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ASSESSMENT OF INTERNAL AND EXTERNAL PRESTRESSING OF PRESTRESSED BOX GIRDERS

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Abstract- Bridges have been built since many years also box girder bridges are being constructed since the 1920s. Initially, only mild steel reinforced box girder bridges were used. But as the requirement of long spans box girders with minimum depths was arising, the use of high strength prestressing steel in the construction of box girders was started. The major work for prestressing or post-tensioning was done by Freyssinet (for internal prestressing). The use of external post-tensioning was evolved in the 1930s in Germany initially. External prestressing had been abandoned in late 1960's and early 1970's, because of several drawbacks, the main one was corrosion of steel. The strengthening capabilities of external tendons became the reason for its re-introduction in late 1970's. In this study, an effort is done to briefly explain the procedure for the design of box girders with internal and external prestressing systems. Also, the cost of the two systems is compared for different spans considering stress limitations and serviceability criteria. This has been done by modeling box girders with spans ranges from 30m to 90m and for the span to depth ratios of 15, 20 and 25. In general, this study quantitatively indicates the prestressing force required for the two systems.

Key Words: Prestressing, Internal & External Prestressing, Post-tensioning, Box Girder

1. INTRODUCTION

The popularity of using box girder bridges is increasing with time. As the name indicates a box girder bridge is basically a box may or may not be of trapezoidal shape with cantilevered top flange extensions on both sides. The top slab width is selected such as that it can accommodate entire roadway width. Box girder bridges can be cast-in-placed or constructed using segmental construction. Old cast-in-place box girders are designed using mild steel reinforcement or post tensioning tendons. For wide roadways, the box portion generally has internal webs and is referred to as a multi-cell box girder. Concrete box girder bridges are typically either single span or continuous multi-span structures. Spans can have a straight or curved alignment and are generally exceed 40 m in length. The first bridge with post-tensioning was been built in Germany [7]. Post-tensioning in concrete bridges started in 1936 in Aue, Saxony.

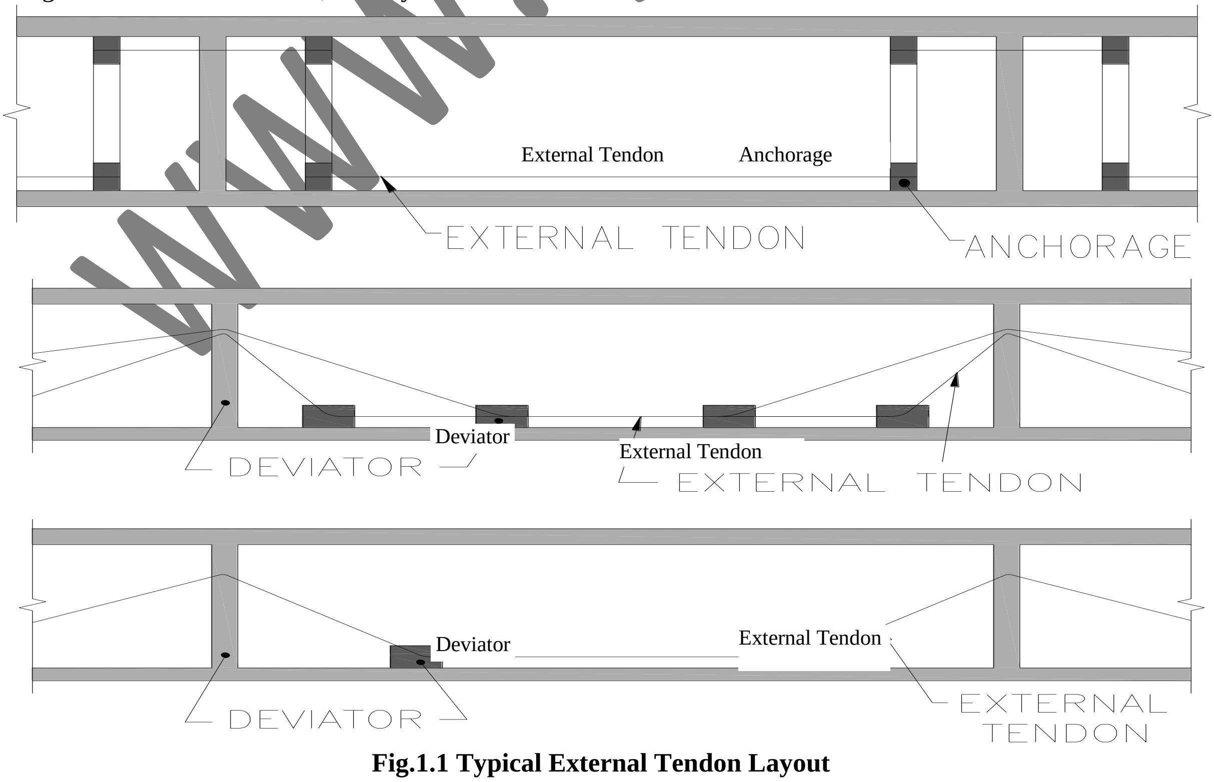


Fig.1.1 Typical External Tendon Layout

German engineers developed a number of construction methods for concrete bridges in same time span. Those construction concepts are efficient and are used across the world [2]. Box girder bridges in the past were mostly designed with the internal prestressing system, in which prestressing tendons are completely embedded inside concrete webs of the box girder. In the recent past, box girders are designed with the combination of internal and external prestressing tendons or only with the external prestressing tendons. External prestressing is a special technique of post-tensioning, which is used to apply prestress forces to the concrete after hardening. External tendons are placed outside of the section being stressed. The forces are only transferred to the anchorage blocks or deviators. In the internal prestressing system, tendons lie within the cross-section of the structure. Internal prestressing can be carried out by using bond between structure and prestressing steel (grouted ducts). The other possibility is internal post-tensioning without a bond between duct and tendon. Prestressing force is transferred through anchorages and contact pressure against the surface of the duct. Typical straight & deviated tendon layouts are shown in Fig.1. As straight tendons do not require deviators, therefore construction is easy, less costly and no loss of prestress occurs due to friction. However, prestressing force produced by the straight tendons does not have a vertical component, thus it is less effective in resisting shear forces. the deviated tendons are usually more practical as they produce bending moments and shear force distributions closer to applied loads compared to straight tendons, also, these tendons can be made continuous in statically indeterminate structures, reducing number of anchorages, when a straight tendon is employed in these structures. However, loss of prestress due to friction occurs when the tendon deviates and also these tendons are susceptible to fretting fatigue problems due to high contact pressure combined with friction and slip during cyclic loads at deviator [2,3].

An external tendon is connected to the concrete only at deviators and anchorage location, where prestress force is transferred to concrete. Deviators are mainly of three types: diaphragm, rib or stiffener and saddle or block as shown in Fig.2. The advantage of using a diaphragm or a stiffener type deviator is a better distribution of tendon deviator forces occur compared to the saddle type whereas more localized stresses due to tendon force occur combined with local bending. Thus, saddles must be properly reinforced and detailed to avoid failure. Diaphragms or stiffeners are bulky and increase the structure weight and pose construction difficulties when compared to saddle type, saddle type deviators are a small block located near the intersection of the web and a bottom slab of box girder Fig.2c. Saddles are easy to construct (less complicated formwork than diaphragms or stiffeners) and are lightweight [6, 7].

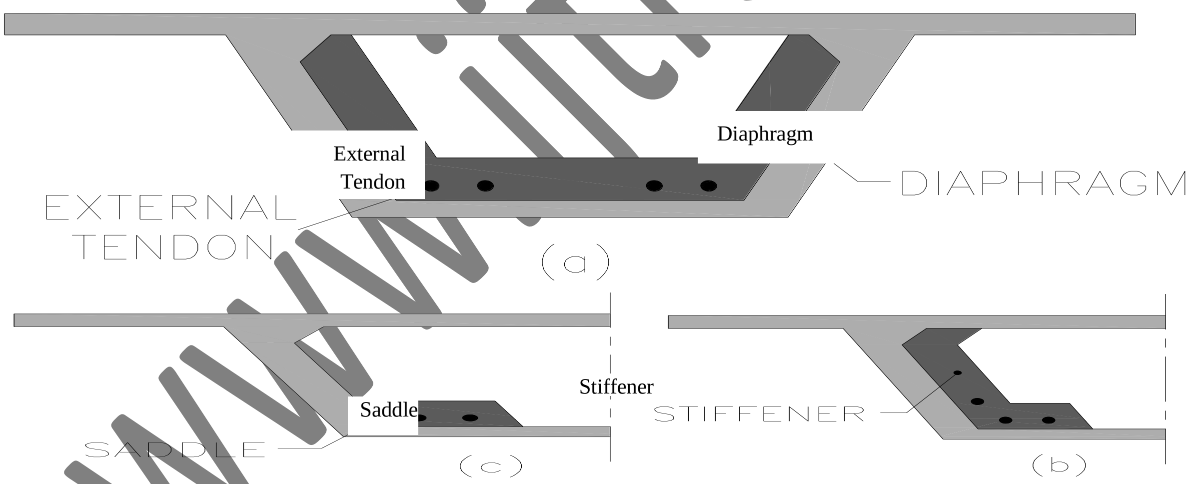
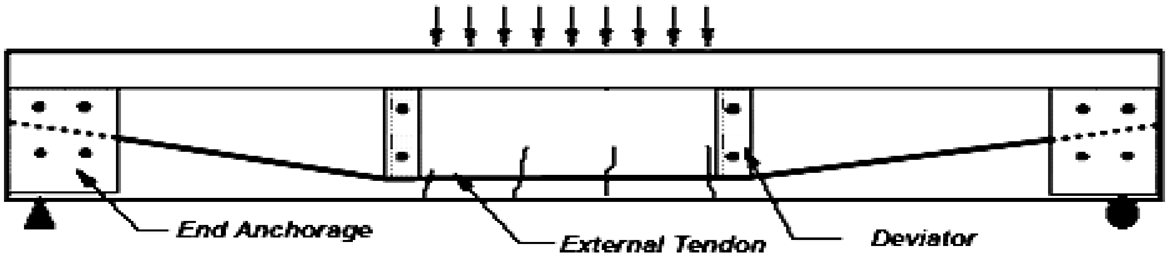


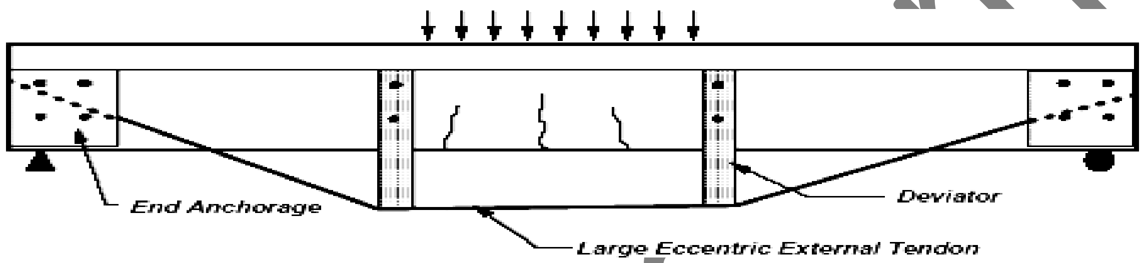
Fig. 1.2 Deviator Types

External prestressing was initially developed for the strengthening of bridges, now a day it is used both for the strengthening of existing as well as for newly built structures. Prestressed concrete bridges with external prestressing are becoming popular because of their advantages like simplicity and cost-effectiveness. External prestressing is when unbonded tendons are placed, and prestressed, outside the structure anchored at the ends and sometimes with one or several deviators during the length of the structure. This method is advantageous for the strengthening of a structural member to obtain improved load carrying capacity. External tendons can be made of steel or fiber reinforced polymer. They provide one of the most efficient solutions to increase the load carrying capacity of existing bridges when the infrastructures are in need of renewal and made of all structural materials, such as concrete, steel, and timber, Håkan Nordin (2005) [4].

Bridges with post-tensioning have been in use since 1950's and there are many examples throughout the world. Mostly load is applied through single prestressing cables or grouped strands. In some cases, stress has applied through high tensile bars. In few cases, the stress is applied using the more unconventional technique. For example, stress in a tendon can be developed by anchoring straightened on in place and imposing deflection at mid span. The deflection is then retained by fixing the deflected points. Prestress can also be developed by applying a load to impose deflection in deck prior to anchoring the tendons or bars. An extension on use of external tendons is to place them at large eccentricities. This is possible only when external prestressing is used since tendons need not be arranged within the concrete section as shown in Fig.3. Sunthavaravidel & Aravinthan, (2005) [9].



a. Conventional Tendon Placement



b. Tendon with Large Eccentricity

Fig. 1.3 Possible Tendon Placement in External Post-Tensioning

Types of external tendons mostly depend on the corrosion protection system and technology adopted, i.e. whether prestressing tendon is bonded or unbonded at deviators. When the tendon is bonded, it cannot be replaced or re-strengthened but construction cost is less compared to unbonded external tendons, this non-replaceable technology is most common in the USA. The bond between concrete and steel tendon is developed in a high-density polyethylene (HDPE) duct embedded in concrete. The duct is filled with cement grout. In France, most of the European countries, the external tendon is unbonded at deviators, thus allowing future replacement and re-tensioning of the tendon. Several methods are available to make the tendon unbonded at deviators. One method is an injection of grease or wax in the HDPE duct at high temperature (80° to 90° C), thus ducts must resist this temperature. Galvanizing strands is another method, which does not require a duct, but long unsupported tendon lengths should be avoided. The most economical solution, as mentioned by Virlogeux (1993), issue of a double tubing system at deviators allowing for the replacement. This consists of injecting HDPE ducts with cement grout, but with the double tubing system at the deviators the replacement of tendons becomes possible.

2. STRESS CALCULATION IN THE EXTERNAL TENDONS

In past five decades, a number of experimental and analytical studies have focused on prediction of unbonded tendon stress at ultimate limit state. Baker (1949) [10] was one of the pioneers worked on this topic, and many methods have been proposed since then. In current investigation, these methods to predict the ultimate tendon stress are reviewed critically and three broad categories are identified, (i) based on bond reduction coefficients, such as the equations proposed by Baker (1949), Pannell (1969), Harajli (1990), Naaman &Alkhairi (1991) and others; (ii) based on regression analyses, such as Warwaruk et al. (1962), Du & Tao (1985) and others; and (iii) method based on member deformation, Ghallab & Beeby (2004) [5].

2.1 Prediction Equations Based on Bond Reduction Coefficients

Baker (1949) [10] expressed the tendon strain at ultimate limit state as a sum of effective prestress f_{pe} and stress increment Δf_{ps} which is determined by using bond reduction coefficient F having value 0.1. Janney et al. (1956) also adopted a bond reduction coefficient for unbonded tendon stress, taken as a ratio of the neutral axis depth c to the depth to the prestressing steel d_p, i.e. $F=c/d_p$. Pannell (1969) investigated experimentally the effect of span-depth ratio, effective prestress & amount of reinforcement on flexural behavior of PC beams with unbonded tendons, his formula based on assumptions, (i) width L_0 of plastic zone at the ultimate is ten times the neutral axis depth `c` at ultimate, i.e. $L_0 = 10c$, (ii) elongation of prestressing tendon in elastic zone is negligible compared with tendon elongation within the plastic zone, (iii) frictional stresses along tendons are neglected and tendon stress is constant between end anchorage, (iv) plane sections remain plane before and during bending. Therefore, ultimate tendon stress f_{ps} can be obtained as

$$f_{ps} = f_{pe} + \frac{E_p e_{cu}\left(d_p - c\right)}{c}\frac{L_0}{L_i} \quad 2.1$$

$$f_{ps} = f_{pe} + \frac{E_p e_{cu}\left(d_p - c\right)}{c}\frac{10c}{L_i} \quad 2.2$$

$$f_{ps} = f_{pe} + \frac{10 E_p e_{cu}\left(d_p - c\right)}{L_i} \quad 2.3$$

Where f_{pe} is effective prestress, E_p is the modulus of elasticity of prestressing tendon, ε^{cu} is an ultimate concrete compressive strain in the extreme compression fiber, L_i is the length between end anchorages. In Eq.2& 3 term L_0/L_i or $10c/L_i$ is bond reduction coefficient. In BS 8110, the ultimate stress of unbonded tendons is predicted from Eq. 1.4.

$$f_{ps} = f_{pe} + \frac{7000}{L_i/d_p}\left(\frac{\pi}{2} - \theta\right)\left(1 - \frac{1.7 f_{pu} A_{ps}}{f_{cu} b d_p}\right) \le 0.7 f_{pu} \text{ Mpa} \quad 2.4$$

Canadian Code A23.3 suggests

$$f_{ps} = f_{pe} + 900\frac{\left(d_p - c\right)}{L_e} \le f_{py} \quad 2.5$$

For which,

$$c_y = \frac{A_{ps} f_{py} + A_s f_y}{\alpha_1 f_c' \beta_0 b} \le f_{py} \quad 2.6$$

$$\alpha_1 = .85 - .0015 f_c' \quad 2.7$$

$$\beta_0 = .97 - .0025 f_c' \quad 2.8$$

Starting from 1998 version of AASHTO LRFD [1] Bridge Design Specifications, a prediction equation A23.3 has been adopted Eq.9 & 10. L_e is effective tendon length, N_s is number of support hinges crossed by tendon,

$$f_{ps} = f_{pe} + 900\frac{\left(d_p - c\right)}{L_e} \le f_{py} \quad 2.9$$

$$L_e = \frac{2L_e}{2 + N_s} \quad 2.10$$

For T-section

$$c = \frac{A_{ps} f_{py} + A_s f_y - A_s' f_y' - 0.85 f_c' \beta_1 (b - b_w) h_f}{0.85 f_c' \beta_1 b_w} \quad 2.11$$

For rectangular section behavior

$$c = \frac{A_{ps} f_{py} + A_s f_y - A_s' f_y'}{0.85 f_c' \beta_1 b_w} \quad 2.12$$

Factor b_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi, exceeding 4.0 ksi, b1 shall be reduced at a rate of 0.05/1.0 ksi of strength in excess of 4.0 ksi, except b1 shall not be taken be less than 0.65.

2.1 Prediction Equations Based on Bond Regression Analysis

In addition to the approach of bond reduction coefficients to determine unbonded tendon stress, regression analysis is used. Warwaruk et al. (1962) studied that on ultimate tendon stress of unbonded prestressed beams increase in tendon stress Δf_{ps} is related to ρ_p / f'c. Eq.13 is based on regression analysis, given by

$$f_{ps} = f_{pe} + 30000 - \frac{\rho_P}{f'_c} x10^{10} \quad 2.13$$

Mattock et al. (1971) showed Eq.2.13 was too conservative and proposed Eq.14.

$$f_{ps} = f_{pe} + 10000 + \frac{1.4f'_c}{100\rho_P} < f_{py} \quad 2.14$$

Eq.2.14 was adopted in 1971 and 1977 versions of ACI Building Code with the modification shown in Eq. 2.15.

$$f_{ps} = f_{pe} + 10000 + \frac{f'_c}{100\rho_P} < f_{py} \quad 2.15$$

With the following limitations
$f_{ps} \leq f_{se} + 60000$
$f_{ps} \leq f_{py}$
$f_{pe} \geq 0.5\ f_{pu}$

Eq.2.16 & 2.17 has been adopted in ACI Building Code since 1983,
For $L/d \leq 35$

$$f_{ps} = f_{pe} + 10000 + \frac{f'_c}{100\rho_P} \quad 2.16$$

For $L/d > 35$

$$f_{ps} = f_{pe} + 10000 + \frac{f'_c}{300\rho_P} \quad 2.17$$

Du & Tao (1985) tested 26 beams under third-point loading with a constant span-depth ratio of 20. A linear relationship was found between the ultimate tendon stress f_{ps} and the combined reinforcement index q_0, the best correlation led to Eq.2.18.

$$f_{ps} = f_{pe} + (786 - 1920q_o) < f_{py} \quad 2.18$$

The limitations are
$q_0 \leq 0.3$
$0.55f_{py} \leq f_{pe} \leq 0.65f_{py}$

2.2 Prediction Equations Based on Bond Regression Analysis

Ghallab & Beeby (2004) [2] proposed a different approach to estimate unbonded tendon stress, applicable to concrete beams externally prestressed tendons. Assumptions are (i) axial shortening of the beam is negligible(ii) displacement of end anchorages are neglected, (iii) friction stresses along the tendons are neglected, (iv) beam deflection is solely due to plastic hinge deformation. The unbonded tendon stress at the ultimate stage is expressed by Eq.19.

$$f_{ps} = f_{pd} + \Delta f_{pr} \quad 2.19$$

Δf_{pr} could be evaluated from deformation of member illustrated in Fig.4. Total initial length of tendon L_t is sum of the lengths of segments AB, BC and CD, i.e.

$$L_t = AB + BC + CD \quad 2.20$$

Where,

$$AB = \sqrt{L^2_{AB} + (y_b - y_a)^2} \quad 2.21$$

$$BC = \sqrt{L^2_{BC} + (y_b - y_c)^2} \quad 2.22$$

$$CD = \sqrt{L^2_{CD} + (y_c - y_d)^2} \quad 2.23$$

The tendon length after loading L^*_t is

$$L^*_t = A'B' + B'C' + C'D' \quad 2.24$$

Where,

$$A'B' = \sqrt{L^2_{AB} + (y_b - y_a + \Delta_b)^2} \quad 2.25$$

$$B'C' = \sqrt{L^2_{BC} + (y_b + \Delta_b - y_c - \Delta_c)^2} \quad 2.26$$

$$C'D' = \sqrt{L^2_{CD} + (y_c - y_d + \Delta_c)^2} \quad 2.27$$

Elongation of the tendon ΔL can be obtained as follows,

$$\Delta L = L^{*}_{t} - L_{t} \tag{2.28}$$

Total tendon strain ε_{pr} is

$$\varepsilon_{pr} = \varepsilon_{pd} + \Delta\varepsilon_{pr} = \frac{f_{pe}}{E_p} + \frac{\Delta L}{L_t} \tag{2.29}$$

For FRP tendon, stress-strain relationship is linear until failure, tendon stress can be calculated from Eq. 2.30.

$$f_{pr} = \varepsilon_{pr} E_p = E_p(\varepsilon_{pd} + \Delta\varepsilon_{pr}) = f_{pd} + \frac{\Delta L}{L_t} E_p \tag{2.30}$$

ΔL can be calculated from Eq. 2.31.

$$\Delta_L = K \frac{\varepsilon_{cu}}{c} L^2 \tag{2.31}$$

2.3 Illustrative model Prestressed Box Girder Bridge

To observe the behavior of prestressed box Girder Bridge with internal and external tendons three span bridge with three span to depth ratios of 15, 20 and 25 for spans range from 30m to 90m was modeled using CSI BRIDGE software.

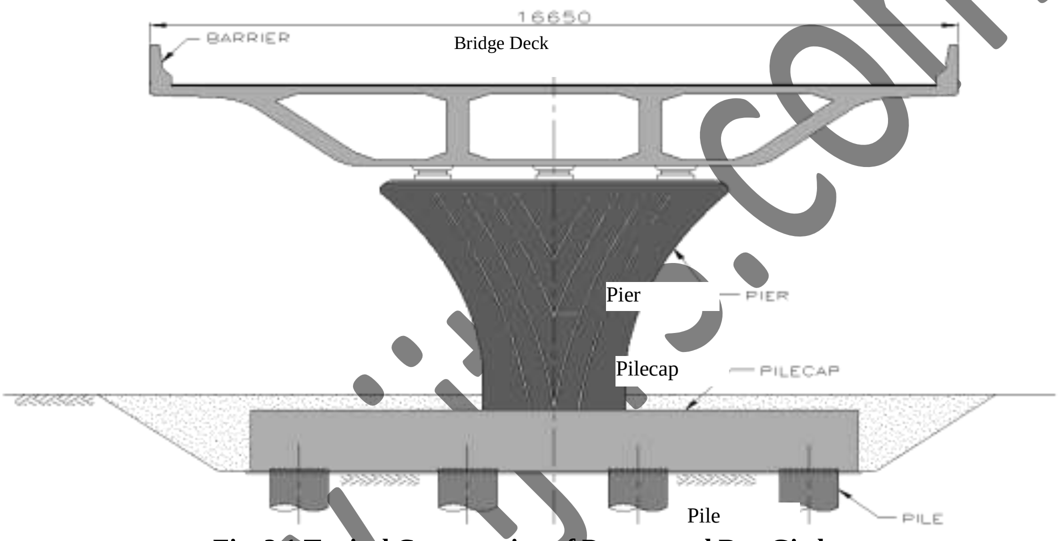


Fig. 2.1 Typical Cross section of Prestressed Box Girder

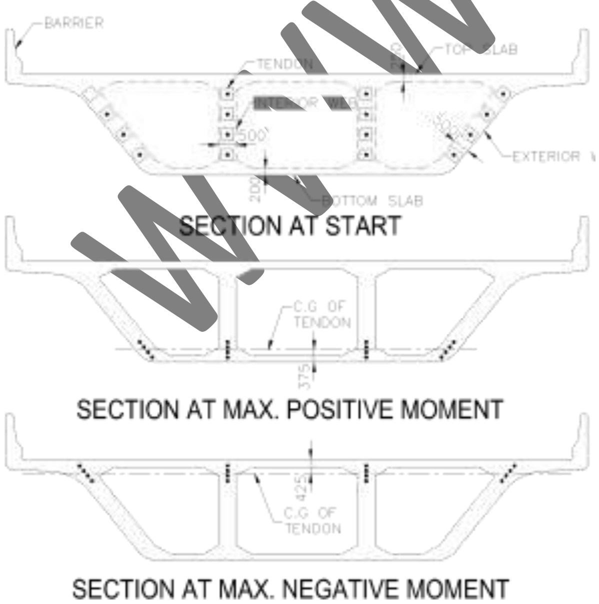


Fig. 2.2 Box Girder with Internal Tendons

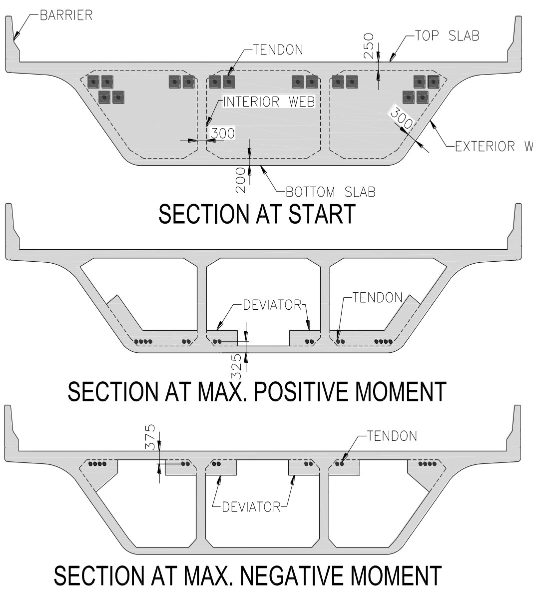


Fig. 2.3 Box Girder with External Tendons

Continuous Box Girder Bridge with internal & external prestressed tendons is shown in Fig.5 & 6. Pier, Pile-cap, & pile have a compressive strength of 28MPa, Box Girder 40MPa, Steel 420MPa & Prestressing steel 1890MPa. A three span continuous bridge has been subjected to various loads including dead, live & earthquake, Design Lanes & Multiple Presence of Live Load as per AASHTO LRFD [1]. Live loads include

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International Journal of Technical Research & Science

Design Truck or tandem & Design Lane. Prestressed box girder properties with internal & external tendons are listed in table 2.1.

Table-2.1 Prestressed Box Girder Properties

Prestressed Box Girder Bridge with Tendons		
Property	**Internal**	**External**
No. of spans	3	3
Span depth ratios	15 to 25	15 to 25
Span length	30m to 90m	30m to 90m
Tendon type	0.6 in dia (bonded)	0.6 in dia (unbonded)
Pier size (mm)	3000x1800	3000x1800
Pile cap thickness	Variable	Variable
Pile diameter	1200 mm	1200 mm
No. of Piles	Variable	Variable

Live Load includes Design truck or design tandem, and Design lane load, design tandem comprises of a pair of 25.0kip axles with a spacing of 4.0 ft., AASHTO LRFD [1] design lane load has taken as0.64klf uniformly distributed in longitudinal direction design truck load is shown in Fig.7.

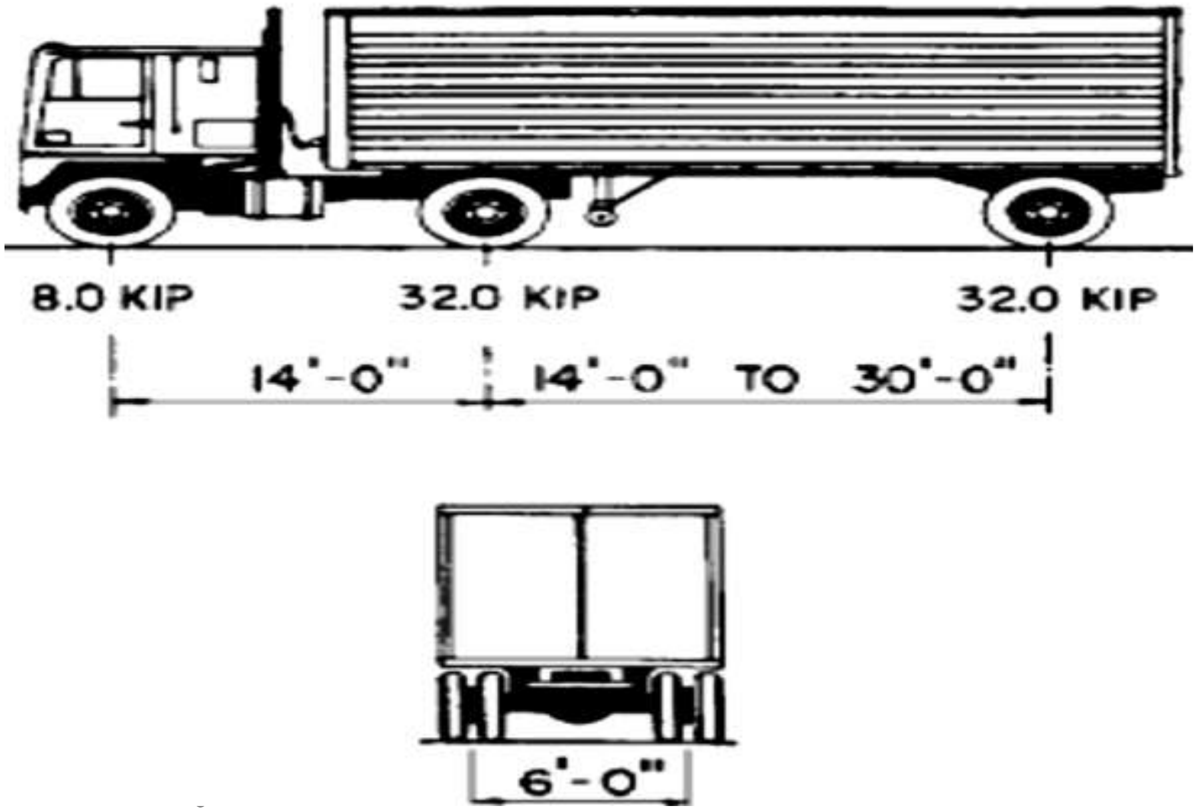

Fig. 2.4 Characteristics of Design Truck

Dynamic load allowance factor applied to the static load is taken as (1 + IM/100). For earthquake loading, Zone 1 of AASHTO LRFD [1] has been considered for both the bridges with internal and external tendons. Response spectrum of AASHTO 2007 has been applied to design the bridge components with applicable response modification factors. CSI Bridge 15.2.0 was used for analysis and design of frame elements purpose, a product of Computer & Structures.

3. RESULTS AND DISCUSSIONS

3.1 Friction Losses for Internal and External Prestressing

Fig.2.8, 2.9 and 2.10 shows a comparison between frictional losses for box girder bridges with Internal& External prestressing arrangement for three different span to depth ratios (L/h) of 15, 20 and 25 respectively. All graphs show that friction losses for all span to depth ratios for the external prestressing system are lesser than the internal prestressing system.

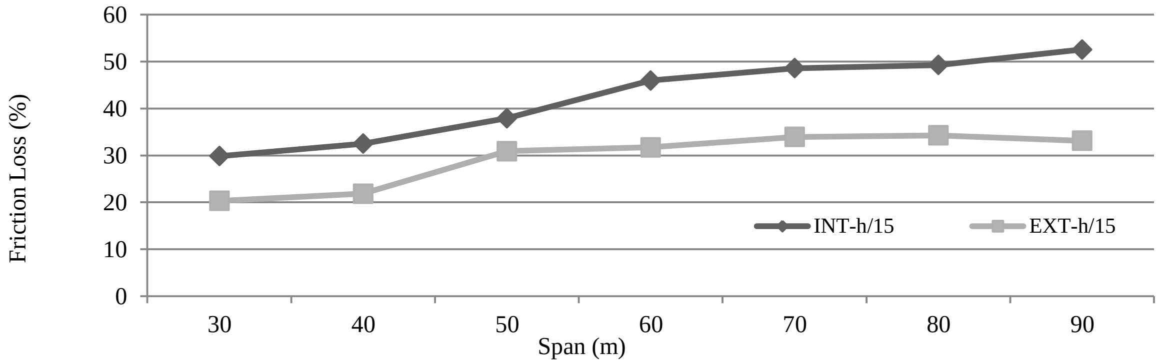

Fig. 3.1 Friction Losses for Int. & Ext. Box Girders, span to depth ratio, L/h=15

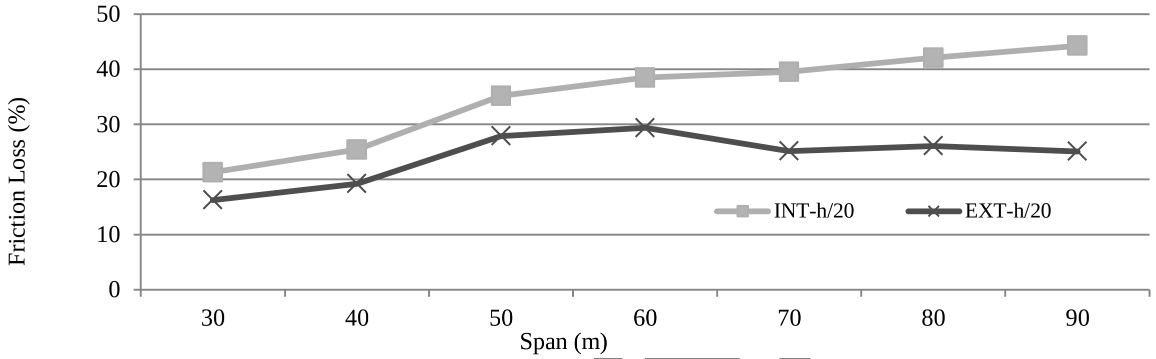

Fig. 3.2 Friction Losses for Int. & Ext. Box Girders, span to depth ratio, L/h=20

Fig. 3.1 indicates maximum friction losses are 30% & 53% for L/h=15, for internal system with L/h=20 are about 22% & 45% as in Fig. 3.2. Fig. 3.3 with L/h=25, friction losses for internal system are about 16% and 39%. For external prestressing figures shows min. and max. friction losses are about 20% and 32%, 17% and 27% and 12% and 20% for L/h=15, 20 and 25 respectively, decrease in friction loss as decrease in section depth.

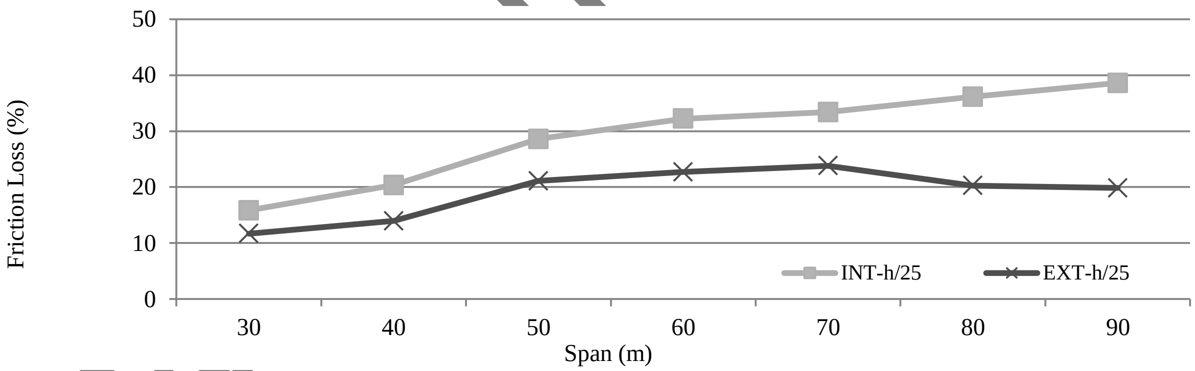

Fig. 3.3 Friction Losses for int. & ext. Box Girders, span to depth ratio, L/h=25

3.2 Jacking Force Required for int. & ext. Prestressing

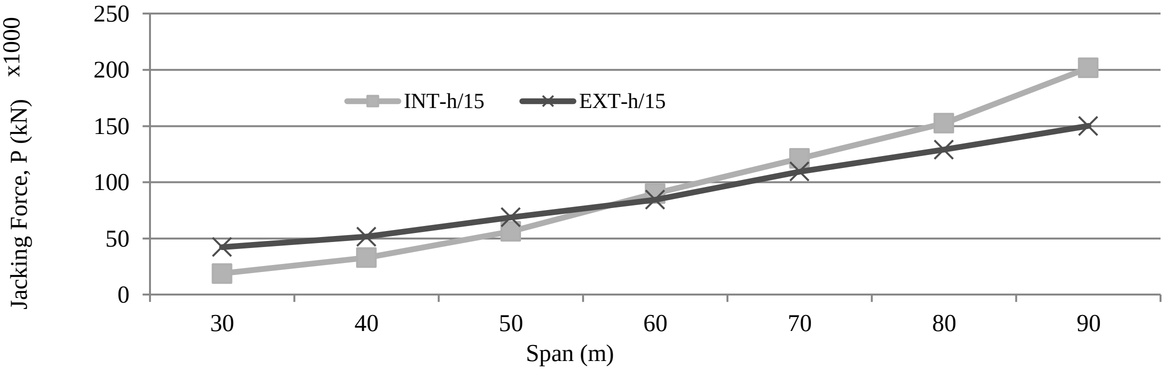

Fig. 3.4 Req. Jacking Force for Int.& Ext. Box Grders, Span to Depth ratio, L/h=15

ISSN No.: 2454- 2024 (online)

International Journal of Technical Research & Science

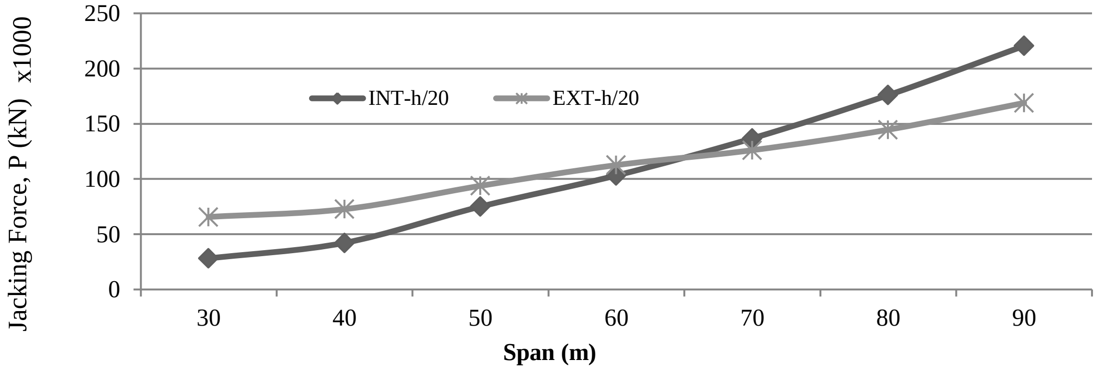

Fig. 3.5 Req. Jacking Force for Int.& Ext. Box Girders, Span to depth ratio, L/h=20

Fig. 3.4, 3.5 and 3.6 shows required an amount of jacking force for internally and externally box girder bridges having different span lengths and span to depth ratios, graphs show amount of jacking friction increases as span increases for both systems. Fig.11 (L/h=15) shows required jacking force for internal system is lesser than external prestressing system up to span of about 58m, Fig.12 (L/h=20) shows required jacking force for internal system is lesser than external prestressing system up to span about 65m, Fig.13 (L/h=25) shows that required jacking force for internal system is lesser than external prestressing system up to span of about 71m. Generally, amount of required jacking force depends on a number of variables but in our study, it majorly depends on eccentricity, friction losses & allowable stress.

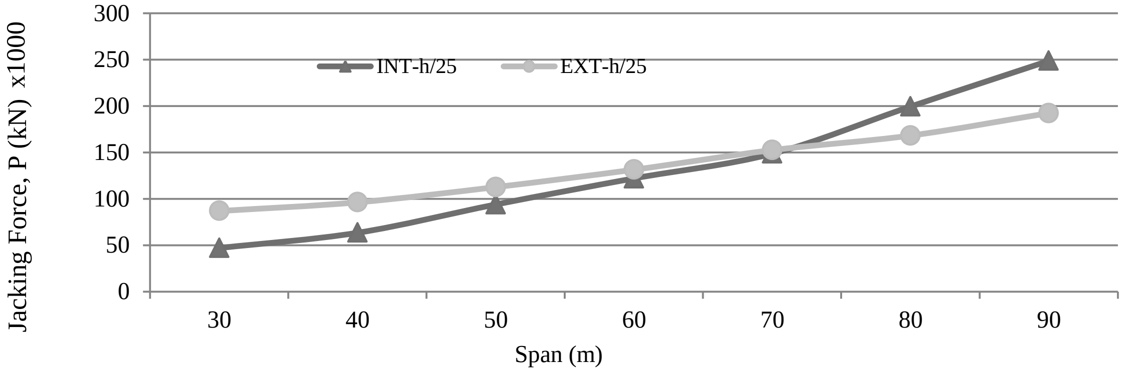

Fig. 3.6 Req. Jacking Force for Int. & Ext. Box Girders, Span to Depth Ratio, L/h=25

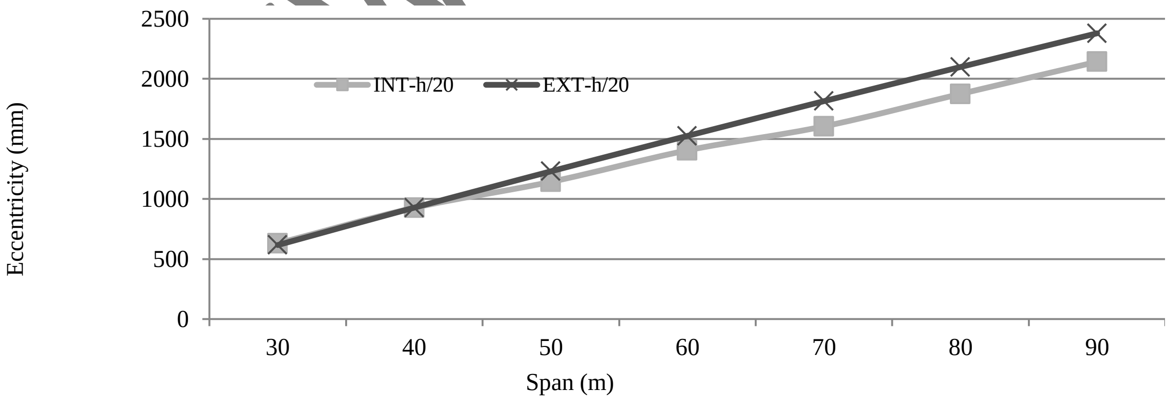

Fig. 3.7 Eccentricity Variation for Int. & Ext. Girders with Span to Depth Ratio, L/h=20

Fig. 3.7 shows variation of eccentricity along span for both int. & ext. prestressing systems. For span 30m external system gives less eccentricity as compared to internal one and at span of 40m eccentricity for both the systems is about equal in magnitude. Based on Fig.12 (L/h=20 four spans 30m, 50m, 65m and 80m were selected for complete analysis and design of prestressed box girder bridge with internal and external tendons.

3.3 Influence of Girder Depth on Required Amount of Prestressing

Fig. 3.8 shows influence of girder depth on the required amount of prestressing for internal and external prestressing systems, for sections with overall section depth up to 2m amount of required jacking force for external is more as compared to the internal system. If 3m section depth is used for spans from 50m to 70m jacking force requirement for external system starts decreasing after 65m span.

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International Journal of Technical Research & Science

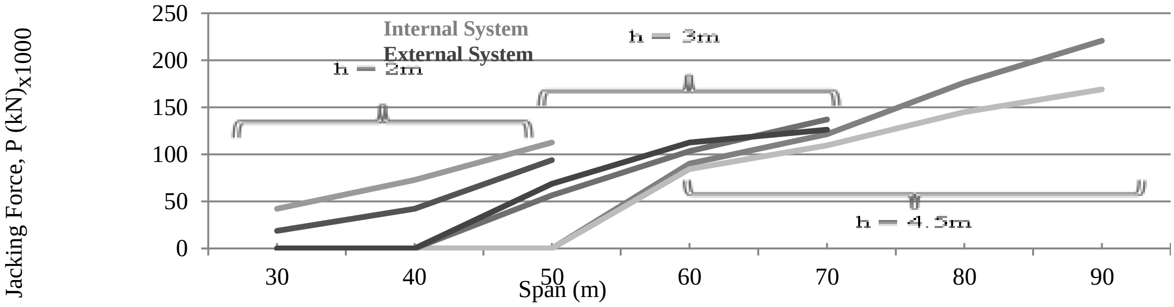

Fig. 3.8 Influence of Girder Depth on Required Amount of Prestressing

3.4 Web Shear Reinforcement Required for Prestressing System

Fig. 3.9 shows variation in the amount of web shear reinforcement required for both internal and external prestressed box girder bridges, external prestressing system shear reinforcement requirement in webs is more as compared to the internal prestressing system.

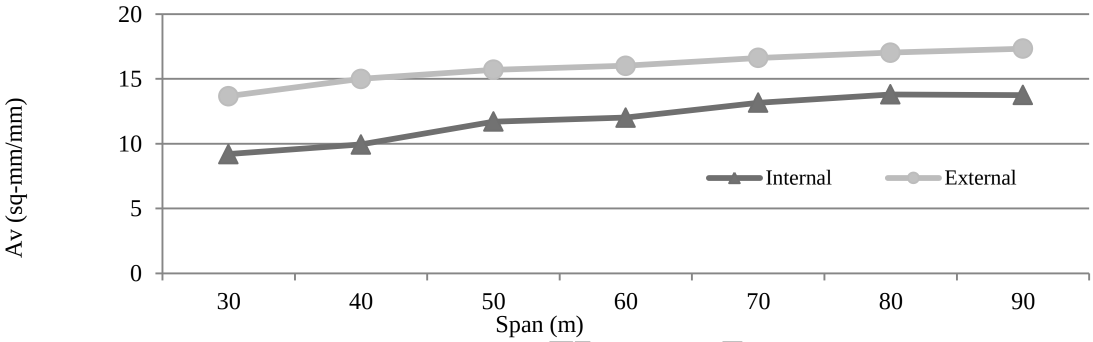

Fig. 3.9 Required Web Shear Reinforcement for int. & ext. Prestressing Systems

3.5 Pier Reinforcement Required for Prestressing Systems

Fig. 3.10 shows variation in the amount of pier reinforcement required for both internal and external prestressed box girders, for external prestressing system main reinforcement requirement in piers of box girder is less as compared to the internal prestressing system. For external system thin webs have been used as tendons are placed outside concrete section, reduction in web thickness reduces overall weight. Lighter superstructure results in lesser load applied to the substructure that reduces the requirement of main pier reinforcement for bridges with external tendons as compared to internal systems.

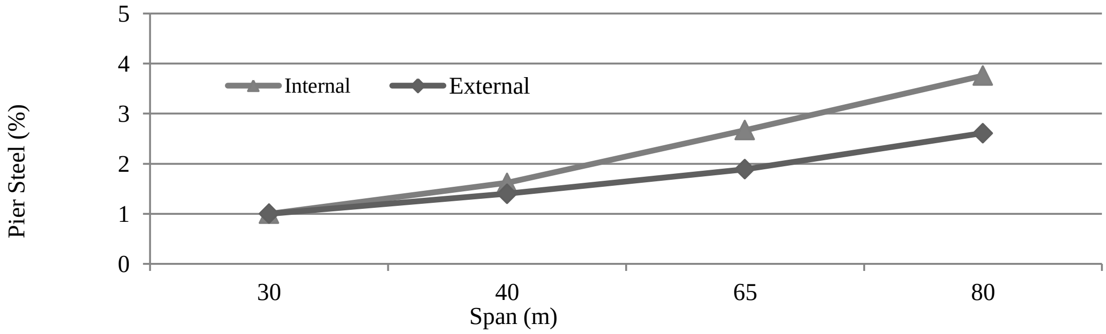

Fig. 3.10 Required Pier Reinforcement for int. & ext. Prestressing Systems

3.6 Pile Load for Internal and External Prestressing System

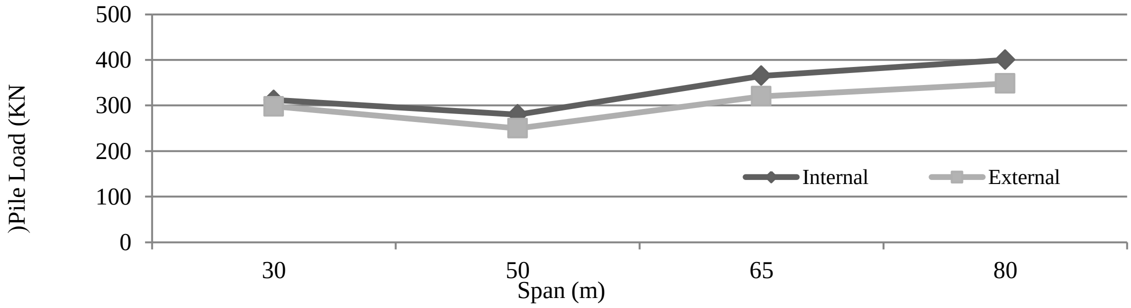

Fig. 3.11 Pile Load for Internal and External Prestressing Systems

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Fig. 3.11 shows variation in pile load required for both internal and external prestressed box girders, for the external prestressing system pile load requirement in piles of box girder is less as compared to the internal prestressing system. For external system thin webs have been used as tendons are placed outside the concrete section, reduction in web thickness reduces overall weight. Lighter superstructure results in lesser load application to substructure that reduces the applied pile load for bridges with external tendons as compared to internal systems.

3.7 Pile Cap Thickness for int. & ext. Prestressing System

Fig. 3.12 shows variation in pile cap thickness required for both internal and external prestressed box girder bridges, for external prestressing system pile cap thickness requirement in pile caps of box girder is less as compared to pile internal system. For external system thin webs have been used as tendons are placed outside concrete section, reduction in web thickness reduces overall weight. Lighter superstructure results in lesser load application to substructure that reduces Pilecap thickness for bridges with external tendons as compared to internal systems.

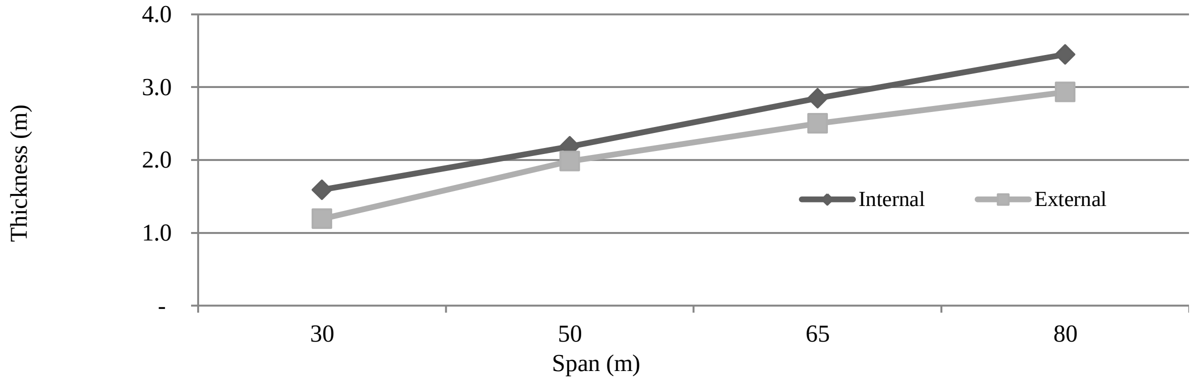

Fig. 3.12 Pilecap Thickness for Internal and External Prestressing Systems

3.8 Pilcap Reinforcement req. for Prestressing System

Fig. 3.13 shows variation in pile cap reinforcement required for both internal and external prestressed box girders, for external prestressing system pile cap reinforcement requirement in pile caps of box girder is more as compared to the internal system. For external system, thin pile caps as compared to internal systems have been used. So the requirement of reinforcement in pile caps for external prestressing systems is more.

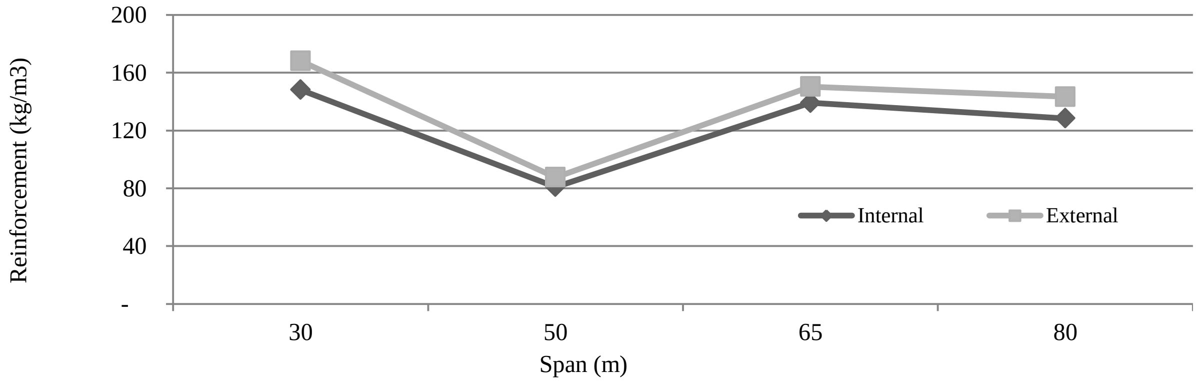

Fig. 3.13 Pilecap Reinforcement for Internal and External Prestressing Systems

CONCLUSIONS

The outcomes and observations made from this study are as follows.

- The super structure model results indicate that for box girder bridges with external tendons only, more jacking force is required for spans up to 60m as compared to for box girders with internal tendons.
- Thin webs can be used for externally prestressed bridges due to the absence of tendons inside the webs, so a reduction in web thickness also reduces the shear resistance of the webs of externally prestressed box Girder Bridge. Therefore, more shear requirements in webs are more for the external system than an internal system.
- Thin webs for the external system also reduces the overall superstructure weight that results in a reduction in pile and pier load and pile cap thickness and their reinforcement ratios.

- For smaller spans up to 30m superstructure and overall cost for the internal system are less than the same required for the external system, but substructure cost is more for all span ranges of the internal system.

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