span length, interior not inspectable, possible fretting	Limited span length, interior not inspectable, fatigue, fretting	Weight of strands may become prohibitive during construction

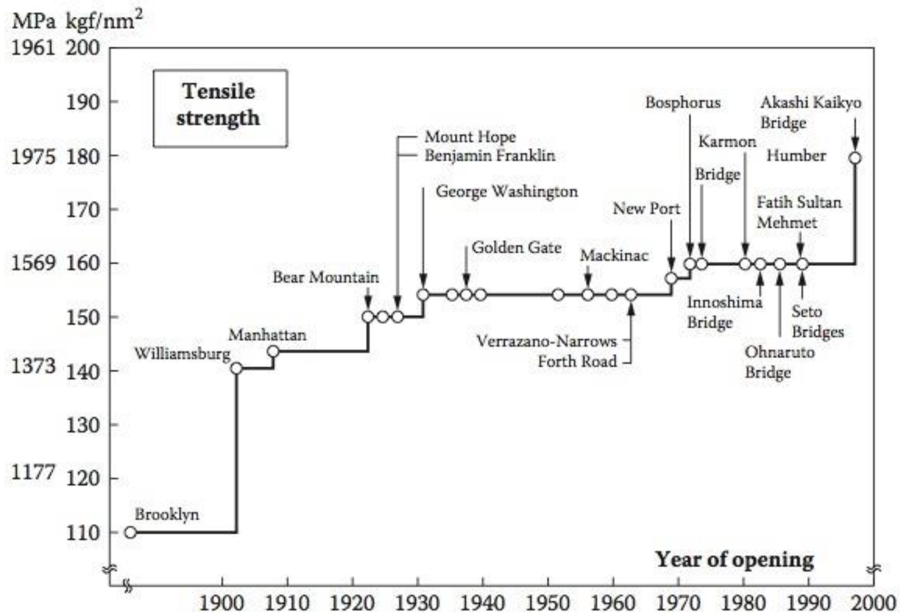

Figure 1.12 Evolution of tensile strength in cable wires from the Brooklyn to the Akashi Kaikyo Bridge.

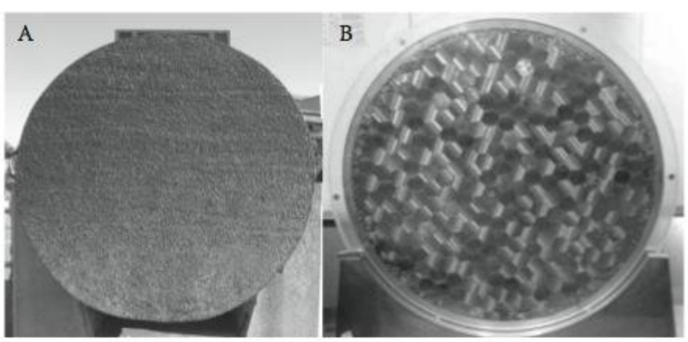

Figure 1.13 Mock-up cross sections of suspension bridge cables: (a) Golden Gate, 924 mm diameter, 27,572 wires, 122 air-spun strands, and (b) Akashi Kaikyo, 1120 mm diameter, 36,830 wires, 290 prefabricated strands.

modulus of such strands as roughly 15–25% lower than that of straight wires, reflecting the compaction of the strands under tension.

Steinman and Watson (1957, pp. 340–342) describe the introduction of "rope-strand cables" in America as follows:

> This new type of suspension bridge construction has been introduced and developed by Robinson and Steinman, commencing with the Grand'Mere Bridge over the St. Maurice River in Quebec, completed in

> 1929 with main span of 950 feet (276 m); and it has since been used by them in the Waldo-Hancock Bridge, the St. Johns Bridge, the Thousand Islands Bridge, the Wabash River Bridge, the Deer Island Bridge, and the Lion's Gate Bridge at Vancouver.... The rope strand type of cable construction offers distinct economy, in saving time and labor in the field, for spans up to about 1,500 feet (457.5 m). For longer spans parallel wire cables remain the most economical form of construction.

Locked-coil strands are helical with outer layers made of Z-shaped wires, intended to protect the interior wires from the intrusion of humidity. They were used in the original cables at Pont de Tancarville and Pont d'Aquitaine (Figure 1.11), to cover internally nongalvanized strands.

Helical strands can have internal protective layers. With adequate external corrosion protection, including dehumidification, helical strands have shown no visible deterioration, for example, at the Little Belt Bridge (main span 600 m, 1970) (Figure 1.14). Fretting and stress concentration, particularly for the super-compacted locked-coil strands, are potential hazards. Helical strands remain the prevalent option for cable-stayed bridges; however, parallel wire strands similar to the suspenders of the Akashi Kaikyo Bridge (Figure 1.15) were used as stays at the record-breaking Tatara Bridge (890 m, 1999).

Prefabricated parallel wire strands were proposed by Jackson Durkee (1966) at Bethlehem Steel. He reported that if a 37-wire parallel wire strand is slightly twisted (or pitched), it can be reeled on drums of larger diameters, akin to helical strands. The PPWSs of 127 wires (Figures 1.14 and 1.15c) were used first in Japan and later worldwide. The two cables

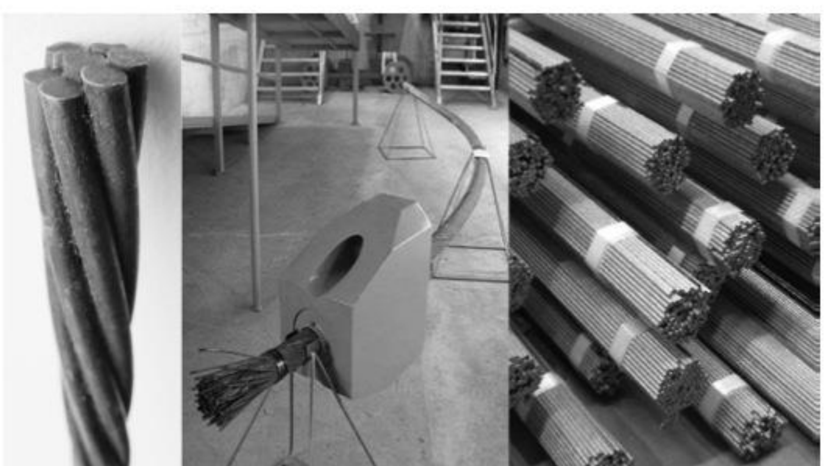

Figure 1.14 (a) High-strength nongalvanized 7-wire steel rope. (b) Helical strand, Little Belt Bridge. (c) Prefabricated parallel 127-wire strands.

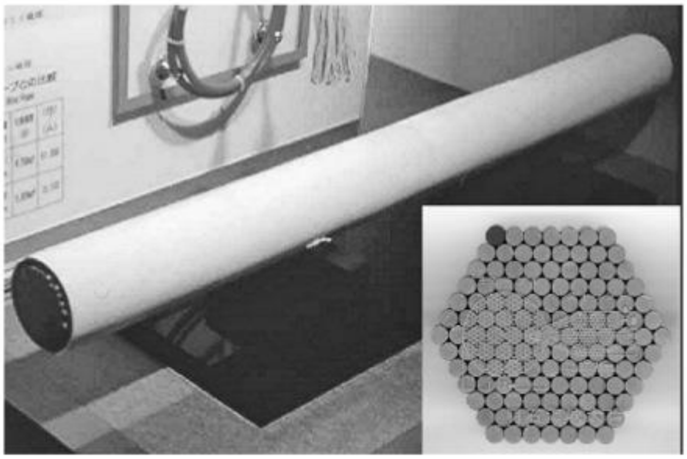

Figure 1.15 Prefabricated strand of 127 parallel wires, used in the Akashi Kaikyo cables and suspenders, and in the stays of Tatara Bridge.

of the Akashi Kaikyo Bridge (Figure 1.13b) are built of 290 PPWSs with a length of 4100 m.

1.4.3 Compaction

Cables are compacted as much as possible both as a protection against the corrosive intrusion of water and as a means of maximizing the clamping effect of adjacent wires. Perfectly compacted wires (except external ones) are in contact with six adjacent wires, and form hexagons, as shown in Figures 1.14 through 1.16.

For T perfectly compacted concentric layers of wires, as in Figure 1.16, the net-to-gross ratio of areas can be approximated as shown in Equation 1.1:

$$2\pi\, r^2\, [3T(T+1)+1]/r^2\, [(2T+1)^2\, 3^{3/2}] \approx (2\pi/3^{3/2})\,(3/4) \approx 0.907 \qquad (1.1)$$

The equilateral triangle defined by the centers of three identical tangent circles is the simplest repetitive unit of the hexagon and directly obtains the same result, as in Equation 1.2:

$$\pi\, r^2/(2r^2\, 3^{1/2}) = 0.907 \qquad (1.2)$$

This maximum compaction is reported for Z-shaped strands (Table 1.2). It is approximated for the heat-straightened strands shown in Figure 1.14c. The voids in aerially spun cables comprise normally about 20% of the gross section area. The compaction should be checked during inspections by comparing the cable's circumference and the number of wires in it. Figure 1.17

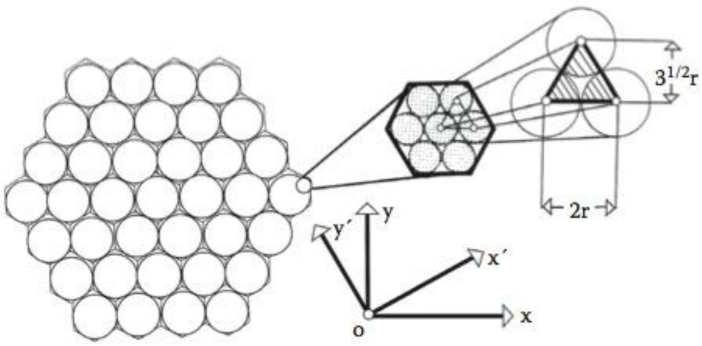

Figure 1.16 Compacted geometry of 37, 6, and 3 circles with radius *r*.

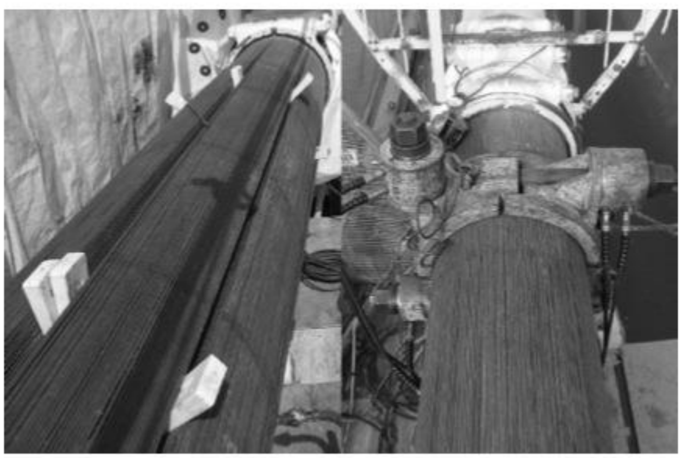

Figure 1.17 Parallel wire AS cable during wedging and compaction.

shows parallel wire AS cables wedged for inspection and compacted for rewrapping with spiral wire.

1.4.4 Parallel Wire Stress–Strain State

Traditionally, the product of the wire strength and the number of wires in a suspension cable was considered its capacity. The ratio of the capacity and the design load acting on the cable was perceived as a safety factor. The implied assumptions of uniaxial tension and linear elastic response obscure the following essential features of suspension cables:

- The wire stress state is complicated by residual stresses induced during galvanization.
- Stress concentration and embrittlement are caused by the presence of incipient cracks and hydrogen.

- During construction and service, the wires are subjected to flexure both by the straightening of their initial curvature and by additional flexure at saddles, strand shoes, and cable bands.

1.4.5 Wire Curvature

The fully extended galvanized wires cool off into a naturally curved shape with a diameter of approximately 2 m. Consequently, compaction into a cable subjects them to bending. Straightening a wire with a section radius r_{wire} and a curvature with radius R induces in it the following bending moment M and corresponding maximum stress σ:

$$M = E\, I/R \tag{1.3}$$

$$\sigma = M/S = E\, r_{wire}/R \tag{1.4}$$

where:

$I = \pi\, r_{wire}^4/4$ is the moment of inertia of the wire section
$S = I/r_{wire}$ is the wire section modulus

Introducing $r_{wire} = 5$ mm and $R = 2000$ mm in Equation 1.4 obtains $\sigma = 500$ MPa (75 ksi), matching the uniform working stress level for which many suspension cables have been designed. Based on x-ray diffraction tests, Mayrbaurl and Camo (2004) estimated that the straightening can induce bending stresses of up to ±240 MPa (36 ksi). Wires invariably crack on the side where straightening has produced tension.

1.4.6 Cable Bands, Saddles, and Anchorages

Gimsing (1997) obtains the local bending at the first cable band past the saddle as a function of the cable tension, diameter, change in the angle over the distance to the saddle, band length, assuming 20% voids. The local bending rapidly declines toward midspan.

To some degree air-spun wires can adjust their own curvature to that of the saddles during construction.

Despite their limited ductility, AS cable wires appear capable of sustaining the plastic deformation caused by wrapping around strand shoes in anchorages, as in Figure 1.18a. At the back of the strand shoe the wires are maximally deformed, as well as compacted, and experience no other load. Prefabricated and helical wire strands are not bent in the anchorages (Figure 1.18b and c); however, their sockets may be vulnerable in other ways.

Figure 1.18 Strand anchorages: (a) parallel air-spun, (b) helical, and (c) prefabricated.

1.4.7 Stress Distribution within the Parallel Wire Cable

The stress is likely to vary between cable wires and strands in a cable to the extent that their initial geometry cannot be perfectly identical. The Honshu-Shikoku Bridge Authority (1998) reports nightly adjustments of the prefabricated strands during the installation to ensure easier compaction.

With the aging of AS cables, it has become necessary (as it did at the Williamsburg Bridge in 1988) to evaluate their remaining strength after a number of wires have broken. It is generally accepted that over a certain development length, a broken wire will regain the stress–strain state of the adjacent wires through friction; however, that length cannot be rigorously quantified under an imperfect compaction and a varied level of stress. Starting with the model shown in Figure 1.16, Gjelsvik (1991) demonstrated that, under perfect compaction, the clamping length can be "typically a few feet," because the broken unstressed wire expands. Under imperfect field conditions, that length rapidly increases. For practical purposes, it has been assumed that three cable bands represent a sufficient development or clamping length for a single wire.

Mayrbaurl and Camo (2004) propose a variety of models for assessing the remaining strength of a deteriorated cable, depending on the available data for the strength, ductility, corrosion, and incipient cracking of tested wires. Since none of that information can be perfectly available, any assessment of the cable's strength must be a probabilistic one. The number of field observations and laboratory tests required for a meaningful analysis increases with the level of deterioration, and with the decrease of the design safety factor.

1.4.8 Cable Stiffness

The effective modulus of a suspended cable E_{ef} can be approximated by Equation 1.5, attributed to Ernst, as that of two springs in series, one with the elastic modulus of steel E, and the other, resulting from the sag, as follows:

$$1/E_{ef} = 1/E + (\gamma\,\ell)^{2}/(12\,\sigma^{3}) \tag{1.5}$$

where:

ℓ is the span length
γ is the specific weight of the cable
σ is the stress in the cable

Since E_{ef} and σ are mutually dependent, an initial assumption and some iterations are required in every case. The effective stiffness E_{ef} decreases with ℓ^2, but increases with σ^3. The stress σ in turn increases with the span length ℓ until the cable can sustain only its weight. Thus, the estimated ratio of the cable strength to the maximum loads for the longest suspension cables has decreased from a factor of 4 at the beginning of the twentieth century to 2.2. That trend implies an increased intolerance to any form of deterioration. Gimsing (1997) refers to E_{ef} as E_{tan} and recommends the use of the secant modulus for cable-stayed bridges.

1.4.9 Number of Cables

John Finley concluded that four chains work better than two. The four cables at each of the three East River suspension bridges in New York City (Figure 1.2) are grouped in different configurations. The inclined cables on the Williamsburg Bridge recall a recommendation made by Roebling (1841). In the same article, Roebling observed that "a single cable will be superior to a pair of cables when displaced by lateral forces." The demands for superior stiffness, aerodynamic stability, and constructability have contributed to the trend of using two cables instead of four, as evidenced in Table 1.1. At the Messina Straits Bridge, which is designed for both vehicles and trains on a single level, four cables were proposed once again.

1.4.10 Suspenders

The evolution of suspenders has followed that of the main cables, resulting in three main alternatives: galvanized wire ropes, helical strands (Figure 1.19), and parallel wire strands (Figure 1.15).

Figure 1.19c shows a combination of wire ropes and strands. Solid rods have a limited use over shorter lengths, as, for example, at the Chelsea Bridge (Figure 1.20). On the Brooklyn Bridge short rods serve as both suspenders and compression struts.

On older bridges, such as the East River crossings shown in Figures 1.2 and 1.25, suspenders are spaced at 6 m. The PWS suspenders at the Akashi Kaikyo Bridge are spaced at 14.2 m and reach a length of 205 m.

Where the stiffening system of the bridge is a truss, the suspenders can be anchored at the top chord (as in more recent bridges) or the bottom

Figure 1.19 Suspenders: (a) wire rope, (b) wire ropes with spacers, and (c) wire ropes and prestressing helical strand.

Figure 1.20 Chelsea Bridge, London.

chord (as on older ones). Mixed anchoring can also become necessary due to geometric constraints.

Wire rope suspenders typically stride over grooved cable bands, as in the examples of Figures 1.3c and 1.19a and b. In the hybrid system shown in Figure 1.2a, the rope suspenders are socketed under the cable bands, in order to form a single vertical plane coincident with the plane of the diagonal stays fanning out from the top of the towers. Helical and parallel wire strands are socketed and pin connected, under equalizer bars, as in Figure 1.19c, or under the cable bands, as in the case of Figure 1.4b. In the latter example, the suspenders are connected with simple or universal pin joints, depending on their length.

Suspenders perpendicular to the longitudinal stiffening system do not contribute to its stiffness. Freeman Fox & Partners designed inclined suspension systems for the Severn (305/988/305 m, 1966), the First Bosphorus (1074 m, 1973), and the Humber (280/1410/530 m, 1981) Bridges. Such suspenders can transmit shear between the main cables and the deck, thus dampening structural vibrations. As a result, they experience higher stress cycles of their own. Nets of intersecting suspenders inclined in opposite directions have also been proposed. Their action within the suspension system is comparable to that of tension diagonals in a truss, suggesting the name *cable trusses*. The five-span San Marcos Bridge in El Salvador (76/159/204/159/76 m) was an example of this system from 1951 to its demolition in 1981.

In the proximity of their sockets, suspenders experience highly variable stresses due to live loads and wind-induced vibrations, as well as the most aggressive corrosion. Consequently, design must anticipate their periodic replacement. To ensure continuous service, the suspenders and the longitudinal stiffening must be designed to redistribute the added load of a predetermined number of missing suspenders without irreversible global deformations. The suspenders on the East River bridges shown in Figure 1.2 have been replaced more than once, under traffic, for example, as shown in Figure 1.9c.

The dynamic characteristics of vertical suspenders running in groups of two or four can be adjusted by connecting them at selected points along their length with spacers, as in Figure 1.19b. The combination of wind and rainwater running down the polyethylene tubes encasing suspenders and stays has caused "galloping vibrations." The smooth surfaces of modern suspenders and stays are corrected in order to guide water flow. Dampers are installed at the bottom anchorages.

1.4.11 Corrosion Protection

Main cables are protected from corrosion by various systems. The galvanization of the wires is the first level of protection. Synthetic and natural corrosion inhibitors (such as linseed oil) are used to fill the voids. Wires have been coated with lead and, more recently, zinc paste. Parallel wire cables are wrapped with spiral wire, polyethylene sheets, and painted.

Many strand cables have no overall wrapping. Individual strands may have protective coating. It is assumed that tension compacts strands sufficiently to prevent water penetration, whereas a wrapping would trap moisture. Examples are the Pont de Tancarville and Pont d'Aquitaine (Figure 1.11) and the Chelsea Bridge (56/101/56, 1937), shown in Figure 1.20.

Suspenders made of ropes, prestressing strands, and parallel wire strands are protected differently. All are galvanized. Ropes are painted. Some helical strands rely solely on galvanization. Polyethylene tubes encase PPWSs, as in Figure 1.15.

1.5 TOWERS (PYLONS)

Gourmelon and Brignon (1989) define suspension bridge towers as the pedestals of the cables. Early towers consisted of individual pylons, as in the case of Roebling's bridge at Niagara Falls and the Chelsea Bridge (56/101/56 m, 1937) in London (Figure 1.20).

Roebling eventually opted for massive portal frames, as at his Cincinnati-Covington (Figure 1.1) and the Brooklyn (Figure 1.2) Bridges. Tower material evolved from wood to unreinforced concrete masonry (Figures 1.1 and 1.2a), steel (Figures 1.2, 1.3, and 1.6), and reinforced concrete (Figures 1.4 and 1.11) with posttensioning. Steel is the material of most American and Japanese bridge towers, whereas concrete is common in European ones. Earlier towers were designed as rigid with sliding saddles. Contemporary steel and reinforced concrete towers are designed to resist the lateral loads transmitted by fixed saddles in flexure. The 210 m towers of the Golden Gate Bridge were designed for movements of "18 in [460 mm] channelward or 22 in [560 mm] shoreward from vertical lines through the centers of the tower bases" (Strauss, 1938).

Towers of small bridges, for example, pedestrian ones, can be inclined and articulated at the base.

Whereas bridge users focus on the fleeting road, two geographically separate communities share similar monumental towers. The height of suspension bridge towers is roughly 10% of the span length; however, that amounts to 254.1 m at the Great Belt and 282.8 m at the Akashi Kaikyo Bridge. Thus, towers and the catenary shape form the trademark images of the suspension bridges. In 1867, Roebling reported to the New York Bridge Company (Reier, 1977) that "the most conspicuous features, the towers" of the proposed Brooklyn Bridge "will serve as landmarks to the adjoining cities, and they will be entitled to be ranked as national monuments." One hundred and fifty-seven years later, that ranking is worldwide. The 184 m tall towers of the George Washington Bridge have been praised for the "honesty" of their (unintended) functional look by critics as demanding as Le Corbusier. In contrast, the brutally utilitarian clunky towers of the Williamsburg Bridge and the deliberately ornamental slender ones of the Manhattan Bridge had to deliver indispensable service for a century in order to gain recognition. Some appearances have benefited from professional help. Irving F. Murrow, consulting architect, designed the striking Art Deco towers and the trademark "international orange" color of the Golden Gate Bridge. Gimsing (1998) states: "The concrete pylons are the most spectacular elements of the East Bridge and the development of their appearance through close collaboration between architects and engineers was a key issue in the designing process."

1.6 SADDLES

If towers are the cables' pedestals, saddles are their bearings. Saddle curvatures are, for example, 7 m at the Great Belt and 9.15 m at the George Washington Bridge. The rotated arrangement of cable strands from horizontal to vertical, shown in Figure 1.16, led to a modification from flat to U-shaped (at the Golden Gate Bridge), and ultimately to "grooved" saddles for prefabricated strands, as shown in Figure 1.21.

Saddles designed to slide tend to "freeze" in fixed positions, over time, and the lateral forces transmitted by the cables to the tower tops increase. During the repair of the saddle rollers at the Williamsburg Bridge in the 1990s, the previously frozen saddles shifted abruptly by almost 150 mm. Fixed cable saddles were designed for the adjacent Manhattan Bridge (1909) (Figures 1.2 and 1.25) and have become prevalent. The 180-ton saddles on the George Washington Bridge were installed on beds of 41 steel

Figure 1.21 Saddles: (a) flat, (b) grooved, and (c) splay saddle.

rollers with 200 mm diameter and fixed after the addition of the lower deck in 1962. The saddles of the Golden Gate Bridge were similarly set on a nest of 200 mm rollers and permanently fixed after the completion of the construction.

If there is a large difference between the cable forces in the main and side spans, the tendency of the towers to lean may be counteracted by strands added to the "backstay" part of the cables. Such strands are typically anchored on top of the saddles. This is the case, for example, at the Tsing Ma Bridge (Figure 1.5), where one side span is not suspended, but also at the Mackinac Bridge, where the ratio of the side to main span is as high as 0.48. Scott (2001) reported that 240 wires were added to the 12,580-wire main cables in the backstays of the latter bridge.

1.7 ANCHORAGES

As suspended spans grow longer, their anchorages gain weight and rigidity. Where the terrain allows, even at record-breaking suspension bridges such as the Bear Mountain and George Washington, anchorages have been drilled into rock. The typical gravity anchorages rely on the weight of massive concrete anchor blocks (Figure 1.22). Key elements of the anchorages include the steel anchor girders, the cable anchor frames, the bent blocks, struts, and splay saddles.

Restrictions on the construction of large anchorages provide an incentive toward longer cable-stayed bridges, such as the Russky in Vladivostok (1104 m, 2012), Sutong over the Yangtze River (1088 m, 2011), Stonecutter at Hong Kong (1018 m, 2009), and Tatara in Japan (890 m, 1999).

Figure 1.22 Typical anchor block with splay saddle and bent strut.

Figure 1.23 Shared anchorage at the San Francisco–Oakland West Bay Crossing.

The importance of the cable bent was aptly demonstrated by the failure at Pont des Invalides in 1826. The splay saddle and strut are exposed in Figure 1.22; however, in many cases they are incorporated in the anchorage monolith. The cable bent at the Great Belt is particularly striking because of its clearly defined function and high-profile setting.

Beyond the strand shoes, eyebar chains or rods fan out into the anchorage monolith or into rock along straight lines. At the Brooklyn Bridge (Figure 1.2), the cables enter the anchorage along a horizontal tangent and the cable bent is entirely avoided. The eyebar chains are embedded in the anchor monolith along a 90° arc, reinforced with bearing blocks at every 10°. Gimsing (1997) points out that "after the introduction of prestressing... it has proved advantageous to use post-tensioned bars or cables to transmit the strand forces to the concrete of the anchor block."

Consecutive suspension bridges, such as the two at the San Francisco–Oakland West Bay Crossing (Figures 1.6 and 1.23), the Kita (264/990/264 m) and Minami (254/1048/264 m) Bisan-Seto Bridges, share an anchorage. The three Kurushima Bridges (Figure 1.6) share two anchorages.

1.8 DECK JOINTS AND BEARINGS

The extreme length and flexibility of suspension bridge decks place extraordinary service requirements on their expansion joints and bearings. Decks on most European and Japanese bridges are continuous between the anchorages and suspended at the towers. On American bridges, they are discontinuous at the towers. The latter option requires two expansion

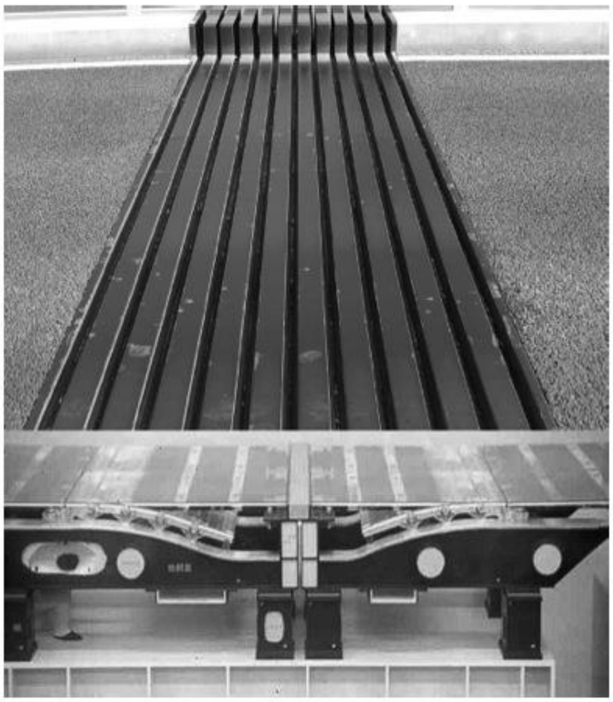

Figure 1.24 Modular and sliding plate joints.

joints and the appropriate sets of bearings per tower, but reduces the movement at the anchorages.

The penetration of water and debris below deck is highly damaging to the sensitive details at towers and anchorages. A variety of joints providing a continuous road surface have been designed as alternatives to the traditional finger joints. Most widespread are the modular joints of various size and displacement capacity. Figure 1.24a is an example. The sliding plate joint, shown in Figure 1.24b, is a model of the joints on the Rainbow Bridge in Tokyo.

Elastomeric, pot, and roller bearings have been used in order to accommodate the large displacements and rotations at supports. Joints and bearings must be designed for high-fatigue stress cycles. They require intensive regular maintenance, consisting of cleaning and lubrication. The high sensitivity to humidity, temperature, displacements, and accelerations makes anchorages, joints, and bearings suitable targets for online health monitoring.

1.9 STRUCTURAL ANALYSIS

1.9.1 The Catenary

In their radically different manner, both Steinman and Ammann acknowledged the role of art and luck in their work. Although recognized as a key ingredient in the engineering of suspension bridges, art endures only with the backing of science. Steinman (1949) states that "the scientific design of suspension bridges dates from about 1898." By that standard, Steinman and Watson (1957) conclude that "John Roebling had built a better bridge than he knew."

Art and science began to converge toward a theory of cable-supported structures, as in most of physics, during the Renaissance. Leonardo da Vinci (1452–1519) speculated about the perfect shapes of both the suspended chain and the voussoir arch. By less than rigorous analogy, Galileo (1564–1642) obtained the shape of a hanging chain as a parabolic arc. Robert Hooke (1635–1703) viewed arches and hanging strings as the reciprocal forms corresponding to pure compression and tension, respectively. The definition of the catenary or funicular as an optimal shape minimizing potential energy owes much to Leonard Euler (1707–1783) and his calculus of variations. According to Irvine (1981), "by the late 17th century the Bernoullis (Jacob [1654–1705] and his brother Johann [1667–1748]), Leibnitz [1646–1716] and Huygens [1629–1695], more or less jointly discovered the catenary." Thus, over roughly 20 centuries, the art and science of bridge building refined their process and expanded their product from compression and the voussoir arch to tension and the suspension structure.

1.9.2 Elastic Theory

Von Karman and Biot (1940), Steinman (1949), Timoshenko and Young (1965), Irvine (1981), and Gimsing (1997) base preliminary estimates of the stresses and shapes of suspension bridge cables on the elastic theory. Steinman (1949, pp. 19–20) stated its five fundamental assumptions as follows:

1. The cable is supposed perfectly flexible, freely assuming the form of the equilibrium polygon of the suspended force.
2. The truss is considered a beam, initially straight and horizontal, of constant moment of inertia and tied to the cable throughout its length.
3. The dead load of the truss and cable is assumed uniform per linear unit, so that the initial curve of the cable is a parabola.
4. The form and ordinates of the cable curve are assumed to remain unaltered upon application of loading.
5. The dead load is carried wholly by the cable and causes no stress in the stiffening truss. The truss is stressed only by live load and by changes in temperature.

Timoshenko (1943) and Timoshenko and Young (1965) analyzed an elastic flexible cable under a vertical load w, uniformly distributed along the horizontal chord with span length ℓ. Equilibrium obtains the tensile force H and the parabolic shape described in Equation 1.6:

$$y = w\,x\,(\ell - x)/(2H) \tag{1.6}$$

at $x = \ell/2$.

$$f = y_{max} = w\ell^2/(8H) \tag{1.7}$$

$$H = w\,\ell^2/(8f) \tag{1.8}$$

where x and y are the abscissa and ordinate of a left-hand coordinate system with origin at the left side support.

The length s of the cable is obtained in terms of ℓ and f by integration and binomial expansion, as shown in Equation 1.9:

$$s = \int_0^\ell \left[1 + (dy/dx)^2\right]^{1/2} dx = \int_0^\ell \left[1 + 64(f\,x/\ell^2)^2\right]^{1/2} dx \approx \ell + 8f^2/(3\ell) \tag{1.9}$$

Differentiating the result of Equation 1.9 with respect to f correlates changes in the sag Δf and in the cable length Δs as follows:

$$\Delta s = 16f\,\Delta f/(3\ell) \tag{1.10}$$

From Equation 1.10, Timoshenko and Young (1965) obtain the sag Δf corresponding to a temperature change $\Delta t°$, as shown in Equation 1.11:

$$\Delta f = c_t\,\Delta t°\,(3\ell^2 + 8f^2)/(16f) \tag{1.11}$$

where c_t is the coefficient of thermal expansion.

Popular literature about the Golden Gate Bridge quotes that for $\Delta t° = 50°C$, $\Delta f \approx 7$ m midspan.

Small elastic elongation Δs and sag Δf caused by a horizontal force H are obtained similarly as follows:

$$\Delta s = H\,[\ell + 16f^2/(3\ell)]/(A_c\,E_c) \tag{1.12}$$

$$\Delta f = H\,(3\ell^2/16 + f^2)/(A_c\,E_c f) \tag{1.13}$$

where A_c is the cross section, f is the sag, and E_c is the cable's elastic modulus.

Timoshenko and Young (1965) obtain H and f for a cable with a horizontal chord, subjected to a superimposed vertical load p, uniformly over limited length $2a < \ell$ and to a concentrated load P.

The authors point out that if p and w represent live and dead loads, respectively, p/w is relatively small. For example, $p/w = 1/6$ at the George

Washington Bridge, presumably without the lower deck. H and f are obtained numerically. Gimsing (1997) further investigates the effect of a horizontal restraint on the cable midspan (as shown in Figure 1.5b).

Without exact knowledge of the preceding information, John Finley recommended practical f/ℓ ratios of 1/6 to 1/7. Gimsing (1997) shows that viable f/ℓ ratios range from 0.1 to 0.15, with 0.25 for just the cable. Theoretically, a cable made of material with design stress $\sigma = 720$ MPa and specific weight $\gamma = 0.09$ MN/m^3 can be expected to carry its own weight up to spans of 10,600 m. By that length, however, the initial assumptions become unrealistic.

Optimizing the global dimensions of a suspension bridge includes the type of tower and stiffening, and the length of the side spans. Cost, local expertise, social demands, and the designer's personal preference are definitive. Table 1.3 summarizes the overall geometry of representative bridges enumerated in Table 1.1. The visual effect of different f/ℓ ratios is illustrated in Figures 1.25 and 1.26. The high f/ℓ ratio of the Golden Gate Bridge appears to enhance its dramatic visual impact.

The 0.58 ratio of the side to main span of the Brooklyn Bridge implies that the decks approach the anchorages above the cables. Roebling countered the greater flexibility entailed by larger ratios of side to main spans with diagonal stays. At the Mackinac, where the ratio of the side to main span is 0.48, Steinman provided an 11.6 m deep space truss. Similar general proportions were selected at the Tagus River Bridge, and again at the Akashi Kaikyo Bridge, where the truss is 14 m deep (Figure 1.4). Side spans are characteristically shorter at Ammann's bridges, up to 0.186 at the George Washington and 0.285 at the Verrazano. Until the lower deck was built in 1962, the George Washington functioned without a stiffening truss, but was stabilized by the superior width and weight of the deck. The depth of the space truss is 7.3 m at the Verrazano Bridge and 7.6 m at the Golden Gate truss (Figure 1.26). The box girder of the Tsing Ma Bridge (Figure 1.5) is 7.4 m deep, to accommodate rail traffic.

Table 1.3 Overall Geometry of Representative Bridges

Bridge	*f/ℓ*	*Side/Main Span*	*Tower Saddle*	*Designing Engineer*
Brooklyn	0.08	0.58	Rollers	J. Roebling
Williamsburg	0.11	0.37[a]	Rollers	L. L. Buck
Manhattan	0.11	0.49	Fixed	R. Modjeski/L. Moisseiff
George Washington	0.10	0.174, 0.186	Rollers/fixed	O. Ammann
Golden Gate	0.16	0.268	Rollers	J. B. Strauss/C. A. Ellis
Mackinac	0.093	0.48	Fixed	D. B. Steinman
Great Belt	0.11	0.332	Fixed	N. Gimsing
Akashi Kaikyo	0.1	0.482	Fixed	S. Kashima/HSBA

[a] Side spans not suspended.

Figure 1.25 Brooklyn, Manhattan, Williamsburg, RFK Triborough (with Hell Gate), Bronx-Whitestone, and Throg's Neck Bridges. *(continued)*

Figure 1.25 (continued) Brooklyn, Manhattan, Williamsburg, RFK Triborough (with Hell Gate), Bronx-Whitestone, and Throg's Neck Bridges.

Figure 1.26 George Washington and Golden Gate Bridges.

1.9.3 Deck Stiffening

The need for deck stiffening was evident to all successful suspension bridge designers since J. Finley; however, theory and practice were slow to converge on the necessary amount and appropriate means of achieving it. The original constraint on the decks was traffic; however, wind soon proved more formidable. To the consternation of nineteenth-century engineering, the suspension structures were not linearly elastic, not homogeneous, and their behavior was not static. Consequently, attempts to extend suspension spans beyond the empirically confirmed lengths or the service to new usage encountered unforeseen behavior.

By realizing the limitations of the pseudostatic elastic analysis, design managed to remain (in both senses of the term) mostly conservative. Theory eventually grasped both large deflections and dynamic response, unfortunately in that nonconservative order.

1.9.4 Deflection Theory

Steinman (1949, p. 19) cautions that "variations from the 4th assumption of the Elastic Theory may be of sufficient amount to require special consideration as is given by the more exact Deflection Theory," summarized in Appendix D therein. His translation from the German published by J. Melan's theory dating from 1888 was published under the title "Theory of Arches and Suspension Bridges" (1913).

The risks of the gained new knowledge become clear in the following introduction to "Fundamental Equations for Stiffened Suspension Bridges" by Timoshenko and Young (1943): "The deflection of the cable produced by live load is small only in the case of heavy long-span bridges. Otherwise, the deflections may be considerable. In order to reduce these deflections, stiffening trusses are introduced."

The statement could be (and was) misinterpreted to mean that whereas the relatively smaller bridges of Finley and even Roebling needed stiffening, the modern larger spans might not. Timoshenko (1943) ominously comments that "Melan's theory has been widely used in analysis of large-span suspension bridges in this country."

L. S. Moisseiff first applied the theory to the design of the Manhattan Bridge and, later, with F. Lienhard, extended it to lateral forces. Gimsing (1997, p. 15) assessed the development as follows:

> The two-dimensional deflection theory developed by Melan had removed the lower bound for the bending stiffness of the girder in the vertical direction, and now the extension of the deflection theory to cover the three-dimensional behaviour implied that a lower bound for the lateral bending stiffness also disappeared.
>
> In the hands of engineers deprived of their intuitive understanding found in the previous century, and trained to trust blindly the results of their calculations, these analytical achievements could, and should, lead to serious mistakes.

Most serious proved to be the neglect of aerodynamic stability.

1.9.5 Aerodynamic Stability

Whereas approximate static analysis is sufficiently accurate for the preliminary estimates of suspension cable strength, the decks and suspenders add up to a structure with complex dynamic behavior, hard to model theoretically and master practically. As if anticipating the Wheeling collapse in 1854, Roebling (1841) cautioned as follows:

> Railings and longitudinal trusses will not prevent these oscillating motions, but a stiff and well-constructed floor will offer a great resistance. The floors of almost all the English suspension bridges are entirely too light; better specimens in this respect are to be found on the Continent.

Against resonance, apart from his signature diagonal stays, Roebling recommended bringing "the weight of the cables into action by connecting them with the floor at intervals, either by timbers or cast iron pipes, which may include the suspenders."

By the time the Tacoma Bridge failed in 1940, Theodore von Karman (1881–1963) had already presented the airfoil theory for nonuniform motion. After the collapse, Bleich et al. (1950) approached the mathematical modeling of vibrations in suspension bridges as follows:

> While the direct stimulus for the formation of the advisory board on the investigation of Suspension Bridges was the failure of the Tacoma Narrows Bridge, it should not be assumed that this was the first occasion wherein dynamic oscillations in suspension bridge structures resulted in damaging stress effects and failure.

The Tacoma failure confirmed that, along with ignorance, misinterpretation and overconfidence are significant vulnerabilities of design. As all major failures, this one was caused by several factors contributing to dynamic instability. Principal among them was flutter.

"Elementary Theory of Wing Flutter" is formulated in Chapter VI. Section 2 of Von Karman and Biot (1940). In Chapter 7, "Flutter Theory," of Bleich et al. (1950), the phenomenon is presented as a form of "self-excitation." Gimsing (1997) defines it as "a harmonic oscillation characterized by a coupling of the vertical and the torsional oscillations occurring when the frequencies of those two basic oscillations coincide." Tacoma became unstable in part because of the shape of its cross section, but also because of its flexibility. Buffeting and vortex shedding may have contributed as well. For members with a dominant mode of oscillation and low damping, resonance is always a threat.

It can be seen from Table 1.1 that practice and analysis eventually converged upon deck stiffening by space trusses (Figure 1.4 and 1.26), and box girders (Figure 1.5). Both systems have been referred to as stiffening girders. Besides the structural performance under the anticipated loads, the choice of one or the other system can be motivated by life cycle maintenance considerations, construction expertise, and aesthetics. Gimsing (1997) considers the introduction of box girders at the Severn Bridge (1966) as "the most important innovation within suspension bridges in the 20th century." They typically imply orthotropic decks, whereas space trusses can be combined with reinforced concrete panels, concrete-filled or open-steel gratings, and so on, as well.

Gimsing (1997) points out examples of aerodynamic stability, achieved by different measures. The Great Belt Bridge, designed mainly to resist wind, has a compact section with all members interacting. The Akashi Kaikyo Bridge, subject to earthquakes as well, has a heavy truss, independent of the deck. The proposed Messina Straights Bridge, which falls somewhere between the two, relies on weight, activates the cable system, and minimizes aerodynamic forces with the shape of the cross section.

The stiffening systems can be discontinuous at the towers, as in most American bridges, or continuous between the anchorages, as in European ones. On many of the latter, such as the Little Belt, the Great Belt (Figure 1.5), and Pont de Tancarville (Figure 1.11), the suspension cables are connected to the stiffening girder midspan with a central clamp. According to Gimsing (1998), a central clamp inhibits the asymmetric torsion mode, transfers axial loads from the girder to the cable, and protects the short suspenders from fatigue.

1.10 VARIATIONS

The suspension and the cable-stayed structural schemes often compete as alternatives for spans of length approaching 1000 m. Under exceptional demands, however, the two systems cooperate and borrow features from each other. Demands qualifying as exceptional have included extreme span length, excessive dynamic loads, environmental constraints, and the desire for uniqueness.

1.10.1 Self-Anchored Bridges

Similarly to bowstring arches and prestressed girders, self-anchored suspension bridges are self-equilibrated structures. The Chelsea Bridge shown in Figure 1.20 is such an example, as are the three chain-link bridges over the Allegheny River in Pittsburgh (Figure 1.8c). The disadvantage of self-anchoring is the need for temporary supports during construction, as shown in Figure 1.7. Consequently, cable-stayed bridges have almost entirely pre-empted this option. The East Bay San Francisco–Oakland Bridge (Figure 1.7) is a notable exception where aesthetic considerations governed.

The vehicular Konohana Bridge (120/300/120 m, 1990) in Osaka (Figure 1.27) is supported by a unique self-anchored monocable.

1.10.2 Hybrid Bridges

Throughout the evolution of cable-supported bridges, hybrids combining stays and suspenders have been proposed whenever the perceived limitations of either system have been exceeded. An early example is the Albert Bridge over the Thames in London, which opened to traffic in 1873 as a cable-stayed structure of the Ordish–Lefeuvre type, was strengthened

Figure 1.27 The self-anchored monocable Konohana Bridge, Osaka. The Albert and Tower Bridges, London.

by a suspension system in 1887, and became simply supported in 1972 (Figure 1.27). The use of trusses as suspension systems before cables had demonstrated their superiority also produced hybrids, notably, the Tower Bridge in London (Figure 1.27). G. Lindenthal's proposal for such a hybrid across the Hudson River was rejected in favor of O. Ammann's George Washington Bridge.

Roebling's Brooklyn Bridge remains the iconic representative of the hybrid suspension–stay system worldwide. In tribute to J. Roebling, J. Schleich, and R. Walther proposed hybrid twin bridges (Walther and Amsler, 1994) for a replacement of the Williamsburg Bridge. A hybrid bridge was considered for the East Bridge crossing in Denmark before the Great Belt suspension bridge was selected. The Third Bosphorous Bridge is a suspension–stay hybrid designed for mixed vehicular and rail traffic.

Historically, stays have either been concurrent with the suspenders throughout the spans (as in Roebling's designs) or acted as sole supports of portions of the spans near the towers. In 1938, Dischinger proposed a combined cable and stay system in which stays fan out in the proximity of the towers toward the deck, and suspenders support only the center of the span. Gimsing (1997) comments that "strangely enough, although Dischinger adopted the idea of combining the suspension system and the cable stayed system, he did not appreciate the original solution of Roebling with the much more continuous lay-out."

Gimsing (1997) attributes to D. Steinman a proposal for a suspension bridge at the Messina Straits with "negative" stays radiating from the towers at deck level toward the cables in the central span. The idea can be extended to a cable net system, combining suspenders, stays, and secondary or trajectory cables. Steinman designed the Tagus River Bridge in Lisbon (632/1013/632 m, 1966) for vehicular traffic on its original upper deck, but anticipating the addition of a lower deck for two train tracks. In contrast with O. Ammann's George Washington Bridge, however, the Tagus would have been reinforced by diagonal stays, thus making it a hybrid. Instead, in 1991, the bridge towers were heightened and two new main cables were added.

1.10.3 Multispan Systems

Four- and five-span alternatives were considered for the San Francisco–Oakland West Bay Bridge, before the two consecutive suspension bridges (Figure 1.23) were selected. In the absence of a shared anchorage, a suspension bridge with more than three spans (and two towers) is multispan. Such bridges were much more common in the nineteenth century than during the twentieth. The most important example of this type was the 5×109 m span bridge at Cubzac (1835–1869) over the Dordone in France (Marrey, 1990). Its ultimate closure was reportedly caused by scour at one of the foundations.

Figure 1.28 (a) Sunniberg Bridge, Switzerland. (b) Viaduct de Millau, France.

The potential instability of intermediate towers in multispan bridges was remarked on by Navier (Kranakis, 1997). Gimsing (1997) demonstrated that a four-span configuration would develop inadmissible deflections unless stiffened by supplementary means, such as horizontal cables between tower tops, or the more common diagonal stays to the deck level of opposite towers, as at Cubzac and, more recently, the four-span cable-stayed Ting Kau Bridge in Hong Kong (127/448/475/127 m, 1998).

Nets of inclined suspenders supported the five-span San Marcos "cable truss" bridge in El Salvador (76/159/204/159/76 m). The spectacular multi-span cable-supported bridges accomplished in recent years have been extrados, such as the Sunniberg in Switzerland (Figure 1.28a), or cable stayed, such as the Viaduct de Millau in France (Figure 1.28b) and Rion-Antirion in Greece. Significantly, the construction of the Viaduct de Millau also required temporary intermediate supports.

Technological advances have revived even steel plates as continuous-tension elements. Passerelle Simone de Beauvoir (304 m, 2006) over the Seine (Figure 1.29) is a lenticular pedestrian bridge supported by two continuous steel plates (1000/150 mm) acting predominantly in tension, as cables might have done.

The Taizhou Bridge over the Yangtze River (2012) has two suspended spans of 1080 m (3540 ft) length. The central steel tower is 192 m (630 ft), and the two concrete side towers are 178 m (584 ft) high. Two 390 m (1279 ft) long side spans are supported on multiple piers. The suspension cables are two. The deck box girder carries six lanes of vehicular traffic.

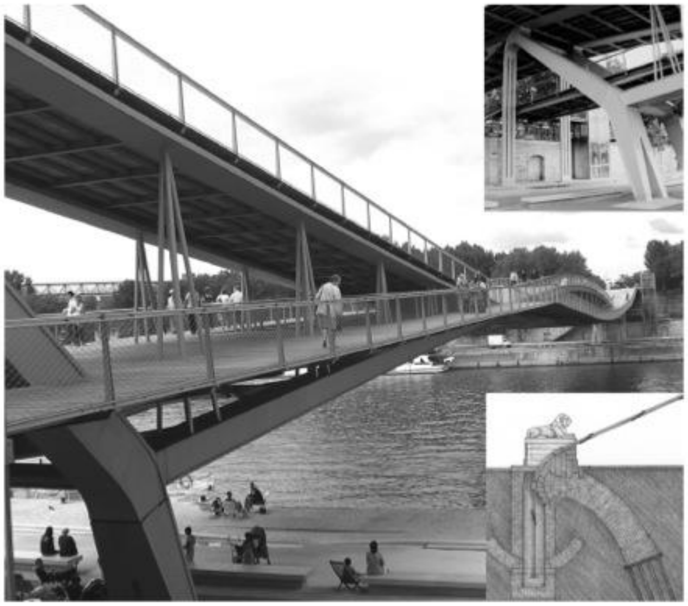

Figure 1.29 Passerelle Simone de Beauvoir, Paris. Insets: Anchorages of the Passerelle and Pont des Invalides.

1.11 LESSONS

Whereas art masterpieces cannot be reproduced, science advances through failures toward understanding. The design and construction of suspension bridges owe much to the lessons of instructive failures.

1.11.1 Failures

Sibly and Walker (1977) and Petroski (1993) discern a cyclic trend in bridge failures at the Dee Bridge in England (1847), the Tay Bridge in Scotland (1879), the Quebec Bridge (1907), and the Tacoma Narrows Bridge (1940). The cycles begin with cautiously successful (partly empirical) designs and culminate by overextending the practice beyond the validity of the model. Petroski (1994) presents the failure of the Dee Bridge at Chester as an example of the "success syndrome," essentially an error of complacency. Structural failures of this type result from applying known routines beyond their valid range. Suspension bridges are particularly vulnerable because their span and function are always at the limit of the technically possible range. After the failure of the Tacoma Narrows Bridge, Ammann (1953) wrote: "[It] has given us invaluable information.... It has shown [that] every new structure [that] projects into new fields of magnitude involves new problems for the solution of which neither theory nor practical experience furnish an adequate guide. It is then that we must rely largely on judgment and if, as a result, errors, or failures occur, we must accept them as a price for human progress."

An early explorer of suspension systems, Leonardo da Vinci (1935), wrote in his *Notebooks*: "Experience is not at fault, it is only our judgement that is in error."

Kranakis (1997) demonstrates how the interpretation of failure causes has evolved concurrently with and sometimes independently from the empirical elimination of incalculable risk.

The following examples continue to advance the art and science of bridge engineering by demanding continuing reinterpretation with every new generation of practitioners.

1.11.1.1 Pont des Invalides (1826)

Navier attributed the failure of Pont des Invalides over the Seine in 1826 to poor material in the anchorages. His drawings suggest a precarious overturning moment as well (Figure 1.29, inset). The massive rigid anchor monoliths of later anchorages (Figure 1.22) eliminate both deficiencies.

Twenty-first-century technology overcame Navier's limitations at the Passerelle Simone de Beauvoir in Paris (Figure 1.29, inset). The geometry of the Passerelle anchorage vaguely resembles the one at Invalides, the better to expose the critical differences. The masonry compression strut is replaced by steel. The tension element approaches the anchorage at an acute, rather than an obtuse, angle. Highly compressed flat arches aboveground and highly pretensioned strands below grade resist the overturning moment. Whereas the nineteenth-century anchorage was entirely passive, the twenty-first-century one can be viewed as, if not active, then at least responsive. Maintaining that response becomes a life cycle responsibility.

1.11.1.2 Wheeling Bridge (1848–1854)

Over a period of 86 years after the collapse of this world's longest span, the need for deck stiffening, by Roebling's hybrid system or by trusses, remained empirically obvious. Then the lesson had to be reformulated and relearned analytically.

1.11.1.3 Tacoma Narrows Bridge (1940)

The failure of the Tacoma Narrows Bridge stimulated both theory and empiricism. The results obtained by the deflection theory were recognized as potentially nonconservative. Von Karman's analysis of airplane wing aerodynamic stability was applied to bridges. Space trusses, box girders, and inclined suspenders henceforth compete as means of deck stiffening.

Large-scale physical models of entire bridges or deck sections are tested extensively in wind tunnels. For the wind tunnel test of the Tsing Ma

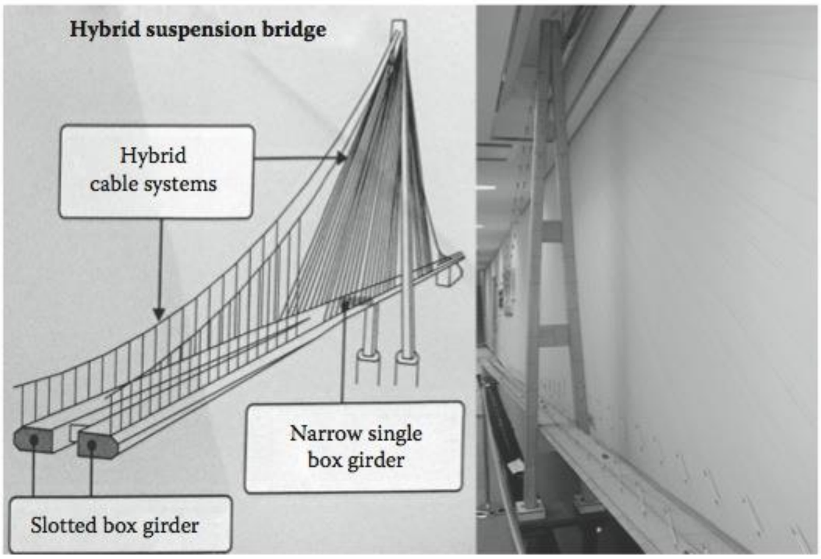

Figure 1.30 Study of hybrid bridge (1100/2800/1100 m), PWRI, Tsukuba City.

Bridge, a 300 m long section was modeled at a scale of 1/90. A 1/100-scale model for the full-length Akashi Kaikyo Bridge was tested in a 41 m wide wind tunnel built for the purpose at Tsukuba City.

Around 1820, the Seguin family tested an 18 m long, 0.5 m wide model of their suspension bridge at Marc Seguin's property near Annonay (Marrey, 1990). Models of hybrid bridges with a total length of 5000 m are tested as of this writing (Figure 1.30).

Weigh-in-motion systems have been installed or are considered for many long-span bridges for accurate assessment of the impact of overweight vehicles.

1.11.1.4 Point Pleasant Bridge (1928–1967)

The Silver Bridge over the Ohio River at Point Pleasant failed in December 1967, due to the brittle fracture of one critically nonredundant eyebar. An uninspectable material defect had initiated the fracture. In direct response, the U.S. Congress launched the National Bridge Inventory (NBI), and thereby modern bridge management.

Recognizing the superiority of high-strength steel wire cables over eyebar chains implies several important lessons:

- Cost is shown to be an insufficient criterion in the selection of essential structures with long spans in space and perpetual life cycles in time.
- Redundancy is understood to imply not only static indeterminacy, but also a viable path of load redistribution. Internal redundancy and ductility become increasingly associated. Fracture-critical structural elements are targeted for elimination.

- John Roebling's singular blend of practical judgment and scientific knowledge reemerges as the prototype for a successful integration of the process and product of the suspension bridge. His designs prefigure the contemporary notions of robustness and resilience. His hybrid suspension–stay system remains relevant every time spans exceed customary lengths and stiffening becomes critical. Most major malfunctions of suspension bridges can be traced to ignoring one or more of Roebling's lessons.

1.11.2 Partial Failures

Another definition of structural failure is nonperformance. Partial performance amounts to partial failure. With the opportunity to examine nonperformance over time come the responsibilities to arrest its progress and eliminate its cause.

Many important improvements and innovations in the design, construction, and management of suspension bridges have been inspired by partial failures. In such cases, it is of great benefit to determine what has prevented the total failure. On the conceptual level, the Brooklyn Bridge and Roebling once again supply the example. Roebling's response to the criticism of his heterogeneous means of load distribution was that if any of his systems fails, the bridge "may sag but shall not fail." Billington (1983) speculates that, lacking the analytic tools, Roebling obtained a working stiffening scheme formalistically. The rigorously analytic Navier had rejected the combination of suspenders and stays (Kranakis, 1997).

Roebling's reasoning in favor of redundancy and robustness was confirmed in 1981 when a diagonal stay with an estimated weight exceeding a ton fell and killed a pedestrian on the Brooklyn Bridge promenade. All stays and suspenders had corroded nearly to failure. Despite the advanced state of deterioration, however, the entire system was replaced under traffic because of its redundancy, as design had anticipated.

Throughout the twentieth century, parallel wire cables have demonstrated a considerable resilience, even when their protection from corrosion has been neither redundant nor robust. The cable wires of the Williamsburg Bridge had not been galvanized on the strength of the argument that if they were not protected, zinc would not save them, and if they were, zinc would be redundant. Besides savings from the deferred galvanization, the benefit was a lighter cable. In 1988, strands of the main cables were found corroded to failure. One strand was broken in the Manhattan Anchorage. Conservative design saved the bridge. The four cables themselves had been designed with a robust strength reserve estimated at 4.2.

Whereas a crack initiation in a fracture-critical element is hard to spot before it causes a global failure, wire breaks in a properly designed and inspected cable-supported structure, as in the cases shown in Figure 1.31, can be treated as localized failures with a reversible effect. Taking into account

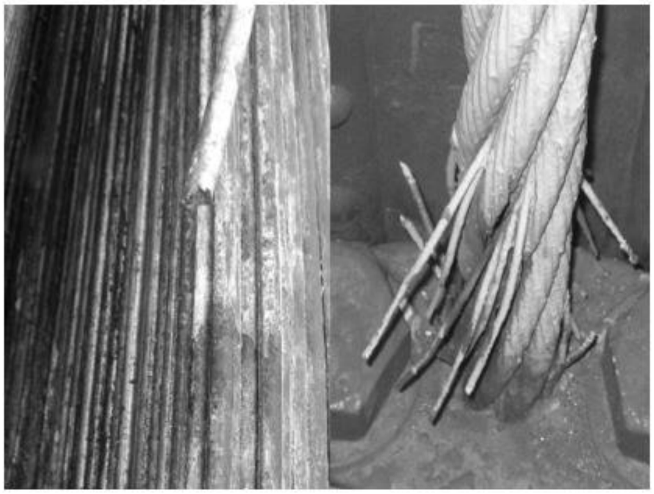

Figure 1.31 Broken wires in a main cable and in a suspender rope.

the observed corrosion (Figure 1.9), as well as internal stress redistribution through friction, the strength reserve of the Williamsburg Bridge cables was found to have dropped to a still acceptable factor of approximately 3. Future maintenance would have no further tolerance for partial failures.

Corrosion of anchorage eyebars necessitated the reanchoring of cable strands at Manhattan, Williamsburg, RFK Triborough, George Washington, and Bronx-Whitestone Bridges. A partial reanchoring of a strand is illustrated in Figure 1.32. The corrosion of the original anchor is attributed to leakage of the finger joints above the anchorage and to lack of dehumidification

Figure 1.32 Reanchoring of a strand.

within. Thus, once again, the product and the process are both at fault for relying excessively on each other.

Certain missteps in the original concept or execution emerge only after years of service. For example, the straps compacting the cable strands, as shown in Figure 1.21a, can have a corrosive effect if made of mechanically intrusive or chemically active material. The latter was true of the original bronze encasement in Figure 1.21c.

Helical strand cables are particularly vulnerable to corrosion concealed within the strands. Since a partial failure manifests itself by a broken strand, rather than individual wires, the strands themselves can be viewed as fracture-critical. The Waldo-Hancock Bridge was demolished in 2013. The new helical strands at Pont de Tancarville (1998) and Pont d'Aquitaine (2002) are galvanized but not locked-coil ones.

As all significant structures, suspension bridges fail because of more than one cause. Failures in the product and the process are investigated by forensics and management, respectively. If the two types of investigations do not converge, their findings also fail to achieve full impact. No structure demonstrates better than a suspension bridge the critical importance of continuity within, as well as between, the process and the product.

The Kukar suspension bridge (100/270/100 m) in Indonesia collapsed on November 26, 2011. At least one analysis has argued that the failure had "plural sources but a single cause." The following facts are reported. At least one anchorage block, sitting on vertical piles, slipped. The tower tilted and the main span sagged. To correct the sag, one of the two trusses was jacked up by midspan suspender 13 (possibly by 10 cm). The cast-iron clamp of the suspender to the main cable fractured, followed by a fracture of the symmetric one on the opposite cable. The remaining clamps, spaced at 10 m, failed successively. Prior to the failure, the suspenders were experiencing increasing vibrations. The two main cables consisted of 19 helical strands. Slippage of the cable clamps had been reported. The bridge had been in service for 10 years.

The effort to identify and prevent causes for partial and total failures from developing in the process and the product has engendered the notions of vulnerabilities and potential hazards. Under the term *risk*, Hovhanessian and Laurent (2006) seek vulnerabilities in critical elements of the product (i.e., the suspension bridge) and in the key stages of the process, comprising design, construction, aging, operation, hazards (i.e., extreme events), and maintenance. Previously vague considerations known to avert potential failures and improve structural performance have become explicit and specific, as, for example, the following.

1.11.2.1 Redundancy/Robustness

The inability to redistribute functional demands makes the difference between the partial and total failure. Thus, robustness can be perceived as a redundancy extended beyond the alternate load paths to encompass unforeseen functional demands. Bridge elements, including suspenders and stays, as well as deck panels, have to be designed for maintenance and eventual replacement without service interruption, as well as for the partial failures of these operations.

1.11.2.2 Inspectability/Maintainability

The inaccessible is unmanageable. Since the main suspension cables cannot be accessible throughout their cross section, particular attention must be paid to their maintenance. The experience with the Williamsburg Bridge inspired two types of countermeasures.

Under project FHWA-HRT-14-024, Columbia University monitored the humidity, temperature, corrosion, and other parameters in a 10,000-wire cable in laboratory conditions and on the Manhattan Bridge, in order to provide online information about the ambient level of corrosiveness. The Honshu-Shikoku Bridge Authority (HSBA) developed the method of dry-air injection (Figure 1.33) under a wrapping of z-shaped high-strength wire (Figure 1.9e). By reducing the cable humidity to 40%, the system precludes any corrosion. The cable dehumidification method is gaining application worldwide. Most anchorages are now considered maintenance-intensive areas and their dehumidification is routine.

1.11.2.3 First/Life Cycle Cost

After the financial constraints of the 1930s precluded a proposed second level at the Triborough Bridge, Robert Moses reportedly commented that in 40 years, New Yorkers would build a bigger bridge anyway. Eighty years later, the indispensable RFK Triborough Bridge was rehabilitated extensively under daily traffic of up to 200,000 vehicles.

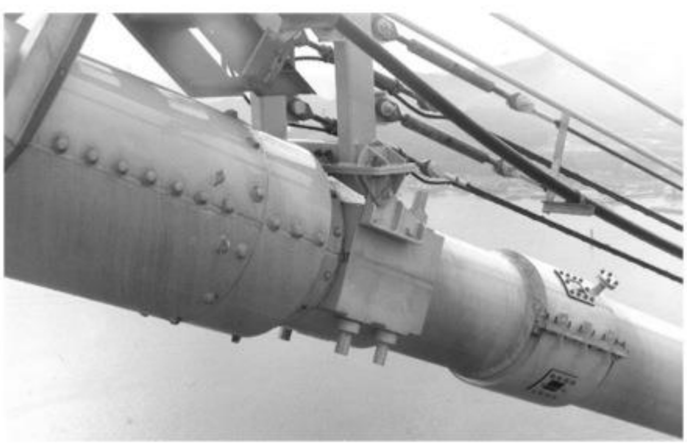

Figure 1.33 Cable dehumidification at the Kurushima Bridge.

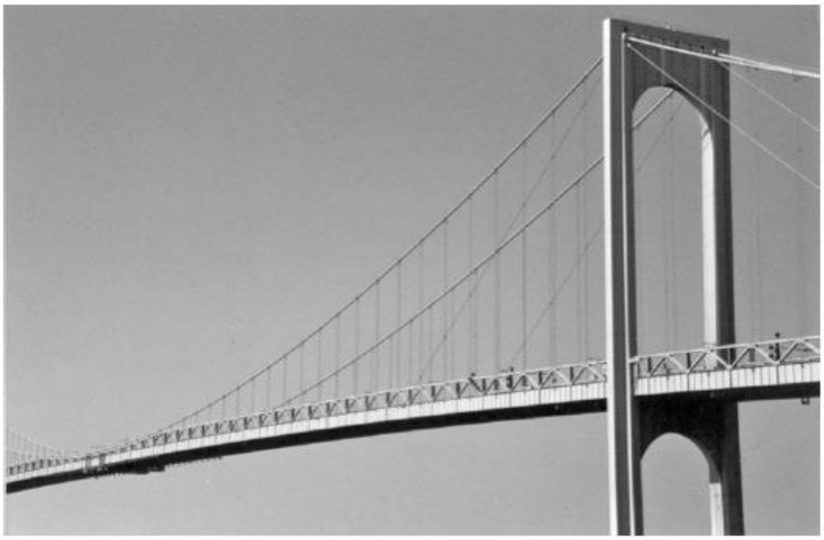

Figure 1.34 The Bronx-Whitestone Bridge, 1990.

At the time of their construction, record-breaking suspension bridges, similarly to tall towers, generate an excitement more appropriate for competitive sports. Once in service, however, they become fixtures of the regional topography and shortcuts in the social brain. Cities cannot exist and countries are not neighbors without them. After more than a century, the required service changes in nature and increases in volume, rendering a closure unacceptable. A million users cross the three East River suspension bridges (Figures 1.2 and 1.25) daily. This comes at a price. Over the last 20 years, the same three bridges have absorbed over US$3 billion in rehabilitation projects. Maintenance costs are supplemental.

The Bronx-Whitestone Bridge (1939), originally similar to the Tacoma Narrows Bridge, was equipped with diagonal stays, stiffening trusses (1946), and a tuned mass damper (1985) (Figure 1.34). Ultimately, all added features were to be scrapped in favor of a modified girder profile and stiffening. Modifications to the roadways, the cables, and the anchorages are under review (Lorentzson et al., 2006). The primary design constraint is clearly not the bridge, but the essential service it has provided for more than 70 years.

The lower deck of the George Washington Bridge was installed in 1962, to accommodate the growing demand for vehicular traffic. The average daily traffic (ADT) reported in 2011 was 276,150 vehicles.

Considerations about the full replacement of the Williamsburg Bridge in 1988 showed that despite the billion-dollar price tag, rehabilitation was preferable if user costs were taken into account.

If suspension bridges are designed for a perpetual life cycle, periodic upgrading must be part of it. Partial rehabilitation is a certainty.

1.11.2.4 *Function/Form*

While artists are balancing form and function, scientists are reconciling determinism and uncertainty. The form of long-span suspension bridges is determined by natural constraints. Function remains uncertain throughout the life cycle. Unforeseen vulnerabilities, as well as benefits, may surface early on or long after a span has made its mark in the rankings for unsupported length.

In the early twenty-first century, as in the nineteenth, random winds and earthquakes, as well as predictable of pedestrians, still excite suspension bridges beyond expectations. In 2000, the torsional stiffness of the three-span Millennium Bridge in London (Figure 1.35), which combines suspension and extrados features, had to be corrected in order to accommodate self-synchronizing pedestrians.

In a statement similar to Roebling's (1841) argument about the importance of science, Eiffel famously argued that his tower would be beautiful because the equations that describe it are correct. At the end of his spectacular career, Ammann (1879–1966) identified luck as his best asset (Talese, 1964). The creativity of these masters, however, is revealed not in their words, but in the unity between their product and process. There is no separation or conflict between the beauty and utility of their structures. The success of suspension bridges at the extreme edge of the possible depends on their organic integration of process and product, form and function, design and performance. Hence, a review of their evolution may contribute to cultivating a taste for it.

Figure 1.35 Millennium Bridge, London.

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