

STRUCTURAL HEALTH MONITORING OF A CURVED GIRDER BRIDGE SUBJECTED TO DIFFERENT BOUNDARY CONDITIONS

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ABSTRACT :

A curved, three-span continuous, steel I-girder bridge in Salt Lake City, Utah was tested in order to determine its dynamic and static response to forced vibration. The bridge supports were altered and its response was monitored for three different boundary condition states. For each of the three boundary condition states, two dynamic forced vibration methods were applied to the bridge in order to determine the feasibility of structural health monitoring. The first forced vibration method used an eccentric mass shaker capable of imparting a horizontal sinusoidal force of up to 89 kN (20,000 lbs). The second method involved horizontally striking the side of the bridge with a 2.47 kN (556 lb) impact hammer. An array of 1-Hz velocity transducers and 1-g accelerometers were utilized to record response of the bridge. The recorded dynamic response of the bridge was analyzed and resulted in mode shapes and natural frequencies for each boundary condition state for both the sinusoidal and impact testing. This paper discusses the resulting changes in relevant dynamic.

KEYWORDS:

Bridge, curve girder, dynamic, mode shapes, damage

Introduction

Steel, curved-girder bridges are a common solution to a wide variety of traffic routing and infrastructural needs. They provide for quick erection (in comparison to a multispan straight-girder bridge where multiple piers are required) and are extremely flexible in their application. Due to these benefits, the use of curved-girder bridges in our infrastructure has steadily increased over the last several decades. However, a better understanding of their in-service performance is still required. One of the largest research studies on the behavior of curved girder bridges was performed by the Federal Highway Administration (FHWA) under the Curved Steel Bridge Research Program (CSBRP) and funded by the (Zureick et al. 1999). This research endeavor was a large scale experimental and analytical program focusing on the improvement of design methods used for curved steel girder bridges. Other researchers, such as Huang et al. (1995), used load testing to identify the importance of centrifugal forces on curved bridges created by vehicle traffic. Most recently, DeSantiago et al. (2005) compared the analysis results of various curved and straight bridges and other researchers (Zhang et al. 2005; Samaan et al. 2005) investigated the lateral-load distribution of curved-girder bridges. A detailed review of the design of

horizontally curved I-girder bridges can be found in Linzell et al. (2004).

In addition to bridge live load studies, considerable effort has also been placed on quantifying a bridge's response when subjected to dynamic loads. During a modal testing program by Bolton et al. (2005), a concrete box girder bridge that had been periodically tested was significantly damaged by the Hector Mine California Earthquake. Because the bridge had been tested just weeks before the seismic forces, it provided an excellent pre and post event opportunity to evaluate a bridge structure. They found that the event caused an overall decrease in natural frequencies of 18% relative to pre-event values. Eberhard and Marsh (1997), performed progressive destructive testing on a reinforced concrete bridge by using six cycles of transverse displacements imposed on the bridge bents. For each progressive test the bridge bearing was weakened by excavating the soil behind the wingwalls and replacing the bridge bearing points with nylon blocks resting on greased, polished, stainless-steel plates. They found the overall system stiffness decreased by 91% comparing the "as is" condition with the fully damaged condition. Dynamic load testing of bridges has also been performed after stiffening repairs. Working on a four-span-arch bridge that had recently undergone stiffening repairs, Proulx et al. (1992) found that while some frequencies and modes were significantly altered others remained largely unchanged. Changes in modal properties have also been related to the amount of repair or damage a structure has experienced. Halling et al. (2001) tested a single span bridge in seven different damage states. A multi-span concrete bridge that underwent stiffening repairs did not see significant changes in natural frequencies largely due to the modest size of repairs performed (Salawu and Williams 1995). In a similar example, bearings were replaced on a 4-span concrete bridge with little effect noted to the modal properties and only a marginal increase in global stiffness (Salawu 1997).

When the I-15 reconstruction project in Salt Lake City, Utah called for the demolition of a curved-girder bridge a unique opportunity presented itself for testing an "as is" curved girder bridge. With the eventual replacement of this bridge, the Utah Department of Transportation (UDOT) allowed researchers the unique opportunity to test the effects that changes in boundary condition states had on the performance of a curved-girder bridge. A variety of testing methods were used for each boundary condition state including sinusoidal forcing and impact loading. A summary of the boundary condition states and an outline of the methods used to collect bridge data for each condition state are presented. In addition, a comparison of the field test results from the sinusoidal and impact testing. This comparison indicates promising testing procedures in the area of structural health monitoring.

Bridge Description

The bridge tested as part of this research was built in 1971 and was designed as a non-composite, three span continuous bridge. The bridge was located at the interchange of 6400 South and I-215 in Salt Lake City, Utah. The continuous superstructure consisted of five curved steel I-girders supported on bronze rocker bearings at the abutments and at the two piers. Fig. 1 shows an elevation view of the bridge. Initial inspection of the "in-service" condition of the bridge showed that a number of the rocker bearings on the bridge piers had been welded when originally installed restraining the intended movement. Additionally, the bearings at the abutments had ceased to function due to corrosion and no longer provided the range of motion that was expected with a bronze bearing system. The inspection also showed that the bridge's concrete deck was found to be integral with the approach slab for nearly 9 meters (30 ft) at both ends of the bridge. These three conditions had resulted in very stiff end and internal bearing conditions that would later be verified by the testing results.

The centerline length of the bridge was 46.3 m (152.3 ft). The two end spans had centerline lengths of 12.6 m (41.5 ft). The middle span had a centerline length of 21.1 m (69.3 ft). The bridge radius was 145.6 m (477.5 feet) and incorporated a vertical curve that transitioned from a slope of -1.5% to -6.1% in 106.7 m (350 ft) along its length. Steel plate girders at a spacing of 2.69 m (8 ft 10 in.) were used to support the deck and had web depths (measured from the inside of the top and bottom flanges) of 0.965

m (38 in.) and widths of 7.94 mm (5/16 in.). Fig. 2 shows a typical cross section of the bridge. The innermost 3 girders (girders 1, 2, and 3) spanning between the abutments and piers had flange thicknesses of 19.1 mm (0.75 in.). The outside two girders (girders 4 and 5) had flange thicknesses of 25.4 mm (1 in.) over the end spans and 41.3 mm (1-5/8 in.) over the center span. All flanges were 304.8 mm (12 in.) wide. Diaphragms were placed at all bearing locations, at third points in the first and third spans, and at fifth points in the second or largest span. The intermediate diaphragms were constructed with partial depth, C 15 x 40 standard sections that were positioned radially to the girders. All supporting piers and abutments were aligned radially to the bridge.

The 12.9 m (42.3 ft) wide bridge deck had a 6.0 percent super-elevation and was composed of 203 mm (8 in.) thick reinforced concrete with a 76.2 mm (3 in.) thick asphalt overlay. Strength tests were performed on several core samples from the bridge deck. These tests showed the concrete to have an average compressive strength ($f'c$) of 35.1 MPa (5090 psi) which was significantly higher than the specified design strength of 20.7 MPa (3000 psi).



Figure 1. Bridge Elevation View

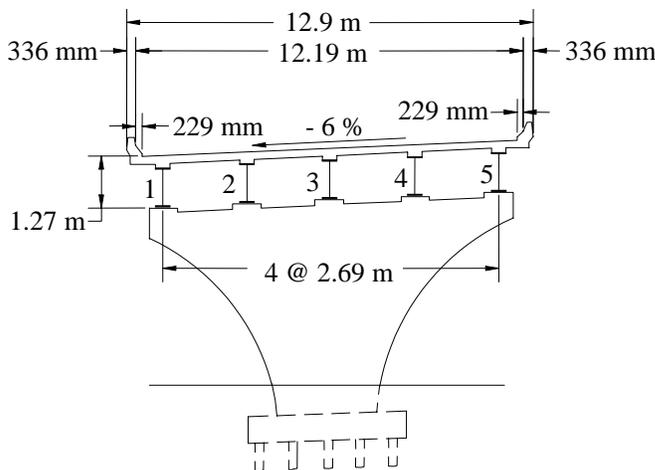


Figure 2. Cross-Sectional View of Intermediate Pier

Boundary Condition States

The dynamic forced vibration testing conducted on this bridge were performed for three boundary condition states. The first state was the “as is” condition which included the welded and frozen bearings along with the integral approach slabs described earlier. This first test was used as a baseline measurement to compare the effects that changes in boundary conditions had upon the bridge behavior. The testing of the second boundary condition state was performed with the same bearing support conditions; however, both integral approach slabs were severed. This left a gap at both ends of the bridge which effectively “freed” the bridge deck from the approach slabs. The third boundary condition state consisted of reducing the translational and rotational resistance at the bearing points of the girders. The original abutment supports which consisted of welded bronze bearings were replaced with Teflon/stainless steel bearings at both ends of the bridge. Additionally, the welds on the intermediate bearings at the piers were removed and the bearings were greased. These boundary condition states provided a verifiable way of adjusting the bridge’s boundary conditions in order to facilitate correlation between the support conditions and changes in the bridge’s dynamic properties.

Forced Vibration Testing

Lateral load testing of the curved-girder bridge was accomplished through the application of forced vibration. Two separate methods of forced vibration testing were used; sinusoidal and impact forcing. Both of these methods were employed for each of the three boundary condition states to allow for a comparison of the bridge’s dynamic response when subjected to lateral forces. An eccentric mass shaker was used to generate the sinusoidal force applied to the bridge. The shaker was capable of imparting a sinusoidal forcing function of up to 89 kN (20 kips) in any horizontal direction. The shaker was mounted on the bridge at a location where it would not coincide with a modal node. A “Radial” test was performed using the shaker to induce sinusoidal forcing in a direction perpendicular to the longitudinal axis of the bridge. The shaker was also used to perform a “Tangential” test which consisted of inducing a sinusoidal force in a direction parallel to the longitudinal axis of the bridge. Both of these sinusoidal forcing tests were performed with the eccentric-mass shaker applying the load to the structure in a frequency range from 0.5 Hz to 20 Hz in 0.02 Hz increments. Fig. 3 shows the location of where the shaker was applied to the test structure.

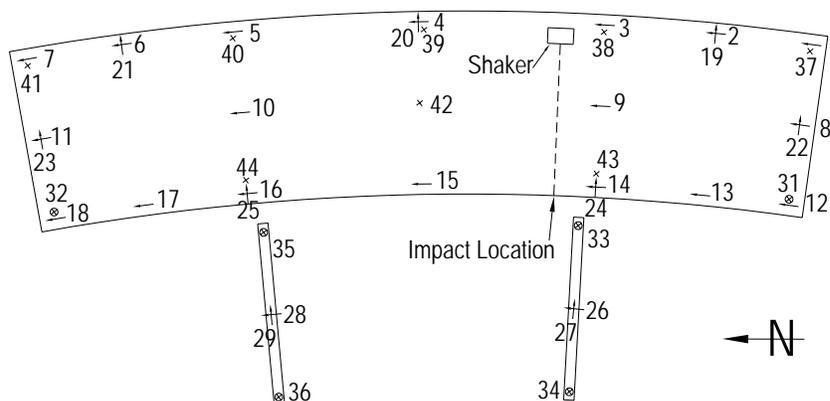


Figure 3. Bridge Sensor Layout

Striking the side of the bridge deck with a 2.5 kN (560 lb) hammer was used to generate an applied horizontal force during the “Impact” test. The location selected for striking the bridge was determined to be on the inside radius of the bridge directly along a radial line coinciding with the location of the eccentric mass shaker. The impact point of the hammer was perpendicular to the concrete deck of the bridge and located below the jersey barrier and above the steel girders. Each impact test consisted of

striking the bridge 32 times to ensure an accurate recording had been taken. Sinusoidal (both radial and tangential) and impact testing were performed for each of the three boundary condition states.

To measure the bridge's response due to the forced vibration, the structure was instrumented with thirty six 1-Hz velocity transducers and eight 1-g vertically oriented accelerometers for a total of 44 channels. Fig. 3 also shows the layout of the sensor array with instrumentation placed at the bearing points, mid-spans, and at selected locations along the centerline of the bridge. The number marks the sensor location and the arrow indicates the direction of the measured response. A 16-bit A/D card with anti-aliasing filters was used to collect the data from the sensor array.

Forced Vibration Testing Results and Evaluation

The first step in evaluating the raw data from the bridge was representing the bridge response due to varied forcing amplitudes and methods. The response for each forcing magnitude was normalized by the amplitude of the applied forcing function. Therefore all displacements are in units of length per unit force. In addition, a fast Fourier transform was performed to translate the data from the time domain into the frequency domain to facilitate identification of the natural frequencies. Once all the data were transformed and normalized the frequency response functions were plotted for each channel.

Figure 4 illustrates a sample of the frequency response function (FRF) plots of selected channels during State 1 for forcing frequencies ranging from 1 to 10 Hz. The plot is based on data from an impact test. The natural frequencies in these plots are represented as peaks that correlate to the structures amplified resonance when resonating at a natural frequency. This approach has been used in several past studies (De Roeck et al. 2000). Some channels, because of their orientation or location, will not exhibit all frequencies. For example, a channel located parallel to the long axis of the bridge will likely miss a mode that contains movement largely perpendicular to the long axis of the bridge. Because of this, the overall natural frequencies reported are a synthesis of the frequencies found at each channel with only the strongest results reported. Consequently, four of the most representative channels were selected for Fig. 4.

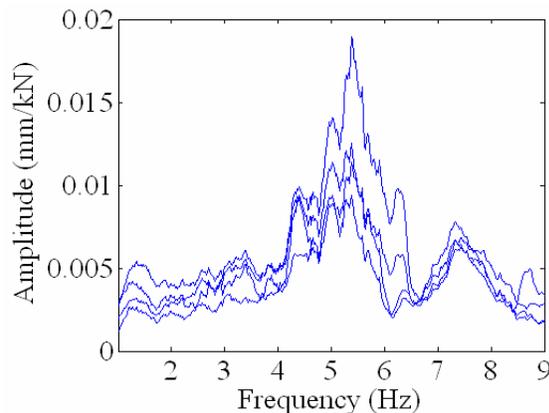


Figure 4. Combined FRF Plots for Impact Test 1

Because of the large amount of data that needed to be analyzed, a database was constructed to compare the frequency response functions recorded at each channel for each boundary condition state. The natural frequencies resulting from each frequency response function were entered into this database and analyzed to determine the overall modal frequencies recorded for each boundary condition state of the bridge. As mentioned earlier, only natural frequencies that were exhibited in a large number of channels and were represented in both impact and sinusoidal testing methods are reported. Table 1 lists the first six modes as determined by this analysis.

Table 1. Forced Vibration Natural Frequencies

Mode	State 1 (Hz)	State 2 (Hz)	State 3 (Hz)
1	4.44	3.84	1.49
2	5.42	4.37	2.95
3	5.96	5.61	5.20
4	7.34	6.09	5.60
5	8.00	6.73	6.27
6	8.72	8.49	8.48

In a comparison of the natural frequencies of the three states, Table 1 shows that as the restraint of the bridge was reduced the natural frequency was also reduced. In general, Modes 1 and 2 experienced the largest change in natural frequencies for the different boundary condition states. Reducing the restraint at the bearing supports (comparing State 2 to State 3) had the largest influence of the natural frequencies of the bridge reducing the natural frequencies for Modes 1 and 2 by more than 61% and 32% respectively. On average, severing the approach slab with a concrete saw from the bridge deck (comparing State 1 to State 2) reduced the natural frequencies by roughly 12% and replacing the bearings (comparing State 2 to State 3) reduced the natural frequencies by nearly 20%.

Once the modal frequencies were determined, the corresponding mode shapes were then plotted using the response data at the natural frequencies listed in Table 1. The plots of the mode shapes provided valuable observations regarding the global behavior of the bridge. For State 1 the bridge responded with a greater amount of torsional movement than in States 2 or 3. This can be attributed to the frozen bearings, welded bearings, and integral approach slab previously discussed. These supports provided for very stiff end bearing conditions which were reflected in the mode shapes. The bridge's tendency to move torsionally in the "as is" state illustrates that the bearings and end conditions were not functioning as a pin and roller. The frozen bearings were stiff enough to force bending and rotation in the bridge structure before allowing sliding or rotation to occur at the bearing points.

For the second boundary condition state, the approach slabs were severed which provided the bridge with a greater ability to translate. While allowing greater translation, this change in boundary conditions also changed the measured mode shapes and their order of occurrence with respect to State 1. The first two modes remained as the first transverse bending and first torsional modes but switched order of occurrence. In addition, both modes decreased in frequency due to the changed boundary conditions. Additionally, the released boundary conditions allowed the bridge to begin exhibiting more rigid body movements. Modes 1 and 3 in the second boundary condition state moved laterally as a rigid body. On the whole, it can be stated that severing the approach slab profoundly altered the response of the bridge. From this, the importance of understanding a structure's boundary conditions and fixities can be illustrated. Changes in the connections within a structure or of the structure's bearing on surrounding surfaces can alter its response significantly.

Boundary condition changes from state 2 to state 3 also significantly altered the bridges response. Because of the reduction in restraint that was induced in this condition state the bridge translated enough to close the saw-cut gap between the deck and approach slab creating a contact point at the North abutment between the approach slab and the bridge deck. This contact occurred during both the radial and tangential sinusoidal forcing tests. In contrast, the closing of the gap did not occur during the impact test or the live-load test. Because of the difference in gap behavior, the mode shapes recorded during the two sinusoidal tests and the impact test were also different. Fig. 5 shows the difference in the recorded mode shapes for the radial and impact test, respectively. The contact on the north abutment caused the formation of a pivot point around which the entire bridge rotated during radial translational modes. The point of contact did not affect all modes in the response. Modes that were torsional or vertically dominant were not affected because the movement was not in the direction of the gap.

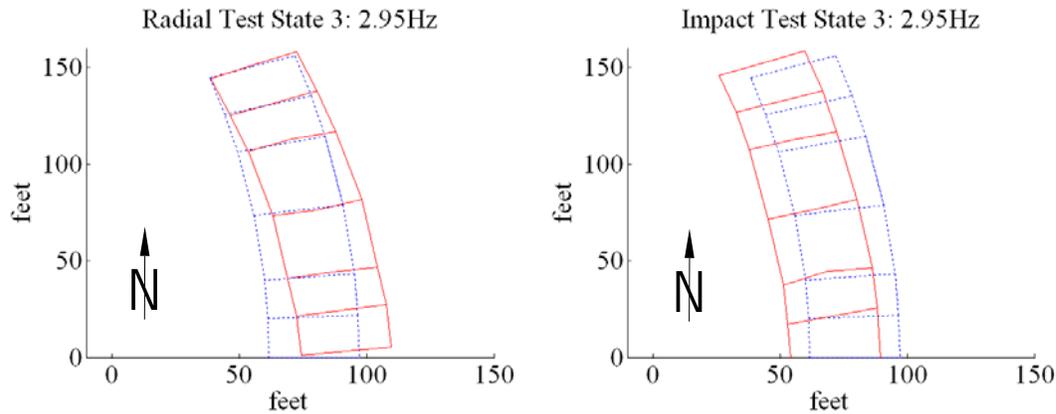


Figure 5. Radial response vs. Impact Test Response for State 3

Two conditions are believed to be the cause of why the contact point was not manifested in the impact test. First, the amount of energy applied to the system during an impact test is much smaller than the energy applied during a sinusoidal test. In many cases, large amounts of information can reliably be extracted by applying reasonably small forces. However, as is illustrated with the results of this test, at times a small amount of energy can fail to extract certain behavior by not having the ability to isolate special structural responses that occur due to the vibration and resonance of the system. Secondly, during the sinusoidal testing the shaker starts by rotating at the lowest frequency for which recording will take place. From there the forcing frequency is increased in increments of 0.02 Hz. At each incremented frequency, the bridge is given 20-30 seconds for the steady state response to stabilize before data recording is initiated. Once begun, the data collection system will record response behavior for a period of approximately 20 seconds. The frequency of the shaker is then incremented higher and the monitoring process starts over again. This testing procedure allows for the effects of resonance to take place when the bridge is excited at its natural frequency. The bridge can be allowed to vibrate at or near its natural frequency for a number of minutes when taking into account the forcing in the frequency steps just before and after the natural frequency. This extended period of resonance allows for more bridge displacement and energy absorption. A sinusoidal test is therefore more likely to promote the formation of contact points than an impact test. This field observation is important when considering the changes in structural response as the level or type of forcing is altered.

Conclusions

The reconstruction of the I-15 corridor through Salt Lake City, Utah provided an excellent opportunity for dynamic testing of a steel, curved-girder bridge under three different boundary conditions. Dynamic testing evolved using both sinusoidal as well as impact excitation with an array of velocity transducers and accelerometers to collect the response. The measured performance of the bridge was analyzed resulting in the following conclusions:

- The results of the dynamic testing showed that changing the boundary conditions produced changes in modal frequencies of up to 34%. Because small changes in modal properties are difficult to attribute to damage, the significantly larger changes in dynamic properties found in this study are an encouraging indication of the feasibility of structural health monitoring using dynamic techniques.
- A reduction in restraint stiffness resulted in a change in the order of modes for each of the tested boundary condition states. For example, changes in boundary conditions led to the introduction of modes that were not present in the original structure. This was verified by a MAC analysis between boundary condition states that indicated many modes changed shape or shifted their

order of occurrence from state to state.

- The test results showed that a few modes increased in natural frequency when a specific mode shape was identified using MAC correlations to ensure similarity. This result verified initial expectations that as boundary conditions changed; altered mode shapes were expected.
- Evidenced in condition State 3, impact testing may not be suitable for testing certain types of structures because it cannot provide enough energy or time for resonance to promote the formation of contact points. A comprehensive testing program should include careful consideration of the strain levels of interest and a recognition of the limitations in small strain testing.

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