기지하중을 받는 교량구조물의 현장 계측 및 해석에 따른 응력분포 연구

Analytical and Field Investigation of Bridge Stress Distribution under Proof Load

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Abstract

The objective of the presented study is to develop an efficient procedure of proof load testing for existing bridges. By analytical methods, some of these bridges are not adequate to carry normal highway traffic. However, the actual load carrying capacity is often much higher than what can be determined by conventional analysis. Proof load testing can reveal the hidden strength reserve and thus verify the adequacy of the tested bridge. Proof load level required for meaningful tests should be sufficiently higher than legal load. In the state of Michigan, the legal 11-axle truck can weigh up to 685 kN. In this study, a combination of two military tanks and two Michigan 11-axle trucks was used. The proof loads were gradually increased to ensure the safety of the test. After each move, measurements were taken. For the considered bridge, stress levels were rather low compared to pre-test analysis results. This is due to incorrect material strength, structural contribution of nonstructural components such as parapets and railings, and partially fixed supports.

요 지

본 연구는 기지하중을 이용하여 교량의 효율적 평가를 하기 위한 것이다. 계산상 내하력이 부족한 것으로 평가된 교량의 하중저항능력은 기존의 방법에 의한 평가능력을 상회하는 경우가 일반적이다. 기지하중을 이용한 실험은 미지의 저항능력을 평가할 수 있으며, 따라서 대상교량의 하중저항능력을 정확히 검증할 수 있다. 실험을 위한 기지하중은 일반적인 통행하중보다 큰 것을 사용하여야 한다. 따라서 본 연구에서는 미시간주에서는 법적으로 허용된 11축 트럭 하중(685kN)을 고려하여, 두 대의 군용탱크와 두 대의 11축 트럭을 이용하였으며, 실험의 안전을 위하여 재하하중의 크기를 점진적으로 증가하면서 계측을 실시하였다. 실험에 의한 대상 교량의 응력수준은 계산에 의한 값보다 다소 작게 측정되었으며, 이는 부적절한 재료강도의 예측, 파라펫이나 가드레일과 같은 비구조요소의 기여, 지지조건의 변화 등에 기인한 것으로 분석되었다.

Keywords : Proof Loads, Bridges, Diagnostic Tests, Live Load

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1. Introduction

The objective of the presented study is to develop an efficient procedure of proof load testing for existing bridges. Proof load test is particularly important for bridges which are difficult to evaluate by analysis (missing drawings, visible signs of deterioration such as cracking, corrosion and/or spalling concrete).

By analytical methods with conservative assumptions, some of those deteriorated bridges are not adequate to carry normal highway traffic. However, the actual load carrying capacity is often much higher than what can be determined by conventional analysis, due to more favorable load sharing, higher material properties than assumed, partial fixity of bearing restraints, unintended composite action, effect of non- structural components (parapets, railing, sidewalks), and other factors that are difficult to quantify (Bakht, B. and Jaeger, L.G., 1990). Some of these favorable structural effects may disappear at higher load levels. However, by proof load testing, such extra safety reserve in the load capacity can be utilized to prove that the bridge is adequate, thus avoiding replacement or rehabilitation.

Proof load level required for meaningful tests should be sufficiently higher than legal load. In the state of Michigan, the legal 11-axle truck can weigh up to 685 kN. Therefore, it is difficult to find a vehicle heavy enough to be used in bridge tests. In this study, a combination of two military tanks and two Michigan 11-axle trucks was used. Each M-1 tanks weighs about 533 kN over the length of 4.6 m.

The proof loads were gradually increased to ensure the safety of the test. After each move, measurements were taken. Increased stress/strain level, or any nonlinearity of response, were considered as indication of inadequate strength.

2. Required Proof Load Level

The proof load level should be sufficiently higher than that from legal maximum truck loads, to ensure the desired safety level. A.G. Lichtenstein(1998) provides guidelines for calculating the target proof load level. It suggests that the maximum allowable legal load should be multiplied by a factor Xp, which represents the live load factor needed to bring the bridge to an operating rating factor of 1.0. A.G. Lichtenstein(1998) recommends that Xp should be 1.4 before any adjustments are made.

It also recommends the following adjustments to Xp, that should be considered in selecting a target live load magnitude: 1) Increase Xp by 15 percent for one lane structures or for other spans in which the single lane loading augmented by an additional 15 percent would govern, 2) Increase Xp by 10 percent for spans with fracture critical details, 3) Increase Xp by 10 percent for structures without redundant load paths, 4) Reduce Xp by 5 percent if the structure is ratable, that is, there are no hidden details, and if the calculated rating factor exceeds 1.0. Application of the recommended adjustment factors, leads to the target live load factor Xpa. The net percent increase in Xp is found by summing the appropriate adjustments given above.

Then

$$Xpa = Xp(1+\Sigma/100) \quad (1)$$

The target proof load (Lt) is then:

$$L_t = X_{pa}(1+DLF)L_r \quad (2)$$

$$1.3 \le X_{pa} \le 2.2 \quad (3)$$

where, Lr is the comparable live load due to the rating vehicle for the loaded lanes, DLF is dynamic load factor, and Xpa is the target live load factor.

Based on the span length, the AASHTO Standard Specifications(2000) specifies the dynamic load factors of less than 0.3. AASHTO LRFD Specifications(1998) specifies dynamic load factors as a constant value of 0.33 for truck load only. However, previous studies by several researchers have indicated that the dynamic load factor is much smaller for heavy loads(Hwang, and Nowak, 1991, Nassif and Nowak, 1995, Nowak et al, 1994). Therefore, for this study, a dynamic load factor of 0.1 was selected in calculation of the proof load level for the selected bridge. The load distribution test prior to the proof load test on this bridge also confirmed that for the most heavily loaded girder (girder No.3), the dynamic load factor does not exceed 0.1 under two trucks side-by-side loading.

3. Selected Bridge for Instrumentation

This bridge was built in 1970. As shown in Fig. 1, there is one lane in each direction. It has five steel girders spaced at 2.82 m. It is a simply supported, composite structure. The length of the tested span is 29.8 m. There is no skew. The bridge has a marginal load rating factor of 1.01, according to the Michigan Structure Inventory. Remountable strain transducers with Wheatstone full bridge circuitry were installed on the bottom flanges of girders at midspan, near selected supports, and between the transverse diaphragms at the centerspan.

The Wheatstone full bridge circuit configuration can eliminate temperature effect. The strain transducers are reusable, and they were calibrated in the lab prior to the bridge test, to ensure the accuracy of the test. LVDT's were also installed at midspan to ensure the safety during the bridge test by monitoring the deflection in real time. Significant efforts were directed to the strain measurements for the evaluation of the bridge strength limit state. The service limit state of the tested bridge was not evaluated in the study.

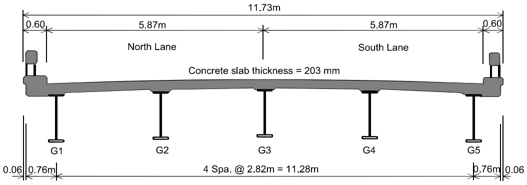


Fig. 1 Cross-Section of Bridge NDN/I69

4. Proof Load Selection

The proof load level should be sufficiently higher than that from legally allowed trucks, to ensure the desired level of safety. It is therefore very difficult, if not impossible, to achieve required proof load level using legal trucks.

The selection of proof load can be a form of concrete block, steel coils, or even building a water tank on top of the bridge. The type of proof load is not important, as long as the required proof load can be achieved. However, it is important that the proof load should be gradually increased. This is because the change of bridge behaviors should be monitored on sight to ensure the safety of proof load test during. the load application

A combination of two M-1 A1 military tanks plus two, fully loaded 3-unit 11-axle trucks were selected as proof load for this study. The tanks were provided by the Michigan National Guard. The maximum lane moment on the selected span (29.8 m) due to legal load (L_r) is 3,766 kN·m, and L_t is determined to be 1.46 according to the guidelines provided by A.G. Lichtenstein. Therefore, the target proof load is decided to be 5,510 kN·m for the considered bridge.

Each M-1 tank used in the proof load test weighs 533 kN (The load is distributed over a track length of 4.6 m). The front and side views of a M-1 tank are shown in Fig. 2.

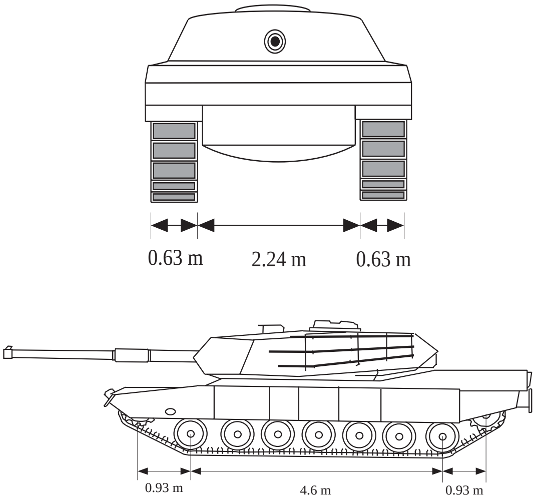


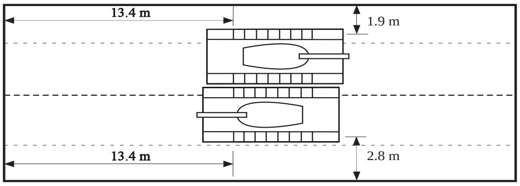
Fig. 2 Front View and Side Elevation of M-1

Table 1 Lane Moment due to Trucks and Tanks for 29.8 m span

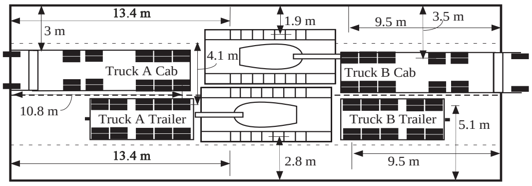
Load Type	Maximum Lane Moment
One 11-Axle Test Truck only	3568 kN·m
One Tank only	3679 kN·m
One Tank + One 11-axle Truck (Truck Detached and Positioned to create maximum moment)	5903 kN·m

Each 11-axle truck used in the test weighs about 688 kN, which is just above maximum legal weight (685 kN). The actual axle weights of the 11-axle test trucks were measured at the weigh stations prior to the test. The trailers of 11-axle trucks were detached from the cabs and positioned separately to cause the maximum bending moment.

Table 1 shows the maximum lane moments caused by the trucks and tanks. The loads were gradually increased to ensure the safety of the proof load test. 16 runs were performed with incremental loading. Detailed proof load position for the most heavily loaded run is shown in Fig. 3.



(a) Run 11



(b) Run 16

Fig. 3 Examples of Detailed Proof Load Positions

5. Pre-Test Analysis

The structural response was constantly monitored, and compared with the pre-calculated analytical results during the proof load test, to verify if the response was within acceptable range to avoid accidental overloading or excessive deformation of the structure.

A preliminary analysis was performed based on the planned loading according to the parameters specified in AASHTO Specifications(2000, 2998, 1994), to calculate the expected maximum strain and the deflection values.

Also, a three-dimensional finite element method (FEM) was applied to investigate the structural behavior of the tested bridge. The concrete slab was modeled with isotrophic, eight node solid elements, with three degrees of freedoms at each node. The girder flanges and web were modeled using three-dimensional, quadrilateral, four node shell elements with six degrees of freedom at each node. The structural effects of the secondary members, such as the sidewalk and parapet, were also taken into account in the finite element analysis models.

Two cases of the boundary conditions were employed in the FEM models. In the first FEM model, it was assumed that the supports could be represented by a hinge at one end and a roller with a hinge at the other end. In the other FEM model, it was assumed that both supports were hinged, with no movement in horizontal direction. Fig. 4 illustrates the mesh of the FEM model for the tested bridge.

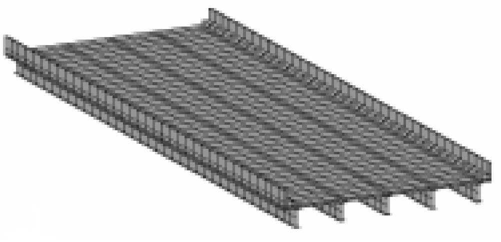


Fig. 4 The Mesh of the Finite Element Model

6. Proof Load Test Results

The target proof load level was successfully reached without any sign of distress to the structure. Fig. 5 shows strains for select side-by-side loadings with tanks and trucks. The load was gradually increased, and run 16 is the most heavily loaded case (Fig. 3). The maximum strain due to the side-by-side loadings is about 400 me. This corresponds to about 80 Mpa, lower than expected value from the pre-test analysis.

The strains were also measured close to the support. Fig. 6 presents the strains recorded at the east support. Negative strain values indicate the strains recorded at the bottom flanges near supports were in compression, due to the partial fixity of support. The strain values near the support were measured when the loads caused the maximum strain at the midspan.

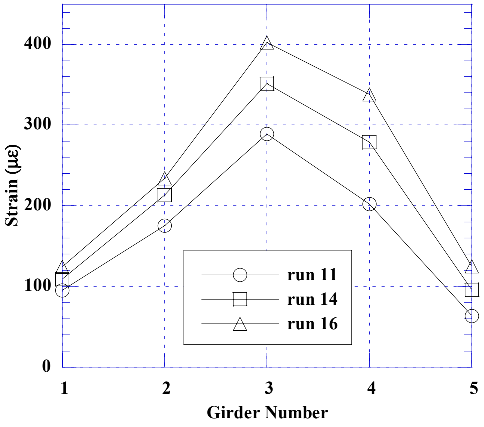


Fig. 5 Strains due to Select Side by Side Proof Loading

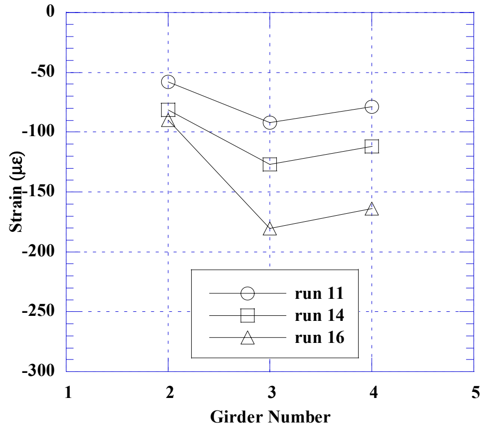


Fig. 6 Strain Recorded near East Support under Side-by-Side Proof Loading

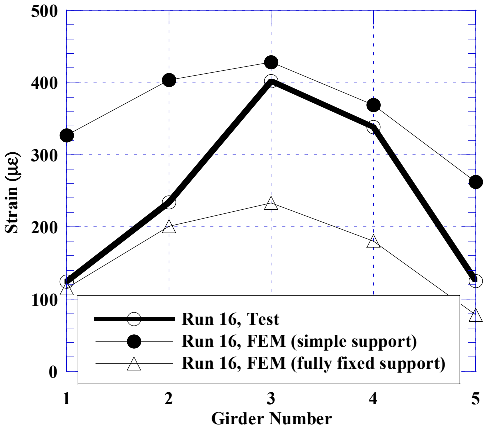


Fig. 7 Test Results Compared with Finite Element Analysis Results for Proof Load Run 16 (Fig. 3)

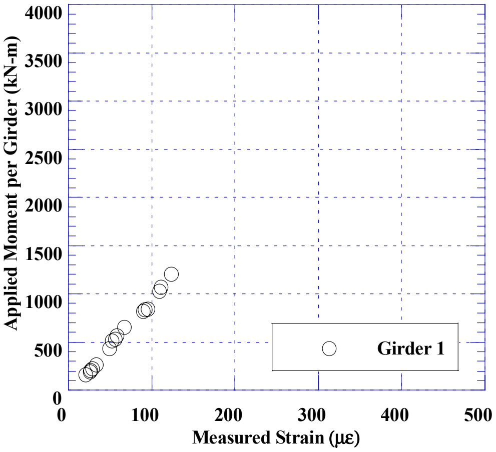
Fig. 7 compares the strain values obtained by the finite element analysis with those from the test for run 16, which is the heaviest run in the proof load test. In the analysis, nonstructural components, such as barriers, are taken into account. The result indicates that the maximum measured strain is still lower than expected from the finite element analysis. Also, the experimental response lies between the two different analytical models. This indicates that the partial fixity exists at the supports of the bridge, as explained before. This may indicate that the partial fixity exists at the supports of the bridge, and decreases .

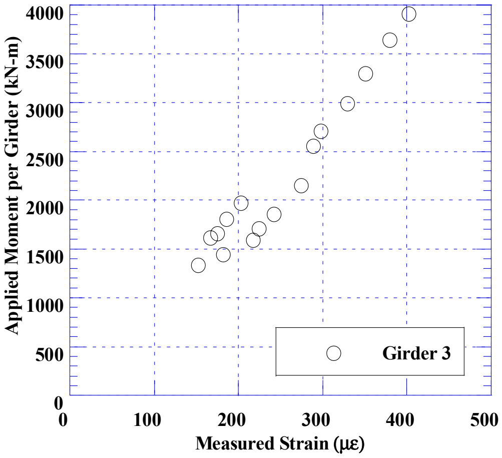
Table 2 shows the results of the finite element analysis for the proof load test, compared with the test results. Only the most heavily loaded girder (girder 3) was compared in the table with test results for run 1 to 16. In all cases, the measured strains are less than the analytical values.

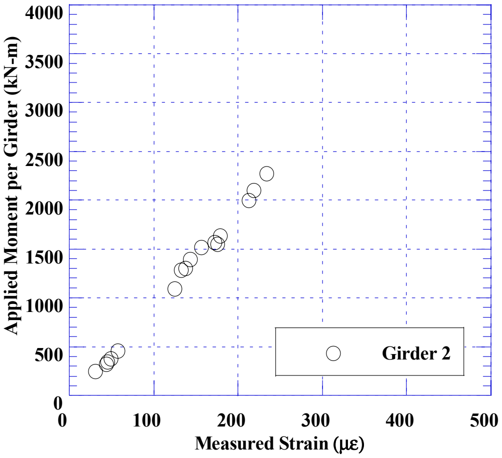
Fig. 8 plots applied moment per girder versus measured strain, for girder 1 to 5. The applied moment per girder was obtained by multiplying, for each load case, the total applied moment due to the load by the GDF from the strain values measured for that load case. All girders showed reasonably linear behavior with the applied lane moment.

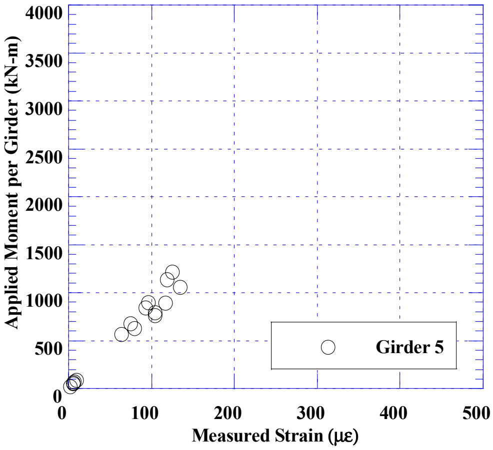
Table 2 Comparison of the maximum strain from the test and FEM results for the poof load test.

Run #	Maximum Measured Strain (10^{-6})	Strain from Finite Element Analysis (10^{-6})	
		Simple Support	Fixed Support
1	153	145	93
2	167	173	101
3	175	177	103
4	186	190	109
5	203	211	119
6	182	145	93
7	218	166	99
8	225	175	102
9	243	189	109
10	275	219	124
11	289	274	171
12	298	288	178
13	330	318	192
14	351	365	213
15	380	397	224
16	402	428	233









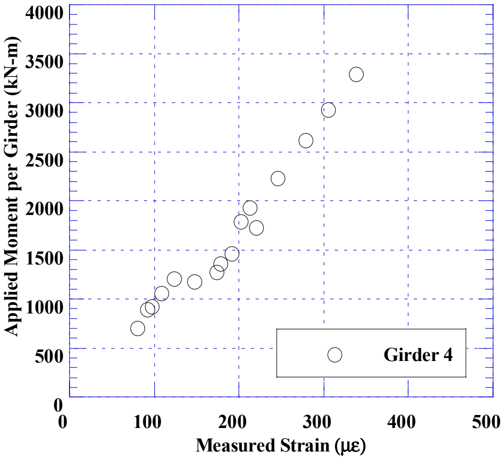


Fig. 8 Moment per Girder Versus Measured Strain for Each Girder

7. Conclusions

A procedure has been applied for proof load testing of a bridge with low operating rating. The proof load tests can be used as an efficient method to verify the minimum load carrying capacity of the structure. Proof load tests can be particularly efficient when the bridge analysis is difficult due to deterioration, repairs, and lack of documentation.

Diagnostic testing uses lower level of loads, such as trucks, and can be used to calibrate or verify analytical model. However, there are some structural parameters that can disappear under very heavy load levels, such as the structural contribution of nonstructural members, the effect of partially fixed support, and the unintentional composite action for noncomposite bridges.

The applied proof load procedure utilized military tanks that are very heavy over a short length of track. For the tested bridge, the target proof load level was reached without any signs of nonlinearity or distress. Therefore, the operating rating factor for 11-axle trucks is decided as more than 1. Also the results show that the tested bridge has a considerable safety reserve in the load carrying capacity.

The absolute value of measured strains is lower than expected from the pre-test analysis. This indicates that the actual stiffness of the tested bridge is higher than expected. This is due to incorrect material strength, structural contribution of nonstructural components such as parapets and railings, and partially fixed supports.

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