PORT SAID ENGINEERING RESEARCH JOURNAL

Faculty of Engineering - Port Said University
Volume 29 No. 2 pp: 1:15

# Behavior of Prestressed Composite Steel Plate Girders in Long Span Applications

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**Corresponding author, DOI: 10.21608/pserj.2025.347001.1386.*

*Received 26-12-2024*
*Revised 26-1-2025*
*Accepted 13-2-2025*



## ABSTRACT

The current research highlights the importance of the use of prestressed composite steel girders due to its numerous applications, particularly in long spans where prestressed concrete is inefficient. However, it is not widely used in the fields and researchers lately focus on prestressed steel beams only. Therefore, this paper investigates the advantages of a composite section of steel and concrete prestressed with tendons. The study includes the efficacy of pre-stressed steel in minimizing the weight of structural steel, raising its maximum capacity by focusing on several parameters. These parameters include the shape of the cables, the locations of tendons placement, the shape of bracing and its locations, and the change in the values of the prestressing force, deflection, and strain. Then, these factors are applied in two loading cases: tensioning and after tensioning. In addition, the effect of modifying the span of a plate girder was investigated. When assessing the data gathered at the top of the slab in the first loading instance, the results show that. The deflection at the top slab of the composite plate girder grows with the length of the plate girder's span because under the same pre-stressing force loading, the deflection for the composite plate girder with span 40 m is approximately 48% larger than the deflection occurred for plate girder with span 30 m at first load case.

**Keywords**: Pre-stressed composite steel plate girders, Concrete slab, Tendons, Bracing, steel ribs, Un-shored Section.

## 1. INTRODUCTION

Prestressing steel is not a new concept in civil engineering. However, despite a long and valuable history of prestressing concrete concept, it has only recently got prevalent consideration [1].

The use of external tendons fastened to the steel I-beam allows the material to be prestressed via this technique. To join steel I-beam ends to external tendon ends, beam stiffeners are utilized. Tendons are given a specific configuration by adding more intermediate stiffeners known as deviators. The prestressing technique is based on a straining action balanced by the straining action induced by external loads [2].

One of the advantages of prestressed steel beams is that their entire cross-section is effective. As pre-stressed steel beams extend a structure's fatigue life, it is indicated for dynamically loaded structures. A beam's temporary load-carrying capacity can be gradually enhanced by prestressing it in stages until it meets the required load-bearing capacity. Prestressing significantly minimizes ultimate deflection, allowing wide-span depth ratios. It is simple to construct replacement tendon assemblies for exterior prestressed beams without significantly raising their cost [1].

The hardships from using pre-stressed steel are many engineers and builders inexperienced on external prestressing of steel technologies. The cost of new equipment is relatively high and as high-strength steel is used, prestressing tendons are brittle. Deviators and anchor plates are sensitive components that require precise placement [1].

Analytical and experimental work by researchers provided a description of prestressed steel and its numerous uses.

Vibhute, [3] studied bonded and unbounded pre-stressed steel tendons used for pre-stressing. For analysis, the triangular tendon profile was used. The impact of the eccentric tendon and pre-stressing force on the beam's flexural natural frequency were investigated using dynamic analysis. The results showed that externally un-bonded prestressing is a useful method for strengthening steel I-beams due to its simplicity of application and economic feasibility. Pre-stressed steel beams are lighter than traditional beams with the same span, making them a more cost effective and practical alternative in a variety of situations.

Ronghe, G. and L. Gupta [4] analyzed and compared different tendon configurations and prestressing parameters related to the overall analysis and design of prestressed steel plate girders analytically. A prototype prestressed steel frame with a straight tendon was developed as a case study. The results showed that tendons should be placed below the bottom flange. According to the case study of the prestressed steel loading frame, the percentage increase in load-carrying capability for the straight tendon over the whole span is around 9% when compared to the non-prestressed loading frame.

Oukaili, N.K.A. [5] studied the short-term loading behavior of externally prestressed composite beams and their flexural load deflection. The results indicate that the entire history of strain and stress distribution along cross-section depth, deflection, and stress increment in the external tendons can be ascertained using the analysis model based on the deformation compatibility of the entire structure. Greater tendon eccentricity raises the beams' ultimate strength. Tendons should therefore be positioned below the bottom (tension) flange. External prestressing increases the yield and ultimate loads of composite beams by about 19.23%. Figure 1 shows the steel beam with concrete slab.

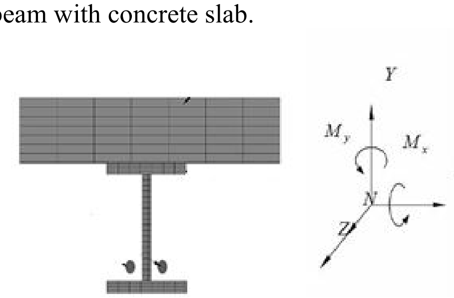



**Figure 1: Cross section of structure element [5].**

In 2010, Belletti, B. and A. Gasperi [6] examined the behavior of steel beams with "I-shaped cross section" that are simply supported and prestressed by tendons. The steel beams are designed to be used as structural parts for the roof, with medium spans ranging from 35 to 45 meters. The study is based on mainly two factors, the amount of the pre-stressing force and the quantity of deviators.

The study's findings demonstrated that forcing tendons to take on the configuration of just two deviators is not recommended. True, beams which have two components of deviators had the least vertical load capacity and prestressing force values. A beam having bracing at both its upper and bottom flanges betters a beam with bracing at just the upper flange. Tensioning after the bracing is installed produces superior results (delayed tensioning); however, the load-bearing capacity of the beam with two deviators is poor, whereas the beam using 11 deviators offers an ideal action, beyond it is more expensive and difficult to construct [2,6].

In 2017 Raju, P.M. [7], the aim of the work is to determine the ideal dimensions of a prestressed steel I-beam with simple support that is laterally unsupported for a specific span and load carrying capability. In this study, the load-carrying capability was limited to 100 kN/m, and the beam's span was 12 m. When tendon losses were ignored and a straight tendon arrangement was assumed for the whole span, the top fiber reached the maximum allowed stress before the bottom flange in the case of simply supported steel beams subjected to gradually applied loads. The greatest prestressing force that could be applied decreases with span. Larger spans were more likely to buckle under significant prestressing forces. For a given load-carrying capacity, the ratio of pre-stress to allowed bending compressive stress levels was between 0.25 and 0.3. The study provided some first guidance on creating optimal specifications for a prestressed steel beam. The authors believed that welding plates with the required dimensions would be an easy solution for the measurements. The primary purpose of the spans and loads considered in this study was to be implemented in residential and commercial buildings.

Abdelnabi [2] in 2013 used numerical analysis to thoroughly evaluate the behavior of beam-column structural members, theoretically and numerically. A mathematical model was created to estimate the maximum capacity of steel I-beams during axial loads and bending moments. That design can accommodate for initial imperfection after conducting a parametric study. The parametric analysis took into consideration variable cross section measurements, length of the span and that are unsupported, the quantity of deviators, tendon design, eccentric behaviors, and pre-stressing force. Beams were characterized using three loading categories: single focused load at mid-span, line load, and uniform bending moment load.

The suggested analytical model accurately predicts the interactive strength, as proved by comparing it to the EC3 design approach and the finite element non-linear analysis. This model is still simple and useful because it proposes a solution to a second order equation that may be obtained with a basic calculator. This suggests that the produced model is a sensible, basic design application. To predict the maximum struggle of beam-columns during contact buckling, an analytical design resembling the Perry formulation and based on Young's equation has been devised. The degree of initial imperfection is considered in this model [2].

In 2005, Dall'Asta [8] used finite element model with non-uniform beam modeling pre-stressed by outside sliding wires. A flexible shear connection was proposed when designing composite beams. Cables can travel long distances and slide smoothly across deviators. For materials, the broad spectrum of nonlinear basic values can be applied. The suggested method can be applied to current nonlinear FE computer programs without the need for extra iterative methods. The recommended approach has been implemented to analyze composite beams made of steel and concrete up to failure. Comparisons with experimental tests verified the numerical results [2, 8].

In 2004, Ryu, H. [9] carried out static tests for bridge using continuous composite box-girder with prefabricated slabs spanning a distance of 20–20 m to assess ultimate strength and estimate flexural performance following cracking. Two prestressing techniques were used on prefabricated slabs: internal pre-stressing prior to external pre-stressing after shear connection [2, 9].

In 2019, Lou, T [10] examines the behavior of continuous pre-stressed composite beams with external tendons of varying cross sections. External pre-stressing improves both the ultimate load carrying capacity and the moment redistribution ability of a continuous steel-concrete composite beam. Higher tendon area results in a somewhat lower stress escalation in external tendons [10].

In 2022, Da Rocha Almeida [11] examines experimental study of the behavior of pre-stressed steel-concrete composite beams with profiled steel decking. Tests were conducted on a beam with a straight tendon. The results suggest that adding pre-stressing can boost the ultimate moment while decreasing deflection. Numerical models reveal that the tensioned steel beam failed due to yielding [11].

In 2023, Wang [12] studied a pre-stressing technique using carbon fiber reinforced polymer (CFRP) plates was investigated to strengthen damaged steel beams. The layer had a significant impact on the strain distributions of the steel-CFRP composite section, but had a minor impact on the flexural behavior of the steel beams. Strengthening damaged steel beams with pre-stressed CFRP plate reduced the effect of the notch depth [12].

In 2024, Xu, Longkang [13] examined the impact of many characteristics, including shear span-depth ratio, pre-stressed tendons degree, height, and damage evolution. A model was developed to predict the shear strength of PSRCC deep beams, taking into account the steel-concrete interaction. The results show. Increasing the pre-stress level increased the cracking load of the specimens, whereas decreasing the shear span-depth ratio increased both the cracking and peak loads [13].

The purpose of this research is to examine how prestressed composite steel hybrid plate girders behave under service and ultimate loads, the prestressing force, and different tendon configurations, different cross section dimension, different plate girder spans are some of the critical parameters that will be the focus of a parametric study of the prestressed plate girders. These variables are also regarded to be important for constructing these girders.

Creating an analytical model with various parameter values for pre-stressed plate girders using non-linear finite element analysis to examine the elastic and inelastic behavior of pre-stressed plate girders with varying parameters and confirming the accuracy of the study results by conducting a thorough parametric analysis of pre-stressed plate girders.

The current research study how to apply these parameters in two different loading, tensioning, and after-tensioning scenarios to get an economic composite steel plate girder section lighter in weight of steel using pre-stressing tendons than non-pre-stressed traditional one.

## 2. FINITE ELEMENT MODELING AND VERIFICATION

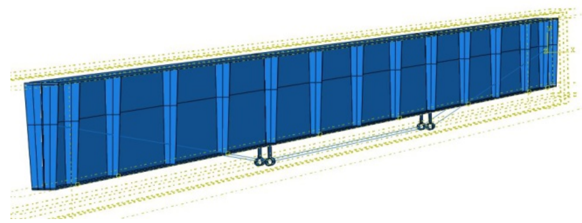
### 2.1. Steel Structure and Material

Belletti, B. [6] used Grade S275, which has Fu (ultimate strength) 430 MPa, Fy (yield strength) 275 MPa for beam with 40 m span, cross section is un-symmetric I beam, two tendons, and fifteen ribs. Tendons are made up of prestressing strands having a diameter of 0.6 inches. The steel used to create the strands has Fu equal 1,860 MPa and Fy of 1,670 MPa [6].

Pipe elements {B31} was employed for deviators having radius of 70 mm and a thickness of 12 mm, four-node shell elements {S4} were used for beam and ribs, tube-tube contact elements {ITT31} were used for tendons which pass inside the deviators [6].

The steel beam's mechanical properties were defined using a plasticity model with isotropic hardening behavior [6]. Figure 2 show the finite element model and mesh control of the verification. Nonlinear finite-element analyses had been done to study the numerical behavior of pre-stressed beams up to failure in response to two separate loading phases: tensioning and after tensioning. During the initial load step (tensioning) pre-stressing force is maximized as the own-weight (dead load of steel beam only) is a single vertical load on the beam. In the last load step (after tensioning), external loads are increased up to the beam's vertical load capacity for a specific pre-stressing force value [6].

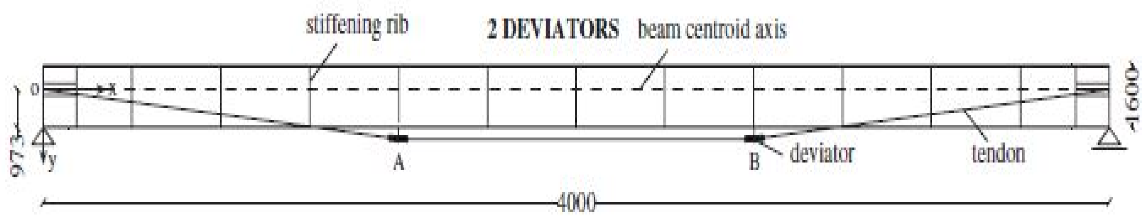
Figure 3 shows the dimensions of the beam with two deviators and the cross-section profile as referenced [6]. The beam was explored till the point of tensioning failure by placing the own-weight load and increasing the pre-stressing until failure occurred then the maximum pre-stressing force was determined. Additionally, the beam's maximum vertical loads were measured after tensioning.



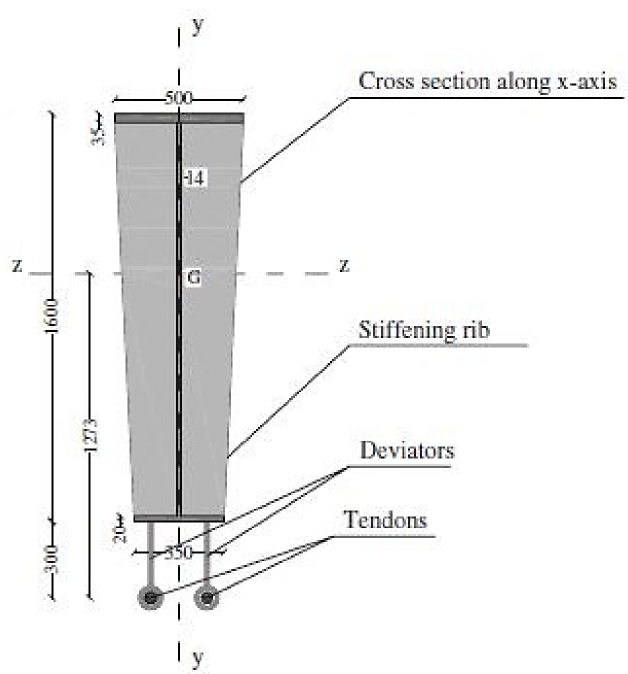
(a)

(b)

**Figure 2: FEM verification (a) the isotropic behavior (b) the mesh control for FEM**




(a)




(b)

**Figure 3: Beam analyzed by Belletti, B (a) beam with two deviators (b) cross section at mid span [6].**

### 2.2. Verification with Belletti, B.

Finite element is used to mode the previous steel prestressed beam with two deviators using ABAQUS program version 6.14-1 [14] and verify the results of the model for tensioning and after tensioning loading instances.

Finding the achieved values are almost very close with an error rate not exceeding 4% except the start of deformed curve. Comparing the results of the previous beam with the results of the model using the same ABAQUS program and at the first loading condition (During tensioning) in which the beam is gradually increased by pre-stressing force in addition to loading the beam with its weight only. According to the study as shown in Figure 4. The results of the comparison of the first load case were in good agreement with the ratio of the horizontal displacement in the mid span of the top flange of the beam corresponding to the maximum pre-stressing force that causes failure, 1559 kN, between Belletti, B. [6] and that obtained from current verification being 99.5%.

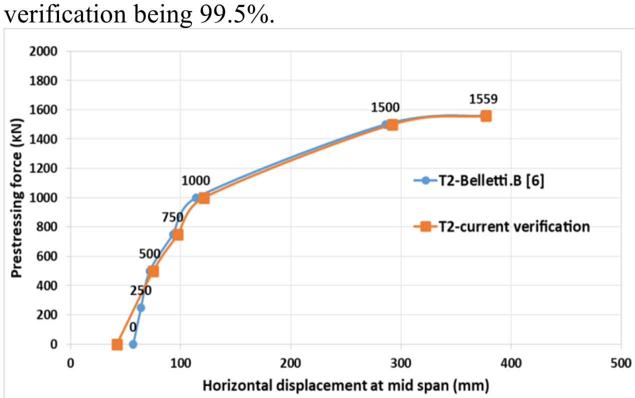



**Figure 4: Comparison between finite element analysis of current verification and Belletti, B. [6] during tensioning.**

Modeling the beam of the analysis in the second loading case (After tensioning), where the beam is loaded with fixed values that is, the one weight plus the prestressing force on the tendon inside the deviator in addition to variable applied load ( external load ) on the beam, (T2-500-BTS) for example mean beam has 2 deviators and pre-stressing force 500 kN with bracing just at the beam's upper flange in addition to variations in the external load values applied to the beam and an examination of the deflection results at the mid span of beam for each of the prestressing forces (0, 500, and 1000) kN.

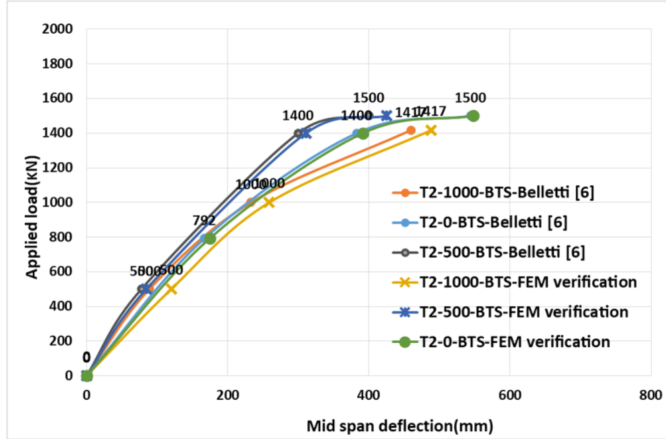



**Figure 5: Comparison between finite element model verification and Belletti, B. [6] after tensioning.**

Tested model (T2-0-BTS) shows that the failure occurs to the beam when applying external loads more than the maximum value ( 1500 kN) and the same situation occurs to the beam (T2-500-BTS), (T2-1000-BTS), but (T2-0- BTS) can withstand higher external load forces than (T2-500-BTS), (T2-1000-BTS) as shown in figure 5, and the beam (T2-500-BTS) is possible to obtain the least deflection and the highest resistance to the beam by using the highest external load value that the beam can be exposed to before collapse, so the capacity of the pre-stressed beam is higher than the non-pre-stressed one.

Comparing the results of Belletti, B. [6] and the current model verification found that the ratios are 0.97, 0.95, and 0.94 for the tested models (T2-0-BTS), (T2-500-BTS), and (T2-1000-BTS), respectively, indicating that the results agreed well in comparison to the second load case (after tensioning).

### 2.3. Verification with Abdelnabi .

The previous model results obtained were verified with Abdelnabi [2] who tested the models (T2-500-BTS) and (T2-0-BTS) for beam with two deviators at second load case (After tensioning) using bracing just at the beam's upper flange. The ANSYS program was used to acquire the results of the deflection in the middle span of the model corresponding to the change in the external load (applied load) values. The external load is the load value distributed along the length of the tested beam.

Figure 6 shows the comparison of the results produced from the finite element analysis model verification with the finite element analysis results obtained by Abdelnabi [2].

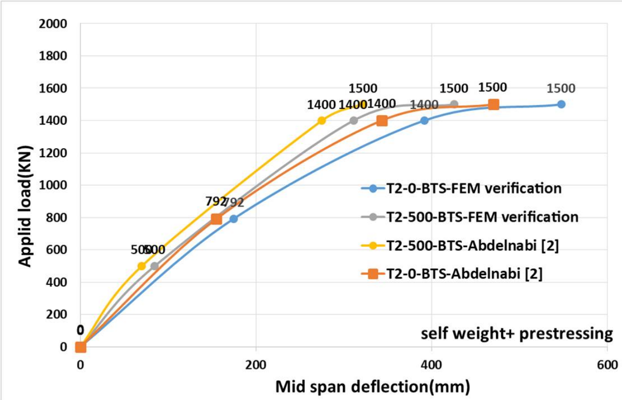


**Figure 6: Comparison between FE model verification and Abdelnabi. [2]**

When testing the model (T2-0-BTS) with zero prestressing force, it is found that from the beginning to the end of the curve expressing the change in the values of the deflection corresponding to the change in the values of the external load, the results of the FE analysis results obtained by Abdelnabi [2] are similar to those of FE analysis model verification. The difference ratio between the ANSYS and ABAQUS analyses is around 90%, which accurately depicts the error ratio and demonstrates the degree of agreement between the ABAQUS and ANSYS results. However, the agreement ratio obtained when evaluating the same model (T2-0-BTS) using Belletti, B. [6] was larger.

Through a comparison of the results from the model (T2-500-BTS) with the prestressing force 500 kN test and the two curves discovered that the values converged in the gradient from the start to the end of the curves. The agreement in the outcomes between FE analysis results obtained by Abdelnabi [2] and the FE analysis model verification was 92%, which is a respectable percentage. However, the verification values that were conducted for the same model with Belletti, B. [6] converged closer than the results obtained from Abdelnabi [2].

The results of finite element modeling and verification showed that:

- Using just two deviators to force the form of tendons is not advised. As the deflection shape of beam is not clear enough so it is preferable to use more than 2 deviators to improve the shape of cables. It works better with bracing at top flange [2].
- This study lacks the use of a composite section of steel and concrete pre-stressing using straight tendons without the use of deviators, attempting to achieve the best dimensions for this section by making it more economical, less expensive, and reducing the weight of steel, resulting in less deflection, which is what will be discussed in this study.

## 3. CURRENT STUDY

There is no doubt about the importance of the concrete section as well as the steel section, each separately. Therefore, a composite section was designed to carry the advantages of both steel and concrete together. The composite section was also prestressed with tensioned steel straight tendons and studied for its effect in long spans of 30 and 40 meters, as reinforced concrete and prestressed concrete are ineffective in this span range.

From here, the composite prestressed steel plate girder is effective for use in two cases, namely a composite section with a span of 30 m and a composite section with a span of 40 m, and the change in pre-stressing force and external load values in the loading scenarios. The first situation is called (Tensioning), in which the section works as a steel section only, and the concrete does not work or resist, and the section is loaded with its own weight and exposed to a prestressing force loaded on the straight tendons. The goal of this case is to make the steel section eliminate its weight and operating loads only, and when loaded with prestressing force on the straight cables, it causes a very weak deflection that is almost close to zero. As for the second case (After Tensioning), the section is work as a composite section and the steel plate girders and reinforced concrete slab work together. After pouring, the concrete hardened and had a weak deflection, which was eliminated in the first case. The composite section is then loaded with external loads, also known as "applied loads," in addition to the tension in the tendons, and the result of deflection is studied in two different span plate girders.

### 3.1. Description of the Material of Prestressed Composite Steel Section.

In this work, the prestressed composite steel plate girder is analyzed; Figure 7 shows the cross section elevation of finite element model, and Figure 8 shows the side view of the composite steel plate girder, which is made up of a concrete slab, plate girders, bracing, ribs, and straight tendons. A span of plate girders was tested at two distances of 30 m and 40 m, and Figure 9 depicts all dimensions of the composite steel plate girders [15].

- Concrete slab: The slab's dimensions are (4000 x 250) mm, with an effective width of 4000 mm and a thickness of 250 mm. The concrete compressive strength is 27.58 MPa, the density is 2400 kg/m$^3$, the modulus of elasticity of concrete is 21000 MPa, and the flooring cover is 0,00048 MPa.

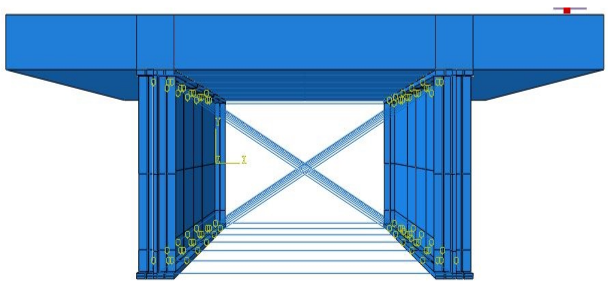


**Figure 7: Elevation of composite steel plate girders.**

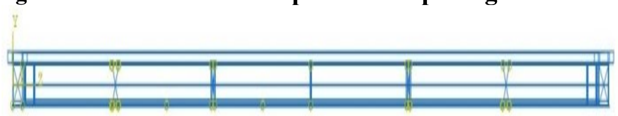


**Figure 8: Side view of composite steel plate girder**

- Steel plate girder: Two sections of steel plate girder are used and their dimensions are shown in Figure 8, the steel has a yielding strength of 345 MPa and an ultimate strength of 450 MPa, plastic strain is 0.2, passion's ratio is 0.3, steel density is $7.85*10^{-5}$ N/mm$^3$, and modulus of elasticity of steel is 200000 MPa [1].

- Tendons: Four straight steel wires are linked to the ribs and placed 400 mm away from the hinged-roller supports, which are 200 mm away from the side of the section. Each tendon has 28 strands with a diameter of 30 mm and covers 19723 mm$^2$. The strands are formed of steel with ultimate stress 1860 MPa and yield stress 1670 MPa, steel density is $7.85*10^{-5}$ N/mm3, modulus of elasticity of steel is 200000 MPa, and passion's ratio is 0.3 [6].

- Ribs: Four solid steel ribs are used in the composite steel plate girder, sized (900*112.5) in mm dimension on each side and 400 mm away from the support formed from the same steel as the plate girder.

- Bracing: Wire element steel bracing with angle shape (L.NO200) was utilized every 5 meters from the length of the plate girder. Seven bracing nodes were used for girders with spans of 30m and 9 bracing nodes for girders with spans of 40m, using the same type of steel as the plate girders.

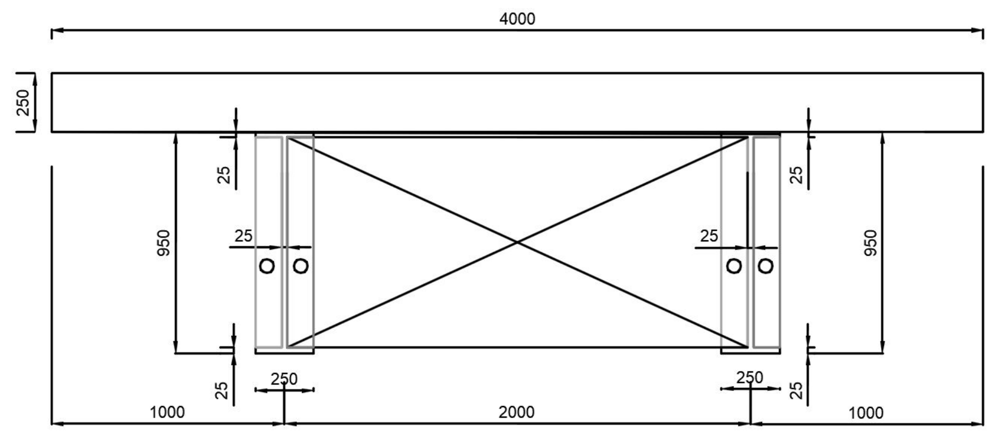
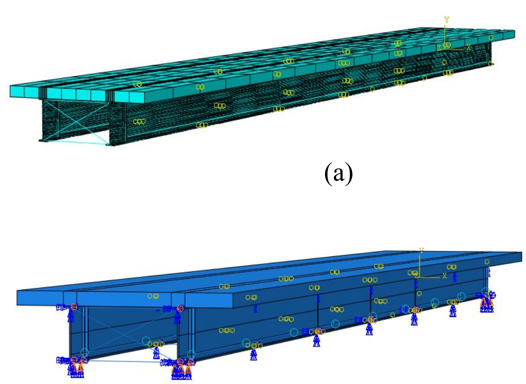



**Figure 9: Cross section showing all dimensions in millimeters**

### 3.2. Finite-Element Analysis of Pre-stressed Composite Steel section

The element used was general isotropic long term in solid homogeneous for steel plate girder type (C3D4): 4-node linear tetrahedron, and use general isotropic long term in solid homogeneous for concrete slab and ribs with element type (C3D8R): an 8-node linear brick, decreased integration, and use element (B31); a 2-node linear beam in space for wire bracing and wire straight tendons. The mesh controls, global and local seeds mesh which used in the model and the boundary conditions for the hinged support at its initial and the roller support at the back end for the model shown in Figure10.



(a)

(b)

**Figure 10: Finite element of composite section (a) mesh controls (b) the boundary conditions for the model**

### 3.3. Results of First Case (Tensioning) In Composite Plate girder (Span 30 m)

In this load case, the un-shored composite prestressed plate girder is called (a steel plate girder) as it works as a steel section only where the concrete slab element does not work and does not resist with us in operation.

At this point, the steel section is loaded with it is own weight only and the operating loads in addition to the prestressing force on straight cables. The purpose of this case is to make the section eliminate the dead load by loading it with the pre-stressing force value which reduces the effect of the deflection occurring from the beam. Deflection can be obtained from Equation (1) to obtain a magnitude of prestressing force (Ps equivalent) that allows the section to eliminate the dead load with a deflection that is almost close to zero and found that the value of deflection is 0.45 mm, and so a very weak deflection value is obtained from the dead loads and the operation load before pouring the concrete in the second stage.

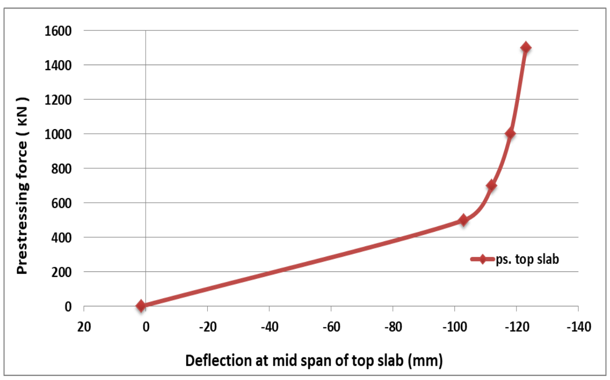



**Figure 11: Vertical deflection at mid-span of top slab versus pre-stressing force curve.**

$P_S$ equivalent $= (5 * W_D*(L)^2) / 48*(e)$ (1)

Where, Ps: the pre-stressing force equivalent, $W_D$: dead load in the section, L: span length of plate girder, and e: eccentricity from the tendons to lower flange of plate girder [15].

Figure 11 shows the relationship between the vertical deflection of the plate girder and the change in the pre-stressing force value in the mid-span of the top slab. The model was loaded with various pre-stressing force values (500– 700–1000–1500) kN, and the deflection was analyzed for each modification. The higher the force value, the greater the deflection, which was 123 mm at 1500 kN . Figure 12 shows deformed shape at first case (during tensioning).

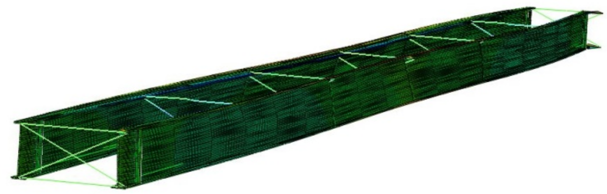


**Figure 12: Deformed shape at first case (during tensioning).**

Figure 13 shows the relationship between the vertical deflection of the plate girder corresponding to the change in the value of pre-stressing force in mid span of bottom steel flange. Testing the model with pre-stressing force and evaluation the deflection results corresponding to the force values at the bottom of the flange, observed that the deflection values increase with increasing forces, although at lower values compared to the results obtained in the middle of the top slab.

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Figure 14 shows a comparison between the values of the deflection corresponding to the change in prestressing force at mid-span in the top slab and bottom steel flange which shows that the deflection caused by a prestressing force of 1500 kN at the bottom steel flange decrease by 7% than the deflection caused by the same pre-stressing force at top slab. This is logical because the concrete slab does not operate or resist under the initial loading condition, and the loading is just on the steel section, in addition to the composite section's own weight. This explains the reason for the difference in the deflection values shown.

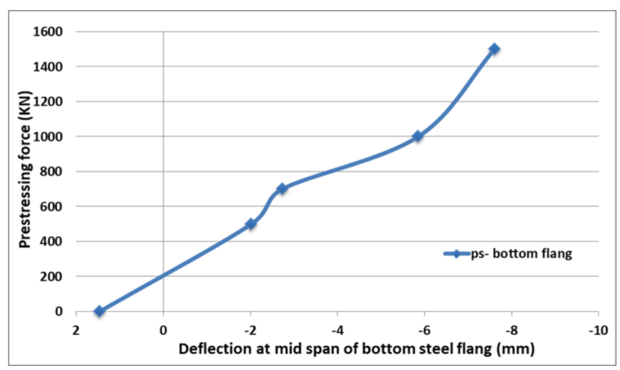


**Figure 13: Vertical deflection at mid-span of bottom steel flange versus pre-stressing force curve.**

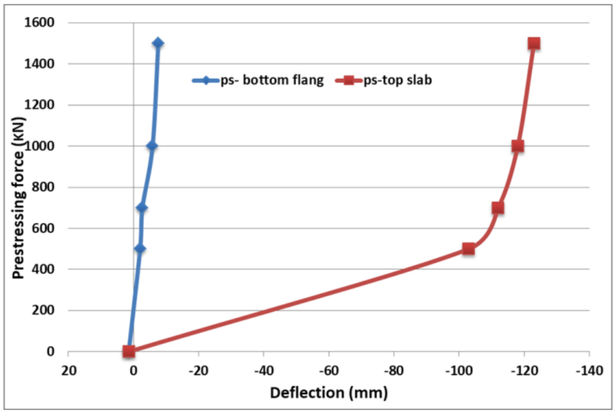


**Figure 14: Deflection versus pre-stressing force at top slab and bottom steel flange.**

### 3.4. Results of Second Case (After Tensioning) In Composite Plate girder (Span 30 m).

In this load case, the section acts as a composite section, with the steel plate girder section and the concrete slab resisting. The section is loaded with both variable values which are the values of the external load (Applied load) on the section, and fixed values, represented by the one weight of the composite section and the value of prestressing force on the straight tendons to be loaded, in each needed loading instance. As a result of changes in the external load values and the stability of prestressing force values to which the tendons are exposed, the section is examined at both the concrete slab and the steel plate girders.

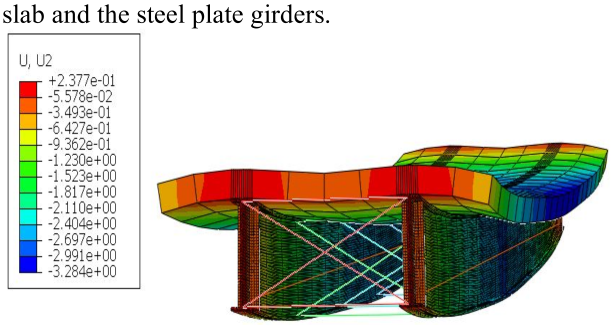


**Figure 15: deformed shape in the second case.**

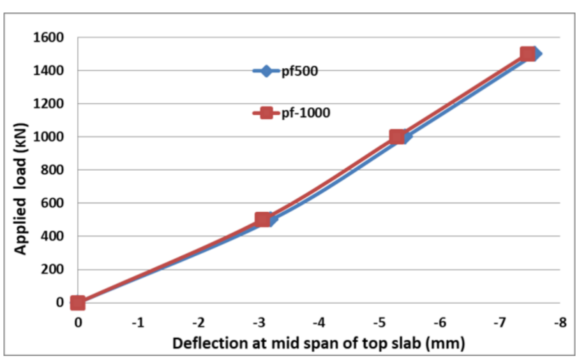


**Figure 16: Deflection versus applied load at top slab.**

The deformed shape at the second load case (after tensioning) is down word as shown in figure 15 as in second load case the model work as a composite section so it can load with more loading so the applied load mean the external additional load on the model is bigger than the own weight and the pre-stressing force on cables which cause deflection down word. Figure 16 shows the relationship between each change in the external load and the variation in the deflection values at the top slab. In addition to the structure's weight, the straight tendons were loaded with 500 kN of prestressing force. The external load values were adjusted to (500-1000-1500) kN. The deflection that occurred in the middle of the top slab was examined for each applied force change. Similarly, the section was loaded with a prestressing force of 1000 kN. Furthermore, the structure's weight is added, and the values of the external load were modified to (500-1000-1500) kN, as well as the amount of deflection that occurred in the center of the top slab. It is discovered that the value of the deflection occurring at the highest external load is nearly identical at both prestressing forces of (500-1000) kN.

Figure 17 shows the relationship between changes in the external load and variations in the values of the deflection at the mid-span of the bottom steel flange. The straight tendons are loaded with a prestressing force of 500 kN, as well as a prestressing force of 1000 kN in addition to the structure's weight. The external load values were adjusted to (500-1000-1500) kN, and for each applied load modification, the value of the deflection that occurred in the middle of the bottom steel flange was evaluated. It was discovered that at both prestressing forces of (500-1000) kN, the value of the deflection occurring at the maximum external load is nearly equal.

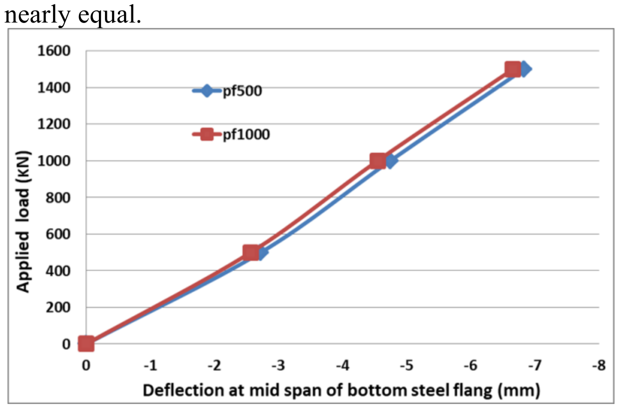


**Figure 17: Deflection versus applied load at bottom steel flange.**

It can be observed that, at a prestressing force 500 kN at the middle of top slab, the value of the deflection is higher than that of bottom steel flange, with an increase of approximately 13% at the largest external live load force, by comparing the values of the results that occurred for the composite section in the second loading case at both of the bottom steel flange and the middle of the top slab, as shown in Figure 18. Furthermore, with a rise of about 10% at the largest external live load force compared to it, the value of deflection is higher than its value in the bottom steel flange when comparing the prestressing force 1000 kN at the mid span of the top slab.

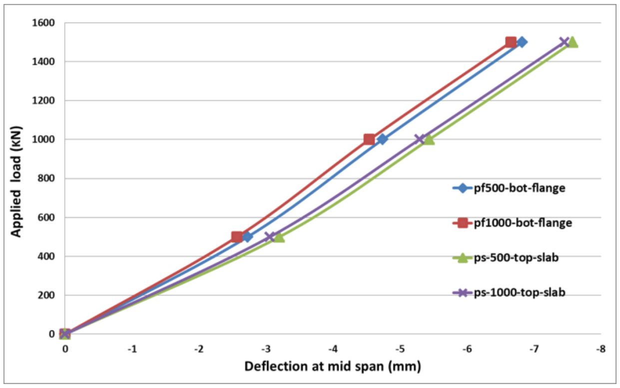


**Figure 18: Deflection versus applied load at top slab and bottom steel flange.**

#### 3.4.1. Strain in composite plate girder 30m

When the tendons are loaded with a prestressing force of 500 kN and 1000 kN, then the stress corresponding to each of the pre-stressing forces and values of the strain at the largest external live load carrying the section shown in Figure 19 for the composite plate girder in second loading condition (after tensioning). The pre-stressing force 1000 kN is the largest prestressing force causing strain on the composite section in span 30 meters when examining the relationship between the prestressing force to which the straight tendons are exposed and the strain occurring at the largest external live load force to which the section is exposed, as shown in Figure 20.

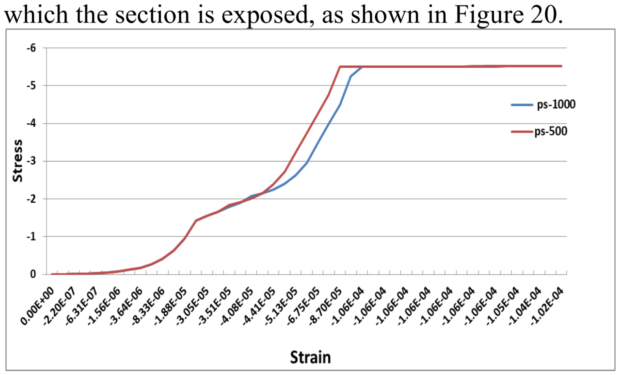


**Figure 19: Stress-strain curve at max external load.**

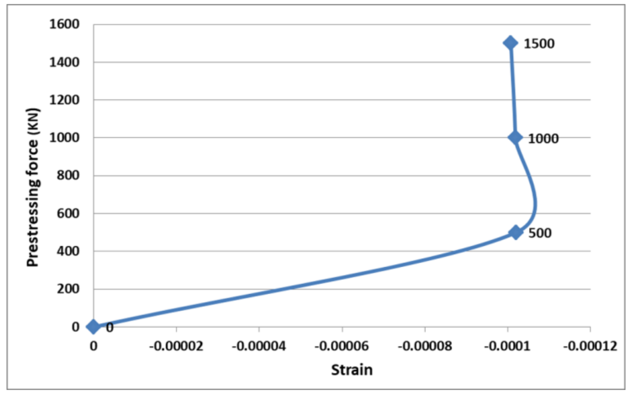


**Figure 20: Pre-stressing force versus the strain for the 30 m span.**

### 3.5. Results of First Case (Tensioning) in Composite Plate girder (Span 40 m)

In this case, the section is loaded in the same condition as the initial load, with the plate girders extended to a distance of 40 meters. In this load condition, the un-shored composite pre-stressed plate girder is referred to as a "steel plate girder" since it works only as a steel section when the concrete slab element fails to function and does not withstand our operation.

Figure 21 shows the relationship between the vertical deflection of the plate girder and the change in value of pre-stressing force (500-700-1000-1500) kN in the middle span of the top slab. The results demonstrate that when the pre-stressing force steadily increased to a maximum of 1500 kN the deflection at the lowest prestressing force of 500 kN normally stabilizes.

However, it is found that when the section is loaded with a P.s of 500 kN, the value of the deflection that happens to the plate girder at the bottom steel flange continuously increases, reaching its maximum (P.S 1500 kN). This appears logical because the steel section withstands the stress that is imparted to it by loading the straight tendons with the values of the varied prestressing forces shown in Figure 22 however the concrete slab does not work or resist under these loading conditions.

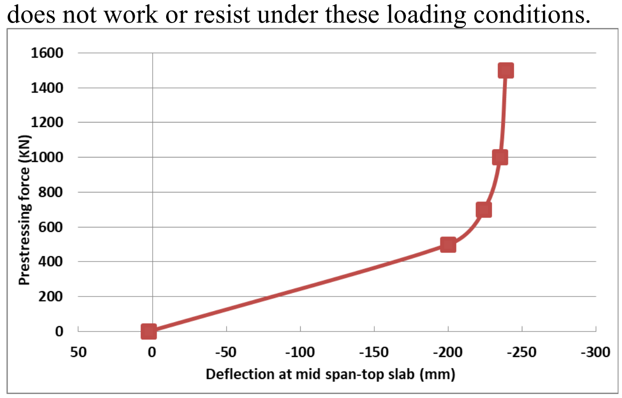


**Figure 21: Vertical deflection at mid span of top slab versus prestressing force curve.**

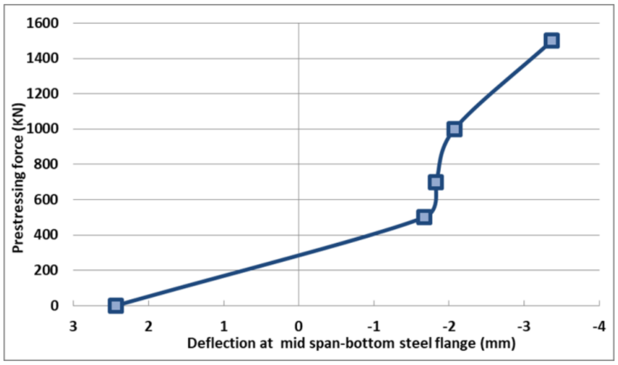


**Figure 22: Vertical deflection at mid span of bottom steel flange versus prestressing force curve.**

By comparing the results of the deflection happening in the top slab and bottom steel flange shown in Figure 23, the value of the deflection increases by nearly 98% in the slab when exposed to the lowest prestressing force 500 kN, While the value of the settlement deflection grows at the same rate as the slab compared to the steel flange when exposed to the highest prestressing force 1500 kN, Figure 24 shows the deformed shape at the first case (during tensioning) in a 40-meter span.

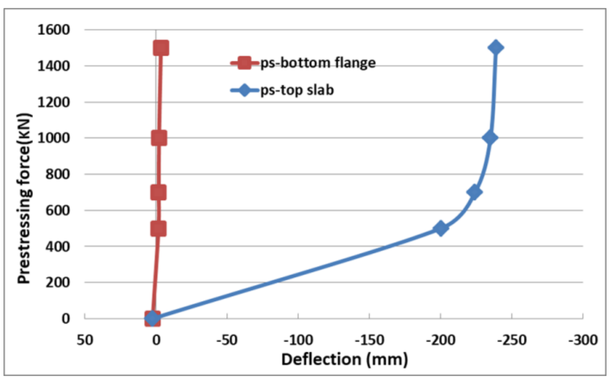


**Figure 23: Deflection versus pre-stressing force at top slab and bottom steel flange.**

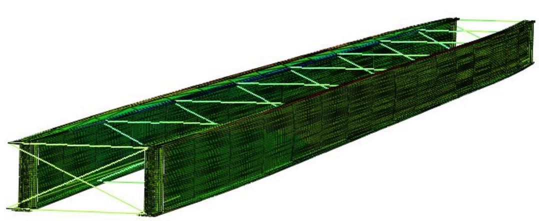

**Figure 24: Deformed shape at first case (during tensioning) in span 40 m.**

### 3.6. Results of Second Case (After Tensioning) in Composite Plate girder (Span 40 m)

Under this load condition, the section serves as a composite section where the concrete slab and the steel plate girder section resist, extending the plate girder's span to 40 meters. The section is loaded with fixed values, which are represented by a single weight of the composite section and the pre-stressing force value on the straight tendons to be loaded, as well as variable values, which are represented by the values of the section's external load (Applied load). Figure 25 shows the relation between variations in the deflection values at the top slab and every change in the external load. The straight tendons were loaded with P.s 500 kN and also try a prestressing force 1000 kN added to the weight of the structure. The external load values were adjusted to (500-1000-1500) kN, and for each applied force modification, the value of the deflection that happened in the middle of the top slab was evaluated, revealing a convergence in the values of the change in the reduction that corresponded to each variation in the external applied load. This study also discovered an approximate convergence in the outcomes with the relationship between deflection and applied load at bottom steel flange, as shown in Figure 26.

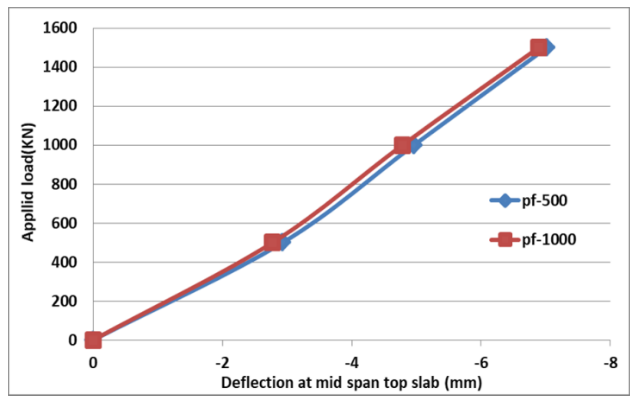


**Figure 25: Deflection versus applied load at mid span of top slab (span 40m).**

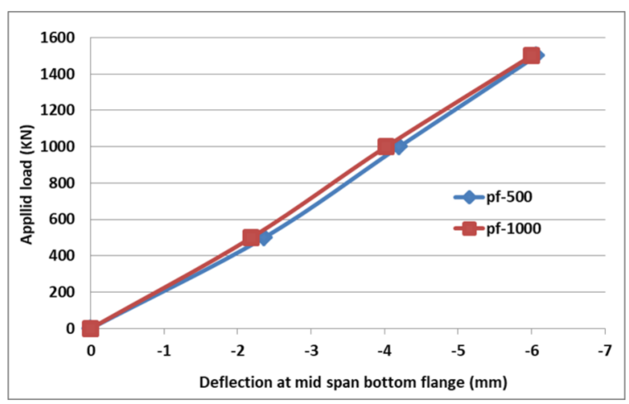


**Figure 26: Deflection versus applied load at mid span of bottom steel flange (span 40m).**

Figure 27 investigates the value of the deflection corresponding to the maximum external applied load (1500 kN) is approximately 14% higher in the top slab than in the bottom flange at both prestressing forces of 500 kN and 1000 kN.

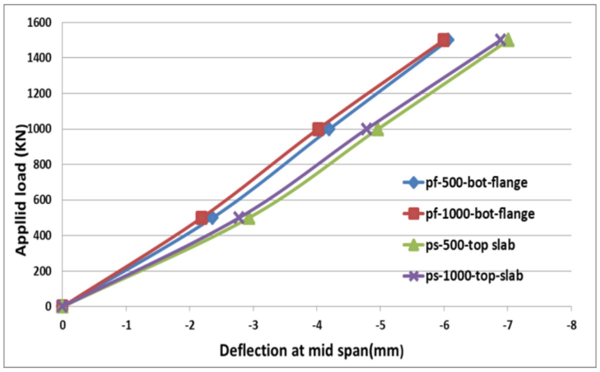


**Figure 27: Deflection versus applied load at top slab and bottom steel flange in span 40 m.**

### 3.7. Comparison between (span 30 & span 40) in first case (During tensioning)

The longer the span of the plate girder, the greater the value of the deflection at the top slab, as the value of the deflection at a distance span 40 meters increases by roughly 48% over the distance span 30 meters when loading with the same value of the prestressing forces for both of them. This is found by comparing the results of the first loading case with the change in the beam distances for each of 30 and 40 meters. In addition to taking the values of the change in the deflection corresponding to the change in the different prestressing forces at the slab and the steel flange, but at the bottom steel flange, the deflection values are steady at the commencement of loading at both distances of 30 and 40 meters. However, they rise to achieve an increase of around 54% at a distance span of 30 m over a distance span of 40 m at the same prestressing force, as shown in Figure 28.

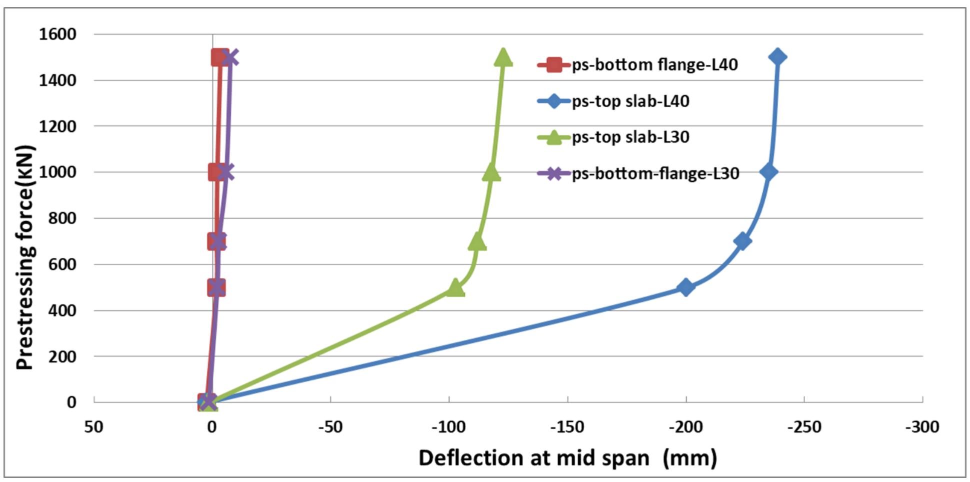


**Figure 28: Deflection (first case) versus prestressing force in top slab and bottom flange in (span 30 & span 40).**

### 3.8. Comparison (span 30 & span 40) in second case (After tensioning)

The deformed shape of the section in the second load case can be shown in figures 29 and 30.The deflection caused by loading the composite plate girder with an external load while keeping the prestressing force value constant at 500 kN and 1000 kN was compared between the two examples of the beam at a distance of 30 and 40 meters. The comparison was made at the middle of the slab and appears in Figure 31. It is found that the deflection resulting from loading the section with an external load increases with the span of the plate girder. The plate girder at 30 meters has a deflection value that is approximately 10% greater than the deflection at 40 meters at the same value of PS force with the increase of the applied loads . Additionally, the subsidence value increases with the decrease in the plate girder's span at the same pre-stressing force value and external load.

When comparing the subsidence caused by loading the section with an external load between the two cases at a distance of 30 and 40 that are taken at the bottom of steel flange section and are shown in Figure 32.

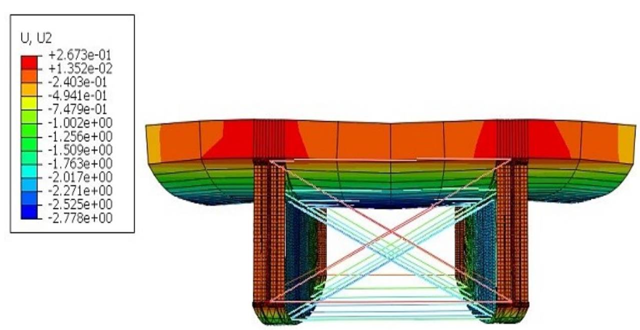


**Figure 29: Deformed shape (elevation) in the second case (span 40 m)**

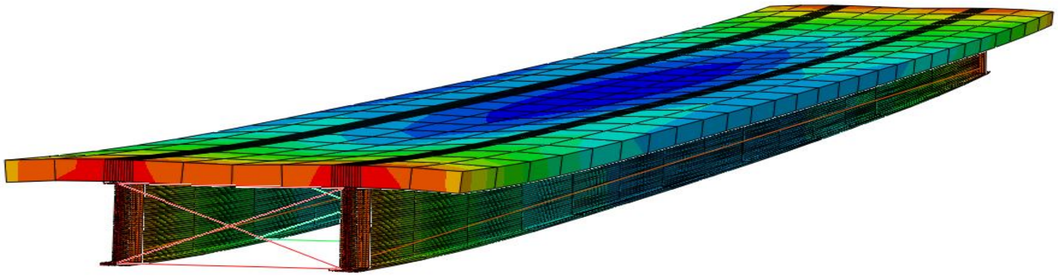

**Figure 30: Deformed shape (side view) in the second case (span 40 m).**

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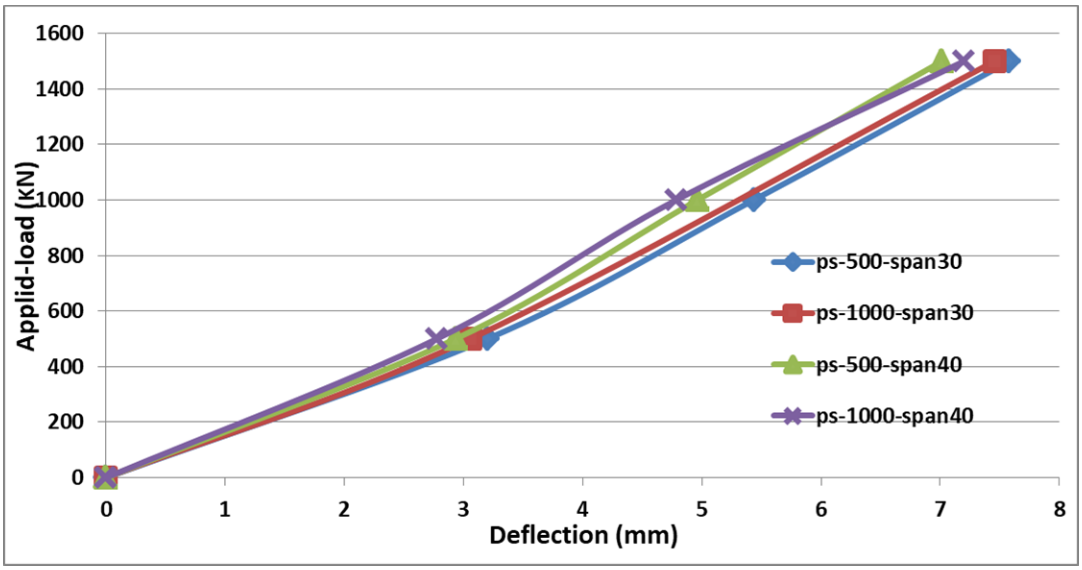


**Figure 31: Deflection versus applied load in top slab (span 30 & span 40) after tensioning.**

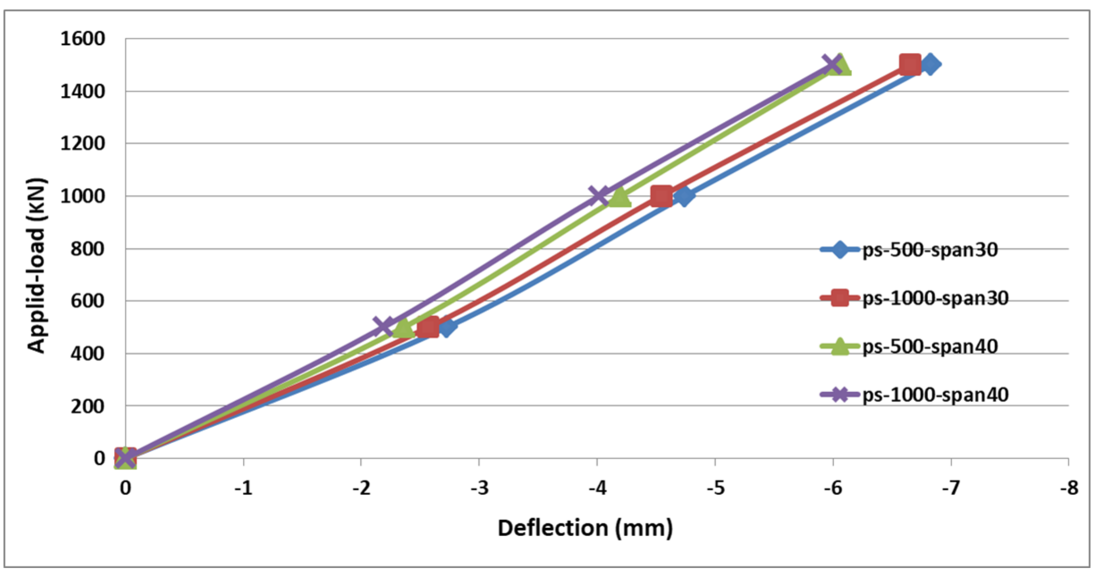


**Figure 32: Deflection versus applied load in bottom steel flange (span 30 & span 40) after tensioning.**

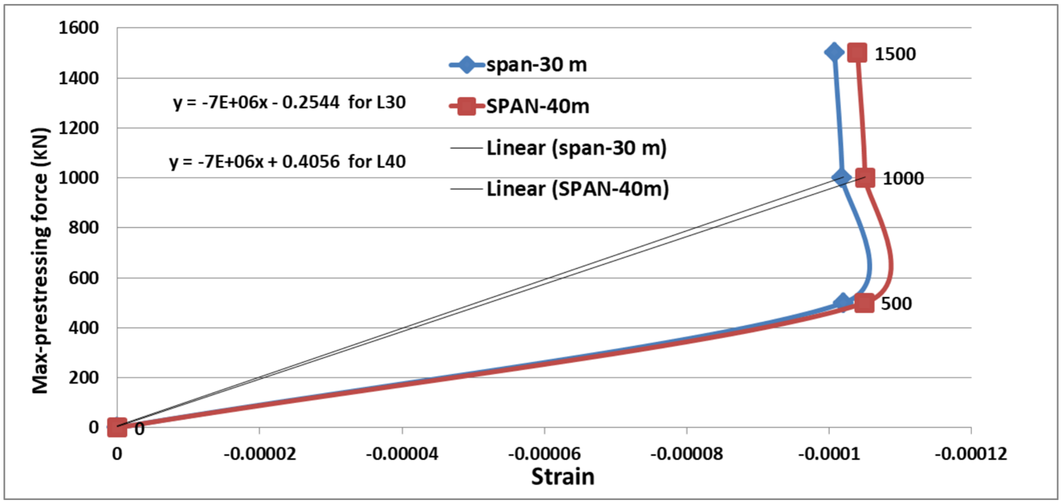


**Figure 33: Max pre-stressing force versus strain (span 30 &40 m).**

For max pre-stressing force and strain in (span 30, 40) m ,figure 33 shows the relation between the maximum pre-stressing force (500,100,1500) kN versus the corresponding strain to prove that the larger the plate girder span, the greater the strain value corresponding to the maximum prestressing force influencing the section. The strain corresponding to the highest prestressing force was compared at 30 and 40-metre section distances.

## 4. CONCLUSION

This research investigates the advantages of employing composite sections that are prestressed using tendons. The shape of the tendons, the location and form of the bracing, the change in the values of the prestressing force, the deflection, and the strain occur while applying them in two loading cases: tensioning and post-tensioning. Some parameters can be used to save on prestressing steel section by using new dimension composite one, increase the ultimate capacity and reduce the weight of structural steel, additionally researching the impact of a variation in plate girder span.

The following presents more detailed evaluation results about pre-stressed composite plate girder:

- The resistance of the section was increased by reducing the deflection of the plate girder using the pre-stressing steel system by 20% of its original value without pre-stressing, so that the deflection value of the plate girder when loaded with its weight and a value of 10 kN operating loads reached from 1.46 to 0.3 millimeters so the pre-stressed steel minimizing the weight of structural steel and raising its maximum capacity.
- The dead load is eliminated in first load case to be 0.45 mm then the effect of deflection that occurs to the plate girder after pouring the concrete in the second stage is reduced to be close to zero by loading the section in the first loading with a prestressing force equal to the deflection that will occur to the section.
- According to the results obtained at the top of the slab, the length of the plate girder determines how much the loading-induced deflection will be. By loading the section with the greatest span on the straight tendons with varying prestressing forces and examining the associated variable deflection values, this equation is obtained.
- As plate girder span increases, deflection values at the bottom of steel flange sections decrease after tensioning.
- The resistance and capacity of the beam section with a span 30 m is higher than the resistance of the beam section with a span 40 m when analyzing the results at the top of the slab in the first loading instance.
- When loading with both prestressing forces of 500 kN and 1000 kN in addition to the section's self-weight with a change in the external load force and examining the deflection values resulting from each load change at the top of the slab and the bottom of the steel flange, the plate girder section with a span of 40 has a higher resistance than the plate girder section with a shorter length.
- The use of prestressed steel tendons resulted in a cost effective composite section with high capacity. This portion is more cost effective than the usual one using non pre-stressed steel tendons.
- The designed model can be used in the creation of bridges that have long spans as well as the structure buildings.
- Recommended study the effect of reducing the height of the plate girder designed in current study by about 20% of its original height and study the possibility of the section in deflection to be less than its original state before the reduction.

- Recommended study the effect of applying the pre-stressing techniques on continues composite plate girder and the effect of using dapped end beam and using over hanging beam.

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