Effect of overloaded vehicles on the performance of highway bridge girder: A case study

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Abstract

This paper is about the analysis of a prestressed highway bridge and its strengthening. The bridge was simply supported I’ Girder structure with four-lanes and it was heavily loaded than its capacity; therefore, bridge was analysed in flexure using software “SAP 2000” for the actual or modified vehicular loading present over the bridge. Flexural analysis was performed in two stages: initially, the bridge was analysed for the loads for which it was originally designed and then the same bridge was checked for the modified vehicular load present over it. Analysis results revealed that the stresses in the I-girders were exceeding in tension side and therefore bridge had a problem in serviceability due to heavy vehicular load (modified load) and therefore required strengthening. After strengthening the bridge using plate bonding strengthening technique, bridge was reanalysed against the modified loading which showed that the flexure capacity of the bridge is sufficient to withstand the heavy vehicular loads.

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Keywords: Highway bridge; prestressing; structural analysis; strengthening

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1. Introduction

Highway bridges are designed to carry traffic loads safely throughout its service life. In Pakistan, National Highway Authority (NHA) is the largest government organization and under its jurisdiction, the allowed axle weights and gross weights of the trucks with different axle configurations are monitored to legally operate traffic on the highways [1]. Other than this, NHA also monitors the axle weights on weigh station located on highways [2]. However, due to continuous rise in fuel prices, the gross weight of the existing vehicular live load (i.e. weight of the transferring goods) on the highways is observed significantly higher than those allowed by NHA [2]; therefore, truck overloading has become very common to save money. In Pakistan, first vehicular live load model (Class A Truck) as shown in Fig.1 was introduced in the code of practice of highway bridges 1967 or West Pakistan Highway Code[3]. This model was based on 1935 load model introduced by the British rulers of that era. Since then to date, enormous change in the loading trucks and overloading has been observed whereas live load model has never been updated. According to NHA, the roads which are designed for 8.2 tons standard axle load are bearing the axle load as high as 24 tons and gross truck weight is about 80 tons [1], which is 1.5 times higher than Class A Truck. Considering the example of USA where HS20-44 was the live load model introduced in 1944 (refer Fig. 2) and in 1993, realising the problem of heavy vehicular loading, this model was upgraded to US live load model as “HL-93” which is almost double the load of HS20-44. Since West Pakistan Highway Code has become old and does not meet the design requirements of existing vehicular loading, most of the bridges in Pakistan are designed based on HS20-44 vehicular load mentioned in the US code of practice of highway bridges [4]. In general, either Class A truck or HS20-44 is used as vehicular live load in Pakistan without any consideration of vehicular live load pattern over the highway bridges. In conjunction with overloading, the distance between the rear
axle of trailer of the first vehicle and front axle of the second vehicle (Class A truck) is quite large (refer Fig. 1) and this distance, specifically in urban areas, based on existing traffic pattern during peak hour, is much lesser than mentioned in the code[3]. These variations in vehicular loads, configuration and traffic congestion during peak hours may result in deficiency in the bridge design.

In United States, all states issue special permits for non-divisible and/or divisible truck overloads more than the weight limit of the highway authority. This causes higher stress levels exceeding the stress level by normal vehicular loads; however, rationales are being documented and procedures of checking overstress before issuing permits has been proposed [5]. Unfortunately, there is no control for overloaded trucks working in Pakistan [1]. Also in developing countries like Taiwan, it was found that there is increase in the size and weight of heavy vehicles due to large-scale constructions projects. A study was conducted to check the bridge safety under the working heavy vehicles and it was found that heavy vehicles significantly affects the bridge safety and the reinforcement in the bridge deck was found 28% lesser than required for heavy loads [6]. In such situation of overstressing and underestimated design section/reinforcement, retrofitting is required for bridges. Around the world, many bridges have been retrofitted (e.g. Menai Straits Suspension Bridge and Tower Bridge in London, etc.[7]) and several strengthening techniques such as external unbonded reinforcement [8], external prestressing [9], carbon fiber reinforced polymer (CFRP) [10, 11], ferrocement [12, 13] and plate bonding [14, 15] are widely studied and applied in order to strengthen inadequacy of bridge in flexure and/or serviceability. Recent work on structural reinforced concrete beams demonstrated that beams can retain much of their strength despite part or all of the main flexural reinforcement being unbonded over a substantial portion of the span [16]. External unbonded reinforcement is based on this principle and is capable of modifying structural behaviour from pure flexure to a hybrid of flexural and tied arch action [8]. The use of external prestressing tendons as strengthening of concrete flexural members upgrade the flexural strength up to 46% and reduce deflection up to 75% [9], but the drawback of this technique is corrosion of external prestressing reinforcement and limitation in prestressing due to concrete member capacity. CFRP on the other hand does not corrode and is 8 to 10 times stronger and 5 times lighter than steel. Carbon fiber composites can make bridges 30 to 60 percent stronger than the original design [10, 11]. The major concern of CFRP is a brittle failure of epoxy and de-lamination. Ferrocement and Ferrocement laminate is advantageous in terms of enhancement of load carrying capacity and serviceability [12, 13]. Ferrocement is low cost and easy to apply technique and mostly suited for small scale members [12]. Plate bonding is used to increase flexural capacity of a reinforced concrete beam by attaching steel cover plates to the tension face of the beam either through epoxy or by bolting [14, 15]. The drawback of this technique is corrosion of the plate.

Current investigation is based on the case study of a medium span prestressed I-Girder Bridge of simply supported span. The serviceability and flexural strength of girder are the performance criteria and the standard truck of HS20-44 [4] and Class A truck [3] has been considered. A convoy of the same trucks has been considered to model the effect of existing traffic congestion and volume on highway bridges of Pakistan specifically in urban areas. At the end, the application of one of the above described strengthening technique has been discussed in the context of the case studied here.

2. Brief Description of the Case Study

A simply supported I-Girder Bridge comprised of four (04) lanes has been selected. Structural system of the bridge is comprised of nine (09) ‘I’ girders and 200 mm thick deck slab as shown in Fig. 3. Further details of the bridge are as follows:

- Span length of the I-Girder = 27600 mm
- Width of carriageway excluding barrier = 14140 mm
- Cross sectional area of the girder (without deck) = 463125 mm²
- Spacing of girders = 1650 mm
- Moment of inertia of girder, \( I_g = 7.387\times10^{10} \text{ mm}^4 \)
- Moment of inertia of (girder + slab) composite section, \( I = 1.79\times10^{11} \text{ mm}^4 \)

Details of section of I-girder and barrier have been given in Fig. 4 and Fig. 5. Following are the material properties used for checking of I-girder Bridge:
Fig. 1. Loading class “A” for highway bridges [4]

Table 1

| Class of loading | Axle Load (Lbs.) | Ground contact area (in) |
|------------------|------------------|-------------------------|
| "A"              | 25,000           | 10                      |
|                  | 15,000           | 8                       |
|                  | 6,000            | 6                       |
| "B"              | 15,000           | 8                       |
|                  | 9,000            | 6                       |
|                  | 3,600            | 5                       |

Note: Class “B” loading will have similar specifications to Class “A” loading with the only difference that the axle loads of class “B” shall be 60% of class “A”.

Table 2

| Clear road width | “J” |
|------------------|-----|
| 16'-8” or less   | 0   |
| 16'-8” to 18'-0” | Increasing uniformly from 0 to 1'-4” |
| 18'-0” to 24'-0” | Ditto 1'-4” to 4'-0” |
| Above 24'-0”     | 4'-0” |

Fig. 2. Loading of HS20-44 design truck [4]
Fig. 3. Deck Plan of I-girder Bridge (All dimensions are in mm)

Fig. 4. Transverse section of I-girder Bridge

Detail “A”
2.1. Concrete

Compressive strength of concrete at 28 days for I-girder is 40 MPa; however for abutments, deck slab, diaphragms and barrier, the compressive strength is taken as 26 MPa.

2.2. Steel

Grade 270 steel is used for post-tensioning of the girder with following properties:
- Ultimate tensile strength of \( f_{pu} \) = 1860 MPa
- Elastic Modulus, \( E = 200,000 \) MPa

Yield strength \( f_{py} \) of post-tensioning steel has been taken as 90% of \( f_{pu} \) for low relaxation steel as per AASHTO LRFD Specifications [4]. Grade 60 steel has been used as non-prestressed reinforcement. The profile of the post-tension tendon has been shown in Fig. 5. Each girder has three (03) tendons and each tendon is comprised of twelve (12) strands. The diameter of a strand is 12.5 mm and the cross section area of tendon \( A_{ps} \) is 1185 mm\(^2\). Each tendon has been crossed through the span in a duct having a diameter of 63 mm. Each tendon has been jacked by applying jacking force \( P_i \) of 1530,000 N in each cable which is equivalent of 0.7\( f_{pu} \) at anchorages immediately after anchor set.

![Fig. 5. Details of I-girder](image-url)
2.3. Bridge loading

Dead loads of bridge include the self-weight of the deck slab, girder and diaphragm. Superimposed loads included the weight of the barrier and wearing course (i.e. finishes). 50 mm think wearing course was provided and weight of wearing course was 7.55 MPa. The weight of the barrier was calculated by multiplying the cross-section area of the barrier (320700 mm$^2$) with density of concrete and it is equal to $1.175 \times 10^{-3}$ N/mm (refer Fig. 4 for barrier section).

HS20-44 and class “A” truck are the most common live loads used for the designing of bridges in Pakistan and these loads are being considered in this case study. The vehicular loading for HS20-44 is shown in Fig. 1. Class “A” truck has been shown in Fig. 2. Four load cases have been considered for analysis of live loads:

| Case  | Description                        |
|-------|------------------------------------|
| Case I| HS20-44 (single) as shown in Fig. 1|
| Case II| Class A truck (single) as shown in Fig. 2|
| Case III| Class A truck (convoy) as shown in Fig. 6|
| Case IV| HS20-44 (convoy) as shown in Fig. 6|

For modified vehicular loading class “A” truck without trailers has been used as a convoy and HS20-44 truck convoy on the bridge having distance between trucks 3900 mm as shown in Fig. 6. These loads are being used by considering the truck load pattern and the distance between two vehicles during peak hour.

![Class A truck Convoy](image)

![HS20 truck Convoy](image)

Fig. 6. Modified truck load configuration (Based on Class A and HS20-44 truck)

3. Analysis, Results and Discussion

Analysis and design of the sample I-girder Bridge has been performed by using SAP2000 and according to AASHTO LRFD 2007 specifications. Superstructure of the bridge is analysed by area object method on SAP2000 whereas the stresses and flexural strength has been checked manually. The detail of analysis and design has been presented in Fig. 7.

The analysis has been performed for I-girder Bridge to obtain the effects of all loadings such as dead load, super dead load, prestress load after all losses, HS20-44 truck load, class A truck load, and modified truck load.
configuration (Based on Class A and HS20-44 truck). The analysis results and Stresses check at control points on service stage have been presented in Table 3. Sign convention for compressive stresses is positive whereas tensile stress is taken as negative. The stresses at the top face of the girder at service stage are calculated by using equation:

\[
f_{\text{top}} = \frac{P}{A} + \frac{M_D}{S_T} + \frac{M_{SD}}{S_{T'}} + \frac{1.55 \times M_L}{S_{T''}} - \frac{M_P}{S_T}
\]

The stresses at the bottom face of the girder at service stage are calculated by using equation:

\[
f_{\text{bottom}} = \frac{P}{A} - \frac{M_D}{S_B} - \frac{M_{SD}}{S_{B'}} - \frac{1.55 \times M_L}{S_{B''}} + \frac{M_P}{S_B}
\]

According to AASHTO LRFD 2007, the allowable stresses for I-girder Bridge in the service stage are:

Compressive Stress = \(0.4 f'_c = 18 \text{ MPa}\)

Tensile Stress = \(-0.5 (f'_c)^{1/2} = -3.16 \text{ MPa}\)
Figure 7 (a to g): Details of Inputs For Analysis

Table 3: Analysis Results for I-girder Bridge and Allowable Stress Check at Mid Span

| Geometric properties | Jacking Force after all Losses, N | Moment due to prestressing, Mp, N.mm | Moment due to Dead Load M0, N.mm | Moment due to Super Dead Load Msd, N.mm | Moment due to vehicular Load M1, N.mm | Stress fbottom using Eq. 1, MPa | Stress ftop using Eq. 2, MPa | Remarks |
|----------------------|----------------------------------|------------------------------------|----------------------------------|----------------------------------------|-------------------------------------|-------------------------------|-------------------------------|---------|
| Area of Girder A, mm² | 4590000                          | 3543356                            | 2.669x10⁹                       | 1.7888x10⁹                            | 2.209x10⁹                          | 1.569x10⁹                    | 2.04                          | 8.38    |
| Section Modulus of Girder, mm³ | 4590000                          | 3543356                            | 2.669x10⁹                       | 1.7888x10⁹                            | 2.209x10⁹                          | 1.827x10⁹                    | 0.2                           | 9.72    |
| S₀                        | 134553734.1                      | 4590000                            | 3543356                          | 2.669x10⁹                            | 1.7888x10⁹                          | 2.209x10⁹                    | -3.58                         | 12.3    |
| Sₙ                        | 113471582.2                      |                                    |                                  |                                        |                                    |                               |                               |         |
| Section Modulus of Girder with Deck, mm³ | 4590000                          | 3543356                            | 2.669x10⁹                       | 1.7888x10⁹                            | 2.209x10⁹                          | 2.353x10⁹                    | -1.213                        | 17.1    |
| S₀'                       | 217483749.5                      |                                    |                                  |                                        |                                    |                               |                               |         |
| Sₙ'                       | 310268321.4                      |                                    |                                  |                                        |                                    |                               |                               |         |

By comparing stresses mentioned in Table 3 with the allowable stresses, it is evident that Class A truck without trailers in convoy is over stressing the girders more than allowable. Hence, in case of heavy traffic and traffic congestion during peak hours, the I-girder Bridge shall be over stressed and shall result serviceability problems such as excessive deflection and large crack width. This stress might not cause any damage as there is non-prestressed reinforcement present that is providing additional moment capacity as the flexural capacity of the I-girder has been checked, the nominal flexural resistance of the I-girder for Case III live load can be calculated by Eq. 3 (Eq. 5.7.3.2.2-1 specified in the US code of practice of highway bridges)[4]:

\[
M_n = A_{ps} \times f_{ps} \times \left( d_p - \frac{a}{2} \right) + A_s \times f_s \times \left( d_s - \frac{a}{2} \right) + 0.85 \times f'c \times (b - b_w) \times h'_f \times \left( \frac{a}{2} - \frac{h_f}{2} \right)
\]  

(3)

Where,

\[
a = \frac{T}{0.85 \times f'c \times b} ; T = A_t \times f_y + A_{ps} \times f_{ps} ; T = 6 \times 122.71 \times 414 \times 3 \times 1185 \times 0.95 \times 1860 ; a = \frac{6586496}{0.85 \times 40 \times 1650} ; a = 117.40 \text{ mm}
\]

\[
f_{ps} = f_{p,e} + 900 \left( \frac{d_p - C}{l_e} \right) \leq f_{py}
\]

\[
f_{ps} = 1767 \text{ N/mm}^2
\]
Then, $M_n = 3 \times 1185 \times 1767 \left(1299.25 - \frac{117^2}{2}\right) + 6 \times 122.71 \times 414 \left(1317.5 - \frac{117^2}{2}\right)\]

$M_n = 8.18 \times 10^9 \text{N.mm}, \quad OM_n = 7.36 \times 10^9 \text{N.mm}$

The ultimate moment due to loads on girder can be calculated as:

$M_u = 1.25 \times (M_D + M_{SD}) + 1.75 \times M_L + 1.0 \times M_P; M_u = 1.25 \times (1.788 \times 10^9 + 2.209 \times 10^8) + 1.75 \times 2.353 \times 10^9 + 1.0 \times (-2.669 \times 10^9); M_u = 6.445 \times 10^9 \text{N.mm}$

It has been found that the I-girders have sufficient flexural capacity to withstand against the modified vehicular load. It is important to note that the I-girder Bridge has been checked with the same standard vehicle in the convoy without considering overload. So, the problem would be more critical with the consideration of illegal gross weights and heavy axle loads of vehicles as reported by Fu, G. et al. and Chou, C. J. [5, 6]. To improve the serviceability of I-Girder Bridge, Plate Bonding technique may be used. There are several possible ways of resolving the serviceability problem. Two cases have been considered here:

- Attaching thick plate at the bottom face of the I-girder with the help of epoxy.
- Attaching thick steel plate at the bottom and side of the girder with the help of bolts

As already stated, plate bonding with epoxy has concerns such as brittle failure of epoxy and de-lamination. The second case is more preferable and analysis and design of the plate can be performed by considering the stress-strain diagram as shown in Figure 8. By using the stress-strain relation, Size of plate can be obtained and the schematic diagram of plate bonding technique with bolting for this particular case study has been shown in Figure 9. The plate having length 5532 mm has to attach at the mid span of the girder in which stresses are exceeding from the allowable stress limit.

4. Conclusions and Recommendations

In this case study, a medium span bridge was checked by considering the standard vehicles in convoy for taking the effect of traffic congestion during peak hours and it has been found that the flexural capacity is sufficient enough to withstand against the standard vehicular load in traffic congestion during peak hours. Based on this result, it may comprehend that the medium span bridges of urban areas of Pakistan which are properly designed and detailed based on standard vehicular loads and codes would be last against the traffic congestion during peak hours. It has also been found that the medium span I-girder bridge had a serviceability problem as the resulting
stress in tension side was exceeding the allowable limit of tensile stress. This overstressing may cause excessive deflection and large crack width. In order to strengthen inadequacy of bridge in serviceability, judicious selection of strengthening technique based on the availability of skilled labours, materials and technology is recommended. This study covers flexural and serviceability of Medium span I-girder Bridge, further studies are recommended to check the behaviour of short and long span bridge in flexure, shear and deflection due to overloading and illegal vehicle loads. It is also proposed to check the effect on substructure due to overloading and illegal vehicle loads. In general, there is need to modify the standard vehicular load by considering the vehicular load running on highways of Pakistan rather adopting the HS20-44 vehicular load mentioned in the US code of practice of highway bridges.

References

[1] http://downloads.nha.gov.pk/index.php?option=com_content&view=article&id=28:axle-load-control&catid=56:services&Itemid=58. , 19 Nov., 2013
[2] Ali, S., Javed, M., Alam B., 2012. A comparative study of live loads for the design of highway bridges in Pakistan. IOSR Journal of Engineering, vol. 2, p. 96.
[3] West Pakistan Highway Department, 1967. Code of practice of highway bridges, Government of West Pakistan Highway Department, Lahore, Tech. Rep. 141.
[4] AASHTO LRFD, 2007. Bridge design specifications, American Association of State and Transport Official.
[5] Fu, G., Hag-Elsafi, O., 2000. Vehicular loads: Load model, bridge safety, and permit checking,” J. Bridge Eng., vol. 5, p. 49-57.
[6] Chou, C. J., 1996. Effect of overloaded heavy vehicles on pavement and bridge design, Transportation Research Record: Journal of the Transportation Research Board, vol. 1539, p. 58-65.
[7] http://www.clevelandbridge.com/sectors.aspx#2, 12 Aug., 2013
[8] Cairns, J., Rafeeqi, S., 1997. Behaviour of reinforced concrete beams strengthened by external unbonded reinforcement. Construction and Building Materials, vol. 11, p. 309.
[9] Harajli, M., 1993. Strengthening of concrete beams by external prestressing. PCI Journal, vol. 38, p. 76.
[10] Khan, A., Ayub, T., 2009. Performance of RC beams strengthened in shear by externally bonded U-shaped wraps, SBEIDCO – 1st International Conference on Sustainable Built Environment Infrastructures in Developing Countries ENSET Oran, Algeria, pp. 151-158.
[11] Khan, A., Ayub, T., 2010. Effectiveness of U-shaped CFRP Wraps as End Anchorages in Predominant Flexure and Shear Region, CICE 2010 - The 5th International Conference on FRP Composites in Civil Engineering, Beijing, China.
[12] Rafeeqi, S., Khan, S., Lodhi, S., 2012. Performance of ferrocement as flexural strengthening in rural areas, 10th International Symposium on Ferrocement and Thin Reinforced Cement Composites (Ferro10), Havana, Cuba pp. 213-221.
[13] Khan, S., Nuruddin, F., Ayub, T., Shafiq, N., 2013. Effects of ferrocement in strengthening the serviceability properties of reinforced concrete structures. Advanced Materials Research Journal, vol. 690, p. 686.
[14] Van Gemert, D., 1981. Repairing of concrete structures by externally bonded steel plates, Proceedings ICP/RILEM/IBK International Symposium on Plastics in Material and Structural Engineering, pp. 519-526.
[15] Täljsten, B., 1997. Strengthening of beams by plate bonding. Journal of Materials in Civil Engineering, vol. 9, pp. 206.
[16] Rafeeqi, S., Khan, S., Zafar N., Ayub, T., 2012. Implication of unbondedness in reinforced concrete beams. Advanced Materials Research Journal, vol. 587, p. 36.