Design and analysis of externally prestressed concrete deck bridges with cables deviated under the deck

Ahmed Khalil[1], Khaled H. Riad[2], Fathy A. Saad[3]

Abstract— Externally prestressed concrete deck bridges with cables deviated under the deck are one of the suitable systems for medium spans ranging from 50m to 80m; using this structural system for medium spans range has several economic and aesthetic advantages compared to traditional solutions for medium spans range such as increasing the deck slenderness, reducing the vertical and seismic loads on piers and foundations, efficient use of materials resulting from the greater slenderness for more economical and sustainable construction, improving the aesthetic consideration of the bridge. However, this structural system is uncommon, and only a limited number of these bridges have been constructed worldwide, probably due to the requirements for vertical clearance below the deck and the limited knowledge about this structural system. In order to contribute towards filling this gap, a nonlinear numerical parametric study has been performed to discuss the most important aspects of the structural behaviour and design criteria of this structural system for simple and continuous span schemes under vertical loads. Also, to provide the designers with some guidance on a preliminary estimate of the required material quantities for the bridge superstructure (deck) and to evaluate the economic impact of using this structural system compared to traditional solutions for medium spans range such as internally prestressed concrete deck bridges and externally prestressed concrete deck bridges with cables deviated inside the deck.

Index Terms— Externally prestressed concrete deck bridges; Under deck cable-stayed bridges; Cable-supported bridges; Bridges supported from below; Nonlinear finite element models.

1. INTRODUCTION

Externally prestressed concrete deck bridges with cables deviated under the deck are a special and innovative bridge typology in which the external prestressing cables follow non-conventional layouts in comparison with those of conventional cable-stayed bridges, the external prestressing cables, which have a polygonal layout under the deck, are self-anchored to the deck in the support sections over piers or abutments and are deflected by struts that, under compression, introduce the upward deviation forces due to the cables into the deck as shown in (Figure 1-1) and (Figure 1-2). The external prestressing cables, which are initially pre-tensioned to compensate for the permanent load and self-anchored to the deck, provide elastic supports to the deck under live loads by means of struts that reduce, in turn, the bending moments acting within the bridge. Externally prestressed concrete deck bridges with cables deviated under the deck present several advantages compared to traditional solutions for medium spans range (from 50m to 80m), such as (a) Increasing structural efficiency of the structure by reducing the flexural demand on the deck and enhancing the axial response as shown in (Figure 1-3). As a result, slender concrete decks can be achieved, reduction in deck self-weight, and efficient use of materials resulting in economic and sustainable construction.; (b) Elimination of a certain intermediate pier. As a result, increasing a certain span length with maintaining the main characteristic of the bridge deck (depth, amount of reinforcement, amount of prestressing steel, etc.) as shown in (Figure 1-4) [3]. However, this structural system is uncommon, and only a limited number of these bridges have been constructed worldwide, probably due to the requirements for vertical clearance below the deck and the limited knowledge about this structural system. In order to contribute towards filling this gap, a nonlinear numerical parametric study has been performed to discuss the most important aspects of the structural behaviour and design criteria of this structural system for simple and continuous span schemes under vertical loads. Also, to provide the designers with some guidance on a preliminary estimate of the required material quantities for the bridge superstructure (deck) and to

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evaluate the economic impact of using this structural system compared to traditional solutions for medium spans range such as internally prestressed concrete deck bridges and externally prestressed concrete deck bridges with cables deviated inside the deck. The results show that Externally prestressed concrete deck bridges with cables deviated under the deck reduce the moments under permanent loads considerably, but they are less effective for live loads due to their small additional cables stiffness for span to-depth ratio of (1/33, 1/25, 1/20 and 1/16). The slight stress variation in cables under frequent live loads reduces the risk of cables fatigue, Therefore, the external prestressing cables can be stressed to higher values safely (similar to the same values of conventional prestress) with span to-depth ratio of (1/33, 1/25, 1/20 and 1/16). Externally prestressed concrete deck bridges with cables deviated under the deck allow a large reduction in the amount of materials, the self-weight of the bridge deck is reduced by 25%, the amount of concrete material is reduced by 20%, and the amount of prestressing steel is reduced by 48%, the total bridge deck cost is reduced by 15% compared to internally prestressed concrete deck bridges and 30% compared to externally prestressed concrete deck bridges with cables deviated inside the deck. therefore, this structural system can achieve a sustainable and economical design.

Figure 1-1 Truc de la Fare overpass, France 1993, Simply supported span road bridge with a span of 53m, prestressed concrete deck with 8m width, span to depth ratio 1/33 [1].

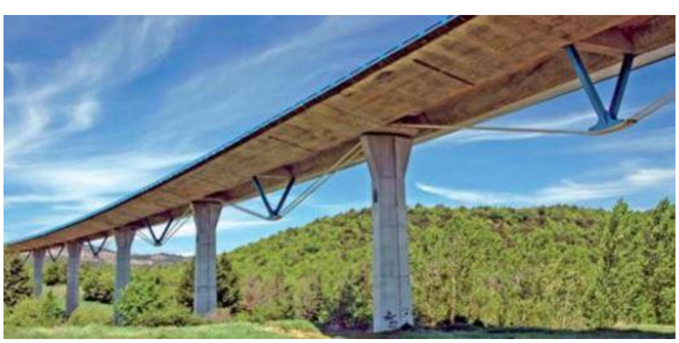

Figure 1-2 Osormort viaduct, Spain 1995, Continuous spans Road bridge with typical spans of 40m, prestressed concrete deck with 12m width, span to depth ratio 1/25 [1].

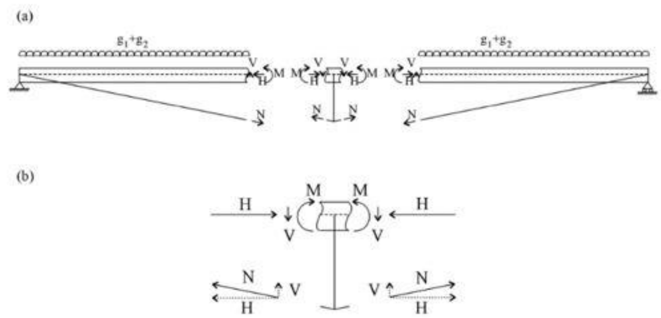

Figure 1-3 (a) Response mechanisms of externally prestressed concrete deck bridges with cables deviated under the deck under live loads; (b) amplification. Moment resisted by the deck = M (flexural response); moment resisted by the cables = H x (strut height) (axial response); the isostatic moment at mid-span = moment resisted by the deck + moment resisted by the stay cables [2].

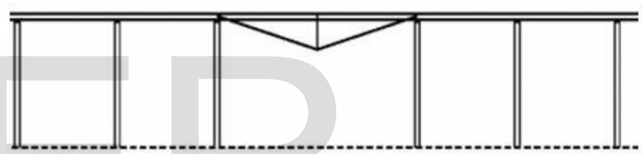

Figure 1-4 Eliminating a certain intermediate pier with maintaining the uniform span distribution and the main characteristic of the bridge deck using under deck cable staying system [3].

2. Historical development and Research achievement

Weitingen viaduct, as shown in (Figure 2-1) is the first under-deck cable-stayed bridge; its construction was completed in 1978 in Germany. It was designed by Fritz Leonhardt, who decided to replace the end piers of the viaduct with a system made up of under-deck stay cables and a strut due to a significant creeping of the valley soil slopes that were complicating their design [1].

Then, several under-deck cable-stayed bridges and combined cable-stayed bridges were designed and built, as shown in (Table 2-1) by well–known engineers worldwide, such as Schlaich, Fritz Leonhardt and Virlogeux; most of these bridges are discussed by (Ruiz-Teran and Aparicio) in [1],[4], [5], [6], [7], [8], [9], [10].

Figure 2-1 Weitingen viaduct, Germany 1978, Continuous spans road bridge with a total length of 900m, the longest span is 263m, the steel deck width is 31m, span to depth ratio 1/43 [1].

Table 2-1 Summary of under-deck cable-stayed bridges (UDCSB) and combined cable-stayed bridges (CCSB) [4]

Bridge	Designer	Year	Country	Type
Weitingen viaduct	Fritz Leonhardt	1978	Germany	UCSB
Gut Marienhof footbridge	Schlaich, Berermann und Partner	1987	Germany	UCSB
Obere Argen viaduct	Schlaich, Berermann und Partner	1991	Germany	CCSB
Truc de la Fare overpass	Michael Virlogeux	1993	France	UCSB
Osormort viaduct	Javier Manterola	1995	Spain	UCSB
Miho Musseum footbridge	Leslie E. Robertson	1997	Japan	CCSB
Jumet footbridge	Jean Marie Cremer	1998	Belgium	UCSB
Losa of the Obispo bridge	José Ramón Atienza	1998	Spain	UCSB
Tobu Recreation Resort footbridge	Toyo Ito & Associates	1998	Japan	UCSB
Glacis bridge	Schlaich, Berermann und Partner	1998	Germany	UCSB
Wacshhaussteg footbridge	Schlaich, Berermann und Partner	1998	Germany	CCSB
Hiyoshi footbridge	Simura & Tanase	1998	Japan	CCSB
Weil am Rheim Viewpoint	Schlaich, Berermann und Partner	1999	Germany	UCSB
Ayumi bridge	CTI Engineering	1999	Japan	CCSB
Takehana bridge	–	2000	Japan	UCSB
Morino-wakuwaku-hashi footbridge	Yosuki Kojima	2001	Japan	UCSB
Torizaki river park footbridge	Civil Eng. Services	2001	Japan	CCSB
Numedalslagen footbridge	Kristoffer Apeland	2002	Norway	UCSB
Montabaur footbridge	Ludwig Müller Offenburg	2003	Germany	CCSB
Haute Provence glass footbridge	Johaness Liess	2003	France	UCSB
Chicago Michigan Avenue Apple store footbridge	Dewhurst MacFarlane & Partners, Inc and A. Epstein and Sons Int.	2003	USA	UCSB
Meaux viaduct	Michael Placidi	2004	France	UCSB
Barajas Airport Terminal 4 Footbridge		2006	Spain	UCSB
Seiryuu footbridge	Asahi Development Consultants Ltd & Oriental Construction Co. Ltd	2006	Japan	UCSB
Fureai footbridge	Taiyo Consultants Co. Ltd and Oriental Construction Co. Ltd	2006	Japan	UCSB
University Limerick Living bridge	Ove Arup and Partners	2007	Ireland	UCSB

3. Parameters Governing the Structural Response of under-deck cable-stayed bridges

In this section, the different parameters that govern the response of under-deck cable-stayed bridges under permanent loads and live loads are discussed [2].

(a) Response of under-deck cable-stayed bridges under permanent loads.

The response under permanent loads is controlled by the compensation level ρ (the compensation level ρ is calculated as the ratio between the vertical permanent load introduced by the struts in the deck and the vertical reaction found on a continuous deck with rigid support in the strut-deck connection section in the permanent state) as shown in (Figure 3-1), For under-deck cable-stayed bridges with concrete decks, compensation levels of ρ 100% of the permanent loads are appropriate to adopt [5], [6]. In a single strut under-deck cable-stayed bridge, if a compensation level ρ 100% is achieved, the effective span of the bridge is reduced to half. Hence, the bending moment diagram of under-deck cable-stayed bridges under permanent load reduced to a quarter of those corresponding bridges without a cable-staying system.

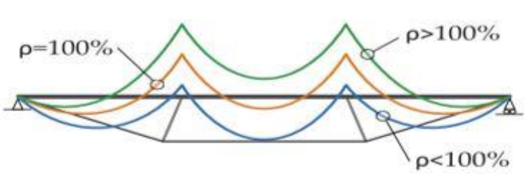

Figure 3-1 Bending moment diagrams of under-deck cable-stayed bridges under permanent loads for different compensation levels ρ.

(b) Response of under-deck cable-stayed bridges under live loads.

The response of under-deck cable-stayed bridges under live loads is controlled by the efficiency of the cable staying system ξ (the efficiency of the cable staying system ξ can be defined as the ratio between the moment resisted by the cable staying system in the form of a couple formed by the tension in the cables and the compression in the deck and the bending moment that would exist on the deck without the cable-staying system). The efficiency of the cable staying system increases by reduction of the flexural stiffness of the deck, increasing the axial stiffness of the cable staying system, increasing the number of struts and increasing the eccentricity of the cable staying system, as shown in (Figure 3-2). The greater the efficiency of the cable staying system, the greater the reduction in the bending moment in the bridge deck under live loads, and the greater the reduction in the depth of the deck compared to conventional bridges without a cable staying system

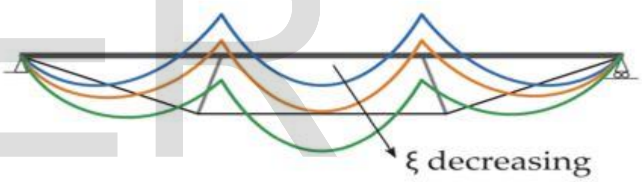

Figure 3-2 Bending moment diagram in the deck for different efficiency values of the cable staying system under uniformly live loads.

Increasing the eccentricity of the cable-staying system using larger struts will increase the efficiency of the cable-staying system under live loads. Nevertheless, other aspects must be taken into consideration in the design process. The cable staying system is recommended to be provided with a max eccentricity of 10% of the total span as a compromise between structural efficiency and aesthetics, as shown in (Figure 3-3).

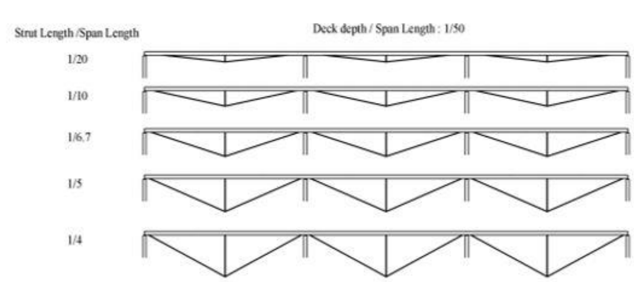

Figure 3-3 The appearance of under-deck cable-stayed bridges for different relative lengths of struts with respect to the span [2].

Increasing the number of struts will increase the efficiency of the cable-staying system under live loads. However, the larger the number of struts, the higher their cost and the more complicated the stressing process becomes. Therefore, the number of struts used should be a compromise of structural, aesthetic, economic, and construction considerations (the recommended number of struts for medium spans range is two).

Increasing the efficiency of the cable-staying system under live loads will increase the stress variation in the cables under frequent live loads increases and become the fatigue limit state, the critical limit state, which controls the cable's cross-sectional area and the type of anchorage used as the following:

- If the stress variation in the cables due to frequent live loads is high, up to 200 MPa, this allows stressing the cables to lower values to ensure adequate fatigue performance (0.5 fpu or 0.45 fpu, where fpu is the ultimate tensile strength of the prestressing steel material) and uses high fatigue strength anchorages which allow a stress variation in the cables up to 200 MPa.

- If the stress variation in the cables due to frequent live loads is small up to 80 MPa, this allows stressing the cables to higher values (0.75 fpu or 0.65 fpu) and uses conventional anchorage, which is used with conventional external prestressing technology, this type of anchorages allow with stress variation in the cables up to 80 MPa.

4. BENCHMARK MODEL DESCRIPTION

Based on the existing research on under-deck cable-stayed bridges with prestressed concrete decks, two benchmark models were developed; all further analyses are performed by taking these benchmark models as a reference. The following is the description of the benchmark models from which we start varying different parameters. The first benchmark model is for simply supported spans, whereas the second benchmark model is for three continuous spans, as shown in (Figure 4-1) and (Figure 4-2). The benchmark models were developed to take into account all geometrical nonlinearity sources (Sagging effects of external prestressing cables, 2nd order effects, 3rd order effects and the effects of cable's slip) and time-dependent effects of materials. Only vertical loads are considered in the analysis,

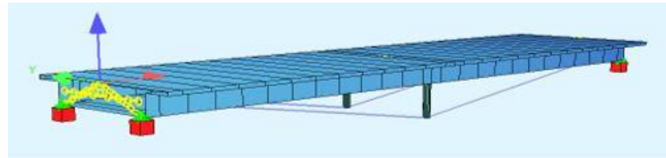

Figure 4-1 Three-dimensional finite element model developed for simply supported spans.

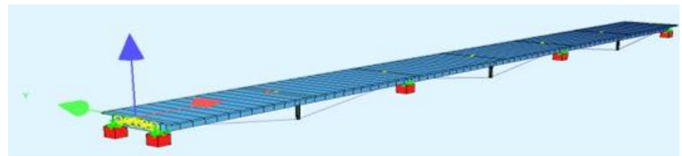

Figure 4-2 Three-dimensional finite element model developed for three continuous spans.

The bridge deck is a concrete box section with a constant depth, 12.5m wide, 0.3m top slap thickness, 0.25 bottom slap thickness, two vertical webs with 0.8m thickness, and the cantilevers are 2m long. the box girder is made of concrete material of compressive strength C50/60, the bridge deck is resisted on bearing, one of the ends of the deck is restrained to longitudinal movements; however, the other is free to move to allow the compression of the deck when the external prestressing cables are pre-tensioned. The diaphragms should be employed in the sections where the struts are connected to the deck.

The strut cross section is considered a circular hollow steel section; the strut is made of steel material with grade S355; the struts are placed along the bisector of the angle formed by the external cables to ensure constant stress in the external cables along their full length to allow the optimum design of the external cables, the struts are pinned to the deck.

The external prestressing cables are considered (seven-wire strands with a nominal cross-section of 140 mm^2 and 1860 MPa ultimate tensile strength); the external prestressing cables are self-anchored to the bridge deck and considered to be continuous cables at strut-external cables connection and at intermediate piers in the case of continuous span schemes, the friction coefficient between the external cable and the deviator depends on many factors, such as deviator type, duct type, etc., and it can only be determined by experimental investigation. However, the friction coefficients were difficult to find in any available literature. For analytical purposes, the friction coefficients at the deviators were assumed to have a certain value, and they were about 0.2 [11].

The internal prestressing tendons are considered (seven-wire strands with a nominal cross-section of 15 0mm^2 and

ISSN 2229-5518

ultimate tensile strength of 1770 MPa).

The concrete box section and the diaphragms are modelled as an eccentric beam element; the bearings are modelled as a spring element; the struts are modelled as a truss element, and the external prestressing cables are modelled as a cable element.

The following loads are defined and applied to the bridge deck as the following:

- Self-weight of the bridge.
- Superimposed dead loads.
- Traffic live loads: load model 1 (LM1) is considered in accordance with EN 1991- 2 - clause 4.3.2.
- Fatigue live loads: fatigue load model 1 (FLM1) is considered in accordance with EN 1991- 2–clause 4.6.2, ECP 201-2015-clause 5-19.
- External prestressing force: the external cables are prestressed to compensate 100% of the permanent loads (Own weight + Superimposed dead loads).

The construction stage analysis was performed by assuming a relative humidity of 70%, an ambient temperature of 20C, and the concrete age at the loading time is 28 days. The following construction stages are considered:

- The bridge deck is constructed on shuttering with an opening for the erection of the steel struts.
- The struts are constructed.
- After 28 days, the shuttering is removed, and the external prestressing cables are prestressed with the required force.
- The superimposed dead loads are added to the structure; during this period, the creep and shrinkage of the concrete material occur under the higher compression force acting on the concrete deck after prestressing the external cables with the required force.
- The creep and shrinkage continue until 30000 days.

The following serviceability and ultimate limit states checks are considered and satisfied according to the design code (Euro code (EN1992-2004)):

- SLS stress limitation of the concrete under characteristic load combination (uncracked section is considered).
- SLS concrete deflection control.
- ULS bending moment in the longitudinal direction, shear and torsion of the bridge deck.
- ULS bending moment of the bridge deck in the transverse direction.
- ULS of axial compression force for the struts.

5. Parametric Study Variables

A nonlinear numerical parametric study has been performed with varying many parameters (span length of 50m, span to depth ratio of 1/33 – 1/25 – 1/20 – 1/16, one and two struts with a strut height of L/50 – L/16 – L/10) as shown in (Figure 5-1), (Figure 5-2).

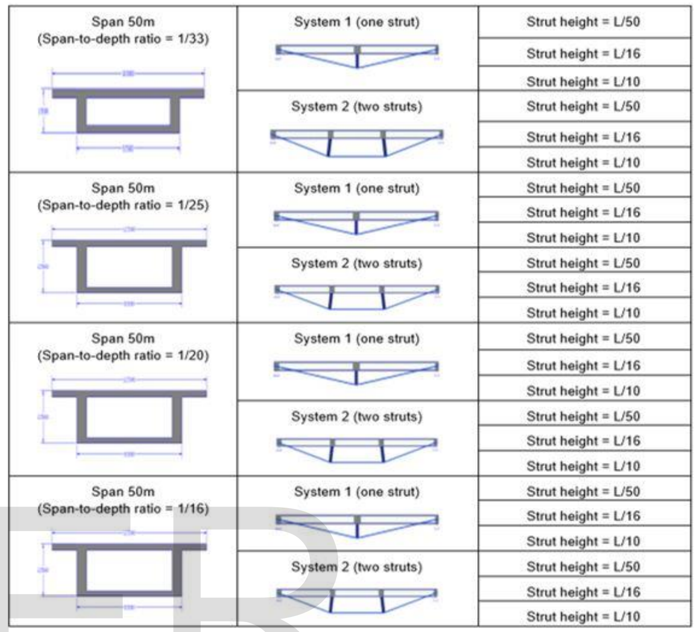

Figure 5-1 The performed parametric study for simply supported spans

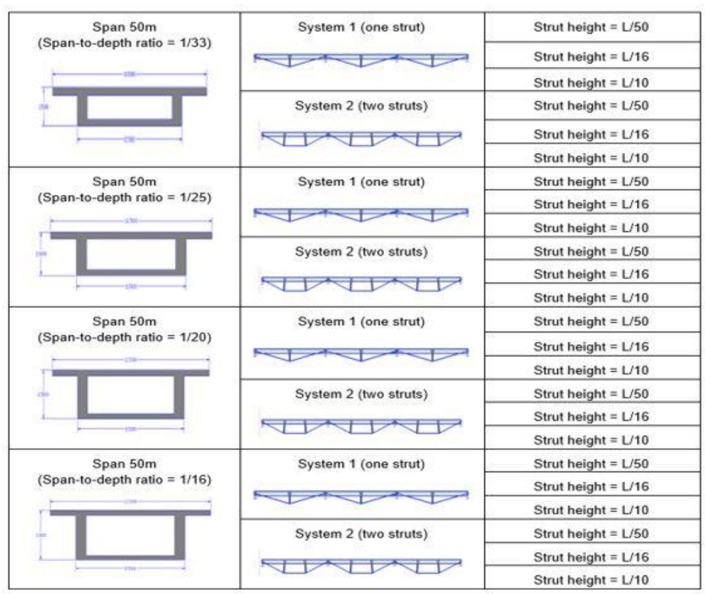

Figure 5-2 The performed parametric study for continuous spans

6. Parametric Study Results

6.1 Structural behaviour of simple span and continuous span schemes under vertical loads.

Externally prestressed concrete deck bridges with cables deviated under the deck reduce the moments under permanent loads considerably, but they are less effective for live loads due to their small additional stiffness for span to-depth ratio of (1/33, 1/25, 1/20 and 1/16).

- under permanent loads, the bending moment of the bridge deck is much smaller than those that would be found in a bridge without external prestressing cables deviated under the deck, Also, the response of the structure in the permanent state is stable over time, Time-dependent effects produce losses of only 5% as shown in (Figure 6-1), (Figure 6-2).

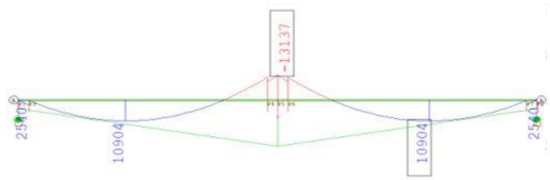

Figure 6-1 The bending moment diagram in the permanent state in (kN.m) after the time-dependent effects of materials.

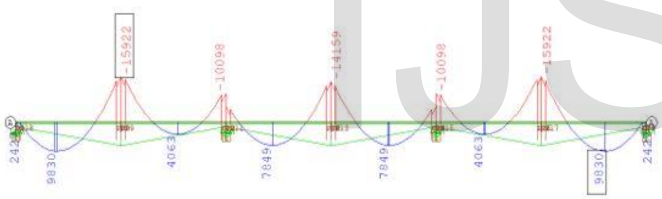

Figure 6-2 The bending moment diagram in the permanent state in (kN.m) after the time-dependent effects of materials.

- Under live loads, the maximum live load ratio carried by the axial response was 23% for the span-to-depth ratio of (1/33) using two struts with a height of (L/10). Therefore, the stress variation in the external cables is small (80 MPa), as shown in (Figure 6-3), (Figure 6-4); the slight stress variation in cables under frequent live loads reduces the risk of cables fatigue. Consequently, the external prestressing cables can be stressed to higher values safely (near the same values as conventional prestress) with span to-depth ratio of (1/33, 1/25, 1/20 and 1/16) and conventional anchorage, which is used with traditional external prestressing technology, can be used. On the other hand, the external cables must be stressed to lower values (near the same values of cable-stayed bridges) with span-to-depth ratios of (1/50) and (1/80). With span-to-depth ratios of (1/50) and (1/80), the maximum live load ratio carried by the axial response was 53% and 80% respectively, Therefore, the stress variation in the external cables is high, 130 MPa and 180 MPa respectively. Consequently, the external prestressing cables must be stressed to lower values to ensure adequate fatigue performance, and high fatigue strength anchorage must be used.

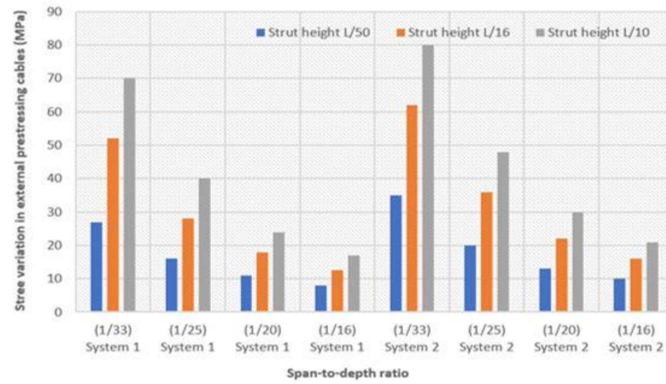

Figure 6-3 Stress variation in the external prestressing cables (MPa) versus different span-to-depth ratios for simply supported spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

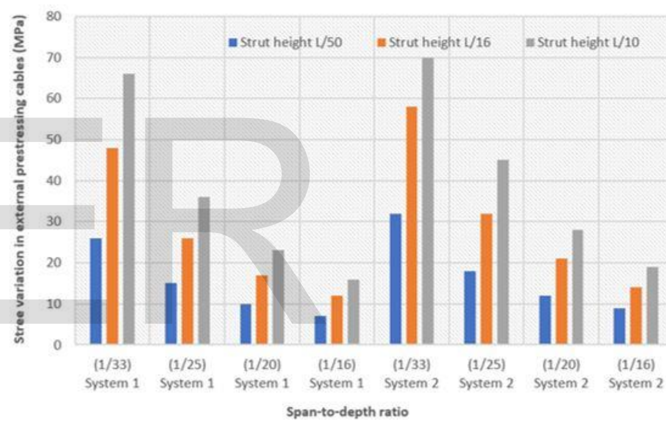

Figure 6-4 Stress variation in the external prestressing cables (MPa) versus different span-to-depth ratios for continuous spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

6.2 Materials quantities

One of the main objectives of the performed numerical parametric study is to provide the designers with some guidance on a preliminary estimate of the required material quantities for the bridge superstructure (deck) for different span-to-depth ratios, struts heights and number of struts, these values of materials quantities needed for the calculation of cost pre-estimates during the feasibility studies of similar bridges conditions and dimensions as shown in (Figure 6-5) to (Figure 6-13).

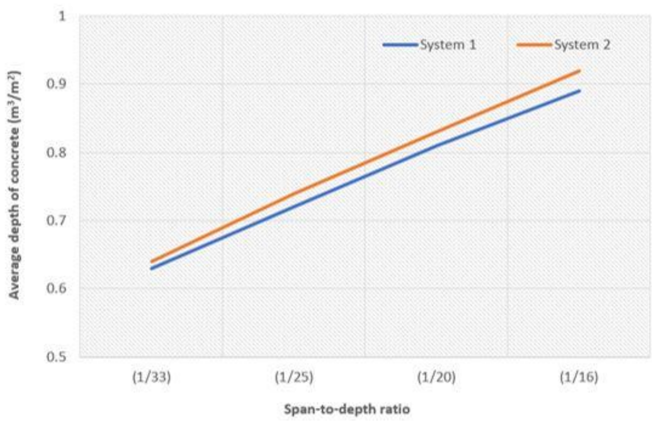

Figure 6-5 Average depth of concrete (m^3/m^2) versus different span-to-depth ratios for simply supported spans and continuous spans for system 1 (one strut) and system 2 (Two struts).

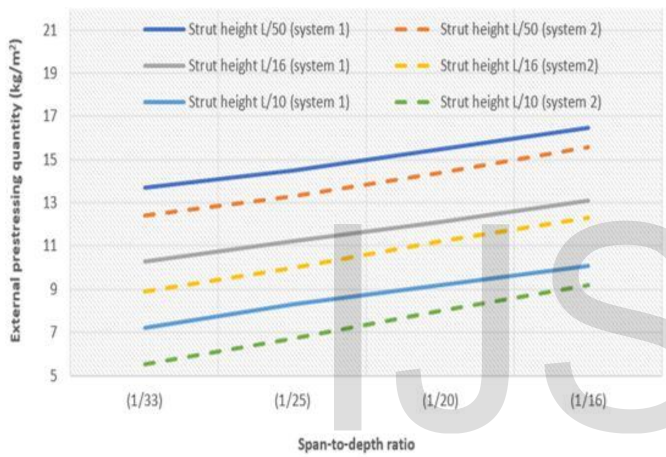

Figure 6-6 External prestressing quantity (kg/m^2) versus different span-to-depth ratios for simply supported spans for system 1 (one strut) and system 2 (Two struts) with different strut height

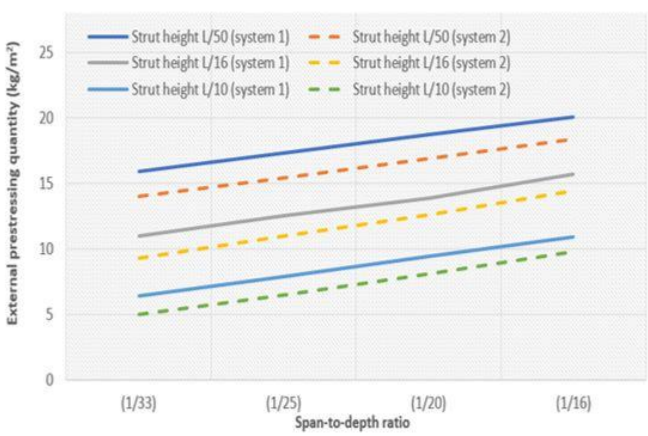

Figure 6-7 External prestressing quantity (kg/m^2) versus different span-to-depth ratios for continuous spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

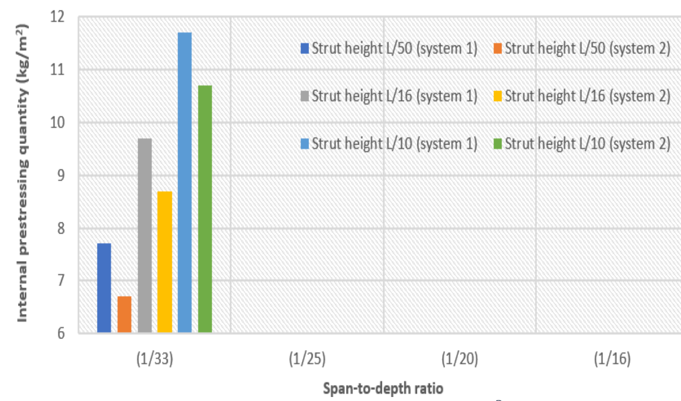

Figure 6-8 Internal prestressing quantity (kg/m^2) versus different span-to-depth ratios for simply supported spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

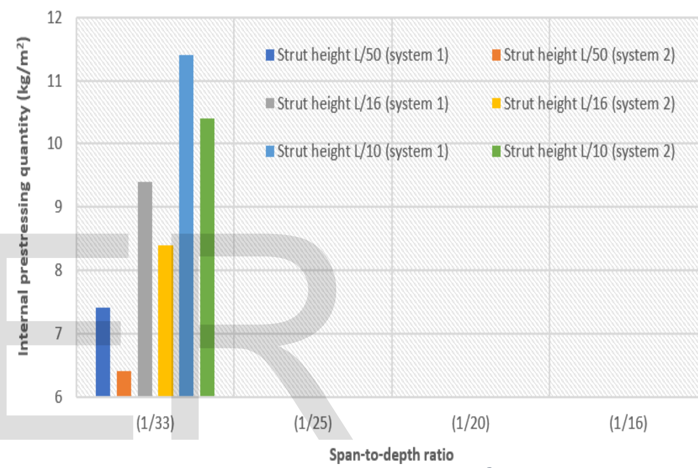

Figure 6-9 Internal prestressing quantity (kg/m^2) versus different span-to-depth ratios for continuous spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

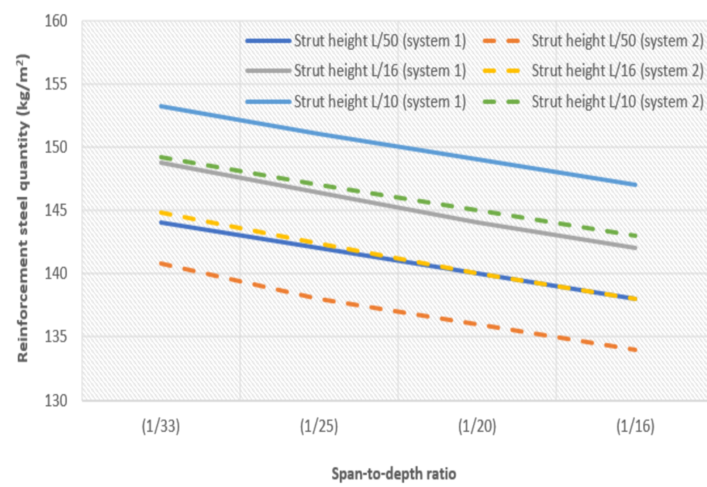

Figure 6-10 Reinforcement steel quantity (kg/m^2) versus different span-to-depth ratios for simply supported spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

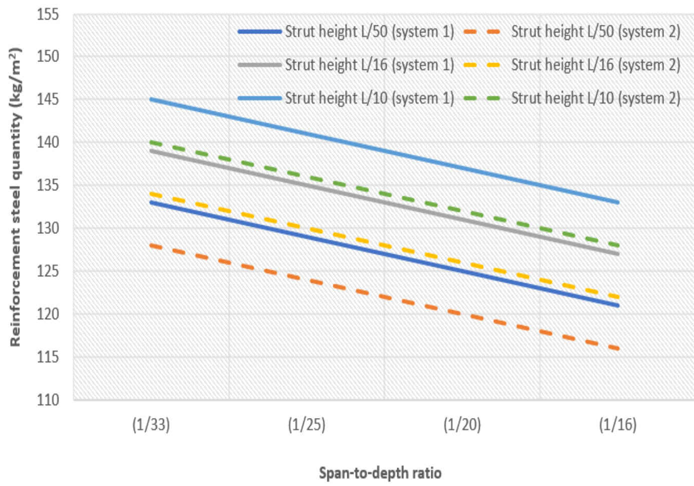

Figure 6-11 Reinforcement steel quantity (kg/m^2) versus different span-to-depth ratios for continuous spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

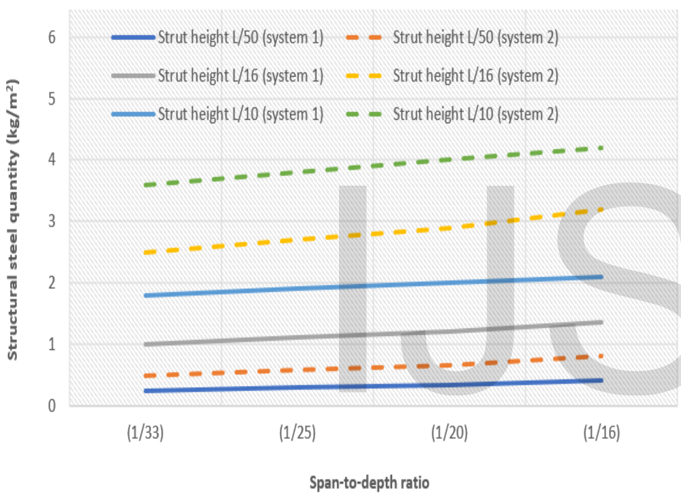

Figure 6-12 Structural steel quantity for struts (kg/m^2) versus different span-to-depth ratios for simply supported spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

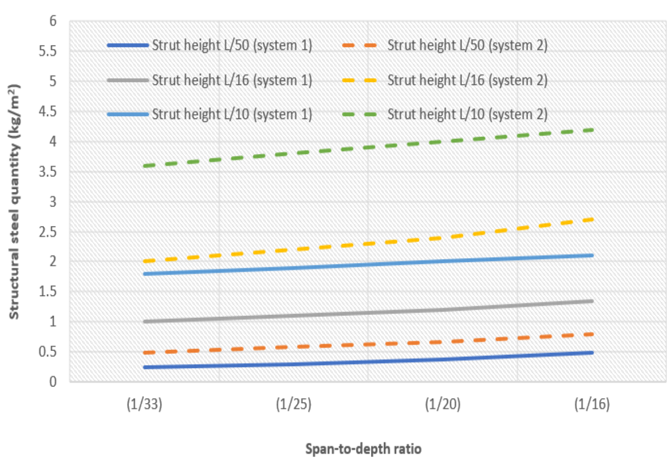

Figure 6-13 Structural steel quantity for struts (kg/m^2) versus different span-to-depth ratios for continuous spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

6.3 The most economical bridge geometry

Using the obtained materials quantities, a simple cost analysis was performed as shown in (figure 6-14), (figure 6-15) to determine the most economical geometry; the cost of main deck components such as (concrete, formwork, internal prestressing strands, external prestressing cables, reinforcing steel and structural steel for struts) have been assessed using the current prices in the Egyptian market in this year (December 2022) as shown in (Table 6-1). the results of the performed cost analysis show that, the most economical geometry uses span to depth ratio of (1/25) with two struts with a strut height of (L/10).

Table 6-1 The current prices of the main component of the bridge deck.

Material	Price
Concrete	3700 L.E / m3
External prestressing steel	90000 L.E / ton
Internal prestressing steel	60000 L.E / ton
Reinforcement steel	29000 L.E / ton
Structural steel for struts	63000 L.E / ton

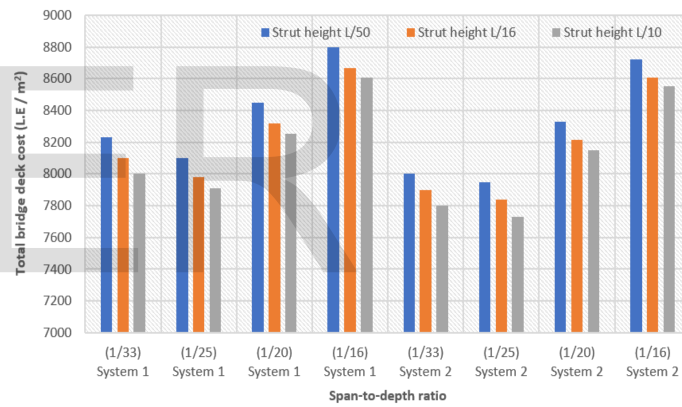

Figure 6-14 Total bridge deck cost (L.E / m^2) versus different span-to-depth ratios for simply supported spans for *system 1 (one strut) and system 2 (Two struts) with different strut heights.*

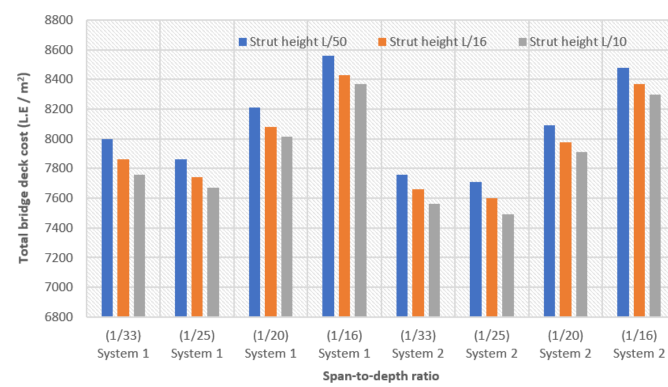

Figure 6-15 Total bridge deck cost (L.E / m^2) versus different span-to-depth ratios for continuous spans for system 1 (one strut) and system 2 (Two struts) with different strut heights.

ISSN 2229-5518

6.4 Economic impact of using externally prestressed concrete deck bridges with cables deviated under the deck.

In order to evaluate the economic impact of using externally prestressed concrete deck bridges with cables deviated under the deck for simple span and continuous span schemes and medium spans range, another cost analysis was performed as shown in (Table 6-2) and (Table 6-3) for three different structural systems (externally prestressed concrete box section deck bridge with cables deviated under the deck, externally prestressed concrete box section deck bridge with cables deviated inside the deck using one deviator at the mid-span section and Internally prestressed concrete box section deck bridge), the results of the performed cost analysis show that, using externally prestressed concrete deck bridges with cables deviated under the deck with span to depth ratio of (1/33) and struts height of (L/50) which is the same structural height of traditional concrete deck bridges (L/20) as shown in (Figure 6-16) allow a large reduction in the amount of materials, the self-weight of the bridge deck is reduced to 25%, the amount of concrete material is reduced to 20%, and the amount of prestressing steel is reduced to 48% and 36% compared to externally prestressed concrete deck bridges with cables deviated inside the deck and internally prestressed concrete deck bridges respectively , the total bridge deck cost is reduced to 15% compared to internally prestressed concrete deck bridges and 30% compared to externally prestressed concrete deck bridges with cables deviated inside the deck. therefore, this structural system is in line with a sustainable and economical design. In addition, the total bridge deck cost is reduced more using the most economical geometry (span-to-depth ratio of 1/25 with a strut height of L/10), but in this case, a suitable vertical clearance under the bridge deck should be provided for the large strut height.

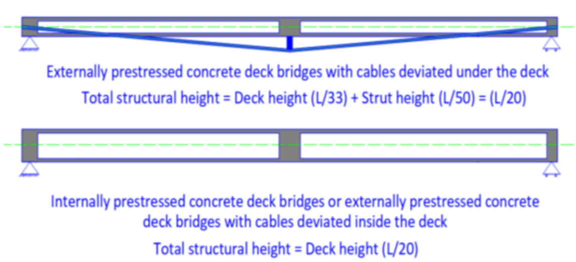

Figure 6-16 The appearance of externally prestressed concrete deck bridges with cables deviated under the deck with the same structural height of traditional solutions for medium spans range (internally prestressed concrete deck bridges and externally prestressed concrete deck bridges with cables deviated inside the deck)

Table 6-2 Cost comparison between three different structural systems with different span-to-depth ratios for simply supported spans.

	Externally prestressed concrete box section deck bridge with cables deviated under the deck		Externally prestressed concrete box section deck bridge with cables deviated inside the deck		Internally prestressed concrete box section deck bridge		
Span to depth ratio	1/25	1/33	1/25	1/20	1/25	1/20	1/33
Number of struts	2	2					
Strut height (m)	L/10	L/50					
Average depth of concrete (m3 / m2)	0.74	0.64	0.72	0.8	0.72	0.8	0.63
External prestressing quantity (kg/m2)	7	12	50	45			
Internal prestressing quantity (kg/m2)		7			42	30	55
Reinforcement steel quantity (kg/m2)	155	140	136	136	136	136	136
Structural steel quantity for struts (kg/m2)	4	0.5					
Total bridge deck price (L.E / m2)	7730	7980	12050	11210	10120	9370	11100

Table 6-3 Cost comparison between three different structural systems with different span-to-depth ratios for continuous spans.

	Externally prestressed concrete box section deck bridge with cables deviated under the deck		Externally prestressed concrete box section deck bridge with cables deviated inside the deck		Internally prestressed concrete box section deck bridge		
Span to depth ratio	1/25	1/33	1/25	1/20	1/25	1/20	1/33
Number of struts	2	2					
Strut height (m)	L/50	L/10					
Average depth of concrete (m^3 / m^2)	0.74	0.64	0.72	0.8	0.72	0.8	0.63
External prestressing quantity (kg/m^2)	6.5	14	43	40			
Internal prestressing quantity (kg/m^2)		6.5			38	33	44
Reinforcement steel quantity (kg/m^2)	140	130	125	125	125	125	125
Structural steel quantity for struts (kg/m^2)	4	0.5					
Total bridge deck price (L.E / m^2)	7490	7760	10850	10150	9510	8760	10100

7. Summary and Conclusions

- For 50m simple and continuous spans, externally prestressed concrete deck bridges with cables deviated under the deck can be used with the same structural height of conventional values and less cost.
- Although the most economical system included the use of struts with a height of (L/10), it is recommended to use struts with a height of (L/50) as the cost difference was less than 5% to provide a suitable clearance under the bridge deck.

- Externally prestressed concrete deck bridges with cables deviated under the deck reduce the moments under permanent loads considerably, but they are less effective for live loads with span-to-depth ratios of (1/33, 1/25, 1/20 and 1/16) due to the big deck stiffness compared to the added cables stiffness. Whereas they are more effective for live loads with span-to-depth ratios of (1/50 and 1/80).

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