ANALYSIS OF THE MEELIYADDA BRIDGE FAILURE IN KURUNEGALA

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ABSTRACT

This paper investigates the failure of the Meeliyadda Bridge, which occurred, when two trucks carrying metal were crossing the bridge. It was reported that some of the lateral bracings of the top chord had removed with the failure of the bridge. Finite element (FE) analysis was carried out using the SAP2000 FE software, where two different FE models were developed: one with all the lateral bracings and the other without three consecutive lateral bracings at the entering end of the trucks. Results from the FE analysis show that removal of the top lateral bracings increases the unbraced length of the top chord members in the lateral direction, reducing their compressive load capacity. This initiates a compression failure in the top chord which could be the reason for the failure in the Meeliyadda Bridge.

***Key words*:** Steel trusses, Lateral bracings, Unbraced length, Finite element analysis

1. INTRODUCTION

According to the RDA database 2015 there are 4210 local highway bridges in Sri Lanka. Out of these 218 are steel bridges, which include about 100 truss type steel bridges. Truss is a simple structure made out of pin ended elements, where the primary forces in the members are axial forces. Since the truss is an open web structure, they can be designed for higher depths with less self-weight when compared with the solid web systems. It could be also noted that steel has high strength to weight compared to concrete and most of the other materials.

Truss type steel bridges can be classified into different types according to the structural systems. Warren, Pratt and Howe are some of the basic truss types. Different truss types have come up later with some modifications to the basic truss types. Varied height truss types became popular with the aesthetical appearance and can be used for higher spans effectively. Warren, Pratt, Double warren and varied height truss are some of the common truss types used in Sri Lanka. Veralgastotupola Bridge, Muwagama Bridge, Nattupana Bridge are some recently built truss type steel bridges in Sri Lanka. Figure 1 shows the Muwagama Bridge located in Muwagama, Rathnapura.

Most of the existing truss type steel bridges in Sri Lanka were constructed during colonial period about 100 years ago. Most of these bridges are outdated for the modern traffic conditions, where the bridges such as Allawa Bridge, Gampola Brige and Katugasthota Bridge have been replaced with newer concrete/steel bridges. Figure 2 shows the old steel bridge located in Gampola [1].

Figure 1: Muwagama Steel Bridge [1]

Figure 2: Gampola Steel Bridge

However, some of the old truss type steel bridges such as Peradeniya Bridge, Kochchikade Bridge and Moratuwa Bridge are still on operation with the modern traffic. Figure 3 shows the steel truss bridge in Peradeniya. It is important to investigate the present condition and suitability of these existing bridges for the modern traffic as a bridge failure could be devastating causing damage to the vehicles and loss of human lives.

Figure 3: Peradeniya Steel bridge

This paper investigates the failure of the Meeliyadda Bridge, which has been occurred recently, when two trucks carrying metal were crossing the bridge. A view of the Meeliyadda Bridge before the failure is shown in Figure 4, while a view of the failed bridge is shown in Figure 5.

Figure 4: Meeliyadda Bridge before the failure

Figure 5: Meeliyadda Bridge after the failure

The finite element (FE) modelling and analysis were carried out using the SAP2000 FE software [2]. Results from this study indicate that the initial failure of the top lateral bracings reduce the compressive load capacity of the top chord members causing a failure in the top chord. Results from the FE analysis for failure load and failure mode agreed reasonably well with those seen during the actual bridge failure. This paper therefore highlights the importance of carrying out a detailed study about the existing old steel bridges in Sri Lanka. This will enable engineers to investigate their suitability for modern traffic conditions, avoiding their failure, saving damage to vehicles and loss of human lives.

2. LITERATURE SURVEY

2.1. History

The history of truss bridges dates back in 1779 when the first truss bridge was built in Coalbrookdale, U.K. Several bridge failures were happened in the history of steel bridges. Ludendorff Bridge failure, San Francisco Oakland Bay Bridge collapse, Ashtabula River Railroad disaster were some examples for the truss type steel bridge failures in the world [3,4].

By observing large number of bridge failures, the common reasons can be noted as,

- Accidental over load and impact
- Force majeure (flood, earthquake etc)
- Structural and Design errors
- Construction and supervision mistakes
- Lack of maintenance

Bridge designers could learn lessons from those failures and new design techniques and improvements were implemented later as a result.

2.2. Bridge Failures in Sri Lanka

Paragasthota Bridge failure, Ehelakanda Bridge failure and Meeliyadda Bridge failure were some of the steel bridge failures happened in Sri Lanka. Both Paragasthota and Ehelakanda Bridge failures were analyzed and the results are presented in a previous paper [1].

Ehelakanda Bridge failed due to its self-weight while it was constructing in 2006. The reason for this failure was identified as the lateral torsional buckling of the top chord members close to the support. This was a pedestrian bridge where the lateral bracings could not be provided for the top chord members close to supports to meet the minimum headroom requirement of 2m.

Paragasthota Bridge failed when a 10 wheel tipper with 5 cubes of metal (weight of about 20-25 tons) was crossing the bridge. The movement of the tipper towards the middle of the bridge increases the axial forces in both top and bottom chords. Results from a previous study [1] showed that the progressive collapse of the bridge was started with the tensile failure of a bottom chord member at the middle of the truss.

Figure 6: Ehelakanda Bridge Failure

Figure 7: Paragasthota Bridge Failure

These bridge failures were predicted reasonably well by using the SAP2000 FE software in a previous paper [1] and hence a similar study has been carried out in this paper to investigate the failure of the Meeliyadda Bridge.

2.3. Meeliyadda Bridge Failure

The Meeliyadda Bridge was constructed in about 1820 by the British government in order to transport rubber to a nearby factory. The bridge was constructed across the Deduru Oya in the Miliyadda area on the road connecting Keppettigala and Kurunegala. This is a Pratt type truss bridge having a length of of 37.2m, width of 4.2m and height of 3.6m.

On 25th of July 2015 this bridge failed when 2 trucks carrying metal was crossing the bridge. It was evident from the villages that the lateral bracings to the top chord were subjected to corrosion and were also damaged time to time by hitting the tippers crossing the bridge. However, exact cause for the failure was not known and hence it was studied in this paper.

3. FINITE ELEMENT ANALYSIS

The failure investigation of the Meeliyadda Bridge was carried out in this paper by using the SAP2000 FE software [2]. By observing the failed bridge it was identified that some of the lateral bracings to the top chord were removed during the failure. By observing the failed bridge and analyzing the information gathered from the villages, it can be predicted that the top lateral bracings would have been broken initially, which might be the reason for the bridge failure.

Two finite element models were prepared in this study, one model with the lateral supports and the other without lateral supports to analyze the effect of lateral supports. The finite element model of the Meeliyadda Bridge with the top lateral supports is shown in Figure 8. When there are no lateral supports, unbraced length of the top chord members in the lateral direction is around 18.6m as illustrated in Figure 9.

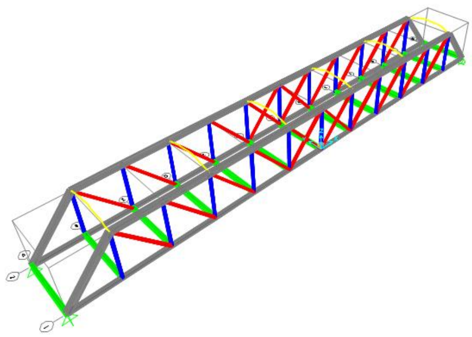


Figure 8: FE model of the Keppettigala Bridge

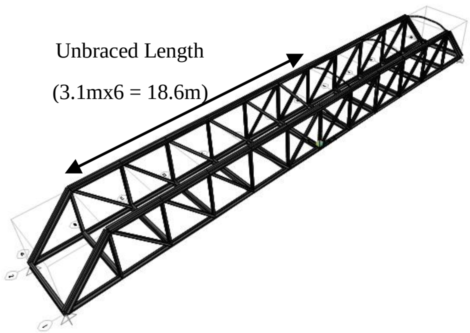


Figure 9: Unbraced length of the bridge

3.1. Material Properties

Following material properties were used in the analysis by assuming all the truss members are constructed with structural steel.

- Weight per unit volume =76.97 kN/m^3
- Modulus of Elasticity, E =205 kN/mm^2
- Minimum Yield Stress =275 N/mm^2
- Minimum Tensile Stress =448.1 N/mm^2

3.2. Loading

Dead load, Superimposed Dead load and live load of the trucks were accounted in the analysis. Dead loads are the self-weight of the structural elements including the weight of the two trusses and the lateral supports. Loads due to Reinforced concrete deck slab and the asphalt layer was considered as the super imposed load, where total thickness of the concrete deck was assumed as 150mm in the analysis. According to RDA details two trucks with metal load was going on the bridge at the failure time.

It was evident from the villages that the truck entered the bridge firstly was close to the remote end of the bridge when the second truck was entering the bridge. This arrangement of the two trucks is illustrated in Figure 10. During the analysis, front truck was kept close to the remote support (without moving), while the rear truck was moved along the bridge. Different load cases were created in the analysis depending on the position of the rear truck.

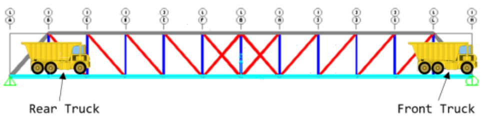


Figure 10: Side view of the two trucks on the bridge

It was estimated that the total weight of a truck with the metal load is 16000kg (W=16tons). About 70% of the load (0.7W) was distributed to the rear axle while 30% (0.3W) was distributed to the front axle in the analysis. The load was inserted as point loads to the lateral cross girders at the bottom chord level, where half of the axle load is distributed through each wheel as shown in Figure 11.

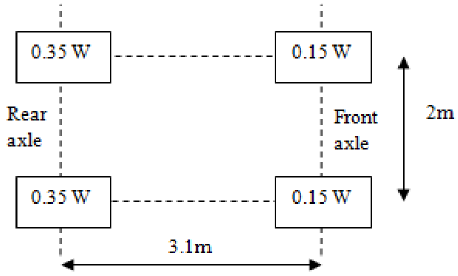


Figure 11: Load distribution through the axles

3.3. Results

Results from the FE analysis for the two FE models: with top lateral bracings and without top lateral bracings are discussed below.

FE model with all the top lateral bracings

Axial force variation of the truss members were analyzed depending of the movement of the rear truck. The load case envelope was developed by considering all the load cases to identify the maximum load in each truss member. Figure 12 illustrates the axial force variation of the truss members for the load envelope in the FE model with all the top lateral bracings. It is clear that the bottom chord members at the middle of the truss have higher tensile forces while those at the middle of the top chord have higher compressive forces.

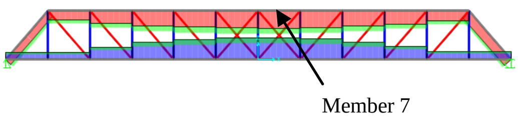


Figure 12: Axial Force Diagram for the load case envelope

Figure 13 illustrates demand/capacity ratios of the truss members for the load case envelope. Demand/ capacity ratio is the ratio between the applied axial force and the axial load capacity of a given member. When the member is under tension, tensile load capacity of the member is used to calculate the demand/capacity ratio, while the compression load capacity is used when the member is under tension.

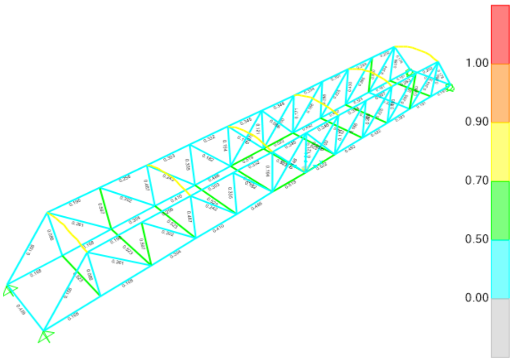


Figure 13: Demand/capacity ratios in the model with lateral bracings

Top lateral bracings have demand/capacity ratio in the range of 0.7-0.9. However, according to Figure 13, none of the truss members have exceeded their demand/capacity ratios over 1, indicating no failure will occur in the truss members with the existence of the top lateral bracings. It is therefore evident that the bridge is safe under the given loading if the lateral supports could securely withstand.

FE model without the first three top lateral bracings at the entering end

By observing the failed bridge it could be seen that the first three top lateral bracings from the entering end of the bridge were completely removed during the failure. FE analysis was therefore carried out by removing the first three top lateral bracings from the entering end. This increases the unbraced length of the top chord members in the lateral direction from 3.1m to 18.6m as illustrated in Figure 9.

The increase in the unbraced length reduces the compressive load capacity of the top chord members from 2403kN to 674kN. The maximum compressive forces could be seen in the members at the middle of the top chord, reaching a value of about 720kN. These members exceed their compressive load capacity of 674kN. This could be seen in the FE model without lateral bracings as shown in Figure 14. However it could be noted that the top chord members do not fail with lateral torsional buckling as the slenderness ratio of the members are less than 180.

SAP2000 FE software also provides design check of the structural members based on the selected design code. In this study design check of the steel truss members were carried out using the BS 5950-1:2000 [5] design standard. Design check data sheet for the critical top chord member (member 7) in the FE model without lateral bracings is illustrated in Figure 15. It is clear that the FE model accurately predicts the failure of the top chord member by giving a warning message as seen in Figure 15. However this warning message could not be seen in the critical top chord members of the FE model without top lateral bracings as illustrated in Figure 16.

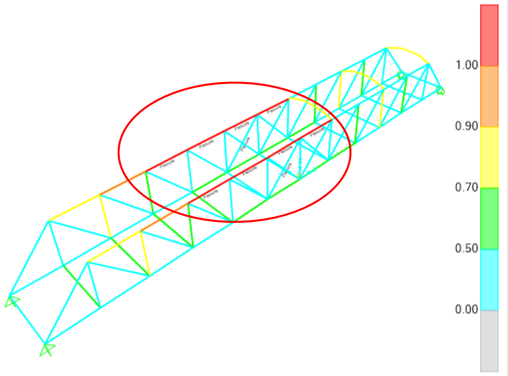
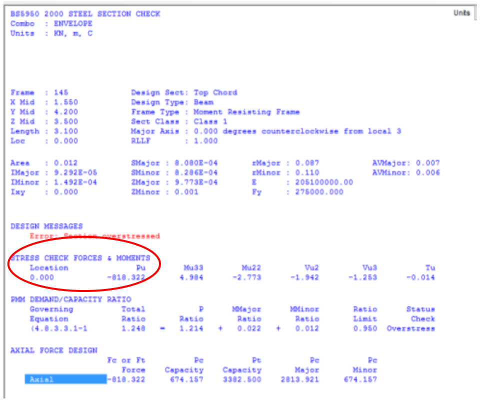


Figure 14: Demand/capacity ratios in the FE model without first three lateral bracings from the entering end



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BS5950 2000 STEEL SECTION CHECK                                              Units
Combo  : ENVELOPE
Units  : KN, m, C

Frame  : 145            Design Sect: Top Chord
X Mid  : 1.550          Design Type: Beam
Y Mid  : 4.200          Frame Type : Moment Resisting Frame
Z Mid  : 3.500          Sect Class : Class 1
Length : 3.100          Major Axis : 0.000 degrees counterclockwise from local 3
Loc    : 0.000          RLLF       : 1.000

Area   : 0.012          SMajor : 8.080E-04     rMajor : 0.087          AVMajor: 0.007
IMajor : 9.292E-05      SMinor : 8.286E-04     rMinor : 0.110          AVMinor: 0.006
IMinor : 1.492E-04      ZMajor : 9.773E-04     E      : 205100000.00
Ixy    : 0.000          ZMinor : 0.001         Fy     : 275000.000

DESIGN MESSAGES
   Error: Section overstressed

STRESS CHECK FORCES & MOMENTS
   Location            Pu        Mu33       Mu22        Vu2        Vu3         Tu
   0.000         -818.322       4.984     -2.773     -1.942     -1.253     -0.014

PMM DEMAND/CAPACITY RATIO
   Governing        Total          P     MMajor     MMinor      Ratio     Status
   Equation         Ratio      Ratio      Ratio      Ratio      Limit      Check
   (4.8.3.3.1-1     1.248  =   1.214  +   0.022  +   0.012      0.950  Overstress

AXIAL FORCE DESIGN
                 Fc or Ft         Pc         Pt         Pc         Pc
                    Force   Capacity   Capacity      Major      Minor
   Axial         -818.322    674.157   3382.500   2813.921    674.157
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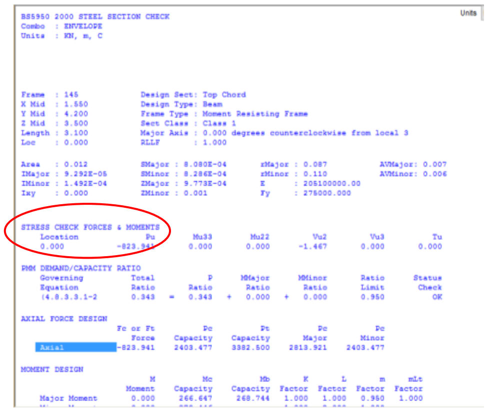
Figure 15: Design check data sheet of member 7 in the FE model without top lateral bracings



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BS5950 2000 STEEL SECTION CHECK                                              Units
Combo  : ENVELOPE
Units  : KN, m, C

Frame  : 145            Design Sect: Top Chord
X Mid  : 1.550          Design Type: Beam
Y Mid  : 4.200          Frame Type : Moment Resisting Frame
Z Mid  : 3.500          Sect Class : Class 1
Length : 3.100          Major Axis : 0.000 degrees counterclockwise from local 3
Loc    : 0.000          RLLF       : 1.000

Area   : 0.012          SMajor : 8.080E-04     rMajor : 0.087          AVMajor: 0.007
IMajor : 9.292E-05      SMinor : 8.286E-04     rMinor : 0.110          AVMinor: 0.006
IMinor : 1.492E-04      ZMajor : 9.773E-04     E      : 205100000.00
Ixy    : 0.000          ZMinor : 0.001         Fy     : 275000.000

STRESS CHECK FORCES & MOMENTS
   Location            Pu        Mu33       Mu22        Vu2        Vu3         Tu
   0.000         -823.941       0.000      0.000     -1.467      0.000      0.000

PMM DEMAND/CAPACITY RATIO
   Governing        Total          P     MMajor     MMinor      Ratio     Status
   Equation         Ratio      Ratio      Ratio      Ratio      Limit      Check
   (4.8.3.3.1-2     0.343  =   0.343  +   0.000  +   0.000      0.950         OK

AXIAL FORCE DESIGN
                 Fc or Ft         Pc         Pt         Pc         Pc
                    Force   Capacity   Capacity      Major      Minor
   Axial         -823.941   2403.477   3382.500   2813.921   2403.477

MOMENT DESIGN
                        M         Mc         Mb          K          L          m        mLt
                   Moment   Capacity   Capacity     Factor     Factor     Factor     Factor
   Major Moment     0.000    266.647    268.744      1.000      1.000      0.950      1.000
```

Figure 16: Design check data sheet of member 7 in the FE model with all the top lateral bracings

Axial force variation of the critical top chord member (member 7) was plotted with the movement of the rear truck while placing the front truck close to the remote support of the bridge. Figure 17 illustrates the axial force variation of the member 7 with the movement of the rear truck in the FE model without top lateral bracings. When the truck reaches 5-10m distance from the entering end, the member reaches its compressive load capacity of 674kN. Maximum axial force can be seen when the truck is at the middle region of the bridge giving a peak force of about 818kN. This is well over 674kN confirming the failure of the top chord

Figure 18 illustrates the axial force variation of member 7 with the movement of the rear truck in the FE model with all the top lateral bracings. Axial force variation of the member is very similar to that observed in the previous FE model. However, it is clear that the maximum axial force in the member 7, which is about 824kN is well below the compressive load capacity of the member which is about 2403kN.

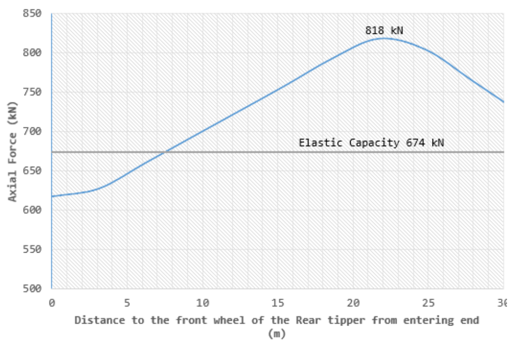

Figure 17: Axial force variation of member 7 in the FE model without top lateral bracings

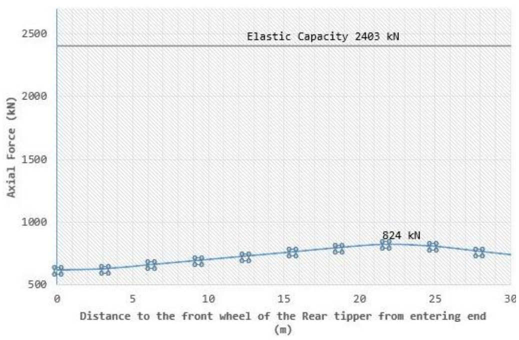

Figure 18: Axial force variation of member 7 in the FE model with all the top lateral bracings

4. CONCLUSION

By analyzing the results from FE analysis it can be concluded that the bridge could withstand the loads of the two trucks if all the top lateral supports were held in their positions without failure. However, failure of the top lateral bracings increases the unbraced length of the top chord members in the lateral direction considerably, reducing the compressive load capacity of the top chord. This could be the reason for the failure of the bridge as the removal of the top lateral bracings was observed in the failed bridge. This paper therefore highlights the importance of the lateral supports in a truss type steel bridge, where their failure can cause the failure of the entire bridge.

5. REFERENCES

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