IMPACT FACTOR OF A PRESTRESSED SCC BRIDGE THROUGH DYNAMIC LOAD TESTS

Eli S. Hernandez, PhD Candidate, Department of Civil, Architectural and Environmental Engineering, Missouri University of Science and Technology, Rolla, MO
Yi Bao, PhD Candidate, Department of Civil, Architectural and Environmental Engineering, Missouri University of Science and Technology, Rolla, MO
John J. Myers, PhD, PE, Department of Civil, Architectural and Environmental Engineering, Missouri University of Science and Technology, Rolla, MO
Genda Chen, PhD, PE, Department of Civil, Architectural and Environmental Engineering, Missouri University of Science and Technology, Rolla, MO

ABSTRACT

The load carrying capacity of a bridge structure can be effectively assessed by means of a field load testing. An important parameter used to determine the load rating of a bridge structure is the dynamic load allowance (impact factor). Bridge A7957is the first bridge superstructure implementation built by the Missouri Department of Transportation (MoDOT) employing normal-strength self-consolidating concrete (NS-SCC) and high-strength self-consolidating concrete (HS-SCC). The aim of this study was to evaluate the impact factor of Bridge A7957 obtained by experimental and analytical methods. To achieve this goal, Bridge A7957 was instrumented with accelerometers at different span locations. For different dynamic load tests, the dynamic response of each span was recorded with the accelerometers and a laser vibrometer. The impact factor was computed using three different design codes and was compared to field measured impact factors. It was found that the impact factors estimated with the design codes provided conservative values regarding the structural response of the bridge under experimental dynamic loads. This difference has direct implications on the rating factor of a bridge.

Keywords: Dynamic Load Allowance, Impact Factor, Self-consolidated concrete, SCC.

INTRODUCTION

Load rating is the strength evaluation procedure employed to obtain the load carrying capacity a bridge structure can withstand without suffering damage or undergoing collapse. This evaluation is a major basis in prioritizing maintenance operations, allocating economic resources, and making decisions concerning load posting and permit decisions. Traditionally, bridge evaluation standards[1] provide two approaches to load rating, namely analytical calculations and field testing. Analytical ratings are based on simplifying assumptions and may not estimate a realistic response of a bridge due to its current physical condition. Conversely, field testing presents a more realistic visualization of the live-load capacity of a bridge because it provides an in-service, as-built characterization of the bridge's performance. Although field testing applications may sometimes be hindered by costs, traffic interruptions, safety, and difficulty to access a bridge to install sensors, it is the most accurate approach.

Field tests have largely confirmed reserves of strength capacity in existing bridges [especially in precast, prestressed (PC/PS) concrete bridges] despite their visual condition and age. Sources that explain these differences are diverse and may be attributed to several field parameters that are not considered during the design or strength evaluation of a bridge's structure. One of these parameters[1, 2] that can be verified by means of field testing is the impact factor (*IM*) which has also been referred to as the dynamic load allowance (*DLA*). The impact factor, as specified in most design codes, considers the dynamic effects applied to a bridge structure by increasing in some fraction the magnitude of the static live load. A precise estimation of the impact factor results in safe and efficient design of new bridges and provides more rational ratings of existing bridges. The complex nature of the factors affecting the dynamic load allowance makes it difficult to estimate its value accurately during the design and strength evaluation process of a bridge[3].

The main objective of this study was to obtain the dynamic load allowance of Bridge A7957 analytically and experimentally as an attempt to quantify differences between both approaches as they are employed in bridge design and evaluation. To achieve this goal, the dynamic response of Bridge A7957's exterior spans was recorded with accelerometers and a laser vibrometer. The *IM* or *DLA* was computed using the analytical provisions given by the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification[4], the LRFD Bridge Design Specification[5], and the Ontario Highway Bridge Design Code (OHBDC)[6]. The experimental impact factor was obtained by comparing the recorded dynamic and static response of the bridge. The following sections present details about the instrumentation plan, and the static and dynamic tests conducted on Bridge A7957 to evaluate its initial in-situ dynamic response.

DETERMINATION OF IMPACT FACTOR

For analytical calculations, the impact factor has been traditionally proposed in design codes as a function of the span length or the fundamental frequency of the bridge. The AASHTO Standard Specifications[4] presents an equation that is expressed in terms of the span length as follows:

$$IM = \frac{15.24}{L + 38} \leq 0.30 \tag{1}$$

Where L = span length (m). In 1994, the AASHTO LRFD Bridge Design Specifications[5] replaced the term impact factor used in the AASHTO Standard Specifications[4] with the term dynamic load allowance. A *DLA* value, independent of the span length, was adopted as equal to 0.33 (33%) for bridge components other than deck joints. In 1983, the Ontario Highway Bridge Design Code (OHBDC) presented a method to estimate the *DLA* in terms of the fundamental frequency of the bridge structure[6] as illustrated in Fig. 1.

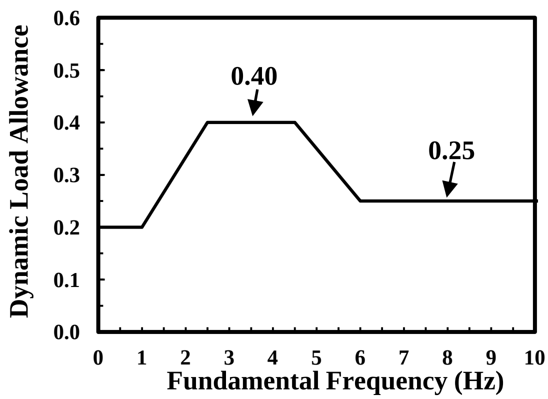


Fig. 1. Dynamic load allowance (*DLA*) vs. fundamental frequency[6, 7]

The experimental estimation of the dynamic *IM* can be achieved by using several definitions proposed in the literature[8]. The value of *IM* is commonly defined as the ratio of the maximum dynamic and static responses regardless of whether the two maximum responses occur simultaneously[9]. Eq. (4) presents this definition:

$$IM = DLA = \frac{R_{dyn} - R_{sta}}{R_{sta}} \tag{2}$$

Where R_{dyn} = maximum dynamic response; and R_{sta} = maximum static response. The estimation of the static response can be obtained by[7, 9]: (1) conducting a quasi-static test where vehicles move across the bridge at a low speed between 5-16 km/h (3-10 mi/h); (2) filtering the measured dynamic response with a low-pass filter to eliminate the dynamic components of signal; and (3) using finite element models (FEM) to calculate the static response when the vehicle weight and loading position are known. In this study, the first option was employed to estimate Bridge A7957's impact factor.

MISSOURI BRIDGE A7957

Bridge A7957 is a three-span, continuous, PC/PS concrete bridge with a 30-degree skew angle (Fig. 2). Each span has PC/PS concrete Nebraska University 53 (NU53) girders[10, 11] [Fig. 2 (b)]. The bridge was built between the summer and fall of 2013 and spans the Maries River in Osage County, Missouri. The first span's girders are 30.48 m (100 ft) long and fabricated with conventional concrete (CC), identified as MoDOT's Class A mixture, with a target compressive strength of 55.2 MPa (8,000 psi). The second span's girders are 36.58 m (120 ft) and were fabricated with high-strength self-consolidating concrete (HS-SCC) of 68.9 MPa (10,000 psi). Girders in the third span are 30.48 m (100 ft) long and employ normal-strength self-consolidating concrete (NS-SCC) with a design compressive strength of 55.2 MPa (8,000 psi). PC/PS concrete panels with a target compressive strength of 55.2 MPa (8,000 psi) span between the girders' top flange underneath the cast-in-place (CIP) reinforced concrete (RC) slab deck in the transverse direction[11] [Fig. 2 (b)]. The CIP deck was cast with a fly ash mixture using 25% replacement of portland cement (MoDOT's Class B-2 modified) and nominal strength of 27.6 MPa (4,000 psi). The superstructure is supported by two abutments and two intermediate bents [Fig. 2(a)] with nominal compressive strength of 20.7 MPa (3,000 psi). The abutments and intermediate bent 2 were cast with a concrete mixture having a 20% fly ash replacement of portland cement (MoDOT's class B). Intermediate bent 3 was built with HVFAC using a 50% fly ash replacement of portland cement with a target compressive strength of 20.7 MPa (3,000 psi).

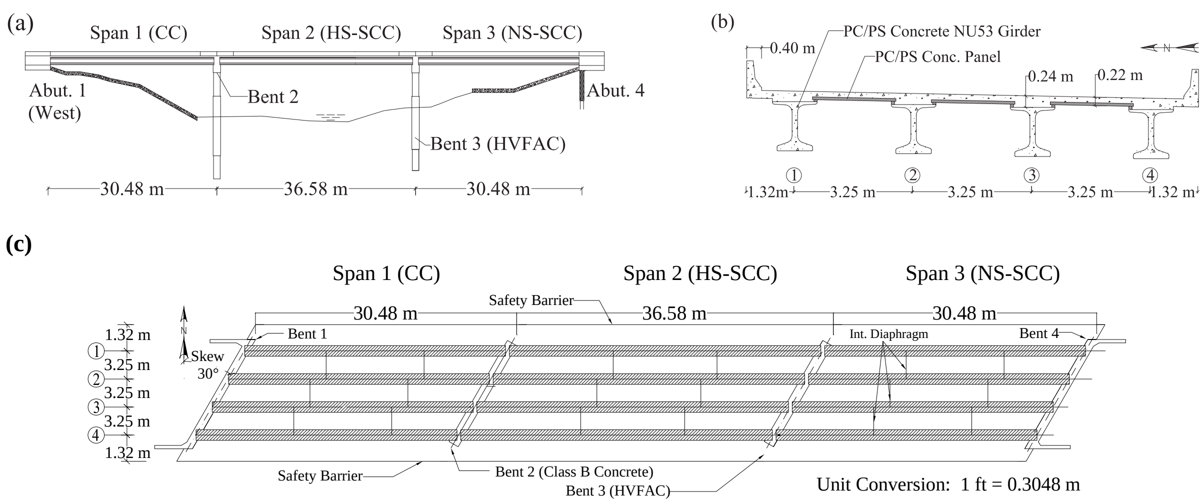


Fig. 2. Bridge A7957. (a) Elevation. (b) Cross-section. (c) Plan view

TESTING EQUIPMENT

The instrumentation program was planned to record: (1) the static vertical deflection at midspan of girders 1-4 (spans 1-2 and 3-4); (2) the dynamic deflection at midspan of girder 3 (spans 1-2 and 3-4); and (3) girder 3 and 4's acceleration at midspan (Fig. 3). The type of sensors employed and details of their installation are described in the next subsections.

AUTOMATED TOTAL STATION (ATS)

An automated total station (ATS), Leica TCA 2003, was employed to record the girders' deflection during the static test conducted on spans 1-2 and 3-4. The ATS recorded the coordinates of targets (prisms) placed at the exterior-span girders' bottom flange (midspan sections) as illustrated in Figures 3 and 4. The ATS has an accuracy of 1 mm ± 1 ppm (distance measurements) and 0.5 arc-seconds (angular measurements). The accuracy of the ATS has been proven to be 0.1 mm (0.008 in.) in vertical deflections measurements as estimated in previous studies[12, 13].

ACELEROMETERS

A total of 6 accelerometers were deployed on Bridge A7957 to record the vertical acceleration of PC/PS concrete girders 3 and 4 at each span's midspan (Fig. 3). Figures 3 and 4 show details of the accelerometers mounted to the girders' bottom flange (midspan sections).

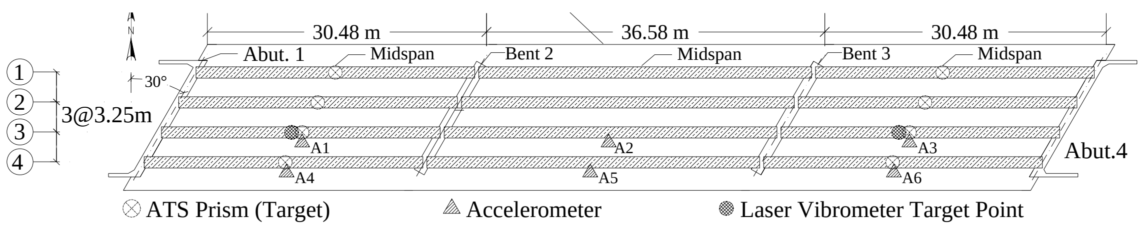


Fig. 3. Bridge A7957 instrumentation layout

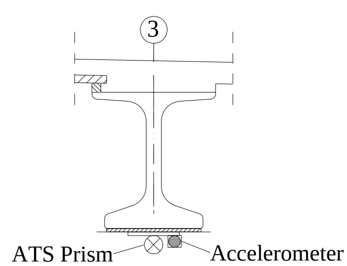


Fig. 4. Girder 3's instrumentation detail (midspan location)

REMOTE SENSING VIBROMETER (RSV-150)

A Remote Sensing Vibrometer RSV-150 (Fig. 5) was employed to record the dynamic bridge response (vertical deflection) of exterior spans' girder 3 (midspan sections). The RSV-150 has a bandwidth up to 2MHz for nondestructive test (NDT) measurements and can detect the vibration and displacement of distant structures.

Fig. 5. Remote sensing vibrometer (RSV-150)

FIELD TESTS PROCEDURE

Large-scale static and dynamic tests were conducted on Bridge A7957. The following subsections describe the test procedure and load configurations planned to obtain the maximum static and dynamic response of the bridge.

STATIC LOAD TEST

For this study, static load tests were conducted on Bridge A7957's exterior spans. One MoDOT H20 dump truck was used to cause maximum static response of the bridge superstructure. Quasi-static tests were carried out by passing the truck at a crawl speed of 16 km/h (10 mi/h). The maximum quasi-static response was compared to the static response obtained with the ATS. The average trucks' dimensions and weight (as reported by MoDOT personnel) are shown in Fig. 6. Figure 7 shows the truck load applied to spans 1-2 and 3-4 during the static tests.

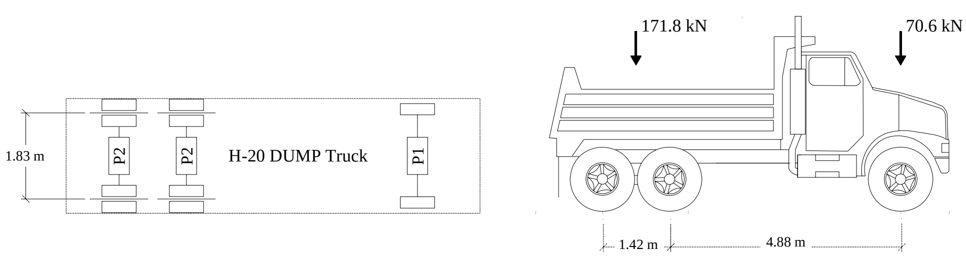
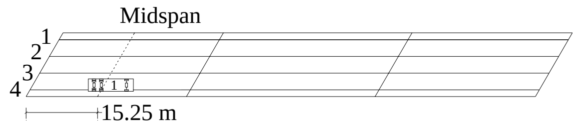
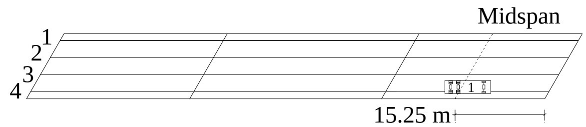


Fig. 6. MoDOT H20 truck employed during static and dynamic tests. Conversion factor: 1 m = 3.28 ft; 1 kN = 0.2248 kip



(a) Static Test 1 (Span 1-2)



(b) Static Test 2 (Span 3-4)

Fig. 7. Static test configurations. Conversion factor: 1 m = 3.28 ft

DYNAMIC LOAD TEST

Varying speeds ranging from 16 km/h (10 mi/h) to 96.6 km/h (60 mi/h) were employed during the dynamic load tests. For each test, the truck speed was increased at a rate of 16 km/h (10 mi/h) until the maximum speed was reached. The maximum dynamic response was compared to the maximum static response to obtain the experimental impact factor. Experimental data was collected at a sampling rate of 120 Hz, and the truck was driven over the south lane (west-east and east-west directions) of the bridge separated 0.60 m (2 ft) from the safety barrier's edge.

LOAD TEST RESULTS

The girders' vertical deflection obtained at midspan of exterior spans 1-2 and 3-4 (Fig. 3) are presented in Fig. 8.

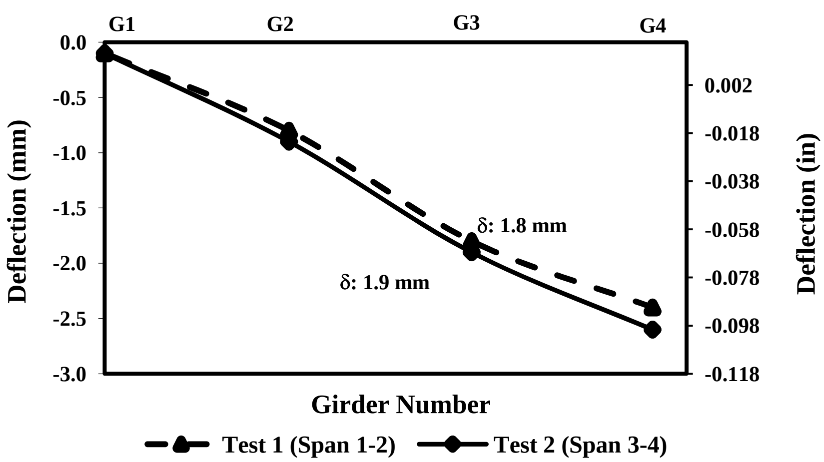


Fig. 8. Girder's vertical deflection at midspan (spans 1-2 and 3-4)

Although both exterior spans have the same geometry (Fig. 2), and these were subjected to the same test load, a 5% difference was observed between the deflections response recorded with the ATS at both spans' midspan. The difference might be attributed to two possible sources: first, a slight variation on the application of the truck load on each span; and second, the accuracy of the automated total station might have affected the measured deflections due to the low level of load applied during the test. This difference might corrected in future tests by taking caution regarding the location of truck loads. In addition, the level of load applied needs to high enough so that the error of ATS' measurements is kept low during data collection.

Figure 9 shows the measured acceleration response recorded from sensor A1 (span 1-2) deployed at girder 3's midspan when the truck was driven from west to east at 96 km/h (60 mi/h). The fundamental frequency was obtained from Fast Fourier Transformations (FFT) of the measured acceleration data. Figure 10 presents the fundamental frequency estimated from a FFT analysis conducted on the acceleration response data shown in Fig. 9. The estimated fundamental frequency corresponds to a value of 3.125 Hz. According to the OHBDC[6], this fundamental frequency yields an impact factor equal to 0.40 (see Fig. 1).

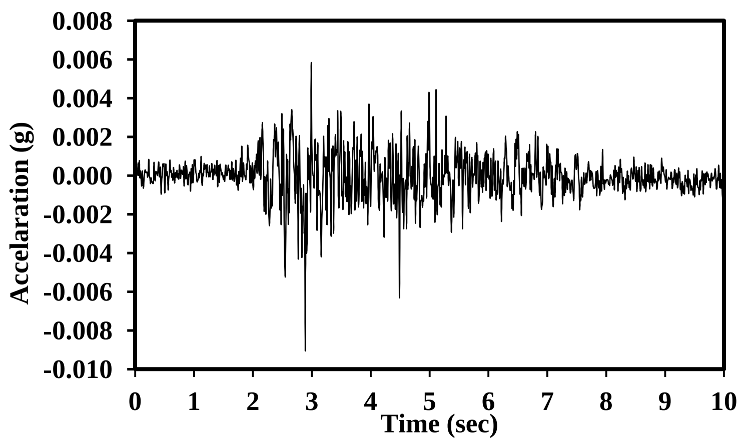


Fig. 9. Measured acceleration response (96 km/h, west-east direction)

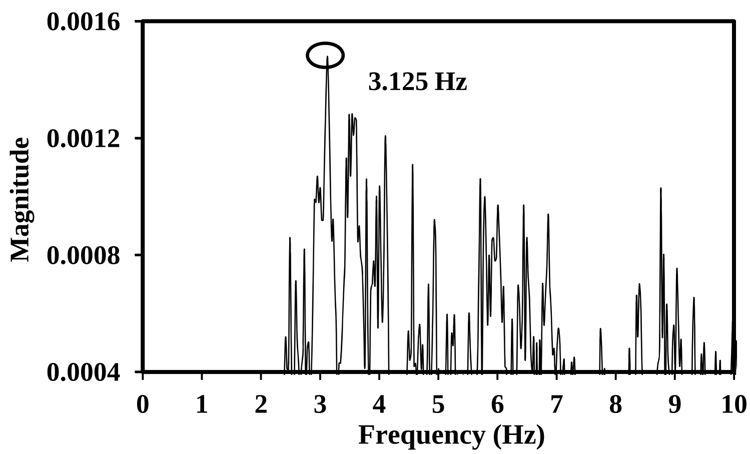


Fig. 10. Natural frequency extracted through FFT

The girder 3's static deflection value at midspan (Fig. 8) was compared to the quasi-static response of the bridge obtained by passing the test truck at a crawl speed [16 km/h (10 mi/h)] over the bridge. This comparison was done to verify if the quasi-static deflection was representative of the bridge's static response and could be used to compute the impact factor. Figure 11 shows that girder 3's quasi-static deflection (collected at crawl speed) was close to the static deflection collected with the ATS (1.77 mm vs. 1.80 mm). Therefore, the filtered quasi-static response of the spans was considered the maximum static deflection value at girder 3's midspan and was used to compute Bridge A7957's impact factor for the different speeds used to test Bridge A7957's superstructure. In addition, Fig. 11 shows the maximum dynamic deflection collected with the Remote Sensing Vibrometer RSV-150 when the truck's was passed over the bridge at speed of 96 km/h (60mi/h).

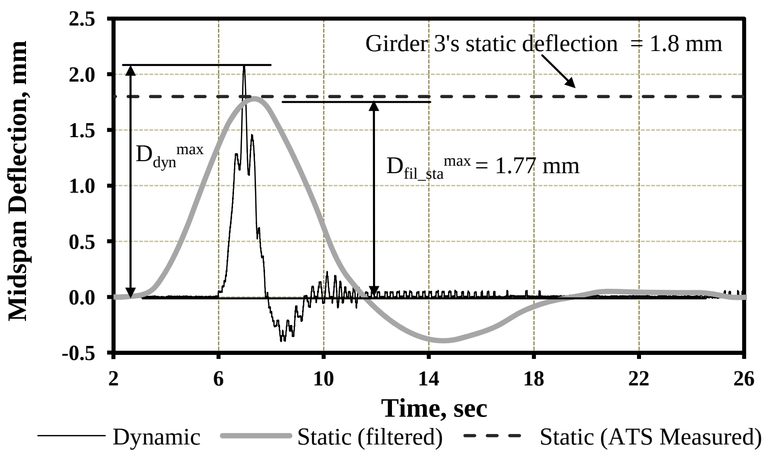


Fig. 11. Maximum static and dynamic deflections. Conversion factor: 25.4 mm = 1.0 in.

The experimental impact factor was computed with Eq. (3). Where IM^{exp} = DLA^{exp} = experimental impact factor; D_{dyn}^{max} = maximum measured dynamic vertical deflection (mm); and D_{sta}^{max} = maximum static deflection obtained from passing the test truck at a crawl speed over the bridge (mm).

$$IM^{exp} = DLA^{exp} = \frac{D_{dyn}{}^{max} - D_{sta}{}^{max}}{D_{sta}{}^{max}} \tag{3}$$

Table 1 presents Bridge A7957's average dynamic and static maximum deflection values collected at different speeds during the dynamic tests (see rows 2 and 3). In addition, the experimental impact factors corresponding to different truck speeds are listed in row 4. The experimental dynamic amplification factor, *DAF* was estimated with Eq. (4).

$$DAF^{exp} = (1 + IM) \tag{4}$$

Where DAF^{exp} = dynamic amplification factor. The dynamic load allowance obtained with the AASHTO LRFD Bridge Design Specifications[5] and the impact factor computed with the AASHTO Standard Specifications[4] are listed in Table 1 (rows 6-8). Finally, the dynamic load allowance estimated as a function of the bridge's fundamental frequency (OHBDC[6] approach) is presented in Table 1 (row 9). The maximum value of the experimental impact factor corresponds to 0.175 and was produced when the test truck was driven over the bridge at a maximum speed of 96 km/h (60 mi/h) (see Table 1 and Fig. 12).

Table 1. Experimental and analytical impact factor.

Speed (km/h)	10	20	30	40	50	60
D_{dyn}^{max} (mm)	1.77	1.79	1.79	1.77	2.03	2.08
$D_{fil\_sta}^{max}$ (mm)	1.77	1.77	1.77	1.77	1.77	1.77
IM^{exp}	0.000	0.010	0.010	0.000	0.150	0.175
DAF^{exp}	1.000	1.010	1.010	1.000	1.150	1.175
DLA (AASHTO LRFD[5])	0.33	0.33	0.33	0.33	0.33	0.33
IM (AASHTO Standard[4]) *	0.222	0.222	0.222	0.222	0.222	0.222
IM (AASHTO Standard[4]) †	0.204	0.204	0.204	0.204	0.204	0.204
DLA (OHBDC[6])	0.40	0.40	0.40	0.40	0.40	0.40

Conversion factor: 10 mi/h = 16 km/h. * Span 1-2. † Span2-3

By comparing the maximum impact factor obtained experimentally to the values obtained analytically using the design specifications described above, it was observed that the three analytical methods provide conservative values of the impact factor. These results are in agreement with results reported by Hag-Elsafi[14] who found an impact factor value slightly less than 0.16 when a similar dynamic test was conducted on a single span bridge. Hag-Elsafi conducted a dynamic test using an HS-20 truck [with a weigh of 320 kN (71.94 kip)] passed over a steel truss bridge at a speed of 96 km/h (60 mi/h).

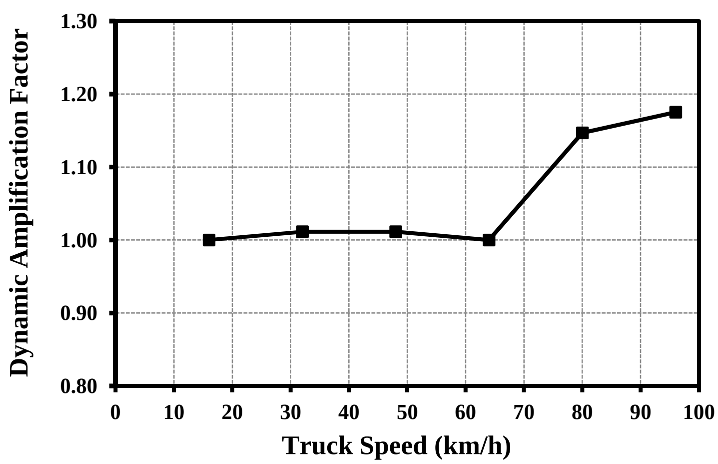


Fig. 12. Effect of speed on *DAF*. Conversion factor: 10 mi/h = 16 km/h

It is important to note that the specifications used to estimate the analytical values of the impact factor[4] or dynamic load allowance[5, 6] propose methodologies that are used for design and evaluation of highway bridges. These specifications tend to be conservative when are used to assess the in-situ dynamic response of existing bridge structures for which field tests have demonstrated to be more appropriate. The difference between the analytical and experimental impact factor has direct implications on the evaluation of bridge structures and might yield to conservative values of a bridge load rating.

CONCLUSIONS

The first series of large-scale static and dynamic load tests was conducted on Bridge A7957 to monitor its initial in-service dynamic response. The impact factor is an important parameter that depends on particular in-situ conditions of a bridge structure. The impact factor (*IM*) or dynamic load allowance (DLA) of Bridge A7957 was obtained from field measurements and using three design specifications. The impact factors obtained with the design specifications resulted in larger values compared to the experimental values. Reasons that explain the difference might be attributed to several in-situ factors not consider by the analytical methods proposed by current design codes. Bridge design specifications intend to estimate the value of the impact factor based on several assumptions as an attempt to cover a large spectrum of bridges fabricated with different materials, span lengths and particular field conditions. The impact factors obtained from field load tests implicitly take into account in-situ parameters such as unintended support constraints and continuity, skew angle, contribution of secondary members and interaction soil-structure which can contribute to improve the bridge's dynamic response behavior. More research needs to be conducted to isolate the influence of these in-situ parameters on the dynamic response of a bridge structure.

ACNOWLEDGEMENTS

The authors gratefully acknowledge the financial support provided by the Missouri Department of Transportation (MoDOT) and the National University Transportation Center (NUTC) at Missouri University of Science and Technology.

REFERENCES

1. American Association of State Highway and Transportation Officials (AASHTO, 2010). *Manual for Bridge Evaluation (2nd Edition) with 2011, 2013, 2014 and 2015 Interim Revisions*, Washington, DC.
2. Cai, C.S. and Shahawy, M., "Understanding Capacity Rating of Bridges from Load Tests," *Practice Periodical on Structural Design and Construction*, V. 8, No. 4, Nov. 1, pp. 209-216.
3. Barker, R.M. and Pucket, J.A. (2013), "Design of Highway Bridges: An LRFD Approach," ed. Hoboken, N.J., John Wiley & Sons.

4. American Association of State Highway and Transportation Officials (AASHTO, 1992). *AASHTO Standard Specifications for Highway Bridges*, Washington, DC.
5. American Association of State Highway and Transportation Officials (AASHTO, 2012). *AASHTO LRFD Bridge Design Specifications, U.S. Customary Units (6th Edition)*, Washington, DC.
6. Ontario Ministry of Transportation and Communications (OMTC) (OMTC, 1983). *Ontario Highway Bridge Design Code (2nd Edition)*, Downsview, ON, Canada.
7. Paultre, P., Chaallal, O., and Proulx, J., "Bridge Dynamics and Dynamic Amplification Factors: A Review of Analytical and Experimental Findings," *Canadian Journal of Civil Engineering*, V. 19, No. 2, pp. 260-278.
8. Bakht, B. and Pinjarkar, S.G., "Dynamic Testing of Highway Bridges. A Review," *Transportation Research Record*, V. No. 1223, pp. 93-100.
9. Deng, L., Yu, Y., Zou, Q., and Cai, C.S., "State-of-the-art review of dynamic impact factors of highway bridges," *Journal of Bridge Engineering*, V. 20, No. 5, pp.
10. Hernandez, E.S. and Myers, J.J., "Use of Self-consolidating Concrete and High Volume Fly Ash Concrete in Missouri Bridge A7957," *Sustainable Performance of Concrete Bridges and Elements Subjected to Aggressive Environments: Monitoring, Evaluation and Rehabilitation*, V. SP 304, No. 6, pp. 85-100.
11. Hernandez, E.S. and Myers, J.J., "Field Load Test and Girder Distribution Factors of Missouri Bridge A7957," *Proc.*, *2016 PCI Convention and National Bridge Conference*, Nashville, TN, March 2016, 2016.
12. Merkle, W.J. and Myers, J.J., "Use of the Total Station for Load Testing of Retrofitted Bridges with Limited Access," *Proc.*, *Smart Structures and Materials 2004 - Sensors and Smart Structures Technologies for Civil, Mechanical, and Aerospace Systems*, San Diego, CA, 2004.
13. Hernandez, E.S., Galati, N., Myers, J.J., and Nanni, A., "Long Term Monitoring of Bridges Using a Robotic Tacheomety System," *Proc.*, *NDE Conference on Civil Engineering*, St. Louis, MO, August 2006, 2006.
14. Hag-Elsafi, O., Albers, W.F., and Alampi, S., "Dynamic Analysis of the Bentley Creek Bridge with FRP Deck," *Journal of Bridge Engineering*, V. 17, No. 2, March, pp. 318-333.