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THE ROLE OF DECK STIFFENED SYSTEM IN THE EVOLUTION OF SUSPENSION BRIDGES

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ABSTRACT

Passing from the earliest suspension bridges to the modern aerofoil deck solutions, this paper wants to trace the design evolution of suspension bridge, looking at the improvements in deck stiffened system, till reaching record slenderness nowadays. The excursion wants to emphasize the effects on structural response that deck characterization could have. To this aim, some peculiar case studies have been analyzed (*back analysis*). The proposed analytical approach leads to synthetize the evolution of suspension bridges through three main design parameters: L_{main}, the length of the main span; (h/L), deck-depth-to span ratio to define deck slenderness; (w/L), width deck-to-span ratio to characterize deck torsional stiffness. The structural optimization process towards slender and lighter structures has been marked by a change in deck cross section type, passing from the early truss-deck stiffened systems to modern streamlined deck box sections. As consequence of deflection theory application and later of aerodynamic studies, great improvements in suspension bridge technology have occurred. Tracing the design evolution of this typology, a classification in three different generations has been proposed, resulting from different ways to conceive stiffened girder: first generation (1883-1940), characterized by rigid deck suspension system calculated on the basis of linear theory (L_{max} = 850m; w/L_{mean} = 1/22; h/L_{mean}= 1/84); second generation (1940-1966), characterized by slender truss-deck suspension system computed in accordance to the deflection theory (L_{max} = 1298 m; w/L_{mean} = 1/37; h/L_{mean} = 1/103); third generation (1966-2022), mainly characterized by aerofoil deck systems as results of studies on aerodynamic stability (L_{max} = 2023 m; w/L_{mean} = 1/40; h/L_{min}= 1/578).

SOMMARIO

Passando dai primi esempi di ponti sospesi alle più recenti realizzazioni, la presente memoria descrive l'evoluzione progettuale dei ponti sospesi, soffermandosi sulle migliorie tecniche che hanno riguardato la caratterizzazione dell'impalcato, fino a raggiungere snellezze record. L'excursus vuole sottolineare come la risposta strutturale di tale tipologia sia fortemente condizionata dalla caratterizzazione dell'impalcato. A tal fine, sono stati analizzati diversi casi studio (*back analy-*

sis). L'approccio analitico ivi proposto mira a sintetizzare l'evoluzione della tipologia sospesa attraverso tre parametri: L_{main}, lunghezza della campata principale;(h/L), rapporto tra altezza di impalcato e lunghezza della campata principale, per parametrizzare la snellezza dell'impalcato; (w/L), rapporto tra larghezza di impalcato e luce della campata principale, per indicizzarne la rigidezza torsionale. Il processo di ottimizzazione verso strutture sempre più leggere è stato segnato da un cambio di configurazione dell'impalcato, passando da rigide travature reticolari a snelli profili aerodinamici, come conseguenza dell'applicazione della teoria del secondo ordine, prima, e degli studi aeroelastici, poi. Tracciando l'evoluzione progettuale dei ponti sospesi, parallelamente al susseguirsi di nuovi approcci teorici, si propone una classificazione in tre successive generazioni: prima generazione (1883-1940), caratterizzata impalcati rigidi calcolati sulla base della teoria lineare (L_{max} = 850m; w/L_{mean} = 1/22; h/L_{mean} = 1/84); seconda generazione (1940-1966) con impalcati a travatura reticolare più snelli, risultanti dell'applicazione della teoria del secondo ordine (L_{max} = 1298 m; w/L_{mean} = 1/37; h/L_{mean} = 1/103); terza generazione (1966-2022) caratterizzata da moderni profili aerodinamici (L_{max} = 2023 m; w/L_{mean} = 1/40; h/L_{min}= 1/578).

1 INTRODUCTION

Some lines written by Steinman in "*The Builders of the Bridge – The Story of John Roebling and His Son*" (1944) [1] well explain the peculiarities of suspension systems. Referring to Niagara Bridge design and construction, he said: […] *From the most primitive swinging spans of twisted vines and fibers, there had of course been successive improvements in materials and in details of construction, but the full potentialities of the suspension type could not be realized as long as it continued to be represented by swaying, undulating structures. As Roebling expressed it: "Suspension bridges have generally been looked upon as loose fabrics hung up in the air, as if for the very purpose of swinging. Repeated failures of such works have strengthened this belief". [..]. His success in the construction of suspended aqueducts – as rigid as stone or cast iron aqueducts-demonstrated the truth of his thesis that cable spans could be built as stiff as desired. […] But it was in the Niagara Bridge that his new concept received its first full expression – the first use of stiffening trusses in all the history of bridge building* (1855)". Passing from the earliest truss-deck applications to the modern aerofoil deck solutions reaching record slenderness nowadays, this paper wants to argue about the role of deck stiffened system in the evolution of suspension bridges. Remarkable collapses [2] [3] [4] of the earliest applications during the 19th century have underlined the importance of having a rigid deck to better exploit the potentialities of this structural system. Meanwhile, the improving studies concerning aerodynamic stability have led to the use of even lighter and slender decks, reducing bridge dead load and construction costs.

Form past examples to nowadays record span structures, suspension bridges have always been characterized by four main components: (1) the deck (or stiffening girder); (2) the cable system supporting the deck; (3) the pylons (or towers) supporting the cable system; (4) the anchor blocks (or anchor piers). Considering that suspension system configuration has had no many changes since the origin, despite a great improvement in cable technologies, there is no doubts that the most influential aspect in suspension bridge design has always been the deck configuration. The process towards its structural optimization has been marked by two main theoretical approaches, firstly the linear theory [5] [6] [7], then the deflection theory by Ritter in 1877 [8], Lévy in 1886 [9], and Melan in 1888 [10]. To better understand the differences between these approaches, a simple load scheme can be considered. It is assumed that upon a cable, suspended between two points, is applied a uniformly distributed dead load, so that the bending moment diagram, as the resulting equilibrium curve of the suspended cable, is a parabola. When a uniformly distributed live load asymmetrically acts only upon one half of the cable, the corresponding funicular curve doesn't match with the initial parabola, leading to an increase of bending effects.

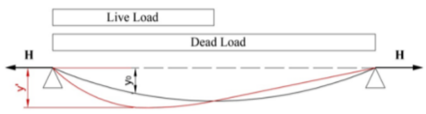

Fig. 1. Linear theory

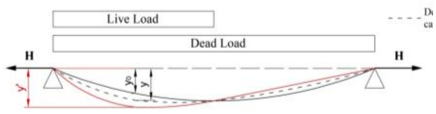

Fig. 2. Deflection theory

In the earliest applications of suspension bridges, in order to restrict these static distortions of the main cable, as consequence of loads transferred by hangers, a stiffening truss was adopted. According to Steinman's theory, it was necessary to use a truss-deck sufficiently stiff to make the deformations of the cable due to live loads practically nihil. Referring to Fig. 1, in this case bending effects can be estimated as M= H·y', where (H) is the horizontal force related to the funicular curve, and (y') is the vertical distance from the cable chord to the final funicular curve. The deflection theory, instead, is a nonlinear elastic theory that takes into account the deformed shape of the main cable under traffic load when calculating the bending moments in the stiffening truss. Thus, the equilibrium is established more correctly for the deflected system. Being connected by the hangers, the deflection of the deck will cause a change in the geometry of the main cable. Referring to Fig. 2, in this case bending effects can be estimated as the horizontal force (H) multiplied by the vertical distance (y'-y) from the funicular curve to the distorted cable. Taking into account the nonlinear effect, the bending moments in the deck is reduced, also to less than half of the one calculated by a linear theory. Nowadays, a great contribution towards even slender decks has derived from studies on aerodynamic stability: streamlined aerofoil deck-box sections can ensure unexpected slenderness (h/L = 1/578) covering record spans (L_{max} = 2023 m).

2 THE EVOLUTION OF DECK STIFFNED SYSTEM

In order to describe how this typology has evolved over the centuries, three main design parameters have been considered: L_{main}, as the length of the main span; h/L, to define deck slenderness; w/L, as index of deck torsional stiffness. As visible in the following excursion, the structural optimization process towards slender and lighter structures is marked by a change in deck cross section type, passing from the early truss-deck stiffened systems to modern aerofoil deck solutions. Looking at technical improvements occurred, the design evolution has been traced. A classification in three successive generations is proposed, resulting from different ways to conceive stiffened girder. Data from the back analysis of 43 existing bridges are discussed (the number in parenthesis that accompanies the name of bridges indicates their position in the whole data base).

2.1 First generation (1883-1940)

As a consequence of the collapses of suspension bridges, occurred from 1818 (Dryburg Bridge) to 1889 (Niagara Bridge), also due to wind oscillations, firstly Roebling voiced his perception that stiffened girder and additional inclined stays could be necessary to make stiffer suspension bridges: Wheeling Bridge (1882) and Brooklyn Bridge (1) (1883) became exemplary cases of this type. The earliest applications that marked the transition from linear to deflection theory (1883 – 1940) till Tacoma Narrow collapse, resulted from this precautionary design approach, that preferred massive stiffened girders, as in the case of Williamsburg Bridge (2). This results in high values of h-to-L ratio (mean of 1/84), as visible in Table 1. Some bridges included in the 1st generation became paradigmatic cases in the evolution of suspension bridges.

Table 1. Synthesis of suspension bridges of the 1st generation (1883-1940)

	year	Bridge	Country	L_{main} [m]	h/L	w/L
1	1883	Brooklyn Bridge	US	486	1/37	1/19
2	1903	Williamsburg Bridge	US	488	1/41	1/14
3	1912	Manhattan Bridge	US	448	1/61	1/12
4	1931	George Washington Bridge -single deck	US	1067	1/628	1/30
5	1937	Golden Gate Bridge-stiffned	US	1280	1/168	1/47
6	1939	Bronx–Whitestone Bridge - unstiffened	UK	700	1/206	1/30
7	1940	1st Tacoma Narrow Bridge	US	854	1/356	1/67

A crucial example to understand the changing role of stiffened girder for suspension bridges is Amman's George Washington Bridge. When it was firstly opened to traffic on October 25, 1931, with a 1,067m long main suspended span, it was the longest bridge at the time. The original structure (Fig. 3) had single deck and did not include a stiffening truss (unlike other suspension bridges built in that period). A stiffening truss was not necessary because the long roadway and cables provided enough dead load to make stable the bridge deck, while cables along the short side spans acted as tendons, reducing its flexibility. With this masterpiece, Amman firstly theorized that heavy stiffening trusses were no necessary for long span suspension bridges [11] [12], pushing towards a new approach in bridge design. Apart from the stabilizing effect of dead loads, the choice of a thickening hanger system (cable spacing-to-L ratio, i.p./L= 1,72%), as well as the increasing number of transversal load-bearing elements gave the possibility to reduce girder sizing. The addition of the lower level (Fig. 4) and of the stiffening truss in 1962, was due to the increase of traffic demand and not to structural deficits in the existing slender deck. Another emblematic case of this generation is the 1st Tacoma Narrows Bridge (7), whose collapse firstly underlined the necessity to take into account aerodynamic stability for suspension bridges. Designed by Moisseff in accordance to the deflection theory, it had a slender open deck with a very low torsional stiffness: in this case, the primary cause of its collapse lies in the general proportions of the bridge and the type of stiffening girders and floor. This event led to improve some other existing bridges, as occurred for the Golden Gate Bridge (5) and Bronx–Whitestone Bridge (6).

Fig. 3. George Washington Bridge (4), 1931

Fig. 4. George Washington Bridge (11), 1962

2.2 Second generation (1940-1966)

The collapse of the 1st Tacoma Bridge (Fig. 5) and its reconstruction firstly led to investigate the issues of aerodynamic stability in designing of suspension bridges. To face this aspect, the 2nd generation of large suspension bridges featured deep and rigid decks. As visible in Table 2, it resulted in the use of truss system, having a higher torsional stiffness in comparison to the one valued for the 1st Tacoma Narrows Bridge.

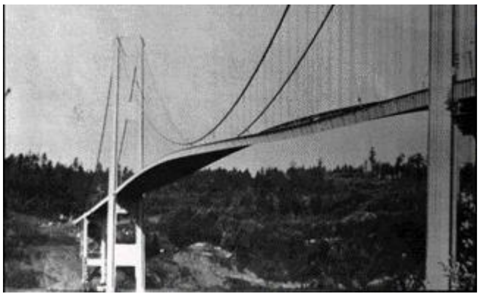

Fig. 5. 1st Tacoma (7) before collapse, 1940

Fig. 6. 2nd Tacoma Bridge, 1950

In particular, looking at the 2nd Tacoma Bridge (9) (Fig. 6), the deck slenderness (h/L) passed from 1/356 to 1/85; at the same time, deck torsional stiffness (w/L) grew from 1/65 to 1/47 while the overall dead load was increased of about 60%.

Table 2. Synthesis of suspension bridges of the 2nd t generation (1940-1966)

	year	Bridge	Country	L_{main} [m]	h/L	w/L
8	1947	Bronx–Whitestone Bridge - stiffened	US	700	1/91	1/30
9	1950	2nd Tacoma Narrows bridge	US	854	1/85	1/47
10	1957	Mackinac Bridge	US	1158	1/100	1/56
11	1962	George Washington Bridge -stiffened	US	1067	1/121	1/30
12	1964	Verrazano Narrows	US	1298	1/178	1/41
13	1964	Forth Road Bridge	UK	1006	1/123	1/30
14	1966	25 de Abril Bridge	Portugal	1013	1/78	1/41

2.3 Third generation (1966-2022)

After Tacoma Narrows Bridge collapse (1940), American engineers realised the problem of aerodynamic stability and further extended span lengths. Two main approaches were used to improve bridge deck stability: (1) adopting a stiffening truss and open grating deck, in order to eliminate the generation of wind vortices; (2) increasing stiffness, adding mass (or weight) to the bridge. A completely different method was introduced by European engineers, adopting streamline-shaped box cross sections, whose aerofoil profile could reduce wind pressure effects, suppressing the emergence of vortices [12] [13]. The first suspension bridge which embodied this revolution was Severn Bridge (15), reaching the unexpected deck slenderness h/L = 1/319. Looking at data in Table 3, the opposite approaches are recognisable by the values of slenderness ratio.

Fig. 7. Yangsigang Yangtze Bridge (39), 2019

Fig. 8. Çanakkale Bridge (43), 2022

Table 3. Synthesis of suspension bridges of the 3rd generation (1966-2022)

	year	Bridge		Country	L_{main} [m]	h/L	w/L
15	1966	Severn Bridge	(aer.)	UK	988	1/319	1/31
16	1973	First Bosphorus Bridge	(aer.)	Turkey	1074	1/358	1/32
17	1981	Humber Bridge	(aer.)	UK	1410	1/313	1/49
18	1985	Ohnaruto Bridge	(t.d.)	Japan	876	1/70	1/26
19	1986	Shimotsui-Seto Bridge	(t.d.)	Japan	940	1/75	1/31
20	1998	Akashi Kaikyo Bridge	(t.d.)	Japan	1991	1/142	1/56
21	1998	Great Belt East Bridge	(aer.)	Denmark	1624	1/406	1/52
22	1998	Fatih Sultan Mehmet Bridge	(aer.)	Turkey	1090	1/363	1/28
23	1999	Jiangyin Yangtze River Bridge	(aer.)	China	1385	1/462	1/47
24	1999	First Kurushima Kaikyo Bridge	(aer.)	Japan	600	1/240	1/22
25	1999	Second Kurushima Kaikyo Bridge	(aer.)	Japan	1020	1/237	1/38
26	1999	Third Kurushima Kaikyo Bridge	(aer.)	Japan	1030	1/240	1/38
27	2005	Runyang Yangtze River Bridge	(aer.)	China	1490	1/497	1/38
28	2007	Yangluo Yangtze River Bridge	(aer.)	China	1280	1/356	1/33
29	2007	Aizhai Bridge	(t.d.)	China	1176	1/157	1/48
30	2009	Xihoumen Bridge	(aer.)	China	1650	1/471	1/49
31	2010	Baling River Bridge	(t.d.)	China	1088	1/109	1/39
32	2012	Yi Sun-sin Bridge	(aer.)	SouthKorea	1545	1/507	1/53
33	2013	Hardanger Bridge	(aer.)	Norvay	1310	1/409	1/72
34	2015	Qingshui River Bridge	(t.d.)	China	1130	1/161	1/33
35	2016	Osman Gazi Bridge	(aer.)	Turkey	1550	1/326	1/43
36	2018	Hålogaland Bridge	(aer.)	Norvay	1145	1/520	1/62
37	2018	Second Dongtinghu Bridge	(t.d.)	China	1480	1/164	1/42
38	2019	Nansha Bridge	(aer.)	China	1688	1/422	1/34
39	2019	Yangsigang Yangtze River Bridge	(t.d.)	China	1700	1/170	1/52
40	2020	Chajiaotan Bridge	(t.d.)	China	1200	1/170	1/44
41	2020	Jin'an Bridge	(t.d.)	China	1386	1/146	1/51
42	2021	Xingkang Bridge	(t.d.)	China	1100	1/134	1/41
43	2022	Çanakkale Bridge	(aer.)	Turkey	2023	1/578	1/45

(t.r.) as truss-deck solution; (aer.) as aerofoil deck solution.

It can be estimated a maximum value of h/L=1/171 for Yangsigang Yangtze Bridge (39) [14] as example of truss-deck solution, Fig. 7, against the lower one, h/L= 1/578, valued for the slender wind sensitive structure of the Çanakkale Bridge (43) [15] visible in Fig. 8.

3. Parametrical evaluations from the "back analysis" of existing bridges

Previous analyses have showed how suspension system has guaranteed to cover longer spans, reaching record length of 2000m. Growing span has often been combined to a reduction of bridge deck depth (as consequence of deflection theory application), until using streamlined-shaped box sections, capable to counteract wind effects. Looking at the following pictures, it is easily to understand how deck stiffened system has changed through the generations. After Roebling's first attempts to remark the need of stiffened deck system for long span suspension bridges (as in Niagara, Ohio and Brooklyn bridges), earliest unstiffened proposals made the way for rigid double deck solutions. A new generation of stiffened truss deck followed: except for Williamsburg Bridge (2), designed according to linear theory, from Manhattan (3) to George Washington Bridge (4), design proposals opted for slender structures, as a consequence of deflection theory application. But, the 1st Tacoma (7) collapse underlined the necessity to take into account also dynamic effects due to acting loads (above all wind). If the more precautionary American approach led to heavy and rigid truss system to cover longer span, European designers proposed the first aerofoil decks, whose streamlined-shape prevented them from aerodynamic instability. Fig.9 shows the evolution in terms of growing main span, valuing: for the 1st generation a mean span length of 760m, with a maximum of 1280m for the Golden Gate Bridge (5); for the 2nd generation a mean value of 1013m, with a maximum of 1298m for the Verrazano Narrows (12).

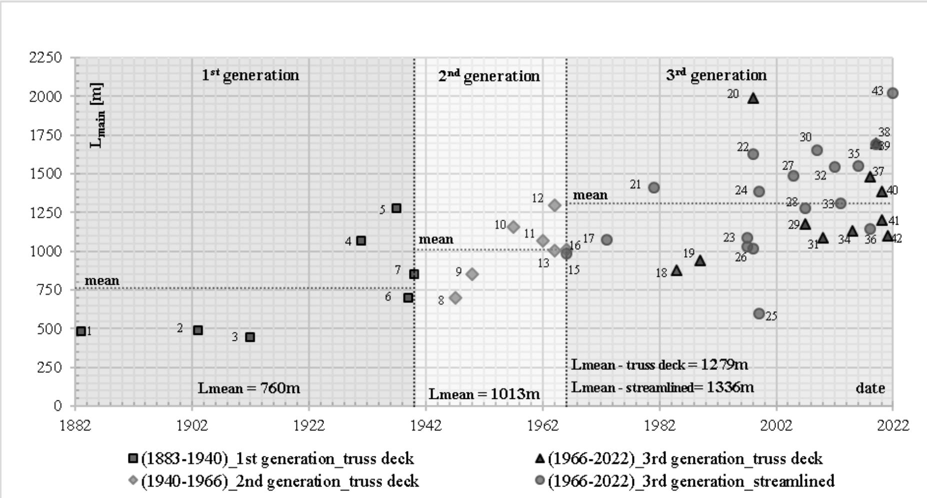

Fig. 9. Growing main span (L_{main}) from 1883 to 2022

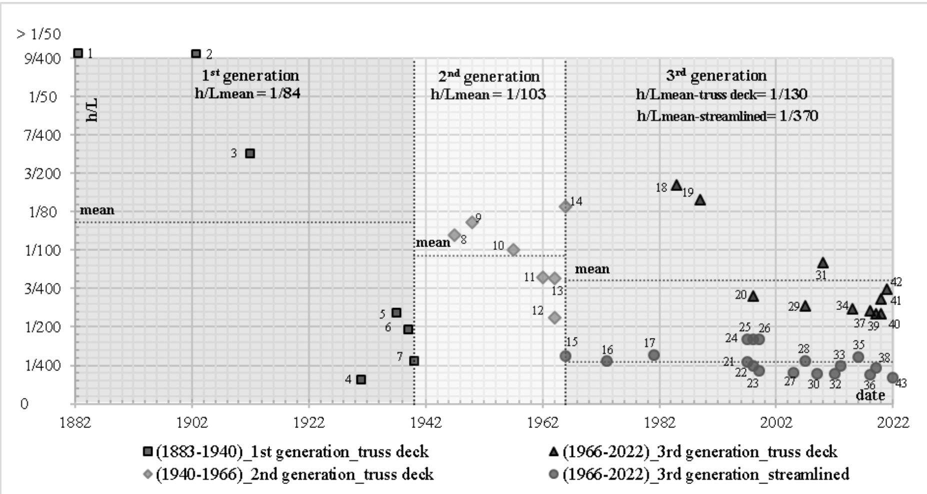

Fig 10. Deck slenderness (h/L) from 1883 to 2022

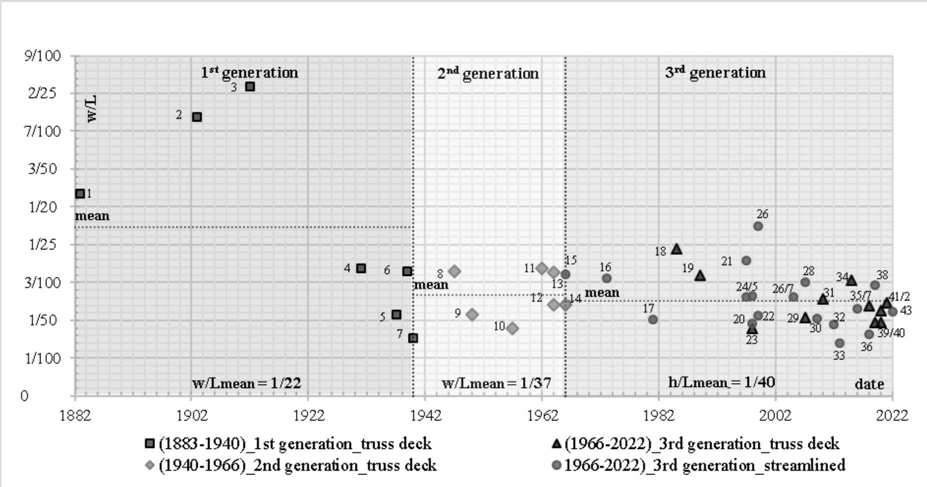

Fig. 11. Deck torsional stiffness (w/L) from 1883 to 2022

For the 3rd generation a comparison between truss-deck systems and aerofoil solutions can be considered: the mean span length for truss deck solutions is 1279m, with a maximum of 1991m for the Akashi Kaikyo Bridge (20); for the aerofoil deck system, the mean L_{main} is 1336m, with the record span of 2023 of the Çanakkale Bridge (43). Fig.10 shows the evolution in terms of deck slenderness. For the 1st generation a mean value of 1/84 can be estimated, with the pioneering George Washington Bridge (4) having h/L= 1/628 before the introduction of the stiffening truss, while the collapsed 1st Tacoma (7) had h/L= 346. The 2nd generation showed a more conscious approach in the application of deflection theory, resulting in slender truss-deck systems, with a mean h/L of 1/103. During the 3rd generation two different approaches are clear: form one side stiffen truss-deck system led to a mean slenderness of 1/130, while the streamlined aerofoil solutions have given the possibility to cover records spans, reaching the unexpected slenderness ratio of 1/578 valued for Çanakkale Bridge (43). Finally, Fig.11 refers to deck torsional stiffness: the 1st generation was characterized by the lowest values, recorded in the case of 1st Tacoma (7) and Golden Gate Bridge (5), later properly retrofitted. After Tacoma collapse, a mean value of w/L = 1/40 has been adopted, until today.

4 CONCLUSIONS

The paper argues about the evolution of suspension bridges, focusing on technical improvements occurred in deck cross section characterization. Through the back analysis of 43 existing structure, a classification into three successive generations has been proposed. In this case, the evolution is traced looking at three main design parameter, as: length of the main span; deck slenderness and deck torsional stiffness. Starting from the lessons learn from spectacular collapses of suspension bridges, occurred in1818 -1940, the passage from one generation to another is marked by the application of different theoretical approaches, from the linear theory to the deflection one, until modern studies on aeroelastic stability. This has led to an evolution in deck stiffened system, passing from the earliest open decks to the massive rigid truss-systems of the 1st generation, to slender and wider truss solutions of the 2nd generation; in continuity, the 3rd generation has seen both the improvement of this last type used to cover record spans, as well as the design optimization by aerofoil deck solutions which have given the possibility to reach unexpected slenderness.

Fig.12 synthetizes the evolution of suspension bridges and refers to the deck cross section characterization of some emblematic examples describing the aforementioned three generations.

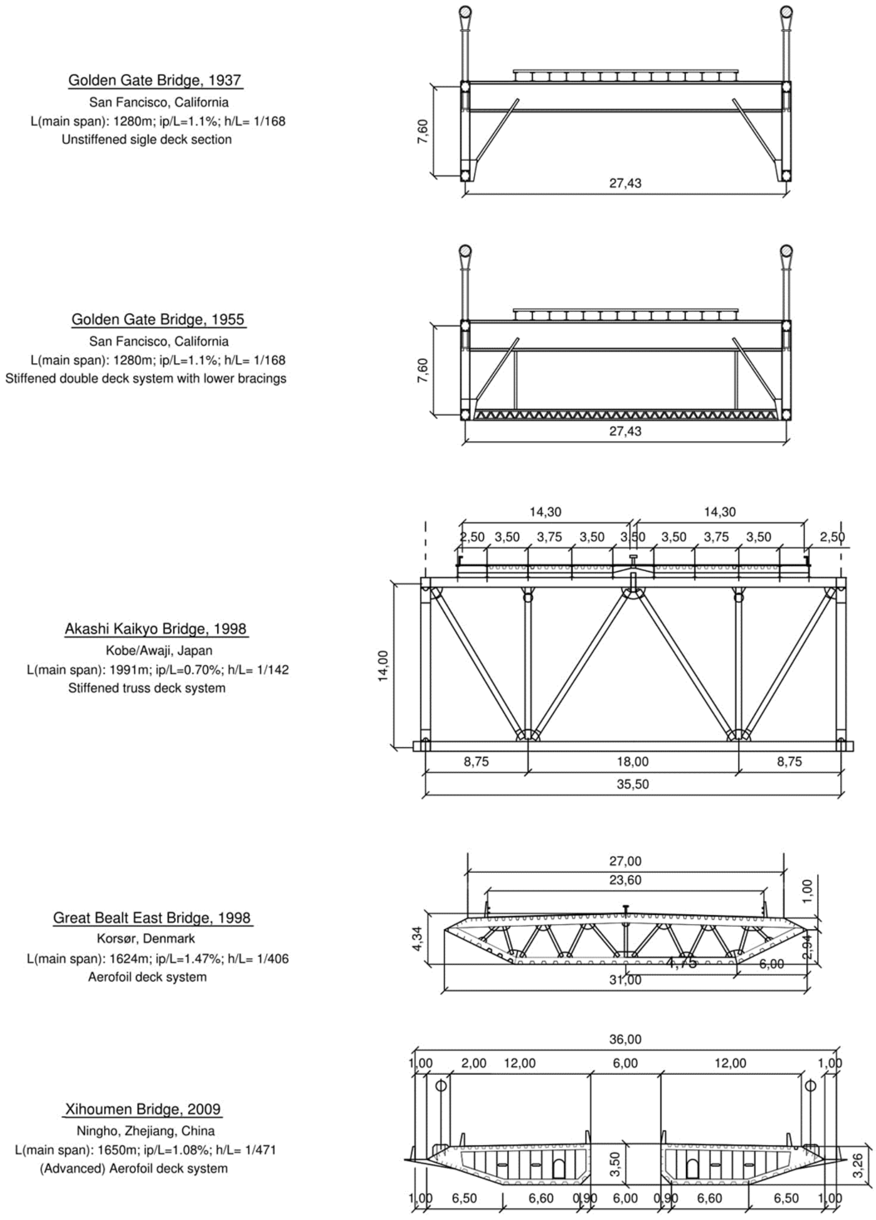

Fig. 12. Suspension bridge historical evolution: deck cross section details

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KEYWORDS

Suspension bridge, deck stiffened system, back analysis, design evolution, parametrical analysis.