Comprehensive Wind Analysis Of Stress Ribbon Bridge

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Abstract— A stressed ribbon bridge (also known as stress-ribbon bridge or catenary bridge) is primarily a structure under tension. The tension cables form the part of the deck which follows an inverted catenary between supports. The ribbon is stressed such that it is in compression, thereby increasing the rigidity of the structure where as a suspension spans tend to sway and bounce. Such bridges are typically made RCC structures with tension cables to support them. Such bridges are generally not designed for vehicular traffic but where it is essential, additional rigidity is essential to avoid the failure of the structure in bending. A stress ribbon bridge of 45 meter span is modelled and analyzed using ANSYS version 12. For simplicity in importing civil materials and civil cross sections, Civil FEM version 12 add-on of ANSYS was used. A 3D model of the whole structure was developed and analyzed and according to the analysis results, the design was performed manually. A comprehensive wind analysis was performed under IRC 6 guidelines for the bridge for spans 45m, 50m, 55m and 60m. Further, depth optimization was performed 45m span for the wind zone located in Mumbai (Zone III)

Keywords—

Introduction

Pedestrian bridges (or footbridges as they are called in many parts of the world) have been used by man since antiquity to cross rivers, deep gorges and narrowmountain passes. One commonly used method has been to suspend a walkway from a fibre rope catenary span somewhat similar to a primitive suspension bridge. The stress ribbon concept borrows the suspension bridge principle but develops it further by using high strength materials and modern engineering technology especially precasting and prestressing methods.

Stress-ribbon is the term that has been coined to describe structures formed by directly walked prestressed concrete deck with the shape of a catenary. The bearing structure consists of slightly sagging tensioned cables, bedded in a concrete slab that is very thin compared with the span. This slab serves as a deck, but apart from the distributing the load locally and preserving the continuity, it has no other function. It is a kind of suspension structure where the cables are tensioned so tightly that the traffic can be placed directly on the concrete slab embedding the cables. Compared with other structural types the structure is extremely simple. On the other hand, the force in the cables is very large making the anchoring of the cable expensive. Fig 1 shows the typical stress ribbon bridge.

Figure1: Typical Stress Ribbon Bridge

It is seen that stress ribbon bridge is used in European countries for people to walk which reduces the traffic intensity on crowded roads. So in developing countries like India stress ribbon bridge can be a solution to traffic reduction in crowded area like Mumbai, Delhi, Bangalore and many more.

Load transfer mechanism of stress ribbon bridge

A typical stress ribbon bridge deck consists of precast concrete planks with bearing tendons to support them during construction and separate prestressing tendons which are tensioned to create the final designed geometicform. The joints between the planks are most often sealed with in-situ concrete before stressing the deck. The prestressing tendons transfer horizontal forces in to the abutments and then to the ground most often using ground anchors. The tendons are encased in ducts which are generally grouted after tensioning in order to lock in the stress and protect them from corrosion. Since the bending in the deck is low, the depth can be minimized and results in reduction in dead load and horizontal forces in abutments. The abutments are designed to transfer the horizontal forces from the deck cables into the ground via ground anchors. Pedestrians, wind and temperature loads can cause large changes in the bending moments in the deck close to the abutments and accordingly crack width sand fatigue in reinforcement must be considered. The soil pressure, over turning and sliding has to be checked for construction as well as permanent condition.

The ideal ground condition for resisting large horizontal forces from the ribbon is a rock base. This occurs rarely but suitable foundations can be devised even if competent soils are only found at so me depth below the abutments. In some case where soil conditions do not permit the use of anchors, piles can also be used. Horizontal deformations can be significant and are considered in the design. It is also possible to use a combination of anchors and drilled shafts. Battered micropilingis another alternative which can resist the load from the ribbon because of its compression and tension capacity.

Literature Survey

In a prestressed concrete traditional stress ribbon bridge high strength steel cables are passed through a series of precast concrete components, the deck assembly of which can be tensioned from stiff abutments. Whereas in a suspension bridge the main load carrying component is the cable with the deck acting as a stiffening element, in a stress ribbon bridge both the cable and the deck can be independently tensioned, thus adding considerable rigidity to the structure.

Lacey et al. (1975) explained the economic advantages of precasting can be combined with the structural efficiency of prestressed concrete box girders for long span bridge structures when erected by segmental construction. The complete superstructure is precast in box segments of convenient size for transportation and erection. These precast segments are erected in cantilever and post-tensioned together to form the complete superstructure.

Strasky (1987) explained that the stress ribbon concept borrows the suspension bridge principle but develops it further by using high strength materials and modern engineering technology, especially precasting and prestressing methods.

Nakazawa (1988) carried out an experiment based dynamic behavior of the cable suspension bridge. This experiment explained that the interest resonance curve such as the shape of hardening type for the first natural frequency and the shapes of softening type for second and third natural frequencies respectively.

Aso et al. (1990) conducted the dynamic analyses of three span continues stress ribbon bridge was clarified by exciting test and eigen value analysis. Results of three dimensional analyses are cross agreement to results of exciting test. The natural frequencies of this bridge up to 10th mode are below 3.0 Hz and are very close to each other due to low flexibility of ribbon. Some modes clearly indicate that tensional vibration and out-of-plane vibration are coupled. These modes are must be considered in aeronautical stability.

Kulhavy (1998)carried out research on the development of hybrid stress-ribbon pedestrian bridges. In this research the classic stress ribbon deck is combined with arches or cables and large horizontal forces that is created near the abutments is eliminated with the help of a slender arch.

Stoyanoff et al. (2000) explained that the lowest natural frequencies of short span footbridges (e.g., less than 30 m) are usually sufficiently high that they are not susceptible to human-induced vibrations. As spans increase (spans up to 200 m have been proposed), their lowest natural frequencies become lower and human-induced vibrations become a concern. Therefore, it is important that this phenomenon is well understood and that reliable theoretical methods are introduced to determine practical solutions.

Arco et al. (2001) presented the structural behavior of prestressed concrete stress ribbon bridge emphasizing the geometrical non linear character based on the preliminary design, the final design can be worked out addressing other loading conditions like pedestrian-induced movements.

Tanaka et al**.** (2002) explained the aerodynamic stability together with the static characteristics of the proposals for the stress-ribbon cable-stayed suspension and stress-ribbon suspension bridges for pedestrian use were examined in the wind tunnel and by numerical analysis.

Newland (2003) summarised his findings as the growth process of the lateral vibration of the girder under the congested pedestrians can be explained as follows. First a small lateral motion is induced by the random lateral human walking forces, and walking of some pedestrians is synchronised to the girder motion. Then resonant force acts on the girder, consequently the girder motion is increased. Walking of more pedestrians are synchronised, increasing the lateral girder motion. In this sense, this vibration was a self-excited nature. Of course, because of adaptive nature of human being, the girder amplitude will not go to infinity and will reach a steady state.

Newland (2004) theorised that the adoption of a non-dimensional number which measures the susceptibility of a bridge to pedestrian excitation. Although currently there are not many good bridge response data, predictions using this non-dimensional number are compared with the data that are available and found to be in satisfactory agreement. Both lateral and vertical vibrations are considered.

Low and Burnton (2004) carried out the studies for the retrofit of the London Millennium Bridge have provided a model of SLE(Synchronous Lateral Excitation) which allows footbridge designs to be checked for SLE phenomenon.

Caetano and Cunha (2004) study focused on the analysis of the dynamic behaviour of a stress-ribbon footbridge, using both experimental and numerical tools. The investigation has demonstrated the complex behaviour of a simple footbridge, and the important role of experimental testing in the characterisation of the corresponding structural behaviour and in the tuning of a finite-element model. On the other hand, the significant variability of the structural response for frequencies of excitation close to natural frequencies and the difficulty in inducing resonance in the prototype, show the advantages of using numerical methods to complement field measurements, in order to extrapolate the most extreme structural response.

Kalafaticet al. (2006)carried out the experimental work todescribe the preliminary static design procedure for one span post-tensioned stress ribbon bridge taking into account real length of the structure and different heights of abutments.

Strasky (2006) elaborates the structural arrangement and static function of a traditional stress ribbon bridge during construction and service stages. The article explains that the super structure of the pedestrian bridge is formed by a prestressed and which is attached to rigid end abutments.

Strasky (2008) describes in detail, primarily about stress ribbon bridges supported or suspended on arches. The study describes about the structural arrangement and the special self-anchoring quality..

Stavridis (2009) proposed an analytical method for the evaluation of static response of a prestressed concrete ribbon pedestrian bridge, the static response of a prestressed-ribbon bridge under live load was obtained using only two dimensionsless design parameters, i.e. ratio of thickness over span length and ratio of prestressing steel area over concrete section area.

Poon et al. (2009) Span-to-depth ratio is an important bridge design parameter that affects structural behaviour, construction costs and aesthetics. A study of 86 constant-depth girders indicates that conventional ratios have not changed significantly since 1958. These conventional ratios are now questionable, because recently developed high-strength concrete has enhanced mechanical properties that allow for slenderer sections.

Strasky (2010) the only unique fact about the research is the mention of Curved Stress Ribbon and Flat Arch Bridges but is not elaborated as it is still in development stage. The author mentions that the curved structures have to be designed in such a way that for the dead load there is no torsion in the deck.

Sandovič and Juozapaitis (2012) the article dwells on a new structural solution for pedestrian steel suspension bridges. This new structural system ofpedestrian stress-ribbon bridges includes suspension members with bending stiffness and a pre-stressed tie.

Wang et al**.** (2012) performed an analysis of the dynamic character, wind-resistant stability and flutter instability are analyzed, especially the analysis of the flutter stability at maximum double cantilever stage during construction phase. This paper analyzes the dynamic characters of the FE model of the whole bridge through response spectrum method to reap the natural frequency, the vibration modes and other parameters, and then checks the flutter stability. The results show that the bridge is very safe. The analysis method provides bases and references for the wind-resistant of Lanqi Song Hua River Bridge.

Cacho-Perez et al. (2013) focused on a stress-ribbon footbridge, the analytic response for the static and modal problem corresponding to a simple suspended steel plate is studied. The typical mechanical response under its own weight (catenary) is modified to take into account the elongation and thermal effects. The equilibrium equations describing the problem are ordinary differential equations that require a pair of coupled nonlinear equations to calculate the value of the stress at one end and then the rest of the parameters. The understanding of the static response is very important not only for the determination of the deformed equilibrium configuration but also for its influence on both the modal analysis and the dynamic response of the suspended cable. Most of the analytical effort is paid for the vertical response of the equivalent suspended cable, that is believed to match, once updated, with the vertical response of the steel-plated stress-ribbon footbridge under study (80m single span Pedro Gomez Bosque footbridge in Valladolid, crossing the Pisuerga River).

Mukherjee et al**.** (2013) The criteria used to judge the acceptability of the wind load and the corresponding structural responses along with the serviceability considerations are also presented. Then based on the given methods the wind forces acting on a continuous bridge whose main span is larger than the 50 meters (i.e. > 50 meter requires dynamic assessment) is studied and compared with the results which could be obtained from the simplified methods recommended in the Euro Code 1.

Findings of Literature Review

The literature focuses primarily on static analysis of stress ribbon bridges for span upto40m. The dynamic behavior of stress ribbon bridge is still scare in the literature. The present work is dealing primarily with a simplified explanation of static analysis with the dynamic aspect of such bridges and the effect of pedestrian excitation has on such slender decks. Multiple spans are taken in order to get clarity under the static and dynamic conditions for various spans.

Problem Definition

For the Static and Dynamic Analysis of Traditional Stress Ribbon Bridge following parameters are considered;
(Note: Winds zones of India are considered considered are as per IRC 6.)
Span = 45m, 50m, 55m and 60m (Wind Zone III), 45m and 60m (All Wind Zones)
Sag (dip) of Bridge deck (*f*) = 0.02*L
Width of footway = 4 m wide (End to end)
Effective width= 3.7m
Deck thickness = 200mm, 225mm and 250 mm (45m and 60m)

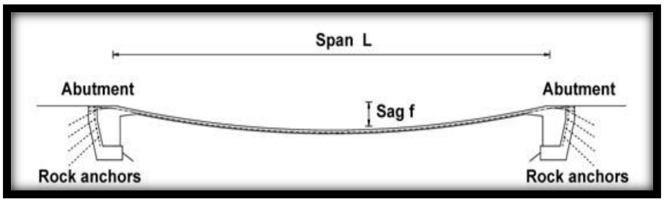

Figure2: Structural arrangement of a Stress Ribbon Bridge

Figure 2 shows the general arrangement of a traditional stress ribbon bridge. Stress ribbon bridges have an advantage that they can be constructed for higher spans. It is this advantage combined with a slender deck that helps the stress ribbon bridge to be a viable option for pedestrian bridges. The deck slab is discreatized into '*l*' no of two nodded rectangular elements. Every element is considered as beam interconnected at an interval of 1m. Here '*l*' is length of span in meters. Size of rectangular element that is modeled in Ansys CIVIL-FEM is of 3.7m x 1m as shown in the Figure 3.

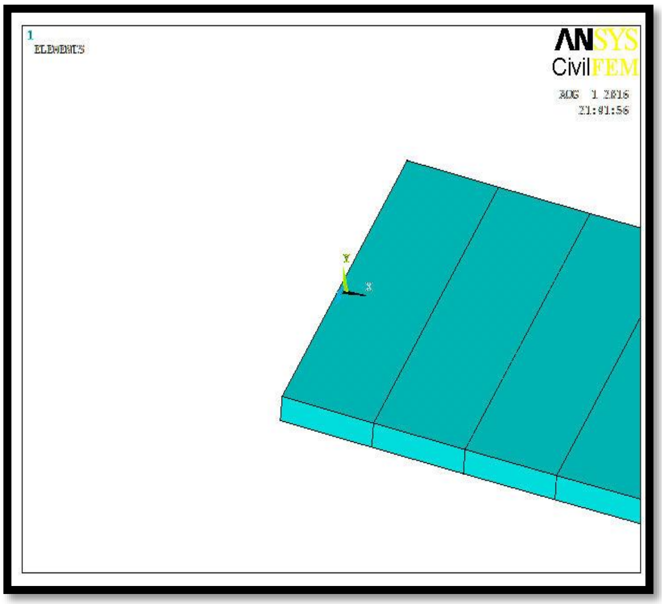

Figure3: Rectangular element in Ansys-Civil FEM

Aims and Objectives

The aims and objective of the work are as follows.

1. To develop a simplified model for a traditional stress ribbon bridge using ANSYS
2. To perform static analysis of traditional stress ribbon bridge for IRC loading conditions.
3. To obtain the permissible pedestrian excitations of the stress ribbon bridge and its mode shapes.
4. To Analyse and check the structure for safety against pedestrian excitation of bridges as per BS 5400.
5. To perform the wind analysis of the stress ribbon bridge of particular spans and to optimize the depth of these spans as per IRC.
6. To decide a suitable abutment for the stress ribbon bridge to transmit the high cable forces to the foundation below.

Scope of Work

Stress ribbon bridges can prove to be an innovative, aesthetic and economical alternative to traditional pedestrian bridges. In the present project, a traditional stress ribbon bridge for a span of 45m, 50m, 55m and 60mis analysed which includes.

1. Analysis of traditional stress ribbon bridge for various spans.
2. Study of effect of variation of depth in stress ribbon bridge for a particular span.
3. To improve the foundation design for the bridge that suits different soil conditions.
4. Analyse the abutment which will support different size of slender deck.
5. Wind tunnel analysis of the same stress ribbon bridge can be performed.

Methodology

Various steps involved in modeling the deck structure, loading it and then analyzing the final structure in the software package ANSYS. An add-on for ANSYS named Civil-FEM was used to import civil material properties and common civil cross sections. Civil-FEM also simplifies the process of selection of commonly used civil elements and association the cross section properties with the element selected. Civil-FEM is also used to simplify the process of view the bending moment and shear force diagrams for the structure.

This topic will describe the various procedures involved in modeling the structure, the selection of element, material selection, cross-section editor, merging the section properties with the element selected and finally creating the structure ready for loading.

Stress ribbon bridges are very slender in nature and have a natural frequency very close to the pedestrian walking frequency of 2 Hz. The synchronous excitation of The Millennium Bridge in 2001 which led to additional cost of £5 million in order to install fluid viscous dampers and tuned mass dampers in order to control the vibrations of the bridge is the best example as to how pedestrian excitation can cause resonance. The codes of choice for structural engineers around the world for pedestrian excitation analysis are The British Standard BS 5400 Part 2 and Ontario Highway Bridge Design Code OHBDCONT 83. Both these codes contain the same loading model which is the single pedestrian load model. For this report, we will be using BS 5400 Part 2 model for pedestrian excitation analysis which has been improved in *fib 32*.

Analysis

The traditional stress ribbon bridge is analyzed in static condition for different loading conditions also for pedestrian excitation. The analysis is carryout using software Ansys CIVIL-FEM. Loading combinations considered for designs are given below:

Types of Loads

1. Dead Load

The dead load taken for the bridge was 26 kN/m which is calculated as

D.L= (Thickness of the deck slab× width of the deck slab× Density of concrete) + 1

The above load includes the cable dead load the load of the railings and the prestressing steel dead load.

2. Live Load

The live load calculations are computed using IRC 6:2010 Clause 206.3. The equation for the live load calculation for effective spans above 30 meters is given below:

$$P = \left(P^1 - 260 + \frac{4800}{L}\right)\left(\frac{16.5 - W}{15}\right)$$

3. Lateral Wind Load

The wind loads are calculated as per IRC 6:2010 clause 209.3.5 for transverse wind loads respectively.

4. Transverse Wind Load

The wind loads are calculated as per IRC 6:2010 clause 209.3.3 for transverse wind loads respectively.

5. Temperature Load (±)

Temperature loads in the structure are calculated as per IRC:6:2010 clause 215.2. The maximum and minimum temperatures in Mumbai are 40ºC and 10ºC respectively. Hence the structure will be analyzed for a mean temperature of (25º ± 10º) C whichever is critical.

6. Prestressed load

The prestressing force is being applied in such a way that the equivalent up thrust generated is 20kN/m.

Loading Combination

1. Normal combination

The normal combination consists of D.L.+L.L only. There is no increase of permissible stresses allowed in this combination.

2. Temperature combination

The temperature combination consists of D.L.+L.L.+Temp(+). The positive value of temperature is taken since it is critical. The IRC 6:2010 also allows an increase in permissible stresses by 15%.

3. Wind and Temperature combination

The wind and temperature combination consists of D.L.+L.L+Temp(+)+Wind. The vertical value of wind is taken since it is critical. The IRC 6:2010 also allows an increase in permissible stresses by 33% in this combination.

4. Dead load and Prestress combination

Stress is also a critical condition during prestressing. IRC 18:2000 mentions that the temporary stresses in compression and tension are 50% of the compression stress at the day of prestressing subject to a maximum of 20 N/mm^{2}and 10% of the permissible compressive stress in concrete respectively.

5. Dead load, live load, Prestress and temperature results

In this condition, the Prestress force is applied to the temperature combination which is the most critical combination for the bridge amongst the three critical combinations.

In dynamic analysis the pedestrian excitation is considered. There are three primary mode shapes for the deck, namely, vertical, lateral and torsion modes. The first three modes are vertical. The first lateral mode is the fourth mode and the first torsion mode is the seventh mode. The first two vertical modes have a frequencies f_1=1.379 Hz and f_2=1.788 Hz respectively. Normal practice suggests that that the second mode shape $f_2$should actually be the first mode shape but, this mode requires an elongation of the cable, hence the corresponding frequency in some cases is higher and mode shape of f_1 becomes the first mode.

Results and Discussions

General

A45m, 50m, 55m and 60m span length of slender deck slab having dept of 250mm was analyzed using Ansys CIVIL-FEM. In static analysis the deflection, bending moment and stress is compared and the same deck is analyzed in different wind zones of India. Similarly in dynamic analysis the pedestrian excitation of the bridges is compared. The results are obtained in the form deflection, bending moments, stresses for various loading combinations by using various parameters. Lastly suitable abutment is also figured out in particular type of soil.

Wind Analysis

The span of 45m and 60m was analysed for the all the wind zones of India and its deflection results are plotted in the graphs shown below in Figure 4 and Figure 5. It is observed that as the wind speed increases the deflection the bridge spans increases. The highest deflection is obtained in the mid span for all bridges which is shown below.

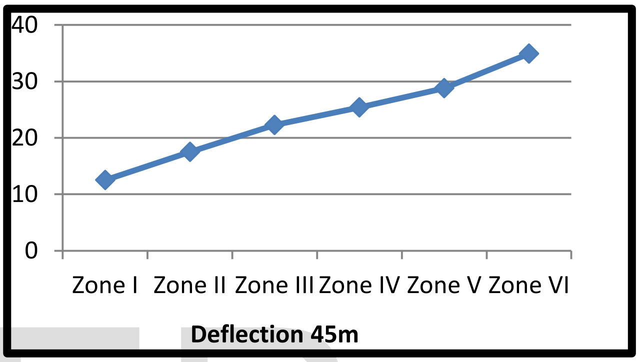

Figure4:Deflection of 45m in all the zones of India

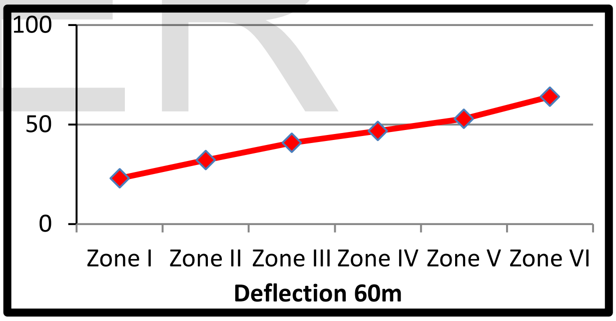

Figure5: Deflection of 60m in all the zones of India

Typical Results under Static Condition

1. Normal combination

Figure 6 and Figure 7 shows the bending moment and deflection of a 45m span deck in normal combination.

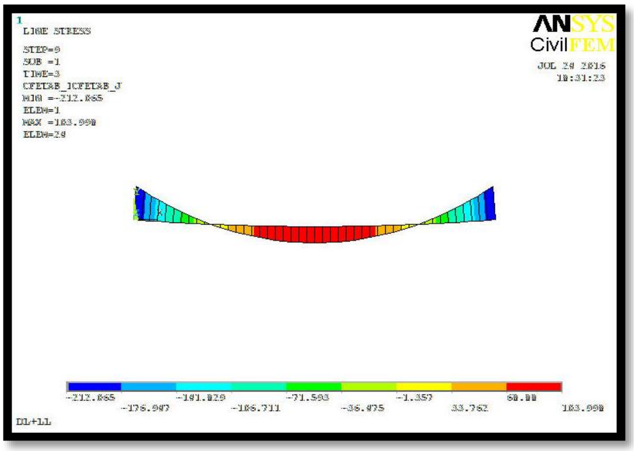

Figure6:Deflection in Normal Load Combination

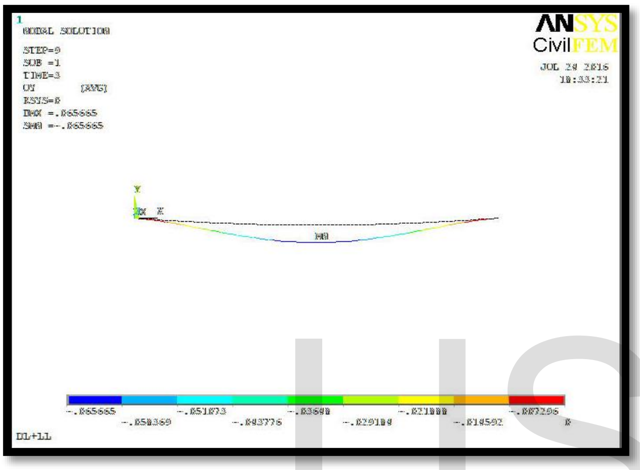

Figure7:Deflection in Normal Load Combination

Figure 8shows the actual deflection for normal combination under static condition. It is observed that deflection goes on increasing as the span increases. Figure 9 shows the sagging and hogging bending moment for normal combination which is nearly constant.

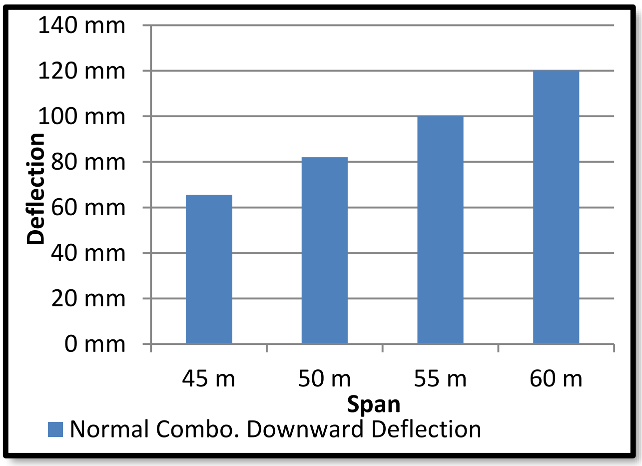

Figure8:Deflection in Normal Load Combination

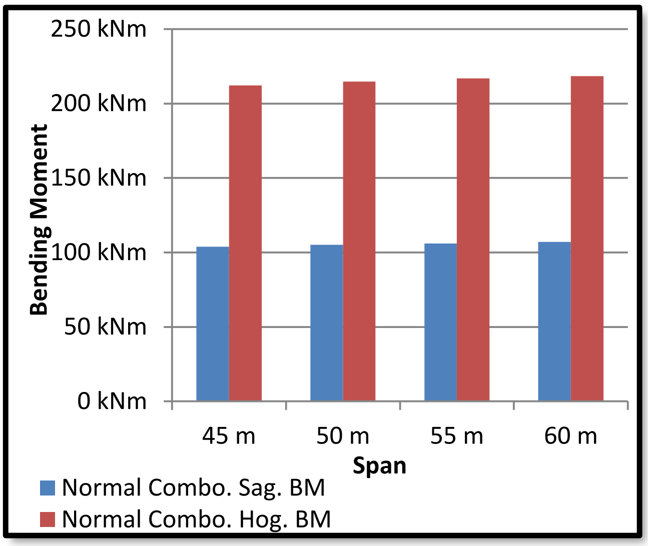

Figure9:Bending Moment in Normal Load Combination

2. Temperature combination

From Table1it is observed that deflection in higher span is more than the permissible limits calculated by using $\delta \leq \frac{l}{360}$where, δ = Permissible deflection and l= length of the span however prestressing is not provided in this combination.

Table 1: Results of temperature combinations

Length of bridge	Bending Moment (kN-m)		Actual Deflection (mm)	Permissible Deflection (mm)
	Sagging	Hogging		
45m	183.13	370.06	114.93	125
50m	177.26	358.87	137.4	138.89
55m	172.55	349.13	196	145.58
60m	168.1	340.6	187.2	171.25

3.Wind and Temperature combination (Wind Zone III)

Figure 10 shows deflection of stress ribbon bridge under wind and temperature combination. It is noticed that the deflection are about 40% more as compared to the deflections for normal combination. From Figure 11 it is observed that the hogging bending moment goes on decreasing where as sagging bending moment is nearly constant. It is worth to note that the increase in bending moments as compared to the normal conditions is about 52%.

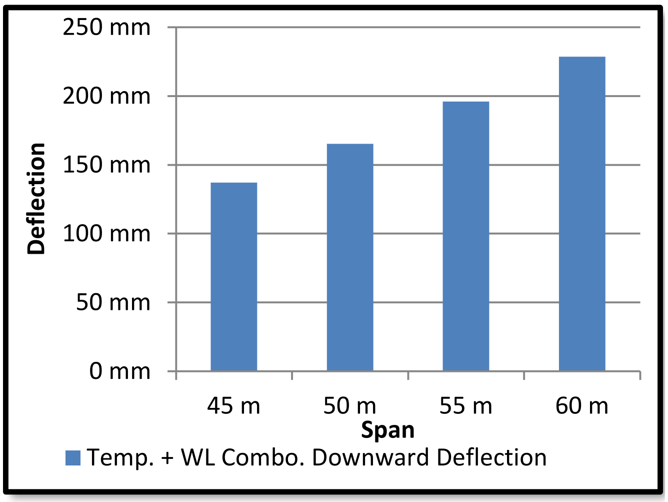

Figure10:Deflection in Wind and Temperature Load combination

Table 2: Results of Dead load and Prestress combinations

Length of bridge	Bending Moment (KNm)		Actual Deflection (mm)	Stress(N/m m^2)	Permissible Stress(N/m m^2)
	Sag	Hog			
45m	387.2	191.5	120.2(upward)	4.30 (T) 14.28 (C)	5.42 (T) 20 (C)
50m	376.28	185.79	144.1(upward)	3.3 (T) 14.73 (C)	
55m	366.7	181.17	170(upward)	2.4 (T) 15.19 (C)	
60m	358.3	176.8	197.6(upward)	1.53 (T) 15.6 (C)	

Note: T- Tensile, C- Compression

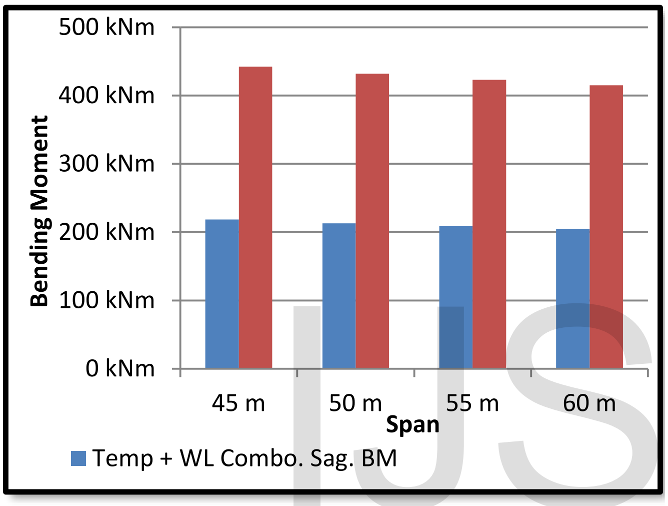

Figure11: Bending Moment in Wind and Temperature Load combination

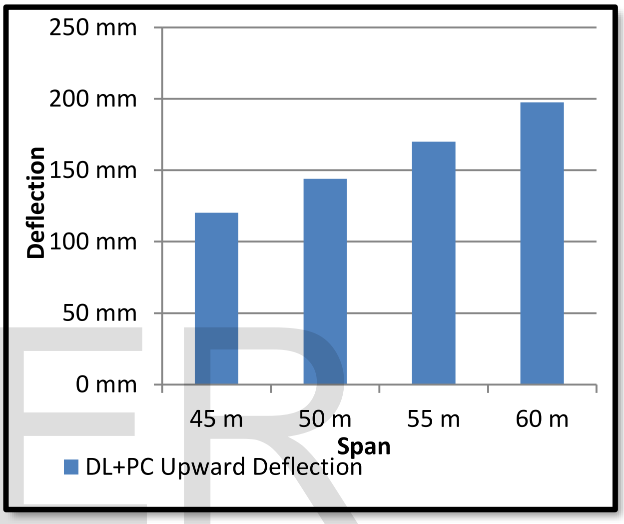

Figure12: Deflection in Dead load and Prestress Load Combination

4. Dead load and Prestress combination

Table 2 shows the moment, deflection and stresses obtained for dead load and prestress load combination. Similarly Figure 12 and Figure 13 shows the deflection and bending moment respectively obtained for dead load and prestress combination. From Table 2 it is observed that the tensile stresses decreases and compressive stress increases with increase in span but values are within the permissible limits as per IRC:18:2010.

5. Dead load, live load, Prestress and Temperature results

From Table 3 it is observed that the tensile stresses decreases significantly from span 45m to 50m and for span 55m and 60m the tensile stresses almost negligible. Figure 14 and Figure 15 shows deflection and bending moment respectively for Dead load, live load, Prestress and Temperature combinations.

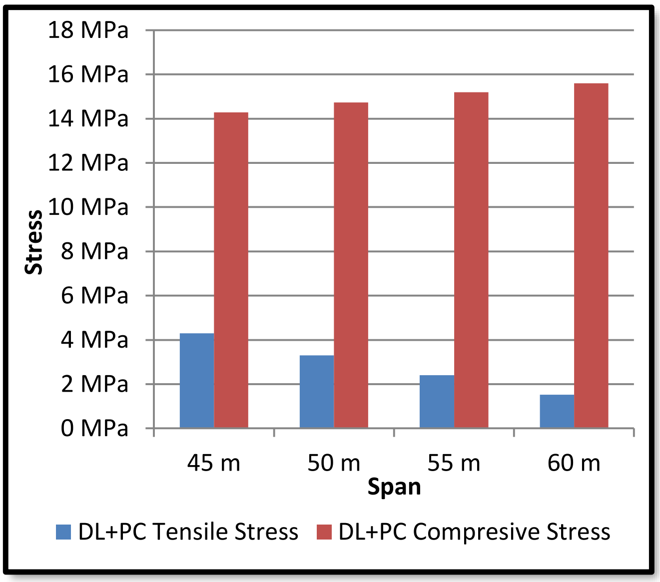

Figure13: Bending Moment in Dead load and Prestress Load Combination

Table 3: Results of Dead load, live load, Prestress and Temperature combinations

Length of bridge	Bending Moment (KNm)		Actual Deflection (mm)	Stress(N/mm^2)	Permissible Stress(N/m m^2)
	Sag	Hog			
45m	17.19	8.7	5.3	0.018 (T) 0.806 (C)	5.42(T) 20 (C)
50m	17.41	8.5	6.6	0.025 (T) 0.861(C)	
55m	17.58	8.62	8.1	0.914(C) 0.0706 (C)	
60m	17.71	8.68	9.7	0.966 (C) 0.11 (C)	

Note: T- Tensile, C- Compression

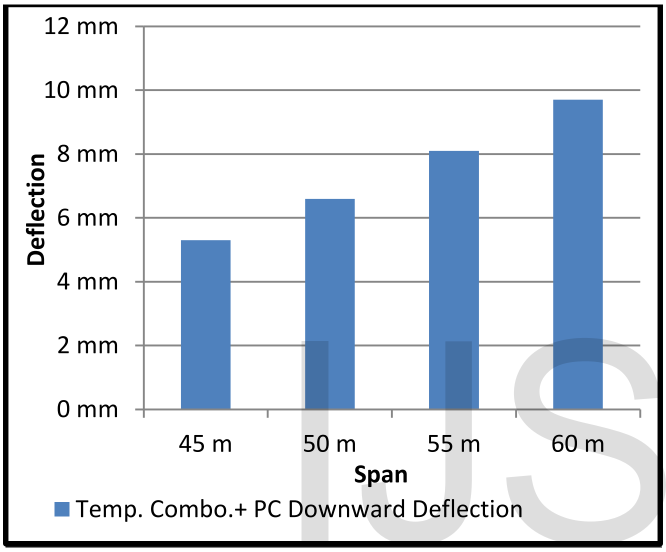

Figure14:Deflection in Dead load, live load, Prestress and Temperature Load Combination

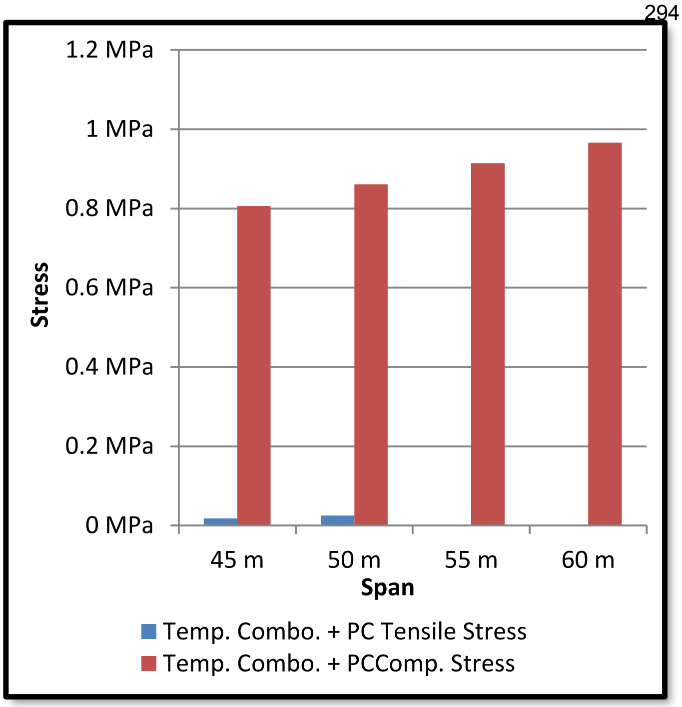

Figure15:Stress in Dead load, live load, Prestress and Temperature Load Combination

Figure 16, Figure 17 and Figure 18 shows the bending moment, stresses and deflection that is obtained of a 45m span bridge in the dead load, live load, Prestress load and temperature load combination. This combination is the critical one.

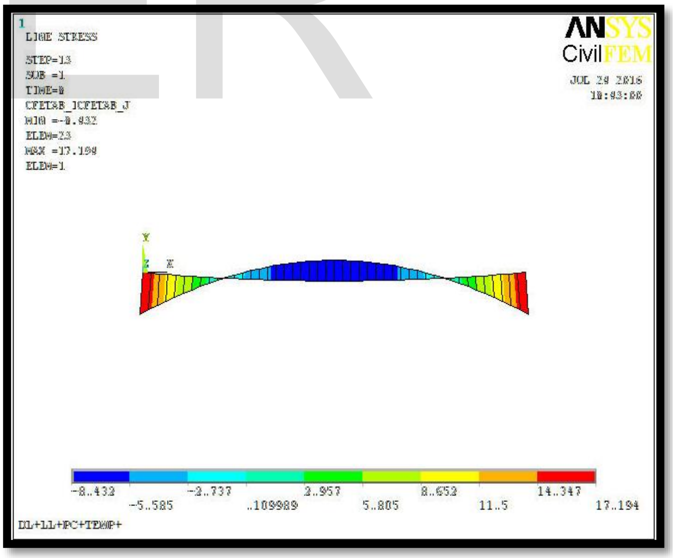

Figure16:Bending moment in Dead load, live load, Prestress and Temperature Load Combination

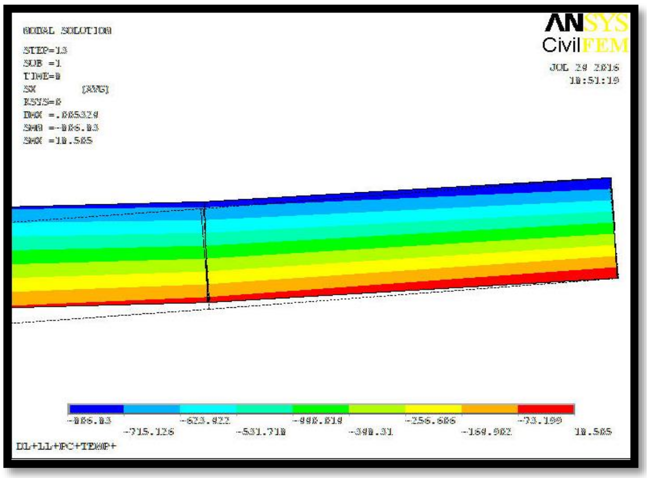

Figure17:Stress in Dead load, live load, Prestress and Temperature Load Combination

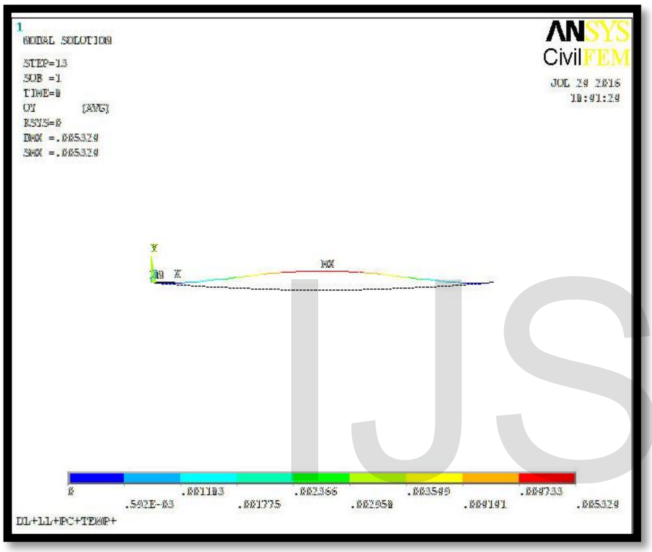

Figure18:Deflection in Dead load, live load, Prestress and Temperature Load Combination

Typical Results of Dynamic Analysis

Vertical acceleration resulting from the passage of one pedestrian walking / running is obtained in the present study by using eq.(1) (BS 5400).

$$a = 4 \times \pi^2 \times f^2 \times y \times \alpha \times \emptyset \quad \text{-------------------------} \quad (1)$$

where,

a= Acceleration in m/s^2,

f= Vertical natural frequency of the bridge in Hz,

y= Static deflection at mid span in m for a force of 700N,

α = Fourier coefficient of the relevant harmonic of the walking or running rate,

$\emptyset$ = Dynamic amplification factor for one pedestrian moving across simple span.

By using BS 5400 Part-II, one can obtain the maximum vertical acceleration resulting from the passage of one pedestrian walking /running with a pace rate equal to the fundamental natural frequency of the bridge. Therefore, it is modeled as an equivalent single degree-of-freedom oscillator.

Figure 19 shows the values of dynamic amplification factor for Number of cycles per span with various damping ratio. The values obtained from Figure 17 are used in eq.(2).

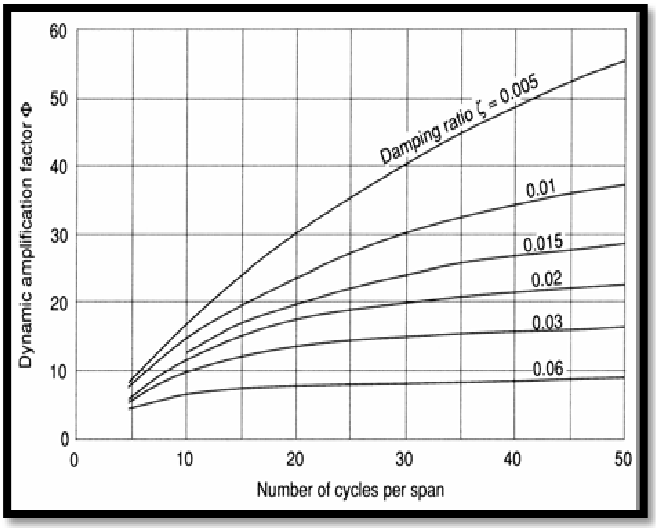

Figure 19: Dynamic amplification factor for different damping ratios (BS:5400)

For given conditions, the value of α would be 0.4 for a step frequency of 2 Hz for the first natural frequency.BS 5400 Part 2 also considers the higher order harmonics of the structure and hence it has a limiting value of frequencies to be considered till 5 Hz. However research has shown that higher order harmonics (frequency above 2.4 Hz) will not produce important oscillations in a structure due to the lower force component of higher harmonics. The permissible vertical acceleration is,

$$a_{v,max} = 0.5 \times \sqrt{f} \quad \text{--} \quad (2)$$

Where, f= Natural frequency of the structure.

The structure is analyzed for natural frequencies of which the first three are f_1=1.379 Hz, f_2=1.788 Hz and f_3=2.905 Hz. The third frequency of 2.905 Hz is higher than the frequency of 2.4 Hz after which there are no important oscillations. The Table 4 shows that the vertical accelerations of the bridge deck for the frequencies are well within the permissible limits and hence the pedestrians will not feel discomfort while walking on the bridge. Hence the bridge will be safe from pedestrian excitation at its critical frequencies and no resonance will occur

Table 4: Results of Pedestrian excitations of various bridges

Span	Mode no.	Frequency (Hz)	y	□	□	a	a_{max}	Safe
45	1	1.379	0.000206	0.4	25	0.155	0.587	**Safe**
	2	1.788	0.000206	0.1	25	0.065	0.669	**Safe**
	3	2.905	No important oscillations so it is discarded					
50	1	1.11	0.000255	0.4	25	0.124	0.527	**Safe**
	2	1.55	0.000255	0.1	25	0.060	0.622	**Safe**
	3	2.415	No important oscillations so it is discarded					
55	1	0.923	0.000316	0.4	25	0.106	0.480	**Safe**
	2	1.36	0.000316	0.1	25	0.058	0.583	**Safe**
	3	2.06	0.000316	0.1	25	0.132	0.718	**Safe**
60	1	0.775	0.000382	0.4	25	0.091	0.440	**Safe**
	2	1.196	0.000382	0.1	25	0.054	0.547	**Safe**
	3	1.799	0.000382	0.1	25	0.122	0.671	**Safe**

Depth Optimization

The depth optimization was conducted for the extreme spans on which the wind analysis is carried out for all the wind zones of India. The main feasibility of the project was to calculate the optimum depth required in wind zone III. The results of the same are shown below:

Check for failure by yield of steel

The formula for this check is given in clause 13 and described below,

$$M_{ult} = 0.9 * d_b * A_s * f_p$$

Where, M_{ult}= Ultimate moment of resistance,

d_b= Depth of beam from maximum compression edge to centre of tendon,

A_s= Area of prestressing steel,

f_p= Ultimate tensile strength for steel or yield stress.

As per the above given formula, for a depth of beam = 175mm, area of steel = 3360mm^2 and ultimate tensile strength = 1420 N/mm^2, the ultimate moment of resistance works out to a value of 751.5kNm. Figure 20 shows the failure moment capacity of the steel of the depth 200mm, 225mm and 250mm.

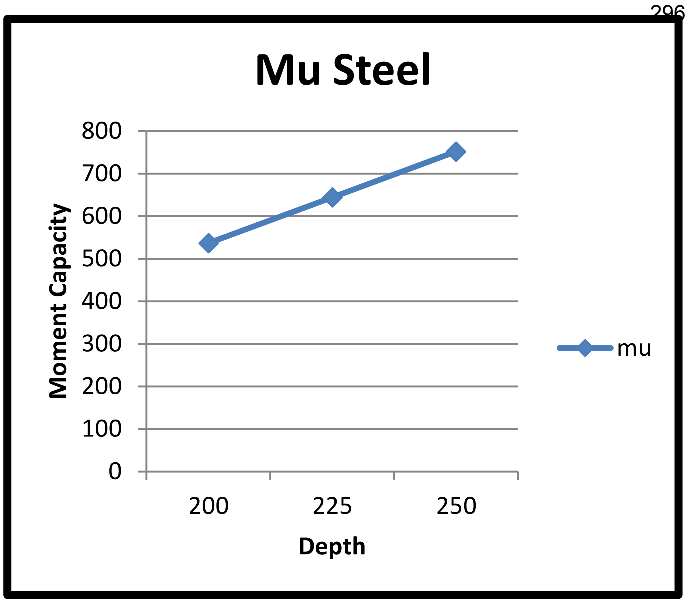

Figure 20: Failure moment capacity of the steel of the depth 200mm, 225mm and 250mm

Check for failure by crushing of concrete

The formula for this check is given in clause 13 and described below,

$$M_{ult} = 0.176 * b * d_b^2 * f_{ck}$$

Where, M_{ult}= Ultimate moment of resistance,

b= Width of beam,

d_b= Depth of beam from maximum compression edge to centre of tendon,

f_{ck}= Characteristic compressive strength of concrete.

As per the above given formula, for a depth of beam = 175mm, width of beam = 4 m and characteristic compressive strength = 60 N/mm^2, the ultimate moment of resistance works out to a value of 1294 kNm. Figure 21 shows the failure moment capacity of the concrete of the depth 200mm, 225mm and 250mm.

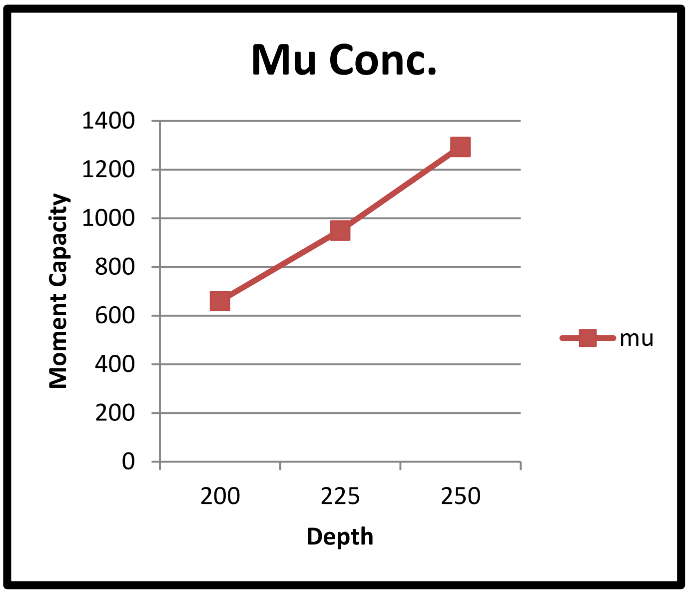

Figure 21: Failure moment capacity of the concrete of the depth 200mm, 225mm and 250mm

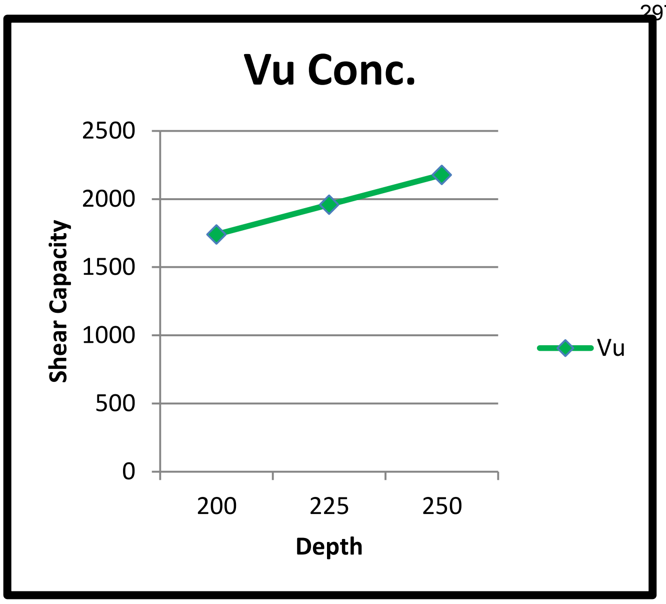

Figure 22: Failure shear capacity of the concrete of the depth 200mm, 225mm and 250mm

Check for shear capacity of concrete

The formula for this check is given in clause 14.1.2 and described below,

$$V_{co} = 0.67 * b * d * \sqrt{f_t^2 + 0.8 * f_{cp} * f_t}$$

Where, V_{co}= Uncracked shear capacity of section,

b = Width of the section,

d = Overall depth of the section,

f_t= Maximum principle tensile stress given by 0.24♦f_{ck},

f_{cp}= Compressive stress at centroidal axis.

As per the above given formula, for an overall depth of beam = 250 mm, width of beam = 4 m, principle tensile stress= 1.859 N/mm^2, compressive stress at centroidal axis = 4.776 N/mm^{2}and characteristic compressive strength = 60 N/mm^2, the ultimate shear capacity works out to a value of 2177kN. Figure 22 shows the failure shear capacity of the concrete of the depth 200mm, 225mm and 250mm.

Abutment Modeling and Analysis

The abutment considered in present study is shown in Figure 23 and Figure 24shows the final model of the abutment created in Ansys Civil-FEM.

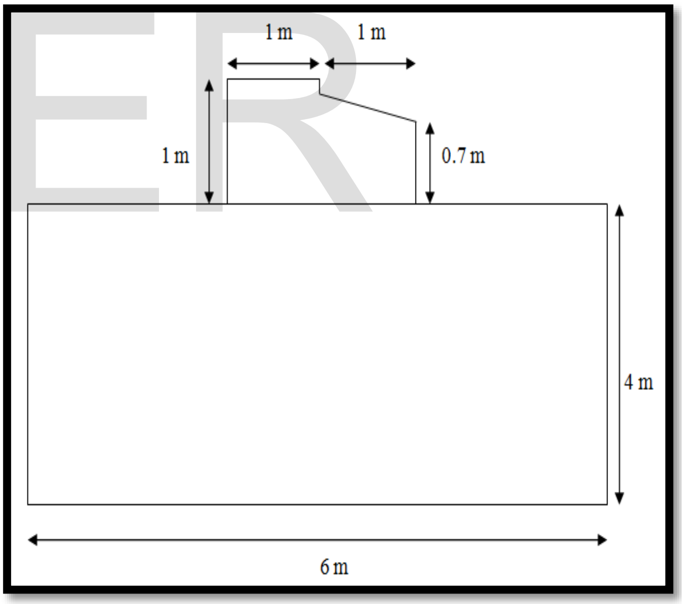

Figure23: Abutment Cross-section

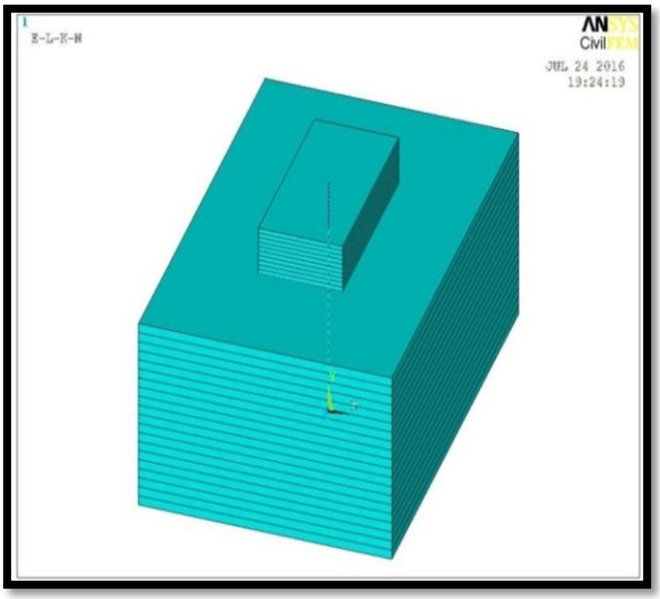

Figure24: Final abutment model without load and support condition

From Figure 25 one can observe that bending moments are minimum where the deck is joint and maximum at the base of the abutment.

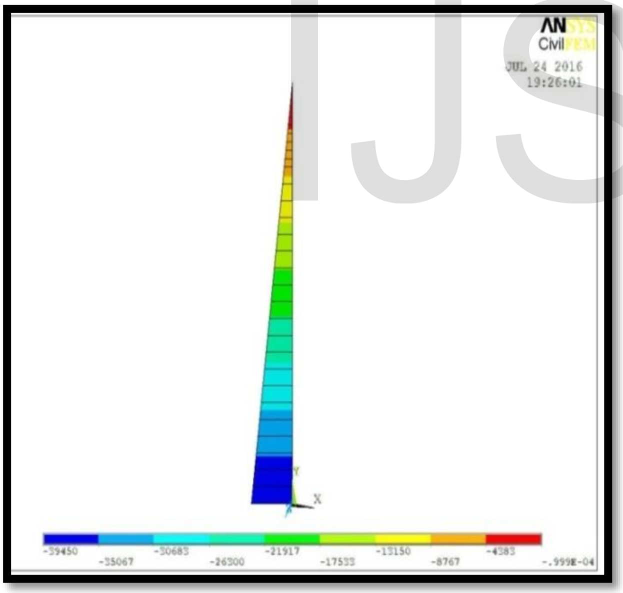

Figure25:Bending moment in abutment due to cable force

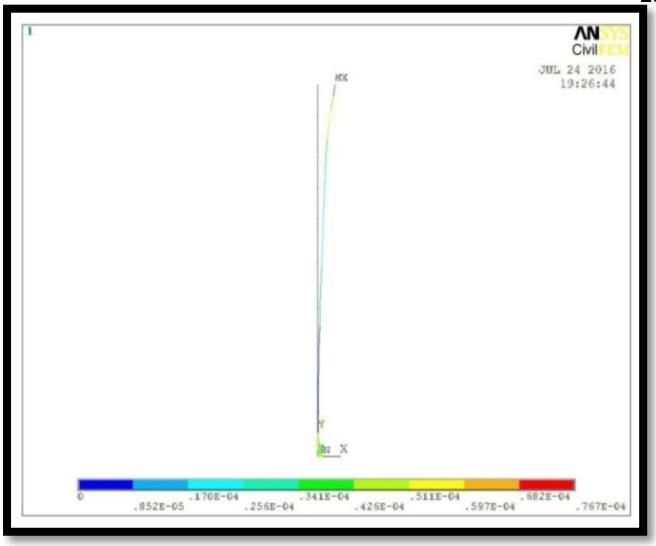

Figure26: Deflection in abutment due to cable force

From Figure 26 it is noticed that the deflection is maximum at the joint where the deck slab is joint to the abutment. Figure 27shows soil pressure result due to cable force for hard rock. It is found that cable forces create a bending effect on the abutment due to which the pressure on the soil is not uniform in nature. While most bridge abutments have only compressive forces on the soil, a stress ribbon bridge abutment will also have tension on its abutment base.

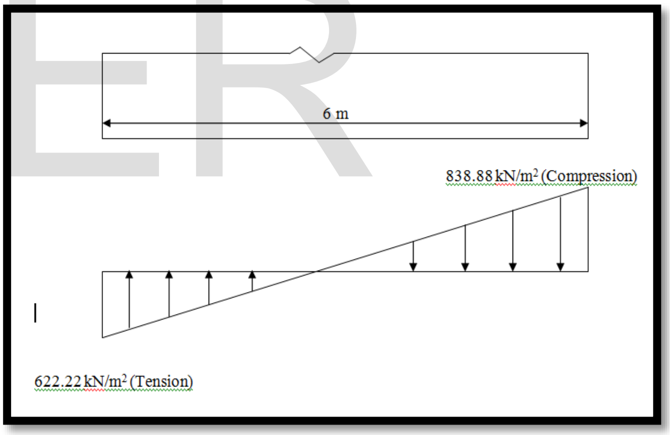

Figure27: Soil pressure result due to cable force

A preliminary analysis of and abutment in cohesion less soil was performed. The results are shown in table 4.

Table 4: Results of abutment Analysis of 45m deck in cohesionless soil

Item	Value	Unit	Remark
Top width	4	m	Transverse width
Top depth	2	m	Depth in Bending
Top height	1	m	
Total height	5	m	
Width of abutment	9	m	Transverse width
Depth of abutment	16	m	Depth in Bending
Height of abutment	4	m	
Cable load	7312.5	kN	
Total Cable Force	7312.5	kN	
Cable Force/m	812.5	kN	
Weight of Bottom	1600	kN	
Weight of Top	50	kN	
Total vertical force	1650	kN	
Moments about toe	9137.5	kN-m	
R from toe	5.537878788	m	
Max stress	198.3398438	kN/m2	
Min Stress	7.91015625	kN/m2	

It is observed that the depth of abutment goes up to16m for satisfactory behavior, which is very difficult and uneconomical to construct an abutment in cohesionless soil.

Conclusions

In the present study, a model of a stress ribbon bridge of 45m, 50m, 55m and 60m span is modelled and analyzed using ANSYS version 12. For simplicity in importing civil materials and civil cross sections, Civil FEM version 12 add-on of ANSYS was used. A 3Dmodels of the whole structure was developed. From the preceding discussions the following conclusions are drawn:

1. With respect to static and dynamic analysis of stress ribbon structures have confirmed that a slender concrete deck supported by an internal and/or external cables can be very efficient.
2. The bending moment in a stress ribbon bridge is much lesser when compared to a beam because of its inverted arch shape.
3. Temperature is a critical condition in a stress ribbon bridge due to its slender deck and flexible nature.
4. In (D.L+L.L+PSC+Temp) condition it is observed that the tensile stresses decrease significantly from span 45m to 50m and for span 55m and 60m the tensile stresses almost negligible.
5. In (Wind+ Temp) combination the deflection is about 40% more and bending moment is 52% more as compared to the deflections for normal combination.
6. As the wind speed increases, the force on the bridge increases which leads to an increase in deflection.
7. The capacities for various depths were calculated and it was concluded that depth of 250mm has better results as compared to 200mm and 225mm for 45m span.

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