Stress in Unbonded Tendons for Post-Tensioned Box Girder Bridges

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Abstract:- **Segmental construction is increasing in bridge industry particularly in urban parts of the country. Ease in construction, less disruption to traffic, fast construction, reduction in dead weight and economy of constructions are the reasons for choosing segmental construction. Segmental box girders with unbonded post tensioning tendons are being widely used in the construction of bridges. METRO bridges, elevated highway bridges and sea links are of this type. It is to mention that ultimate flexural behaviour of externally prestressed segmental bridges are different from that of internally prestressed bridges. Segmental box girder bridges with the combination of external unbonded tendons and internal bonded tendons are also being constructed. In such bridges, internal bonded tendons could improve ductility and the external tendons could be convenient for maintenance. The load carrying capacity of box girder bridges of external unbonded tendons and internal bonded tendons is greater than that of a segmental box girder with only external tendons. In view of this, it is necessary to study the ultimate flexural behaviour of segmental box girder with external and internal unbonded tendons. Generally the ultimate flexural behaviour of any unbonded tendons is evaluated by the stress at ultimate in unbonded tendons. Therefore, the paper intends to review the literature on the ultimate flexural behaviour of the stress in unbonded tendons at ultimate for segmental box girder bridges with external and internal unbonded tendons, and to perform parametric studies. Accordingly, performance of prediction equations developed for calculating the stress-increase in unbonded tendons at ultimate Δf_{ps} by various researchers, has been carried out. Data published in the literature has been used for the studies. Δf_{ps} (predicted) by the prediction equations has been compared with Δf_{ps} (experiment), and performance of them have been found out. Parametric studies on equivalent plastic hinge length and Δf_{ps} (experiment) also have been performed, and concluded.**

INTRODUCTION

The segmental construction of concrete box girder bridges represents one of the most fascinating examples of the advancement of structural engineering towards the 21st century. Improved efficiency of materials, better handling of environmental concerns during construction and improved aesthetics, along with the inherent safety and durability of the structures, have become new requirements for future structural designs. The segmental construction method uses external or internal prestressing. Prestressing with unbonded tendons behave differently, when compare to bonded tendons. Because the tendons are unbonded, loading produces an increase in the stress of the tendon beyond the effective prestress which is lower than that of bonded prestress bridges. Therefore it is necessary to evaluate the nominal flexural resistance using the parameter: stress at ultimate in unboned tendons f_{ps}. Researchers have proposed equations to calculate the stress at ultimate in unbonded tendons. It is necessary to review them and their performances. Also, influencing parameters on stress at ultimate in unbonded tendons need to be identified by parametric studies.

LITERATURE REVIEW

Rabbat and Sowlat (1987) tested three decked bulb T section segmental concrete girders. The three girders consist of i) post-tensioned with internal tendons; ii) post-tensioned with external tendons; and iii) post-tensioned with external tendons, but the second stage of casting at top of the bottom flange. They tested with two loading cycles, the first load cycle consists of loading upto the mid span deflection of about 76 mm and unloaded. Then they burned the anchorage wedges to simulate the loss of prestress due to severe earthquake. Subsequently, they did the second load cycle till the destruction. Then the They concluded that i) girder with internal tendons (dry joint) attained strength predicted by the classic bending theory; ii) despite the loss of bond of the top two strands, the girder of external tendons and modified external tendons exceeded the strength predicted by the AASHTO specification. In the first load cycle, the girders with internal tendons and modified external tendons behaved similarly, and the girder with internal tendons exhibited larger deflections than that of the one with modified external tendons. Amar (1988) carried out experimental studies on the static and dynamic behaviour of simply supported two-cell prestressed concrete box girder bridges. He concluded that i) the bridge behaved linearly up to approximately 2 times the service load and allowed no signs of cracking and showed the mid span deflection as L/4400; ii) the bridge exhibited load capacity 3.5 times more than the load predicted by the Ontario Hoghway Bridge Design code, and therefore the bridge design procedures show conservatism. Ramos and Aparicio (1996) developed a numerical model for the analysis of monolithic and segmental externally prestressed concrete bridges of simply supported and continuous conditions. They claimed that the model can calculate the real stress in the prestressing steel at any load level and for any type of externally prestressed bridge, and the joint model gives very good predictions of the behavior of segmental beams with internal and external prestressing. Aparicio et al. (2002) tested three segmental box girders of 7.2 m span with box section. They concluded that the segmental box girder failed due to crushing in the upper slab of the girder. At the failure load, the stress at ultimate in unbounded tendons for all the girders was lower than the yield stress of the tendons. Stress increase in unbounded tendons was large, which resulted in ductile failure with high deflections and without yielding of strands. Saibabu et al. (2010) had conducted experimental investigation and studied the behaviour of the segmental joints at decompression, cracking and ultimate stages on a scaled model of a prototype box girders. They concluded that in the elastic load level, the segmental girder behaved satisfactorily and was nearly identical to the monolithic girder, and epoxy joint behaved linearly up to the elastic limit. Yuan et al. (2013) conducted experimental investigations on the behavior of box beams with hybrid tendons. They tested three scaled specimens with different ratios of the number of internal tendons to the number of external tendons under bending. They concluded that the ratio of tendons had a significant effect on the load carrying capacity and ductility of the beams. Joints next to the pure bending zone and nearest to the applied load were opened. External tendons showed lesser stress increment than that of internal tendons due to second order effects. The moment-strain of the plain bar of the mid span segment is two straight lines. From zero to turning point, the initial compression strain produced by prestressing was released and from turning pint to failure, the bar acquired tensile strain, caused by difference in strain between internal tendons and the surrounding concrete. Overall, the beams failed with concrete crushing in the top of the critical joint. Yuan et al. (2014) have conducted experimental investigations on Segmental prestressed-concrete box girders with external unbonded and internal bonded post-tensioning tendons. They studied four different tendon ratios and two types of loads combined loads. The analysis of unbonded post tensioning prestressed segmental box girders presents a difficulty in comparison with the analysis of conventional method because the tendon stress at ultimate cannt be calculated accurately with section analysis. The simply supported specimens consist of 12 numbers of segments of five types. They tested under static load until failure. The authors concluded that the stress increase in the external tendons Δf_{ps} is approximately 30% to 50% of the effective prestressing stress f_{pe} at the ultimate failure load. Most of the proposed design equations for evaluating the stress increase in unbonded external tendons at ultimate stress are conservative and encounter scatter in predicting experimental data. The openings between the segments and crushing of the concrete compression region are the main reasons for the nonlinear behavior. Gupta et al. (2010) have done analytical studies on box girder bridges considering three cross-sections namely Rectangular, Trapezoidal and Circular. They considered three dimensional 4-noded shell elements for discretization and analyzed the complex behavior of different box-girders. They also carried out parametric studies using SAP 2000. Authors have investigated on the effect of increase in depth of rectangular box girder and did linear analysis on the behavior of these box girders as per Indian Road Congress provisions. They considered a span of 20.0 m for the analysis of the bridge for all types of cross sections and three depths such as 2.0 m, 2.4 m and 2.8 m. In the parametric study, they compared the different loadings such as dead load and live load for maximum eccentricity at mid-span. From the results of linear analysis of three box girder bridge cross-sections namely Rectangular, Trapezoidal and Circular of varying depths, the authors concluded that i) the detailed study carried out using SAP software has clearly brought out the effectiveness of 4-noded shell elements for analysis of box girder-bridges; and ii) the simple beam theory is a crude approximation for analysis of box sections. Bhivgade (2000) had conducted the analysis and design of two lane simply supported box girder bridge made up of prestressed concrete as per IRC-6 and IRC-18. He did the analysis of box girder with clear span of 30 m and roadway width of 7.5 m using SAP 2000. The author concluded that i) the box girder shows the better resistance to the torsion of superstructure; ii) various trail of l/d ratio, deflection and stress criteria satisfied the permissible limits; iii) as the depth increases the prestressing force decreases and the number of cables decreases; and iv) because of prestressing the more strength of concrete is utilized and also well governs the serviceability. Osimani (2004) reviewed the literature available on prestressed concrete bridges with external tendons. Externally prestressed bridges are becoming popular because of their advantages such as simplicity and cost-effectiveness. Previous investigations reported that the stress in the unbonded tendons slightly increases with applied load before cracking. Whereas the stress increases significantly after cracking takes place. The increase in the stress level depends on the amount of tension reinforcement. Higher the reinforcement index, lower the magnitude of the stresses. Also, the span to depth ratio is one of the most important parameters that affect the trend of the stresses in the unbonded tendons. He summarized from the above review that i) the prediction of increase in stress in unbonded tendons beyond the effective prestress is the major issue in the post-tensioned concrete members; and ii) even though dedicated efforts or the computational method of the unbonded tendon stress have been made for more than four decades, it is still questionable if any proposed method are consistent at all.

DISCUSSION

Δf_{ps} is taken as a tool for assessing the prediction equation because Δf_{ps} is directly related to the formation of plasticity. To determine the nominal flexural resistance of the box girder bridges with external unbonded tendons, the parameter stress at ultimate in unbonded tendons f_{ps} is being used. Various researchers have developed expression or equation to predict the f_{ps}. Eleven equations were considered and five were selected out of which, on the basis of the analytical approach and generality in application. They are Tam and pannell (1976), Du and Tao (1985) Du and Liu (2003) Harajli (2006) and ACI (2008) are as follow:

Tam and Pannell (1976)

$$f_{ps} = f'_c \frac{\dfrac{q_e+\lambda}{1+\dfrac{\lambda}{\alpha}} - \dfrac{q_{s\lambda}}{\alpha+\lambda}}{\rho_p} \qquad (1)$$

where

$$q_e = \frac{A_{PS} f_{pe}}{bd_p f'_c}$$

$$q_s = \frac{A_s f_y}{bd_p f'_c}$$

$$\varphi = 10.5$$

Du and Tao (1985)

$$f_{ps} = f_{pe} + [786 - 1920 q_o] \qquad (2)$$

where

q_o = reinforcement index $\leq$ 0.3

Du and Liu (2003)

$$\Delta f_{ps} = \frac{4E_{PS}\varepsilon_m \delta_{mid}}{L^2} \qquad (3)$$

where

ε_m = tendon eccentricity at box girder midspan

δ_{mid} =box girder mid span deflection .

Harajli Equation (2006)

$$\varepsilon_{ps} = \varepsilon_{pe} + \varepsilon_{cu}\left(\frac{d_p - c}{\frac{L}{n}}\right)\left(\frac{20.7}{f} - 10.5\right) \qquad (4)$$

$f = 3$ for single concentric load

$f = 10$ for two pint load and uniformly distributed load

The American concrete institute (ACI) code equation (ACI 2008) is given as

$$\Delta f_{ps} = f_{ps} - f_{pe} = 70 + \frac{f'_c}{\kappa \rho_p} \qquad (5)$$

Where

f'_c = concrete compressive strength

K = 100 for L/d$_p$ $\leq$ 35 and K=300>35

ρ_p = ratio of prestressing steel.

SPECIMENS

The equations were validated using the experimental data published in the literature, the data comprise of Aparicio et al. (2002), Yuan et al. (2013), Amar (1988) and Saibabu (2010) the effective prestress f_{pe} of all the data are in the range of 800 MPa to 978 MPa and the compressive strength of concrete f'_c are in the range of 28 MPa to35 MPa. Three specimens of Aparicio et al. (2002) namely D2, DS1 and DS2 of size 7.20 m x 0.60 m with simply supported span , three specimens of Yuan et al. (2013) namely SPCB2-4-4, SPCB2-6-2 and SPCB2-2-6 with simply supported span of 5760 mm x 600 mm and B1 with simply supported span of 3531 mm x 257 mm. Graphs representing Δf_{ps}(exp) vs Δf_{ps}(predicted) have been made for all equations. The general approach of the ACI to determine f_{ps} is that $f_{ps} = f_{pe} + \Delta f_{ps}$. It is to mention that at the stage of the f_{pe} the effective prestress member does not undergo any deformation due to external load. All the behavioral changes happen only at the stage beyond effective prestress. On this basis it was intended to consider only Δf_{ps} for the comparison. Accordingly Δf_{ps}(exp) vs Δf_{ps}(predicted) were compared. Fig. 1 shows the performance of the equation Tam and Pannell (1976). Its performance are satisfactory and the data are nearer to the perfect correlation line.

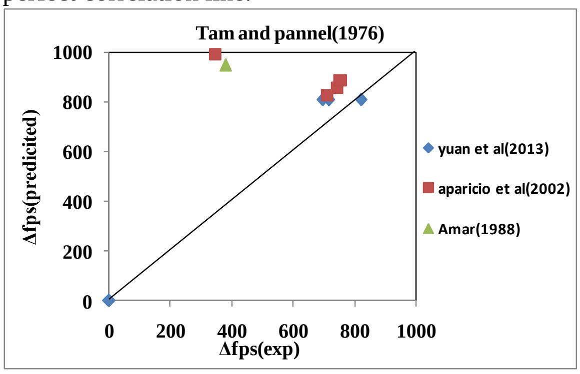

Fig.1Prediction of Δf_{ps} by Tam and pannell (1976)

However the equivalent plastic hinge length assumed in the equation is constant as 10.5, which is empirical. As for as performance of equation of Du and Tao (1985) shown in Fig. 2. Some of the data are correlated and some have not correlated. Moreover, the equation formed in the empirical basis. The performance of next equation is ACI (2008) is shown in Fig. 3 which shows the scattering and the equation uses 70 as constant and limited by k value for two different span to depth ratio.

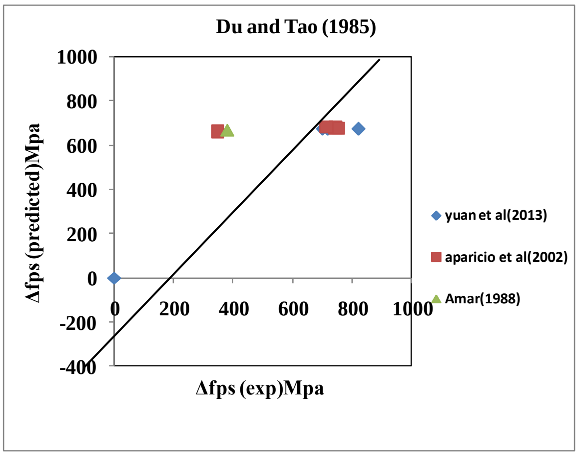

Fig. 2 prediction of Δf_{ps} by Du and Tao (1985)

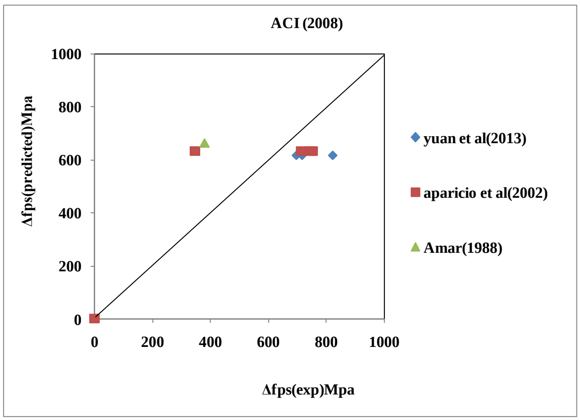

Fig. 3 prediction of Δf_{ps} by ACI (2008)

The equation of Harajli (2006) is shown in Fig. 4 and this shows the many of the points concentrated on perfect correlation line and some of the points scattered. However, the equation has an analytical background. It has incorporated a defined equivalent plastic hinge length L_o in the equation. In view of this, the equation of Harajli (2006) could be observed as a satisfactory equation. The equation of Du and Liu (2003) is shown in Fig. 5, which is much scattered from the perfect correlation line and therefore the performance is not satisfactory.

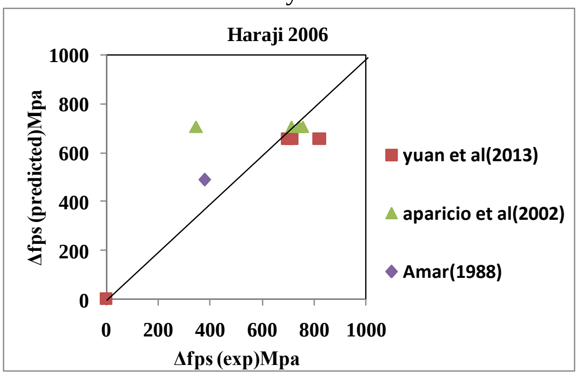

Figure 4 prediction of Δf_{ps} by Harajli (2006)

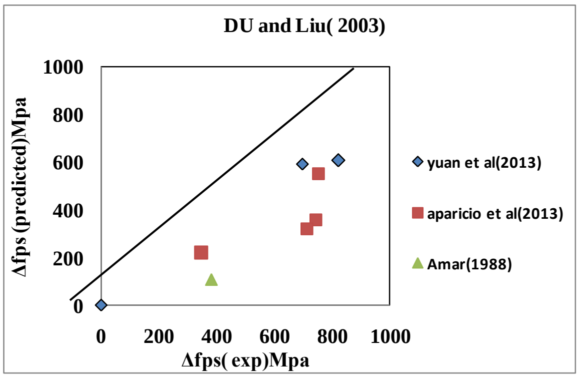

Fig.5 prediction of Δf_{ps} by Du and Liu (2003)

In the comparison of all the equation and their performance it can be understood that the analytical background is the basis for the successful prediction of an equation. In view of this, the status of equivalent plastic hinge length with regard of Δf_{ps} is studied. From the above figures, the scattering of points is due to the unsatisfactory performance of equations which is due to not predicting the plastic hinge formation properly. The most important parameter that influences the increase in stress in unbonded tendons at ultimate is the equivalent plastic hinge length. So by predicting the plastic hinge formation and length correctly, it is possible to predicted Δf_{ps} accurately. Therefore a parametric study on equivalent plastic hinge hinge length needs to performed to observe the status. Accordingly, the equivalent plastic hinge length is compared with stress-increase in unbonded tendons at ultimate Δf_{ps} (exp). The very old definition and a empirical form for equivalent plastic hinge length $L_O = 1.5d_p$ has been used for comparison in Fig. 6 to observe the trend of the data. It is correlating with perfect correlation line. However, it is empirical. In Fig. 7, Pannell's plastic hinge length $L_O = \varphi.c$ has been used and it is observed that it is not correlated with experiment data. Next, the equivalent plastic hinge length of Li- Hyung Lee et al (1999) ie $L_o = \left[\dfrac{1}{f} + \dfrac{1}{L/d_p}\right]$ is used and shown in Fig. 8. It is nearer to the correlation line. Also L_o of Harajli and Hijazi ie $L_O = d_p\left[\dfrac{s}{d_p}\left(\dfrac{0.95}{f} + 0.05\right) + 1\right]$ is also nearer to the correlation line, which is shown in Fig.9. It can be observed that $1.5d_p$, L_o of Li-Hyung Lee et al (1999) and Harajli and Hijazi have correlated with experiment data of Δf_{ps} (exp). From the performance of Fig.6, Fig 8 and Fig.9, it can be concluded that the stress-increase in unbonded tendons at ultimate is directly related to the equivalent plastic hinge length. It can also be believed that the reasons for the unsatisfactory performance of the prediction equations are due to the lack of defining the equivalent plastic hinge length. Equivalent plastic hinge length needs to be defined from the plastic rotation theory of reinforced and prestressed concrete structural members. Graphs representing Δf_{ps} (exp) vs L_o have been made for all equations.

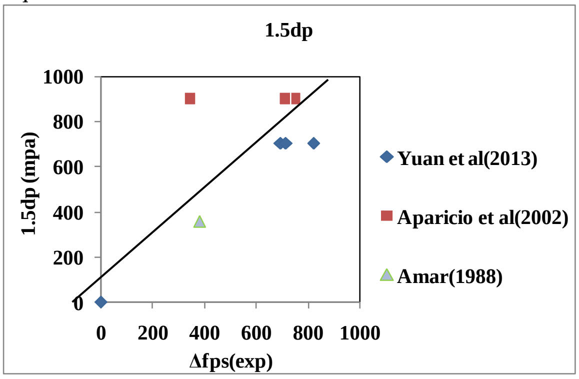

Fig 6 Comparison of Δf_{ps} (exp) with L_O of 1.5d_p

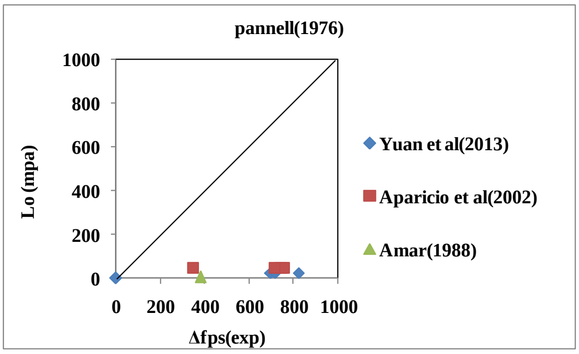

Fig 7 Comparison of Δf_{ps} (exp) with L_O of Pannell (1976)

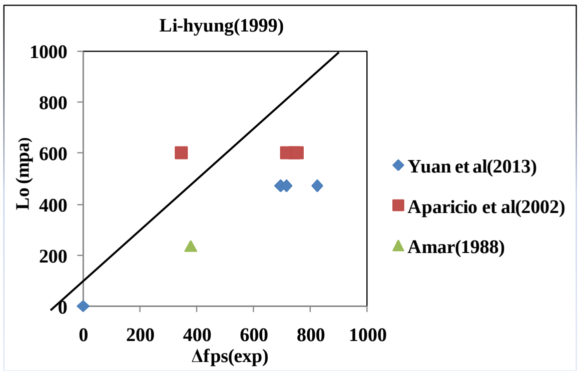

Fig. 8 Comparison of Δf_{ps} (exp) with L_O of Li-hyung (1999)

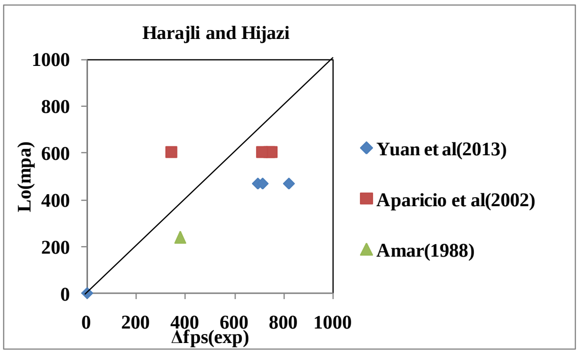

Fig 9 Comparison of Δf_{ps} (exp) with L_O of Harajli and Hijazi

CONCLUSION

1. Equivalent plastic hinge length is having almost a linear relation with stress-increase in unbonded tendons at ultimate for simply supported box girder bridges.
2. Reason for the unsatisfactory performance of the prediction equations are due to the lack of defining the equivalent plastic hinge length.
3. Equivalent plastic hinge length needs to be defined from the plastic rotation theory of reinforced and prestressed concrete structural members.

NOTATIONS

A =Area of bonded prestressed reinforcement

A_{PS} =Area of unbonded prestressed reinforcement

A_S =Area of non-prestressed tensile reinforcement

A'_S =Area of non-prestressed compressive reinforcement

b =Width of the section

b_w =Web thickness for flanged sections

c =Neutral axis depth measured from extreme top concrete fiber

d_p =Depth of the unbonded prestressed reinforcement measured from top concrete fiber

E_C =Modulus of elasticity of concrete

E_{PS} =Modulus of elasticity of related prestressed tendon

f =Load Geometry Factor

f'_c =concrete compressive strength

ρ_p =ratio of prestressing steel.

e_m =tendon eccentricity at box girder midspan

δ_{mid} =box girder midspan deflection

f_{py} =Yield stress of prestressing tendons

f_{pe} =Effective prestressing force in unbonded tendons

h_f =Flange thickness of section

L =Span length

L_p =Plastic hinge length

δ =Elongation of unbonded tendon length between end anchorages

ε_{cu} =Strain in the top fiber of concrete at ultimate

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