Nondestructive Evaluation of the I-40 Bridge over the Rio Grande River

David V. Jáuregui, A.M.ASCE,1 and Paul J. Barr, A.M.ASCE2

Abstract: A nondestructive strength evaluation of the I-40 Bridge over the Rio Grande River in Albuquerque, N.M. was completed for the New Mexico Department of Transportation (NMDOT). The I-40 Bridge is a precast, prestressed concrete girder bridge located within 3 mi. of the “Big I” interchange carrying Interstates I-40 and I-25. Because of its location, the I-40 Bridge is subjected to large amounts of heavy truck traffic. The primary objective of the study reported herein was to determine a more accurate capacity rating for the I-40 Bridge and thus, reduce the number of overweight vehicle permit denials. To achieve this objective, a conventional rating analysis is first performed based on American Association of State Highway and Transportation Officials (AASHTO) guidelines. A diagnostic load test and a finite-element analysis are then completed. Details of the AASHTO rating analysis as well as the approach by which measured girder strains from the load test and finite-element results were considered in the capacity rating of the I-40 Bridge are discussed. Findings from the study confirmed that the capacity ratings of the I-40 Bridge could be safely increased by a factor of 1.7.


CE Database subject headings: Bridge tests; Finite element method; Load distribution; Nondestructive tests; Permits; Strain measurement; New Mexico.

Introduction

This paper reports on the analytical and experimental strength evaluation of the I-40 Bridge, a precast, prestressed concrete girder bridge that carries the east and westbound traffic on Interstate-40 (I-40) over the Rio Grande River in Albuquerque, N.M. During the past 7 years that the I-40 Bridge has been in service, a large number of overweight vehicles have been denied an overload permit. These permit decisions were made by the New Mexico Department of Transportation (NMDOT) and were based on the results of a capacity rating analysis. Before the study reported herein was completed, an overload was approved only if the force effects (e.g., shear, moment, etc.) of the vehicle were shown not to exceed those caused by the equivalent of 1.67 times the force effects caused by HS-20 truck loading. If denied a permit, the designated detour for the overload vehicle typically results in approximately 8.0 km (5 mi.) and 30 min of additional travel on city streets for the truck driver. This change in itinerary is an inconvenience to the trucking industry that results in detour related expenses. As a result, the NMDOT considered the I-40 Bridge a major bottleneck in the state. The primary objective of the study reported herein was to determine a more accurate capacity level for the I-40 Bridge, one which better represented the true capacity of the bridge so that the NMDOT would avoid unnecessarily rejecting a permit request for an overweight vehicle.

In this study, a systematic approach was followed to arrive at a more accurate capacity rating for the I-40 Bridge. A conventional rating analysis was first performed based on the (load factor design) (LFD) method specified in the American State Highway and Transportation Officials (AASHTO) manual for condition evaluation of bridges (2000). To confirm and possibly further improve the LFD load rating, a diagnostic load test and a detailed finite-element analysis of the I-40 Bridge were completed. Details of the LFD rating analysis as well as the procedures for integrating girder strain measurements from the load test and finite-element results into the capacity rating of the bridge are discussed in this paper.

Bridge Description

The I-40 Bridge consists of twin structures that carry eastbound and westbound traffic over the Rio Grande River. Each structure is composed of a series of three precast, prestressed concrete bridges (two three-span continuous bridges separated by a four-span continuous bridge) that span a total distance of 378 m (1,240 ft). The two exterior bridges are three-span continuous bridges with span lengths of 39.4, 30.5, and 39.6 m (129 ft 2 in., 100 ft, and 130 ft, respectively). A four-span continuous bridge with equal span lengths of 39.6 m (130 ft) is located between the two three-span bridges. The bridge was designed based on the LFD method given in the AASHTO standard specifications. Construction of the I-40 Bridge was completed in 1995, at which time it replaced a fracture-critical, plate girder steel bridge that had previously been in service for approximately 30 years.

Figs. 1 and 2 each show a design drawing and photograph (looking west) of the exterior three-span continuous bridge unit located on the eastern end of the westbound structure. The spans

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Note. Discussion open until April 1, 2005. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor.

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are designated as Span 1, Span 2, and Span 3 in the east to west direction and are supported by fixed (labeled F) and expansion (labeled E) bearings as shown in Fig. 1(a). The cross section of the I-40 Bridge consists of 12 (labeled 1–12 from north to south) bulb tee BT-1830 (BT-72) girders spaced 2.2 m (7 ft 3 in.) apart as shown in Fig. 2(a). The girders were designed as simple spans for noncomposite dead loads (including the girder self-weight, slab/haunch weight, stay-in-place metal decking, and diaphragms) and as continuous spans under composite dead loads (including the future wearing surface and traffic barriers) and vehicular live load. Structural continuity under composite dead loads and live load was achieved at the interior piers by providing cast-in-place closure diaphragms and negative moment reinforcement in the deck slab. The precast, prestressed girders were fabricated using a harped strand layout with hold-down points located at the 1/3 points along the girder length. The girders were designed to act

![Image](image_url)

**Fig. 1.** Tested three-span continuous unit of I-40 bridge

![Image](image_url)

**Fig. 2.** Cross section of I-40 bridge: (a) design details and (b) transverse truck positions
Conventional Load Rating Analysis Using Load Factor Design

In accordance with NMDOT bridge design provisions, engineering consultants are required to specify an inventory (INV) and operating (OPR) rating for the bridge. These two measures of bridge capacity are recorded in the PONTIS Bridge Management System and used to maintain the safe use of the bridge and to arrive at posting and permit decisions. The decision to issue a permit is primarily made based on a comparison between the maximum bending moment caused by the overload vehicle \( (M_{\text{overload}}) \) and the maximum bending moment caused by HS-20 truck loading \( (M_{\text{HS-20}}) \). A permit is granted if the moment ratio \( (M_{\text{overload}}/M_{\text{HS-20}}) \) does not exceed the OPR rating factor for the bridge. When the moment ratio is greater than the OPR rating factor, the bridge is considered unsafe to carry the overload and the vehicle is rerouted along an alternate route. According to customary practice, the engineering consultants had originally assigned the I-40 Bridge an inventory rating of an HS-20 (i.e., INV rating factor=1) and an operating rating of an HS-32 (i.e., OPR rating factor=1.67) before the present study was completed.

Based on the flexural capacities of the girders, separate load ratings were determined for the three- and four-span continuous bridge units of the I-40 Bridge. Load ratings were determined for an interior and exterior girder in both the positive and negative moment regions of the three- and four-span bridges. In the LFD method (AASHTO 2000), a rating factor is determined using the equation

\[
RF = \frac{R_n - \gamma_D D}{\gamma_f L(1+I)}
\]

where RF = bridge load rating factor (either operating or inventory); \( R_n \) = nominal member capacity (based on flexural strength); \( \gamma_D \) = dead load factor=1.3; \( D \) = nominal dead load effect (including noncomposite and composite dead loads); \( \gamma_f \) = live load factor=1.3 (for operating rating) and 2.17 (for inventory rating); \( L \) = nominal live load effect (caused either by HS-20 truck or lane loading); and \( I \) = live load impact factor=15.24/(L+38) where L is the individual span length (taken as 39.6 m)=0.196 (equal to zero for lane loading).

The inventory rating is the smaller of the two rating levels and is defined in the AASHTO Manual for condition evaluation of bridges (2000) as the “live load that can safely utilize an existing structure for an indefinite period of time.” The operating rating factor corresponds to the “maximum permissible live load to which a structure may be subjected” (AASHTO 2000) but on a limited basis such as an occasional overweight vehicle. The rating factors represent the multiple of the force effects caused by either the HS-20 vehicle or lane loading that the bridge can safely carry at the inventory and operating levels. For example, an operating rating factor equal to 3 (computed based on the live load effects caused by HS-20 loading) indicates that the bridge can safely carry a vehicular load that causes a bending moment equal to three times that caused by HS-20 loading.

For the I-40 Bridge, the magnitudes of flexural capacity, dead load moments (noncomposite and composite), and live load moments for an interior girder, as computed according to the AASHTO standard specifications (1996), are given in Table 1. These quantities were determined using information taken from the design drawings of the I-40 Bridge. Both the positive and negative moment regions of the three- and four-span continuous bridge units were evaluated. For each bridge unit, the critical section for positive moment occurred close to mid-span of the 39.6 m (130 ft) exterior span, while the critical negative moment section occurred at the interior support of the exterior span. In the case of the three-span bridge [see Fig. 1(a)], for example, the critical sections are close to mid-span of Span 3 (for positive moment) and at the interior pier between Spans 2 and 3 (for negative moment).

Using the data given in Table 1, an LFD rating analysis was performed for an interior girder, results of which are summarized in Table 2. The table lists the inventory and operating rating factors for the positive and negative moment regions which were computed by substituting information from Table 1 into Eq. (1). Note that rating factors are reported for HS-20 vehicular loading and lane loading. For positive moment, the four-span bridge had slightly smaller rating factors (INV=1.93 and OPR=3.22) under HS-20 vehicular loading compared to the three-span bridge which had rating factors of INV=1.96 and OPR=3.28. The rating factors computed for lane loading were about 25% higher than those computed for HS-20 loading and thus, HS-20 loading controlled for positive moment. In the negative moment region, the rating factors were smaller for the three-span bridge (INV=2.03 and OPR=3.38) than the four-span bridge (INV=2.61 and OPR=4.36) based on HS-20 vehicular loading. When lane loading was considered, however, the factors decreased to roughly INV=1.70 and OPR=2.85 for both the three- and four-span bridges.

### Table 1. Load Rating Information for Interior Girder of Three- and Four-Span Bridge Units

<table>
<thead>
<tr>
<th>Property</th>
<th>Positive moment</th>
<th>Negative moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Three-span bridge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexural capacity</td>
<td>13,720</td>
<td>5,610</td>
</tr>
<tr>
<td>Noncomposite dead load</td>
<td>4,360</td>
<td>0</td>
</tr>
<tr>
<td>Composite dead load</td>
<td>552</td>
<td>541</td>
</tr>
<tr>
<td>Total dead load</td>
<td>4,910</td>
<td>541</td>
</tr>
<tr>
<td>HS-20 vehicular loading</td>
<td>1,440</td>
<td>933</td>
</tr>
<tr>
<td>Lane loading</td>
<td>1,360</td>
<td>1,320</td>
</tr>
<tr>
<td>Four-span bridge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexural capacity</td>
<td>13,720</td>
<td>6,600</td>
</tr>
<tr>
<td>Noncomposite dead load</td>
<td>4,360</td>
<td>0</td>
</tr>
<tr>
<td>Composite dead load</td>
<td>493</td>
<td>685</td>
</tr>
<tr>
<td>Total dead load</td>
<td>4,850</td>
<td>685</td>
</tr>
<tr>
<td>HS-20 vehicular loading</td>
<td>1,480</td>
<td>842</td>
</tr>
<tr>
<td>Lane loading</td>
<td>1,390</td>
<td>1,550</td>
</tr>
</tbody>
</table>

*Close to mid-span of exterior span.

*At interior support of exterior span.
also evaluated girder, AASHTO the load distribution factor for bending moment of an exterior flange width approximately 76 mm difference between the two girders, however, was the load distribution capacity and dead load moments where 0.659. Since the load distribution factor was smaller for the exterior girder, which results in smaller live load moments, and the prior girder, which results in smaller live load moments, and the reserve strength (i.e., flexural capacity minus dead load moments) was about the same for both girders, the exterior girder had larger rating factors than the interior girder (see Table 2) and thus, did not control the load rating.

Nondestructive Live-Load Testing

Although not usually considered in design, certain attributes of bridge behavior have been observed in numerous load tests to favorably influence the load distribution and safe load-carrying capacity of slab-on-girder bridges. Some of the factors that affect bridge performance include end restraint, unintended continuity, flexural participation of curbs and/or railings, and two-way slab action (Burdette and Goodpasture 1988). The standard AASHTO LFD rating procedure, however, employs concepts and assumptions similar to those used in design, which ignore these factors and thus, often underestimate the bridge’s true capacity. As a result, several state highway departments have adopted nondestructive load testing to better quantify the response of a bridge to live load. Starting in the 1980s, the New York Department of Transportation has occasionally used load testing to determine the safe load level for a bridge. In the last 10 years, experimental bridge rating activity has expanded to other state departments of transportation such as Florida (Shahawy 1995) and Alabama (Conner et al. 1997). The two types of nondestructive load tests commonly used to evaluate the live load response of an existing bridge are diagnostic and proof tests. These two methods differ in terms of the level of load applied to the bridge, the quantity and significance of measurements taken, and the manner in which the experimental findings are used to arrive at a load rating (Lichtenstein 1998; Pinjarkar et al. 1990). In a diagnostic test, the bridge is subjected to a known load below its elastic load limit or diagnostic test load as shown in Fig. 3. Strain and/or deflection measurements are taken at strategic locations to determine the load distribution and stiffness characteristics of the bridge. Following the test, the measurements are then used in combination with an analytical model to confirm the behavior of the bridge and better estimate its capacity. In a proof test, increasing loads (labeled 1–5 in Fig. 3) are applied to the bridge until a target proof load is reached or nonlinear behavior is observed. When either of these two events occurs, the load test is stopped and the maximum load carried by the bridge is adjusted to determine the load rating. In both the diagnostic and proof test, the measurements can be used to adjust or refine the load rating of the bridge. Specific procedures for conducting nondestructive load tests and incorporating the field measurements into the capacity rating have been developed by several researchers including Stallings and Yoo (1993), Chajes et al. (1997), Lichtenstein (1998), Goble et al. (2000), Cai and Shahawy (2001), and Barker (2001).

Load testing of the I-40 Bridge offered some unique challenges. According to the NMDOT, the I-40 Bridge carries an average daily truck traffic of over 45,000 vehicles. This large number of trucks is primarily due to the bridge’s close proximity to

<table>
<thead>
<tr>
<th>Girder and moment region</th>
<th>Rating factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inventory</td>
</tr>
<tr>
<td>Three-span bridge</td>
<td></td>
</tr>
<tr>
<td>Interior girder, positive moment</td>
<td>1.96a</td>
</tr>
<tr>
<td>(2.49)b</td>
<td>(4.15)</td>
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<tr>
<td>Interior girder, negative moment</td>
<td>2.03</td>
</tr>
<tr>
<td>(1.71)</td>
<td>(2.85)</td>
</tr>
<tr>
<td>Exterior girder, positive moment</td>
<td>2.04</td>
</tr>
<tr>
<td>(2.60)</td>
<td>(4.33)</td>
</tr>
<tr>
<td>Exterior girder, negative moment</td>
<td>2.13</td>
</tr>
<tr>
<td>(1.78)</td>
<td>(2.97)</td>
</tr>
<tr>
<td>Four-span bridge</td>
<td></td>
</tr>
<tr>
<td>Interior girder, positive moment</td>
<td>1.93a</td>
</tr>
<tr>
<td>(2.46)b</td>
<td>(4.10)</td>
</tr>
<tr>
<td>Interior girder, negative moment</td>
<td>2.61</td>
</tr>
<tr>
<td>(1.70)</td>
<td>(2.84)</td>
</tr>
<tr>
<td>Exterior girder, positive moment</td>
<td>2.01</td>
</tr>
<tr>
<td>(2.57)</td>
<td>(4.28)</td>
</tr>
<tr>
<td>Exterior girder, negative moment</td>
<td>2.72</td>
</tr>
<tr>
<td>(1.77)</td>
<td>(2.96)</td>
</tr>
</tbody>
</table>

aBased on HS-20 vehicular loading.
bBased on lane loading.

These rating factors were found to be the smallest magnitudes and thus, represented the new load capacity of the I-40 Bridge at the inventory and operating levels (note that the original ratings were INV = 1.0 and OPR = 1.67).

To complete the LFD rating analysis, an exterior girder was also evaluated (see Table 2). The exterior girders had the same prestressing strand layout as the interior girder, but an effective flange width approximately 76 mm (3 in.) larger. For rating purposes, this small difference in the effective flange width was ignored and exterior girder parameters (including the flexural capacity and dead load moments) were assumed to be equal to those reported earlier in Table 1 for the interior girder. One major difference between the two girders, however, was the load distribution factor. The AASHTO standard specifications (1996) state that the load distribution factor for bending moment of an exterior girder “shall be determined by applying to the stringer or beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between stringers or beams.” This approach, also known as the lever rule concept, resulted in a distribution factor for the exterior girder equal to 0.632. For the interior girder, AASHTO (1996) specifies a distribution factor of (S/5.5) where S=average girder spacing in feet, which amounted to 0.659. Since the load distribution factor was smaller for the exterior girder, which results in smaller live load moments, and the reserve strength (i.e., flexural capacity minus dead load moments) was about the same for both girders, the exterior girder had larger rating factors than the interior girder (see Table 2) and thus, did not control the load rating.

Fig. 3. Hypothetical load–deflection response of bridge (based on research in Pinjarkar et al. 1990)
the “Big I” interchange, located at the junction of Interstates I-40 and I-25. This interchange is the busiest in New Mexico with an average daily traffic of over 300,000 vehicles/day (note that the original 1966 design was for 50,000–60,000 vehicles/day). At the time the I-40 Bridge was to be load tested, a major reconstruction project was in progress on the “Big I.” In order to avoid disrupting the “Big I” construction and traffic flow on the I-40 Bridge, the load test was performed during the early morning hours (between 12:00 am and 4:00 am) on 2 separate days. A diagnostic load test was performed since it could be completed in less time than a proof test and the design plans of the I-40 Bridge were available to create a representative analytical model. Furthermore, two of the four lanes could be left open to traffic during the diagnostic load test and a smaller truck could be used to apply load. A proof test typically requires closing all the lanes on the bridge until completion of the test, which was not an option on the interstate and also involves applying large levels of load in a careful and time-consuming manner.

Strain transducers were installed on eight of the twelve girders at two longitudinal locations along Span 2 of the three-span bridge unit; close to the interior support or negative moment region and at the mid-span or positive moment region [labeled NEG and POS, respectively, in Fig. 4(a)]. This particular bridge unit was considered the most practical for load testing because of its easy access and low vertical clearance compared to the other bridge units. Also, as discussed previously, the LFD rating analysis showed no major differences in the capacity levels between the three- and four-span bridge units. Therefore, it was believed that load testing and finite-element analysis of the three-span bridge could be used to adequately obtain a comprehensive load rating of the I-40 Bridge. The instrumentation locations proved to be the most accessible since there was a catwalk installed at the interior pier and a levee at mid-span (see Fig. 1). At both locations, a pair of strain transducers was installed on girders 1–8; on the bottom of the girder and below the top flange as shown in Fig. 4(b). This instrumentation layout resulted in a total of 32 strain measurements (16 at each instrumented section).

Using a 16-channel data acquisition system, the bridge was tested over a 2 day period (July 12 and 13, 2001) with a six-person crew starting with the positive moment region. Traffic control was arranged through the NMDOT, which involved closing the two north lanes of the I-40 Bridge and leaving the two south lanes open to traffic (refer to Fig. 2 for lane positions). A single, three-axle water truck weighing 238 kN (53.4 kips) was used to apply load to the bridge since it was the heaviest vehicle the NMDOT could provide at the time of testing. The truck was driven over the entire length of the three-span bridge starting at the east abutment at a speed of approximately 8 kph (5 mph) along three separate paths (labeled Paths 1, 2, and 3), as shown in Fig. 4(b). Each truck pass was completed with either minimal or no traffic on the open lanes. Before each run, surveillance of oncoming traffic was made. Once the I-40 Bridge seemed to be clear of traffic (i.e., no westbound traffic could be seen approaching from the east side of the bridge), all the strain transducers were balanced and the truck pass was made. To ensure good data records, three separate runs were completed for each truck path. Strain measurements were collected at a rate of 32 Hz. With the truck traveling at approximately 8 kph (5 mph), a typical run required a total time of approximately 1 min. As a result, there were instances when there was normal traffic on the tested bridge while the water truck was on course along the load path. The most notable changes in the strain record occurred when an 18 wheeled tractor trailer crossed the bridge on the north, open lane during the second (Path 2) and third (Path 3) truck paths. If a large truck was on the bridge during a test, the test was redone until three tests were completed free of heavy truck traffic to avoid affecting the girder strain measurements. Due to the time of the test, passenger cars and trucks were seldom present and had very little effect on the recorded girder strains.

Fig. 5 shows the average top and bottom strains (from three separate runs) recorded at the positive moment region of Girder 1 as the truck was driven along Path 1. For this path, Girder 1 was the most heavily loaded girder and as a result, had the largest recorded change in strain. The figure shows that when the truck was in Span 1 or Span 3, the mid-span moment in Span 2 was negative. When the truck was in Span 2, the moment is positive and largest when the truck was near mid-span. These trends confirmed that the bridge was acting as a continuous system; however, the degree of continuity was not obvious solely from the strain record. Furthermore, since the location of the top gauge was near the neutral axis of the composite girder (as computed based on a transformed section analysis), the small compression...
strains in the top gauge and the large tensile strains in the bottom
gauge indicated that the girder and deck were acting composite
and no unexpected loss of composite action had occurred. The
averaged measured neutral axis was within 5% of the calculated
composite neutral axis based on material properties determined
using the design concrete strengths and tributary width recom-
manded by AASHTO. Similar trends were recorded for the other
instrumented girders for each load path in both the positive and
negative moment regions. These observations related to the gen-
eral behavior of the I-40 Bridge were used to develop an analyti-
cal model (which incorporated both continuity and composite ac-
tion) for the finite-element analysis of the bridge discussed in the
following section.

Refined Load Rating Using Finite-Element Analysis

A finite-element model of the tested three-span bridge unit of the
I-40 Bridge was developed using SAP2000 (Computers and
Structures Inc. 2000) to determine the bridge’s theoretical re-
sponse and allow for direct comparison with the measurements
from the live load test. The accuracy of the model was first vali-
dated with the test measurements and then used to calculate a
theoretical live-load distribution factor. Using the test validated,
theoretical distribution factor, a refined load rating was deter-
mined by adjusting the live-load expression in the denominator of
the AASHTO rating equation [refer to Eq. (1)]. Details regarding
the finite-element modeling, the computation of the theoretical
distribution factor, and the integration of the results in the load
rating of the I-40 Bridge are discussed in the following subsections.

Finite-Element Model of I-40 Bridge

A finite-element model of a concrete slab-on-girder bridge can be
created in many different ways; however, the modeling assump-
tions can have a significant influence on the results. Various re-
searchers have investigated different modeling techniques for
slab-on-girder bridge types. Hays et al. (1986) used quadrilateral
shell elements to model the deck and space frame elements to
model the girders. The nodal locations for the shell and frame
elements of this bridge model were coplanar. Tarhini and Freder-
wick (1992) developed a three-dimensional (3D) model using
eight-node brick elements for the deck and shell elements for the
girders. A 3D bridge model was also developed by Zokaie et al.
(1991); however, the deck and the girders were modeled using
shell elements and eccentrically spaced frame elements, respec-
tively. In a study by Mabsout et al. (1997), each of the finite-
element models just described was used to calculate live-load
distribution factors for slab-on-girder bridges. The authors con-
cluded that the models yielded similar load distribution factors.

Based on a review of the work from the aforementioned re-
searchers, the modeling scheme of Zokaie et al. (1991) was cho-
sen because it was relatively simple and it was also used in the
development of the live-load distribution factor equations given
in the AASHTO (1998) “load and resistance factor design speci-
fications.” Hence, the precast girders were modeled using frame
elements placed at their geometric centroid and the bridge deck
was modeled using four-node shell elements. Because the cen-
troids of the girder and deck are not coplanar, rigid-body con-
straints were enforced to connect the two separate components
and simulate composite action. The constraints were applied in
the longitudinal direction between the nodes of the girder frame
elements and the overlying, respective nodes of the deck shell
elements. Fig. 6 shows a cross-sectional view of the frame and
shell finite-element model. The dimensional and material proper-
ties for the girders and the deck were taken from the design plans.
The stiffness of the respective bridge components (i.e., modulus
of elasticity) was calculated using the elastic modulus equation
provided at the American Concrete Institute (ACI 1999) with the
specified concrete strength. The nodes for the deck elements were
spaced 368 mm (14.5 in.) in the transverse direction to fit the
2.2 m (7.25 ft) girder spacing and 305 mm (12 in.) longitudi-
nally. A 305 mm (12 in.) longitudinal, nodal spacing was also
used for the girder elements in order to match the node locations
of the shell elements. Although this spacing produced over 20,000
nodes, the nodal arrangement was considered necessary in order
in order to obtain a relatively fine mesh of elements (with an aspect ratio
close to unity) and to provide accurate placement of the wheel
loads. Although the I-40 Bridge was constructed with intermedia-
tate diaphragms, previous researchers (Sithichaikasem and
Gamble 1971; Barr et al. 2001) have found that intermediate dia-
aphragms have little impact on the live load distribution and thus,
were not included in the finite-element model of the I-40 Bridge.

As mentioned earlier in the paper, unintended continuity has
been shown to influence the actual response of many bridges
(Burdeette and Goodpasture 1988). Thus, in order to evaluate the
effect of the stiffness of the interior piers on the longitudinal load
distribution, three different support conditions were considered.
In the first model, the three-span bridge was supported with con-
ventional pins at the fixed bearing locations and rollers at the
expansion bearing locations [labeled F and E, respectively, in Fig.
1(a)]. This model ignored the effect of pier stiffness on the bridge
response. In the second bridge model, the four circular columns of
the interior piers [see Fig. 1(b)] were modeled with frame
elements to simulate the actual pier stiffness. The columns were
fixed at their base and connected to the girders by means of rigid
constraints. Gross section properties (i.e., uncracked) based on
design data given in the plans were assumed for the column elements. For the third bridge model, the ends of the girders in Span 2 were fixed (i.e., no translation and no rotation) at the interior pier locations. This model was intended to represent an upper bound on the interior pier stiffness. The weight of the water truck was applied as six wheel loads on the bridge deck. Because the locations of the wheel loads did not coincide with nodal locations, simple shape functions were used to distribute the wheel loads to the four nodes of the “loaded” shell elements. Although continuous measurements were made at 32 Hz during the live-load test, the wheel loads applied to the finite-element model were positioned at 3.0 m (10 ft) increments along each of the truck paths. Similar to the field test, the truck was positioned transversely along three separate paths delineated by the front wheel locations given in Fig. 4(b).

**Comparison of Live-Load Test with Finite-Element Model**

The measured live-load response of the three-span bridge test was compared with the theoretical response calculated with the three finite-element models. At each truck location, the bending moment carried by each of the 12 composite girders was calculated in the positive moment region of Span 2 [labeled POS in Fig. 4(a)]. The tributary width used to calculate the composite section properties for the interior girders was assumed to be equal to the girder spacing. For the exterior girders, the tributary width was taken as half the girder spacing plus the overhang distance.

Finite-element output generated by the bridge models included axial forces and moments for the frame elements (girder), and top and bottom stresses for the shell elements (deck). Composite, cross-sectional moments for the interior and exterior girders were found by first calculating the stress at the bottom of the girder caused by the theoretical forces in the frame element using Eq. (2). The resulting stress was then multiplied by the theoretical, composite section modulus as shown by Eq. (3) to obtain the girder moment

\[
\sigma_g = P/A + M_g/S_g
\]

\[
M_g = S_g \sigma_g
\]

where \( \sigma_g \) = stress at bottom flange of girder; \( P \) = axial force in frame element; \( A \) = cross-sectional area of BT-72 girder (noncomposite) = 495 \times 10^3 \text{ mm}^2 (767 \text{ in.}^2); \( M_g \) = bending moment in frame element; \( S_g \) = bottom section modulus of BT-72 girder (noncomposite) = 244 \times 10^6 \text{ mm}^3 (14,915 \text{ in.}^3); \( M_c \) = composite cross-sectional bending moment; and \( S_{gc} \) = bottom composite section modulus = 583 \times 10^6 \text{ mm}^3 (356 \times 10^6 \text{ in.}^3).

The composite, cross-sectional bending moments in the girders from the live-load test were calculated from differences in strain readings with the bridge in its loaded and unloaded condition. Prior to driving the truck along a load path, the strain transducers were balanced to set the strain readings equal to zero with the truck off the bridge. After a run was completed, any offset from the initial zero reading was assumed to be a result of thermal effects. In these cases, individual zero readings for every load location were found by linear interpolation (based on the truck position) between the initial and final zero reading. The measured composite, cross-sectional bending moment was calculated by first multiplying the change in strain \( (\varepsilon_c) \) by the modulus of elasticity \( (E) \) to obtain the state of stress \( (\sigma_c) \) and then using Eq. (3).

Fig. 7 compares the positive mid-span moments for Girder 1 calculated from the measured strain values (labeled TEST) with those determined from the three finite-element models (labeled CONTINUOUS, FRAME, and FIXED) with the truck traveling along Path 1. The figure shows that the CONTINUOUS model (with the pin and roller supports) overestimated the absolute value of the measured mid-span moment (i.e., TEST) when the truck was on any of the three spans. The maximum positive moment from this model was about 34% larger than the mid-span moment measured from the live-load test. The FIXED model produced moments that were close to the measured moments when the truck was on Span 2; however, since the girder deformations were restrained at the ends of Span 2, the truck loading on Spans 1 and 3 caused no bending moment at mid-span, which did not agree with the live-load test. This model was intended to provide an upper limit on the rotational stiffness at the pier and yielded mid-span moments that were nearly 8% smaller than the measured moments with the truck on Span 2. The FRAME model (with columns at the interior pier supports) provided a good correlation between the measured and predicted moment values when the truck was in all three spans. The mid-span moment from this model was slightly larger (by about 3%) than the measured moment. This strong correlation between the measured moment and those from the FRAME model was observed for each girder and for each load case. Although the interior piers were not designed to produce this type of restrained behavior, the pier stiffness contributed to measured moments about 25% smaller than those calculated based on design assumptions (i.e., pin and roller supports or zero pier stiffness).

Fig. 8(a) shows a comparison of the mid-span moments for Girders 1–8 with the truck positioned at the critical location for maximum positive moment along Path 1. The finite-element moments plotted in the figure were calculated using the FRAME model. The figure shows that for Path 1, the truckload was supported mainly by Girders 1–4 with negligible contributions from Girders 5–8. In both the live-load test and the finite-element model, the largest mid-span moment occurred at the exterior girder (i.e., Girder 1) and decreased in magnitude in a nearly linear manner to Girder 4. The FRAME model slightly overpredicted the mid-span moment for Girder 1 and underpredicted the moments for Girders 2–4. Fig. 8(b) shows the same comparison as that given in Fig. 8(a) with the truck positioned transversely in Path 2. For this truck path, two additional girders (i.e., Girders 5 and 6) were involved in resisting the truckload. Furthermore, the load was distributed more uniformly between the girders compared to that observed for Path 1. Girders 3 and 4 carried the largest amount of the load; however, the magnitude of the moment carried by these two interior girders was smaller than the
tions were investigated using the AASHTO HS-20 truck. In all live-load distribution factors are intended to represent the envelope to obtain more accurate live-load distribution factors. Because the I-40 Bridge were used in lieu of the AASHTO standard equation results 

The FRAME and CONTINUOUS finite-element models of the I-40 Bridge were used in lieu of the AASHTO standard equation to obtain more accurate live-load distribution factors. Because the live-load distribution factors are intended to represent the envelope of all possible loading scenarios, many loading configurations were investigated using the AASHTO HS-20 truck. In all load cases, the positive moment in Spans 2 and 3 and the negative moment at the interior pier separating these two spans were considered.

The critical longitudinal positions of the AASHTO HS-20 trucks were established by finding the locations that produced the maximum positive or negative moment on an isolated, three-span continuous beam. The simulated truckloads were placed longitudinally in the same positions on the two finite-element models. The critical transverse locations of the AASHTO HS-20 trucks were found by dividing the bridge into as many 3.7 m (12 ft) wide lanes as possible (six) and then systematically moving the individual trucks within their respective lane. Lanes were also systematically moved in order to simulate many transverse-loading conditions as in the study by Zokaie et al. (1991). In developing the load cases, it was assumed that the center of the wheels could be placed no closer than 0.61 m (2 ft) from the inside edge of the traffic barrier. More than 75 load cases were evaluated and the maximum positive and negative moments for each girder were recorded for each load case. The critical load case for the exterior girder is shown in Fig. 9.

Finite-element results were used to calculate live-load distribution factors as follows. The maximum bending moments carried by each of the 12 composite girders under HS-20 truck loading were first determined. The AASHTO (1996) multilane reduction factor was then applied to the calculated moments based on the number of loaded lanes (equal to 1 for one or two lanes, 0.9 for three lanes, and 0.75 for four lanes). Finally, distribution factors for the interior and exterior girders were found by dividing the maximum girder moment by the total statical moment caused by the AASHTO truck loading when placed on an isolated, 3-span continuous beam. The results from the CONTINUOUS and FRAME models are listed in Table 3. For each model, the distribution factors for positive moment in Spans 2 and 3 as well as those for negative moment at the intermediate, interior pier are given. Although including the columns in the FRAME model decreased the total moment at mid-span by about 25% as discussed earlier, the distribution factors given in Table 3 show that the pier stiffness did not significantly influence (i.e., there was less than 5% difference in the values between the CONTINUOUS and FRAME models) the load distribution between the girders in the transverse direction. In comparison to the AASHTO standard distribution factors (i.e., 0.659 for the interior girder and 0.632 for the exterior girder), the finite-element values were constantly smaller. The differences between the finite-element and AASHTO distribution factors, which ranged from 3 to 13%, are given in parentheses in Table 3.

Table 4 lists the new load ratings computed using the finite-

Fig. 8. Measured versus finite element moments at mid span of I-40 bridge: (a) path 1 and (b) path 2

Fig. 9. Critical load case for exterior girder
element distribution factors. To obtain the new ratings, the AASHTO Standard distribution factors were simply replaced by those determined from finite-element analysis (using the larger estimates for the distribution factor italicized in Table 3). Compared to the rating factors determined using the AASHTO Standard distribution factors (noted at the bottom of Table 4), the new factors were larger. For positive moment, the new rating factors were 15 and 9% larger, while for negative moment, the rating factors were only 10 and 3% larger for the interior and exterior girders, respectively. These percentages indicate that the AASHTO standard distribution factor was the most conservative in the case of positive moment of an interior girder. In this case, the finite-element distribution factor of 0.575 was well below the AASHTO distribution factor of 0.659, which resulted in the large increase of 15% in the load rating factors. The cases of positive moment of an exterior girder and negative moment of an interior girder showed about the same improvement (10%) in the rating factors. For negative moment of an exterior girder, however, the distribution factors from AASHTO (0.632) and the finite-element model (0.615) correlated very well and as a result, there was not a significant change in the rating factors (i.e., only 3%). Thus, the distribution factors from the finite-element analysis of the I-40 Bridge further verified the AASHTO Standard distribution factors but did not result in any significant improvements beyond the AASHTO LFD capacity ratings. However, the 25% reduction in the longitudinal moment caused by the pier stiffness may improve the capacity ratings even more but was not investigated in this paper.

Conclusions

A load rating analysis was performed on the I-40 Bridge over the Rio Grande River in Albuquerque, N.M. The analysis showed that the inventory and operating factors of the I-40 Bridge could be increased by a factor of 1.7–1.70 and 2.85, respectively. A diagnostic load test was performed to validate this increase in the capacity ratings. The results of the load test showed that the stiffness of the interior piers affected the longitudinal distribution of live load. The pier stiffness resulted in a mid-span moment that was about 25% smaller than the moment a designer would obtain assuming the girders were supported by pins and rollers. A finite-element model of the bridge was then used to calculate a more accurate live-load distribution factor. The finite-element analyses resulted in distribution factors ranging from 3 to 15% smaller than the AASHTO standard distribution factors. Results of the finite-element analysis also showed that the pier stiffness did not have a significant influence on the value of the distribution factor. No significant changes occurred in the rating factors as a result of the finite-element distribution factors; however, the reduction in the longitudinal moment in the girders caused by the stiffness at the interior piers may improve the capacity ratings. Based on these findings, the New Mexico Department of Transportation safely increased the inventory and operating ratings of the I-40 Bridge to 1.70 and 2.85, respectively.

Acknowledgments

The writers would like to thank the New Mexico Department of Transportation (NMDOT), in particular Jimmy Camp (State Bridge Engineer) and Wil Dodge (District III Bridge Engineer) for the support that made this research possible. Special thanks goes to Jim Pate (Research Associate, New Mexico State University) and the NMDOT Bridge Maintenance Division for their assistance during the live-load testing of the I-40 Bridge.

References


Table 3. Finite-Element Distribution Factors for CONTINUOUS and FRAME Bridge Models

<table>
<thead>
<tr>
<th>Moment region</th>
<th>Distribution factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior girder</td>
</tr>
<tr>
<td>CONTINUOUS model</td>
<td></td>
</tr>
<tr>
<td>Negative moment at interior pier</td>
<td>0.590</td>
</tr>
<tr>
<td>Positive moment in span 2</td>
<td>0.565</td>
</tr>
<tr>
<td>Positive moment in span 3</td>
<td>0.565</td>
</tr>
<tr>
<td>FRAMES model</td>
<td></td>
</tr>
<tr>
<td>Negative moment at interior pier</td>
<td>0.600</td>
</tr>
<tr>
<td>Positive moment in span 2</td>
<td>0.575</td>
</tr>
<tr>
<td>Positive moment in span 3</td>
<td>0.560</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Moment region</th>
<th>Rating factor</th>
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<tbody>
<tr>
<td></td>
<td>Inventory</td>
</tr>
<tr>
<td>Interior girder</td>
<td></td>
</tr>
<tr>
<td>Positive moment</td>
<td>1.95</td>
</tr>
<tr>
<td>Negative moment</td>
<td>1.87</td>
</tr>
<tr>
<td>Exterior girder</td>
<td></td>
</tr>
<tr>
<td>Positive moment</td>
<td>1.93</td>
</tr>
<tr>
<td>Negative moment</td>
<td>1.82</td>
</tr>
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</table>

aCorrelation of bridge

<table>
<thead>
<tr>
<th>Moment region</th>
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<tr>
<td></td>
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<td>Positive moment</td>
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<td>1.93</td>
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<tr>
<td>Negative moment</td>
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</tr>
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</table>

bFor interior girder, rating factors equal 1.70 (inventory) and 2.85 (operating) based on AASHTO distribution factors.

bFor exterior girder, rating factors equal 1.77 (inventory) and 2.97 (operating) based on AASHTO distribution factors.


