

Effect of Overweight Vehicles on the Design of PC Girder Bridges



R. S. Tomal and K. M. Amanat

Abstract Overloading has become a major concern for engineers in recent years. The mentality of logistics service providers towards increasing their profit margins and higher transportation and fuel costs in middle-income countries like Bangladesh induce overloading of freight traffic. These result in an increase in the number of accidents due to braking system failure and infrastructure damage such as roads and bridges. Increased frequency of overloads may trigger shorter service life of critical components and perhaps catastrophic bridge failure. This study aims to assess the impact of overweight vehicle on the design of prestressed concrete girder bridges, which comprise the majority of highway bridges. For this purpose, an overweight vehicle has been selected investigating monthly overweight truck status report from a previous study. Bridges of four different spans (30 m, 35 m, 40 m and 45 m) have been modeled and designed in CSiBridge for strength and serviceability combination. To explore the effects of overloading, series of selected overweight vehicles were driven over those bridges. Parameters considered for studying the effect are maximum mid span moment, D/C ratio, maximum reactions at supports for Strength I combination and girder bottom tensile stress at Service III combination. It has been observed that maximum moment has been increased by 20.46% for 30 m bridge, 25.74% for 35 m bridge, 31.59% for 40 m bridge and 37.89% for 45 m bridge. D/C ratio has been increased by 19.76% for 30 m bridge, 35.33% for 35 m, 41.44% for 40 m bridge and 32.41% for 45 m bridge. Substantial changes have been noticed in girder bottom tensile stresses. Stress has been increased by 625.8% for 30 m bridge, 1995% for 35 m bridge, 884.5% for 40 m bridge and 763.8% for 45 m bridge. Maximum support reaction has been increased by 44.33% for 30 m bridge, 37.83% for 35 m bridge, 48.77% for 40 m bridge and 41.63% for 45 m bridge for interior girders and for exterior girders 32.55% for 30 m bridge, 32.48% for 35 m bridge, 37.20% for 40 m bridge and 35.79% for 45 m bridge. This study provides quantitative evidence

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of damages caused by overweight vehicles to highway bridges and represents the significance of overloading in the design of pc girder bridges.

Keywords Overloading · CSiBridge · Service life · Strength and serviceability combination · Prestressed concrete bridge · Damage · Failure

1 Introduction

1.1 Background and Motivation

Increasing freight movement especially overweight freight is gradually becoming a great concern in Bangladesh. Two main reasons contributing to this increased overweight freight movement are: (1) Major development projects and activities generating movement of freights heavily loaded with construction materials; (2) Recent trends in online trades and shopping generating large numbers of freight heavily loaded with consumers goods. Most of this freight transportation is happening through trucks. The main concern for structural engineers here is that bridges are designed for a certain design traffic load (e.g., HS 20–44, HS 15–44, HL-93 K) which may be crossed by this overweight traffic and cause significant damage. In Bangladesh, although there is axle load rules and axle load control centers, lack of proper implementation of rules and enforcement often permits overweight freight to move through the bridges. Moreover, Government often relaxes this load control rules during special occasions when the highways become congested. This necessitates a requirement of study that can quantitatively show how much damage is occurring due to those overweight freights/trucks.

1.2 Objectives and Scope of This Study

This focus of this study is to determine the extent to which these overweight trucks are crossing different resistance limits provided by AASHTO (American Association of State Highway and Transportation Officials) and to quantify the damage occurring due to heavy truck movements on PC Girder Bridge. If it becomes possible to quantify this damage or if it is possible to represent this mathematically, it will work as evidence and it is expected that this will be helpful in raising consciousness of Government authority, Highway agency and Bridge authority about the severity of this overloading problem. The objectives and scopes of this study can be summarized as:

- (1) To study the process of analysis, design, and performance evaluation of prestressed concrete bridges by the methods specified in AASHTO LRFD 2012.
- (2) To conduct a parametric study by varying pc girder bridge span and develop a comparison between design truck and overweight truck on different parameters like maximum moment, demand to concrete shear capacity ratio and stresses at service III condition.
- (3) Focuses mainly on the superstructure modeling, analysis and design
- (4) Substructure elements will not be modeled in detail.

1.3 Findings of Previous Studies

Researchers have investigated the effects of overweight vehicles, but most of these studies were limited in scope. The researchers' observations were only provided during and after the overloading passages. The long-term consequences of overweight cars have not been studied. The following is a list of notable studies on overweight vehicles. Duncan [1] investigated superload effects on bridges in South Africa, emphasizing the significance of precise methodologies for assessing superload effects on bridges in order to use lower margins of strength for regulated superload passages. The effects of overload trucks on deck damage were explored by Kostem [2], who highlighted the necessity of including material nonlinearities in the analysis to depict how forces are redistributed across the structure. According to Kostem [2], The deck slab's flexural cracking as a result of the bridge's overloading is what causes the damage. Ohio's three steel-girder bridges were put to the test by Turer and Aktan [3] to see whether they could handle an 817-kip superload truck with a cooling unit. Cross-frames, which link the girders together and offer stability, were revealed to be the most severely strained parts with a stress range of 10 ksi. Additionally, it was determined that the transverse deck stress produced by transverse bending was significant. On the other hand, the girders' stress level remained at or around 5 ksi. The three bridges were all confirmed to be in good condition. Though this type of damage was not seen, the loss of composite action was described as a frequent damage mechanism. Additionally, the deflections were far lower than the typical limit of the span length divided by 800 [4]. Before, during, and after a superload passage, by Ziehl and Lamana, [5] a Louisiana bridge made of prestressed concrete (PC) had its stresses and deflections examined. The acoustic emission method was also used to evaluate damage. The researchers found no deterioration or alteration in the bridge's response following the passage of the superload.

2 Model Development and Validation

2.1 Model Data

1. Span (c/c distance of bearing), $L = 104.5$ feet (31.86 m)
2. Roadway width = 24 feet (7.32 m)
3. Total width of the bridge = 31 feet (9.45 m)
4. Numbers of Lane = 2
5. Number of longitudinal girders = 4
6. Future wearing course = 4 inch thick
7. c/c spacing of girders = 7.75 feet (2.36 m)
8. AASHTO Girder type = V
9. Post tensioning steel: 7 wire strand, low relaxation, Grade 270, Nominal diameter = 0.6 inch (15.2 mm)
10. Curb height = 12 inch (3.66 m)
11. Maximum initial concrete stress at transfer: $f_{ci}' = 0.8 f_c'$
12. Concrete for Deck slab, $f_{c(Deck)}' = 5$ ksi (34.5 MPa)
13. Concrete for Girder, $f_{c(Girder)}' = 6$ ksi (41.4 MPa)
14. Elastic Modulus for Slab Concrete, $E_{c(Deck)} = w_c^{1.5} \sqrt{f_{c(Deck)}'} = 3904$ ksi = 26,920 MPa
15. Elastic Modulus for Slab Concrete, $E_{c(Girder)} = w_c^{1.5} \sqrt{f_{c(Girder)}'} = 4276$ ksi = 29,480 MPa
16. Elastic Modulus of Prestressing steel, $E_p = 28,500$ ksi (Fig. 1).

2.2 Modeling Steps

See Fig. 2.

2.3 Validation

The model validation will show variation of different parameters from CSiBridge to manual calculation according to Code. This will ensure that the model has been correctly modeled. Validation for section properties, moment, support reaction, stress in tendon after all losses, stress check at transfer, check for final condition (Table 1).

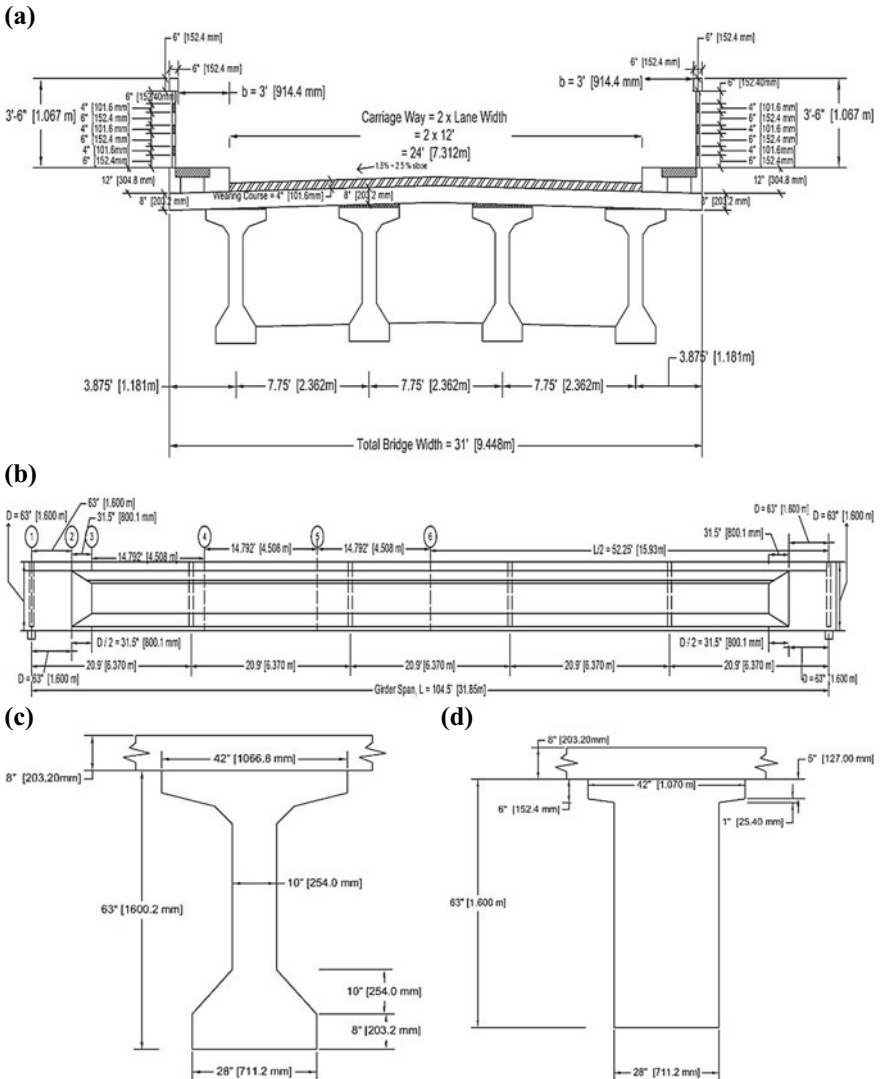


Fig. 1 a Deck configuration of model bridge; b Longitudinal section of AASHTO Type V girder, c Cross section of Mid-section and d Cross section of End-section

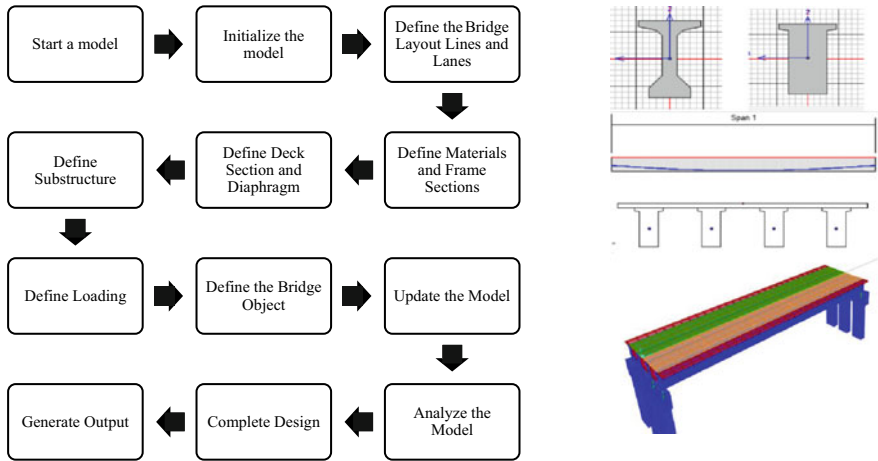


Fig. 2 a Steps of implementing the 3D FE model of bridge in CSiBridge; b 3D FE model in CSiBridge

	Location	Distance (m)	Manual calculation (KN-m)	CSiBridge (KN-m)	Variation (%)
Check for moment due to girder self-weight	1	0	0	0	0
	2	6.911	1464	1425	2.664
	3	11.42	1974	1935	1.976
	4	15.93	2144	2107	1.726
Check for moment due to Dead load (Girder Self weight + Diaphragms + Deck Self weight)	1	0	0	0	0
	2	6.911	2700	2564	5.000
	3	11.42	3645	3487	4.335
	4	15.93	3955	3795	4.046
Check for moment due to Sidewalk and railing	1	0	0	0	0
	2	6.911	638.8	627.8	1.722
	3	11.42	864.8	859.0	0.671
	4	15.93	940.3	947.3	0.739
Check for moment due to wearing course load	1	0	0	0	0
	2	6.911	454.9	445.3	2.110
	3	11.42	616.4	606.4	1.622
	4	15.93	670.1	660.4	1.448
Check for moment due to pedestrian live load	1	0	0	0	0
	2	6.911	141.5	139.0	1.767
	3	11.42	191.5	190.3	0.627
	4	15.93	208.1	209.8	0.810

(continued)

(continued)

	Location	Distance (m)	Manual calculation (KN-m)	CSiBridge (KN-m)	Variation (%)
Check for stresses in tendon after all losses	1	0	1125	1130	0.4425
	2	6.911	1143	1147	0.3487
	3	11.42	1155	1159	0.3451
	4	15.93	1168	1172	0.3413
Stress check at transfer at bottom fiber of concrete (- means compression)	1	0	-7.786	-7.731	0.7064
	2	6.911	-17.98	-17.85	0.7230
	3	11.42	-20.39	-20.22	0.8337
	4	15.93	-21.36	-21.20	0.7491
Stress check at transfer at top fiber of concrete (- means compression)	1	0	-4.754	-4.678	1.599
	2	6.911	-3.376	-3.367	0.2666
	3	11.42	-1.240	-1.295	4.247
	4	15.93	-0.5000	-0.5259	4.925

From this validation part, it can be summarized that the model has been almost correctly modeled as most of the parametric variations are within 5%. This implies that the model can be used further for parametric studies.

3 Parametric Study

3.1 Overweight Truck Selection and Modeling

Due to time and resource constraints, it was not possible to conduct an axle load survey. Hence previous survey data has been studied and the most overweight vehicle has been selected. From such a previous survey found in a study by Ullah [6], the most overweight vehicle observed to have a total weight of 33.2 ton which is a two-axle truck having respectively 9.5 ton and 23.7 ton on axle 1 and axle 2. For modeling purpose, it has been assumed that the axle-to-axle distance is 14 feet similar to design truck (Fig. 3).

For parametric study purpose, 4 different length bridges have been selected: 30 m, 35 m, 40 m, and 45 m. They have been designed (Design procedure will be described in the later portion of this chapter). It has been assumed that the total bridge length will be occupied by a series of selected overweight trucks. Truck to truck distance and other dimensions are shown in the figures (Figs. 4, 5 and 6).

In CSiBridge these series of 4 and 5 trucks have been defined as a single vehicle of respectively 8 and 10 axles and having an impact factor of 1.15 moving in both lanes at a speed of 50km/h (13.889 m/s).

Table 1 Validation of model bridge

	Parameters	Manual calculation	CSiBridge	Variation (%)
Section properties for non-composite section	Section modulus of mid-section for bottom fiber	$2.729 \times 10^8 \text{ mm}^3$	$2.729 \times 10^8 \text{ mm}^3$	0
	Area of mid-section	$7.097 \times 10^5 \text{ mm}^2$	$7.097 \times 10^5 \text{ mm}^2$	0
	Section modulus of end-section for bottom fiber	$3.230 \times 10^8 \text{ mm}^3$	$3.230 \times 10^8 \text{ mm}^3$	0
	Area of end-section	$1.188 \times 10^6 \text{ mm}^2$	$1.188 \times 10^6 \text{ mm}^2$	0
Section properties for composite section	Section modulus of mid-section for bottom fiber	$3.780 \times 10^8 \text{ mm}^3$	$3.780 \times 10^8 \text{ mm}^3$	0
	Area of mid-section	$1.148 \times 10^6 \text{ mm}^2$	$1.148 \times 10^6 \text{ mm}^2$	0
	Distance of centroid of mid-section from bottom fiber	1155 mm	1155 mm	0
	Section modulus of end section for bottom fiber	$4.813 \times 10^8 \text{ mm}^3$	$4.813 \times 10^8 \text{ mm}^3$	0
	Area of end section	$1.626 \times 10^6 \text{ mm}^2$	$1.626 \times 10^6 \text{ mm}^2$	0
	Distance of centroid of end section from bottom fiber	1065 mm	1065 mm	0

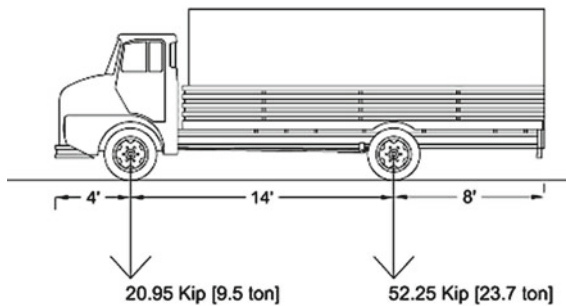


Fig. 3 Two axle overweight truck

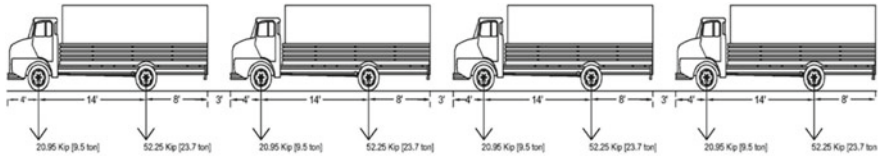


Fig. 4 Series of 4 overweight trucks for 30 and 35 m bridges

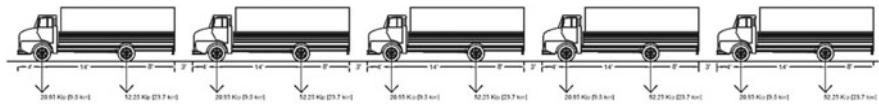


Fig. 5 Series of 5 overweight trucks for 40 m and 45 m bridges

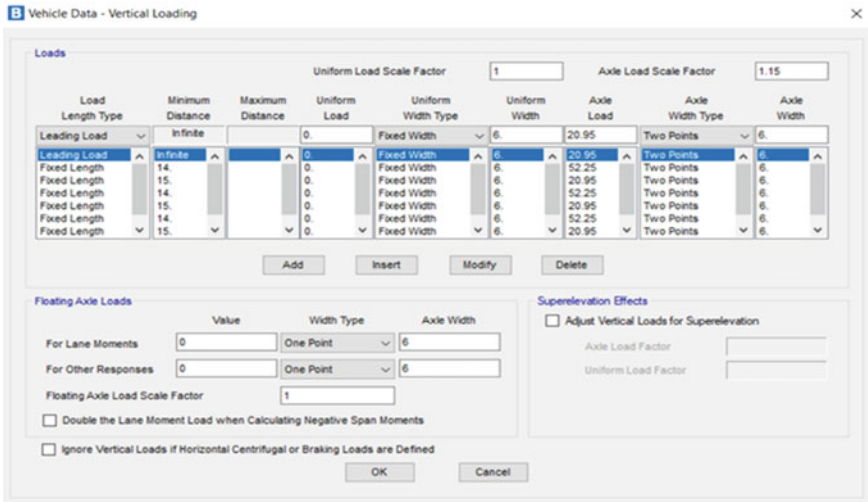


Fig. 6 Vehicle data for vertical loading

4 Results and Discussions

The observations found from this study can be summarized as follows (Fig. 7):

1. Flexural moments for strength I combination using a series of overweight trucks have been increased by 20.46% for 30 m bridge, 25.74% for 35 m bridge, 31.59% for 40 m bridge and 37.89% for 45 m bridge from moments produced using Design HL 93 Live load and these moments have also crossed the positive flexural resistance limit.
2. D/C ratios for strength I combination using a series of overweight trucks have been increased by 19.76% for 30 m bridge, 35.33% for 35 m bridge, 41.44% for

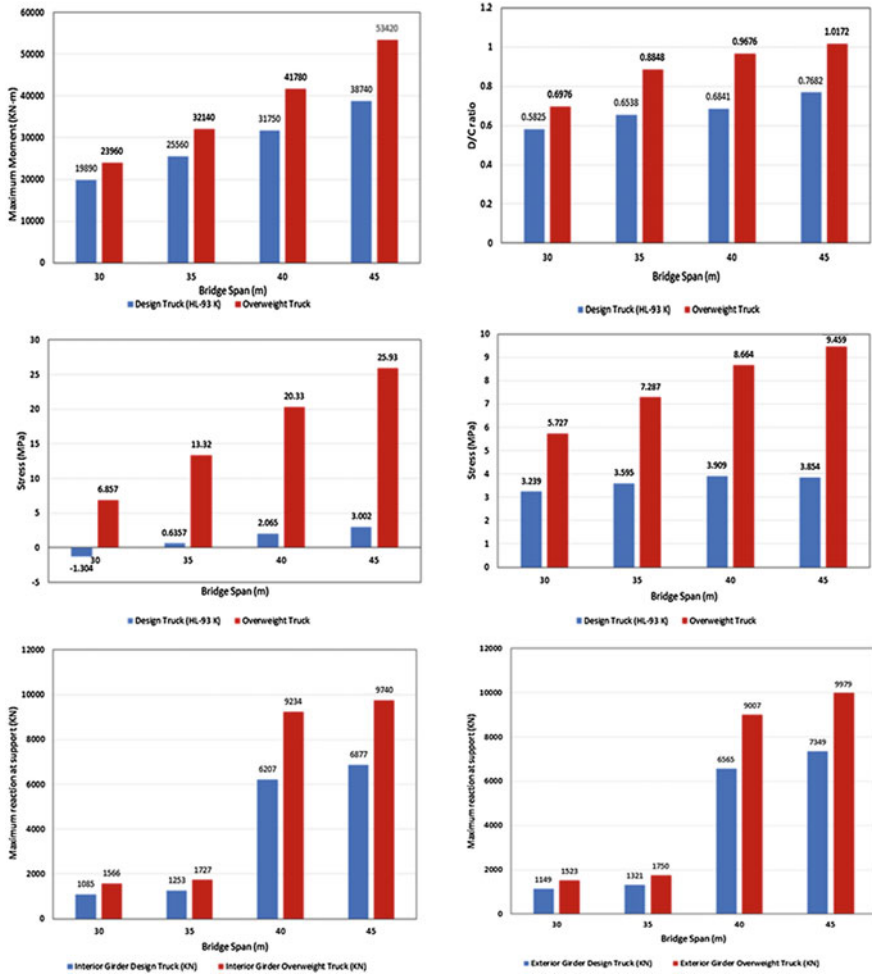


Fig. 7 a Maximum moment for strength I combination versus Bridge span; b Demand to maximum concrete shear capacity ratio for strength I combination versus Bridge Span; c Girder bottom stress for service III combination versus Bridge span; d Girder bottom stress for live load versus Bridge span; e Maximum reaction at support for strength -I combination for interior girder versus Bridge span; f Maximum reaction at support for Strength -I combination for exterior girders versus Bridge span

40 m bridge and 32.41% for 45 m bridge from D/C ratios produced by using design HL-93 Live load. For 30, 35 and 40 m bridges this ratio is within limit but for 45 m bridge it crosses the limit (Figs. 8, 9 and 10).

3. Tensile stresses at girder bottom for Service III combination using a series of overweight trucks, have been increased by 625.8% for 30 m bridge, 1995% for 35 m bridge, 884.5% for 40 m bridge and 763.8% for 45 m bridge from stresses produced by using HL 93 vehicle and also, they have crossed the limit by 2.143

Fig. 8 Flexural moment check due to overweight truck for Strength I combination (including prestress force) for 30 m bridge

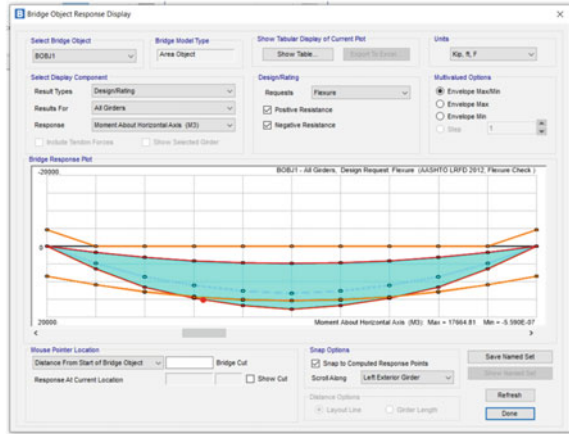


Fig. 9 D/C ratio check due to overweight truck for Strength I combination (including prestress force) for 30 m bridge

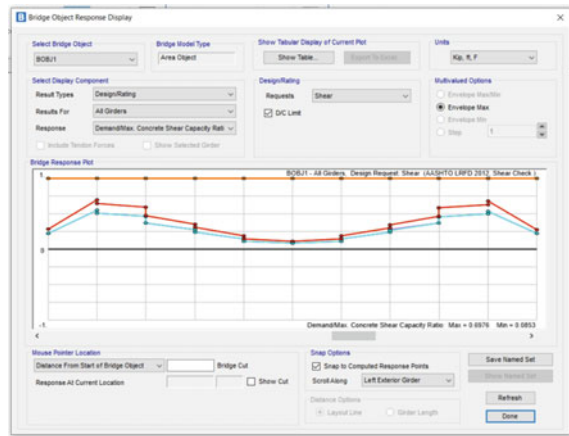
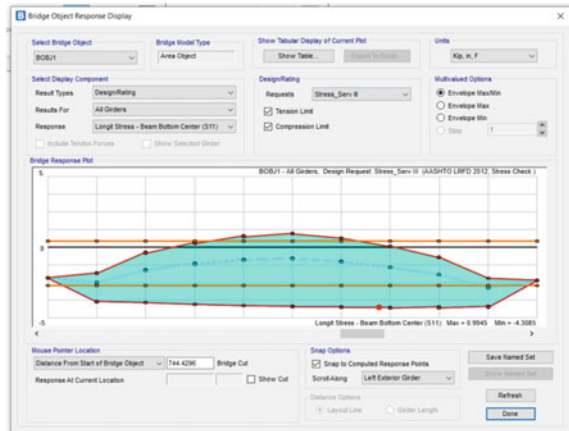


Fig. 10 Stress Check due to overweight truck for service III combination for 30 m bridge



times by 30 m bridge, 4.163 times by 35 m bridge, 6.353 times for 40 m bridge and 8.103 times by 45 m bridge.

4. Maximum reaction at supports for Strength I combination using a series of overweight trucks, have been increased by 44.33% for 30 m bridge, 37.83% for 35 m bridge, 48.77% for 40 m bridge and 41.63% for 45 m bridge for interior girders and for exterior girders 32.55% for 30 m bridge, 32.48% for 35 m bridge, 37.20% for 40 m bridge and 35.79% for 45 m bridge.

5 Conclusion

5.1 Findings

The findings of this **study** can be summarized as:

- (1) Bridges designed considering single AASHTO Standard HL-93 vehicle in each lane as Live load are not capable of withstanding series of overweight vehicles (both lanes) considered in this study.
- (2) All the bridges considered in this study failed in flexure moment check crossing the flexural resistance limit for the series of overweight vehicles considered.
- (3) For 30, 35, 40 m bridges overweight vehicles couldn't not cause shear failure, but 45 m bridge failed in D/C limit check due to overweight vehicle. D/C ratio increased with the increase in the bridge span.
- (4) All the bridges studied in this **study** failed in Girder bottom tensile stress check. Though these bridges have passed this stress check for design live load, they have failed to meet the same limit when series of overweight moved through them.
- (5) Due to series of overweight truck movement, bearing support reaction increased than that is required for design live load. So, bearing pad needs to be redesigned considering overweight vehicle movements.

5.2 Limitations and Recommendations

This **study** has a very limited scope to observe all the things about overweight vehicle problems. The fields in which study can be further extended are discussed below:

- (1) Due to time and resource constraint axle load survey has been skipped and overweight vehicle data has been collected from previous studies which may not be the real scenario of the recent time. So, a comprehensive survey can be conducted in future to get a more realistic result.
- (2) In this study, CSiBridge v23.3.1 has been used for finite element modeling and analysis of bridges. This software has many limitations. It was not possible to check all the stress limits recommended by AASHTO LRFD 2012 code. So, in

future a more accurate and detailed study can be conducted using other software like Midas Bridge, Ansys etc.

- (3) It was not possible to replicate the effect of accumulative damages due to live load in highway bridges which is very important in studying the fatigue damage and fatigue life of bridges. So, consideration of this effect in future studies can bring a very interesting and significant result.

From the trend of damages occurring due to a range of overweight vehicles, a **study** can be conducted to predict the reduction in service life of bridge.

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