Dynamic Behavior of Steel Deck Tension-Tied Arch Bridges to Seismic Excitation

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Abstract: The dynamic responses of steel deck, tension-tied, arch bridges subjected to earthquake excitations were investigated. The 620 ft (189 m) Birmingham Bridge, located in Pittsburgh, was selected as an analytical model for the study. The bridge has a single deck tension-tied arch span and is supported by two bridge piers, which in turn are supported by the pile foundations. Due to the complex configuration of the deck system, two analytical models were considered to represent the bridge deck system. Using the normal mode method, seismic responses were calculated for two bridge models and the results were compared with each other. Three orthogonal records of the El Centro 1940 earthquake were used as input for the seismic response analysis. The modal contributions were also checked in order to obtain a reasonable representation of the response and to minimize computational cost. Displacements and stresses at the panel points of the bridge are calculated and presented in graphical form.

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Introduction

Behavior of highway bridges under seismic excitation is an important design consideration. The inadequacy of many highway bridges to withstand dynamic effects of seismic ground motion has been isolated as the reason for considerable damage to a number of those bridges during the 1971 San Fernando, 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe earthquakes. In response to this concern, considerable research has been conducted on the safety of highway bridges under seismic loads (Tseng and Penzioni 1973; Imbsen et al. 1978; Fleming and Egeseli 1980; Abdel-Ghaffar and Rubin 1982, 1983; Torkamani and Koubha 1991; NEHRP 1998). However, similar studies concerning the seismic response analysis of arch bridges have been scarce. Arch bridges are used throughout the world in both short and long span situations. If arch bridges are to be designed safely or guidelines developed to predict possible damage, it is essential that the designer have information concerning their behavior under static and dynamic loads.

This research presents the results of an investigation concerned with the dynamic response of tension-tied arch bridges to seismic loads. The Birmingham Bridge, located in Pittsburgh, Pa., was selected for the analytical model of the bridge type in this study. An arch bridge is a complex three-dimensional system for which it is difficult, if not impossible, as for most such three-dimensional systems, to obtain an exact algebraic solution for the displacements and stresses predicted by seismic ground motions. In seismic evaluation, it is desirable to develop an appropriate analytical model by considering a number of proper assumptions that will adequately represent the desired dynamic characteristics of the structure. Two computer models were considered for representing the bridge deck system: one was a series of simple beam models; the other was modeled by using substructures. The maximum seismic responses at panel points for the two bridge models were calculated by using the normal mode method. The validity of the simple beam model was discussed by comparing the results of seismic responses of the two models to each other.

The other emphasis in the study was modal contributions on the seismic response analysis of the bridge type. Generally there are many modes in a low frequency range for a complex three-dimensional bridge model in which the frequencies are also closely spaced. It is questionable how many modes should be derived and used in analysis to accurately predict the structure’s responses. Thus, it is necessary to check and derive the modal contributions in order to obtain a reasonable representation of the response while minimizing computational cost. In the present study, the modal contributions were investigated on the second bridge model under seismic ground motions.

The El Centro earthquake of 1940 was chosen in this study, since its accelerograms are readily available in three directions and because its peak ground acceleration also occurs within the first several seconds. The vertical ground motion record was used as vertical inputs while the S90W component and the S00E component were used as longitudinal and lateral inputs, respectively.

Description of Birmingham Bridge

The Birmingham Bridge—which is a six-lane, solid-ribbed, steel-deck-tied, through-arch span—was constructed across the Monongahela River in Pittsburgh, in the late 1970s. The bridge superstructure consists of a 620 ft (189 m) arch span and a 100 ft 9 in. (31.71 m) wide roadway. At the middle of the arch span, the height of the arch ribs is 150 ft (45.7 m) from the deck level. The bridge superstructure is supported, at the ends of the arch span, by
two bridge piers, and each pier foundation is supported, by a large
group of piles, on a type of sand with medium density.

The arch span, shown in Fig. 1, consists of two tension-ties,
floor-beams, deck slab, two arch ribs and their lateral bracings,
and a set of suspenders. The segmental arch rib consists of two
rectangular steel box girders spaced 108 ft 9 in. (33.15 m) apart
and hinged to the tension-ties at the ends of the arch span. The
deck system for the arch span, which consists of 20 panels, each
31 ft (9.45 m) in length, is designed in accordance with AASHTO
Standard Specifications for Highway Bridges (1965) with
HS20-44 loading. A 7 1/2 in. (19.05 cm) two-way reinforced con-
crete slab is supported longitudinally by wide flange floor string-
ers that in turn are supported laterally by wide-flange floor-beams
located at each panel point. These floor-beams are then supported
by two rectangular steel box tension-ties, which are hung to the
arch ribs at each panel point by a set of cables. The concrete slab
is spliced along the centerline of the bridge, and the continuity of
each end-floor-beam is broken at the middle of the beam. There
are two piers located at the ends of the arch span. The tension-ties
at the south end are connected to the pier by hinge joints, while
the tension-ties at the north end are connected by roller joints. All
necessary information for bridge modeling was drawn from blue
prints, which were provided by the Pennsylvania Department of
Transportation, Pittsburgh Section.

General Modeling Notes

A realistic model of an arch bridge is a damped three-dimensional
system with three translational degrees of freedom (DOF) at the

masses. However, there are many factors that may be considered
in developing a lumped mass mathematical model for an arch
bridge. The dynamic properties of the mathematical model de-
pend upon the following: (1) the number and location of the con-
centrated masses to be considered in order to include all impor-
tant DOF of the bridge; (2) the DOF that are included at the
lumped masses during the analysis; (3) the development of the
stiffness matrix and the type of elements that are used in this
development; (4) the effect of foundation deformation upon the
response of the superstructure and how it may be incorporated in
the mathematical model; and (5) the magnitude and type of damp-
ing incorporated in the mathematical model.

The general description that is used in the development of the
mathematical models is as follows:

1. The global coordinate axes are oriented from the southwest
end of the arch span, where the x-axis is horizontal and
to the tension-ties, the y-axis is vertical and perpen-
dicular to the deck surface, and the z-axis is horizontal and
perpendicular to the tension-ties.

2. The cross-sectional areas of members, moment of inertia,
and torsional constants are based on the typical member
cross sections with stiffeners, splice plates, etc., neglected.

3. The cross-fall and longitudinal slope of the bridge deck is
also ignored.

Analytical Modeling

The arch elements, tension-ties, and suspenders were modeled
individually. The arch elements consist of ribs, struts, and arch
diagonals. The segmental ribs are laterally connected by wide-
flange struts and diagonals, as shown in Fig. 1. There are seven
different arch rib cross sections along the bridge. One typical
section at the quarter point of the arch span consists of an 84
× 54 in. (2.13 × 1.37 mm) steel box girder with thickness of 1 3/4
in. (4.445 cm) for web plates and 1 1/2 in. (3.81 cm) for flange
plates.

The deck system consists of reinforced concrete slab, string-
ers, and floor-beams. The modeling of a deck system is relatively
ambiguous due to its complex configuration. For example, the top
flange of the stringers are embedded in the concrete slab and the
lower flanges of the stringers are connected to the floor-beams
with high-strength bolts. Composite section was assumed in the
modeling with regard to the flexural rigidity of the deck system.

Each pier foundation was represented by an elastic boundary
matrix at the center of the bottom plane of the pier footing. The
dynamic coefficients of the matrix were computed according to
the studies by Novak (1974), Novak and El Sharnouby (1984),
Roesset (1980), El Sharnouby and Novak (1984), Poulos (1979),
Novak and Beredugo (1971), and Prakash (1992). The masses of
arch elements were properly lumped at panel points on the arch
ribs along the bridge.

There is some uncertainty in choosing an analytical model that
is capable of appropriate representation due to the complex con-
furation of the deck system. In many cases, the deck system was
represented by a series of simple beams in their analytical models
of the structures. However, the adequacy of the simple beam
model is questionable for representing the bridge deck system in
the seismic response analysis of the bridge type. To investigate
the accuracy of seismic responses calculated from the simple
beam model, a second analytical model for the deck system was
considered in the study, and the results of the responses from the
two models were compared with each other.
**Bridge Model I**

The bridge deck was modeled using some overly simplifying assumptions about bridge deck behavior, such as treating the bridge deck as a series of lateral beams connected to the tension-ties at panel points. A lateral beam served to replace the given floor-beam and the associated portion of the concrete slab. Because of the rigid action of the bridge deck in its own plane, similar to the floor system in building analysis, the axial displacements at the ends of the lateral beams would have been nearly equal. Therefore, assuming rigid motion in its axial direction, a large value for the cross-sectional area of the beams was assigned in the computer model. The flexural rigidities of the stringers are neglected in the calculation of the equivalent beam rigidities. The lumped masses for the simple-beam model are shown in Fig. 2. This model has 235 dynamic DOF. There are two roller joints between the tension-ties and the pier at the north end. Since the x-axis force was assumed nontransferable through these joints, it was necessary for the longitudinal translation to lump the mass of deck portion and pier portion separately. For the computer model, three mass less rigid members were placed between the deck and the pier at the panel points.

**Bridge Model II**

Rather than attempting a simplified analysis treating the deck components as an equivalent beam element, a substructure that may be referred to as a “super-element” was chosen so that a more accurate representation in determining properties of the deck system could be achieved. This model has 40 super-elements, with each super-element having 24 DOF. Each super-element consists of 19 subelements. Two lateral beams that are parallel to the z-axis are interconnected by 11 longitudinal stringers parallel to the x-axis. One longitudinal girder, which is also parallel to the x-axis, is part of the tension-tie system, reinforced concrete slab, and four tie bracings. The tie bracings are 14BP73 sections, which run in a zig-zag fashion between the floor-beams, as shown in Fig. 1. A typical super-element model is depicted in Fig. 3. The tie bracings were modeled as truss elements. One should note that two additional tie bracings may be omitted from any position simply by specifying zero properties in the computer model.

A lateral beam served to replace the given floor-beam and associated portion of concrete slab. Similarly, a longitudinal beam served to replace a stringer and associated portion of concrete slab. Each substructure is connected to the adjacent substructures at panel points and at the center of the bridge along the floor-beams. As noted in bridge model I, the lateral and longitudinal stiffness of the bridge deck are usually higher than the overall vertical stiffness of the deck. Several joints in the bridge deck would have nearly equal displacements along their own plane. Therefore, the lumped masses for the lateral and longitudinal DOF were represented at certain panel points on the bridge deck. In addition, three mass less rigid members were placed at the
panel points between the deck and the pier at the north end as in bridge model I. Fig. 4 depicts the dynamic DOF assigned at panel points along the bridge for bridge model II. This model has 205 dynamic DOF.

**Tension-Ties**

Each tension-tie consists of a 138 in. (350.5 cm) rectangular steel box girder with a 7/8 in. (2.222 cm) web thickness and with the flange varying in thickness from 1 1/4 to 2 in. (3.175 to 5.08 cm) section by section. These tension-ties, hung to the arch-rib by a set of bridge strands at each panel point, support the bridge deck system. Each tension-tie lying between the two panel points was modeled as a three-dimensional frame element.

**Suspenders**

The suspenders run between the tension-ties and arch ribs, located at each panel point, are composed of four 2 in. (5.08 cm) bridge strands. They are tensioned between 27 and 32 ksi (186.2 and 220.6 MPa) by the dead load of the deck and have a minimum breaking strength of 1,904 kips (8.47 MN).

With respect to the transfer of the y-axis forces from the tension-ties to the arch ribs, the actual connections allow the transfer of tension forces only. Because of the large tension forces on the suspenders, resulting from the dead load of the bridge deck, it was assumed that these dead loads would exceed any calculated compressive forces due to moderate earthquake accelerations. Therefore, the assumption was made that all of these suspenders be modeled as truss elements that resist both tensile and compressive loads.

**Bridge Piers**

Two reinforced concrete bridge piers, as shown in Fig. 5, are located at the ends of the arch span. A bridge pier consists of four columns having a circular hollow cross section with a 3 ft 6 in. (106.68 cm) thickness, a 14 ft 6 in. × 9 ft (4.42 × 2.74 m) rectangular cross section of pier cap with a 2 ft (60.96 cm) wide beam placed on the top of the pier cap, a tapered solid wall at the bottom of the columns, and a rectangular footing at the base. Each pier has a similar form except for the height of the columns and the size of the solid walls. In modeling the pier, six DOF at each joint were condensed out except for joints 1–4 (see Fig. 6).

In the model, the tapered solid wall and the footing, as shown in Fig. 6, were assumed to be one rigid block. The stiffness matrix of each column is transformed to node 1 at the center and the bottom of the rigid block. For example, the stiffness matrix of column \(i\) with nodes \(j\) and \(k\) may be expressed as

\[
[K_{ii}] = [T_i]^T[K_i][T_i]
\]

(1)

where

\[
[T_i] = \begin{bmatrix} RT_j \\ RT_k \end{bmatrix}
\]

(2)

and

**Fig. 4. Lumped mass for bridge model II**

**Fig. 5. Bridge pier profile**
The dynamic stiffness of a pile group is most often evaluated using the transformation matrix and obtained as 14.7 for the long direction and 12.4 for the short direction. With the given properties of the piles and soil, the horizontal interaction factor based on Poulos’ (1979) solution was computed and obtained as 14.7 for the long direction and 12.4 for the short direction.

**Lumped Masses**

The mass of the bridge superstructure was calculated and lumped at the panel points in the structure. The mass lumped at the center of one arch rib at a given panel point includes one-half the mass of the arch rib lying between the given panel point and the adjacent panel points with or without one-half the mass of the strut.
depending on the given point. The mass of the arch diagonals lying between two arch ribs is also lumped properly on the given panel point.

The mass of the bridge deck and the tension-ties was calculated and distributed to all panel points on the deck level along the longitudinal direction of the bridge. The mass of the pier cap and one-half of the columns was lumped at their joints and linearly redistributed to the proper panel points. The bottom portion of each column mass was lumped as point mass at the top of the solid wall where they are fastened. The mass moments of inertia were then calculated for the rigid block and point masses with respect to the \( x \)-, \( y \)-, and \( z \)-axes passing through the center at the bottom plane of the rigid block. The mass of the pile group was added to the mass point of the rigid block for the horizontal response, while the total masses of the pole group and the soil enclosed were included for the vertical response. More information regarding the mass calculations and distribution is given in Lee and Torkamani (1998).

### Dynamic Characteristics

Computation of the dynamic characteristics of the structure is necessary to investigate the response to dynamic loads. However, the following questions should be posed before calculating stresses and displacements of the selected nodes: (1) How many modes or modal frequencies should be derived for an arch bridge in order to accurately predict the structure’s responses? and (2) How significantly does each mode contribute to the response of the arch bridge for seismic loading?

The three-dimensional model of the Birmingham Bridge has many physical DOF. Masses were assigned to the selected DOF (dynamic DOF); therefore, a reduction technique was needed for the assembly of the stiffness matrix. Within the computer programs, which generate the stiffness matrix of the bridge model, the frontal solution (Hinton and Owen 1977) was employed to reduce core storage requirements. The assembly and reduction process was carried out systematically from the left side of bridge to the right. The dynamic characteristics of translational and torsional vibrations of the three-dimensional bridge models were calculated. The frequencies of the first 15 modes are listed in Table 1.

The computed natural frequencies of the bridge models revealed many modes of the translational and torsional vibrations in the lower frequency range. The fundamental modes for the bridge models consist mainly of a single wave of vertical translation. Their frequencies are 0.39236 and 0.39152 Hz for bridge models I and II, respectively. These fundamental frequencies of vibration for the bridge models, as usual in most bridges, seem much smaller than those of most buildings. It is interesting to note that mode 3 for bridge model I and mode 2 for bridge model II show a rigid displacement of the arch ribs and bridge deck in the longitudinal direction (Lee and Torkamani 1998).

### Seismic Response Analysis

From an earthquake-engineering point of view, one of the most important problems is to discover which are the critical sections of a structure and to estimate where the maximum displacements or the maximum stresses may occur. Assuming linear elastic behavior, the maximum responses for the two bridge models were calculated at panel points along the bridge by using the response-spectrum method [ten percent method recommended by the ASCE Standard for Seismic Analysis of Safety Related Nuclear Structures (1986)]. The maximum stresses at the ends of the bridge members were computed by the square root of the sum of the squares method. Three orthogonal records of the El Centro 1940 earthquake were used as support excitation for the seismic response analysis of the bridge.

The overall damping for a given mode of the soil-structure system is a complex value made up of the energy dissipated by the structure, energy losses from internal friction, and wave radiation into the foundation medium. In the structure itself, damping is usually specified as damping ratios for the structure modes, while energy dissipation in the foundation is measured by the coefficients of equivalent frequency-dependent dampers. Bielak (1976) developed an approximate procedure for estimating modal damping ratios in soil-structure systems. This procedure was used for estimating damping ratios in the modal analysis of the bridge considered in our study. Table 2 shows the computed damping ratios for both bridge models assuming 4% critical damping in the superstructure of the bridge.

#### Table 1. Modal Frequencies of Bridge Models

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Bridge model I</th>
<th>Bridge model II</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.39236 0.39152</td>
<td>0.39152 0.39152</td>
</tr>
<tr>
<td>2</td>
<td>0.54857 0.68849</td>
<td>0.54857 0.68849</td>
</tr>
<tr>
<td>3</td>
<td>0.67260 Longitudinal</td>
<td>0.83291 Longitudinal</td>
</tr>
<tr>
<td>4</td>
<td>0.78323 Torsional</td>
<td>0.83474 Torsional</td>
</tr>
<tr>
<td>5</td>
<td>0.82445 Vertical</td>
<td>0.85488 Torsional</td>
</tr>
<tr>
<td>6</td>
<td>0.84433 Torsional</td>
<td>1.08003 latera</td>
</tr>
<tr>
<td>7</td>
<td>1.14933 Lateral</td>
<td>1.34698 Lateral</td>
</tr>
<tr>
<td>8</td>
<td>1.21836 Vertical</td>
<td>1.54538 Vertical</td>
</tr>
<tr>
<td>9</td>
<td>1.37443 Vertical</td>
<td>1.59463 Vertical</td>
</tr>
<tr>
<td>10</td>
<td>1.58416 Torsional</td>
<td>1.68011 Torsional</td>
</tr>
<tr>
<td>11</td>
<td>1.58657 Vertical</td>
<td>1.73406 Torsional</td>
</tr>
<tr>
<td>12</td>
<td>1.72327 Torsional</td>
<td>2.43153 Lateral</td>
</tr>
<tr>
<td>13</td>
<td>1.75841 Lateral</td>
<td>2.50884 Vertical</td>
</tr>
<tr>
<td>14</td>
<td>2.21074 Lateral</td>
<td>2.5394 Lateral</td>
</tr>
<tr>
<td>15</td>
<td>2.36088 Lateral</td>
<td>2.62023 Torsional</td>
</tr>
</tbody>
</table>

#### Table 2. Modal Damping Ratios for Bridge Models

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Bridge model I</th>
<th>Bridge model II</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.04260</td>
<td>0.04240</td>
</tr>
<tr>
<td>2</td>
<td>0.04590</td>
<td>0.39263</td>
</tr>
<tr>
<td>3</td>
<td>0.37197</td>
<td>0.04734</td>
</tr>
<tr>
<td>4</td>
<td>0.05055</td>
<td>0.04112</td>
</tr>
<tr>
<td>5</td>
<td>0.04255</td>
<td>0.04499</td>
</tr>
<tr>
<td>6</td>
<td>0.04400</td>
<td>0.07126</td>
</tr>
<tr>
<td>7</td>
<td>0.04765</td>
<td>0.04171</td>
</tr>
<tr>
<td>8</td>
<td>0.05242</td>
<td>0.04680</td>
</tr>
<tr>
<td>9</td>
<td>0.07449</td>
<td>0.04110</td>
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<td>10</td>
<td>0.04819</td>
<td>0.04966</td>
</tr>
<tr>
<td>11</td>
<td>0.04110</td>
<td>0.04492</td>
</tr>
<tr>
<td>12</td>
<td>0.04297</td>
<td>0.21817</td>
</tr>
<tr>
<td>13</td>
<td>0.05271</td>
<td>0.04092</td>
</tr>
<tr>
<td>14</td>
<td>0.13508</td>
<td>0.12382</td>
</tr>
<tr>
<td>15</td>
<td>0.05188</td>
<td>0.05397</td>
</tr>
</tbody>
</table>
Note that “stress” represents the maximum stress in the following discussion. In addition, “major axis stress” or “minor axis stress” refers to the total stress—the combined-axial stress plus the major axis bending stress or the minor axis bending stress, respectively. Similarly, “displacement” represents the maximum displacement at a given panel point.

**Adequacy of Bridge Deck Model**

Given a set of earthquake load specifications, the goals of seismic analysis are to ensure design adequacy in terms of requirements, such as allowable stresses and displacements, and to improve reliability and economy within these requirements. However, if the properties of the structure are not adequately represented in its analytical model, the computed response from the model will not provide true responses of the system.

One of the simplest analytical models for bridge analysis is that the bridge deck be modeled as a series of simple beams as assumed in bridge model I. To investigate the adequacy of the simple beam model, the maximum responses from bridge model I under seismic motions were computed, and the results were compared with those from bridge model II in which the bridge deck is represented more realistically in its analytical model.

Fig. 7 depicts the directional displacements of the arch rib and the tension-tie at panel points along the bridge computed from the bridge models. Similarly, comparisons of the maximum stresses in the bridge members were made and are shown in Fig. 8. In Figs. 7 and 8, the responses from bridge models I and II are represented by line with solid-circles and line with hollow-circles curves, respectively. From the figures, the principal differences in the responses between the two bridge models are:

1. Arch rib and tension-tie displacements were found to be larger in bridge model II than in bridge model I, especially vertical displacements, which were significantly different between the two models;
2. As the result of large vertical displacements in bridge model II, larger major axis stresses in the arch ribs and the tension-ties and larger

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**Fig. 7.** (a) Maximum x-axis rib displacements for bridge models I and II; (b) Maximum y-axis rib displacements for bridge models I and II; (c) Maximum z-axis rib displacements for bridge models I and II; (d) Maximum x-axis tie displacements for bridge models I and II; (e) Maximum y-axis tie displacements for bridge models I and II; (f) Maximum z-axis rib displacements for bridge models I and II.
axial stresses in the suspenders were obtained from bridge model II than those from bridge model I; and (3) the tension-tie members were significantly displaced in the lateral direction in bridge model I, and, subsequently, their minor axis stresses were greater than the stresses computed from bridge model II.

In the modeling of tall buildings, many analysts assume the floor systems in the structure to be rigid diaphragms. However, the deck flexibility for bridges would be a significant factor in the calculation of the bridge’s response. Thus, in the modeling of the bridge deck, neither an assumption of rigid diaphragm nor simple beams, as assumed in bridge model I, would be correct. Therefore, the bridge deck system should be modeled by considering rigidities of the deck system, both in lateral and longitudinal directions, in order to obtain accurate seismic responses of the bridge. In other words, a series of simple beams would inadequately represent the bridge deck in the calculations of bridge seismic responses.

Fig. 8. (a) Maximum major axis rib stress for bridge models I and II; (b) Maximum minor axis rib stress for bridge models I and II; (c) Maximum major axis tie stress for bridge models I and II; (d) Maximum minor axis tie stress for bridge models I and II; (e) Maximum axial suspender stress for bridge models I and II

Fig. 9. Relative longitudinal displacement between panel points A and E
The seismic responses of the bridge deck in the longitudinal direction were also calculated from bridge model I under the seismic loads to obtain some measure of the effects of reducing the longitudinal dynamic DOF on the bridge deck. Under the first 10.22 of the El Centro earthquake, the relative displacements in the longitudinal direction between the left end panel point, A, and the left fourth panel point, E (see Fig. 1), were calculated. The maximum relative displacements between two panel points were found to be 0.137 in. (0.35 cm). Note that in the computation of the relative displacements of the bridge deck, panel points A and E were considered. This is because the longitudinal dynamic DOF on the deck level were considered for every fourth panel point in bridge model II, enabling one to see the significance of this reduction of dynamic DOF in the bridge deck modeling for the seismic response analysis of the bridge. Fig. 9 depicts the relative longitudinal displacements between panel points A and E in the time domain. As shown in Fig. 9, the relative longitudinal displacements between panel points on the bridge deck were so small that a few reductions for the longitudinal dynamic DOF on the bridge deck would not result in serious errors in the calculation of the seismic responses of the bridge. Moreover, the bridge deck was modeled more realistically in model II than in model I. In order to reduce the size of a full-scale model, it was assumed that the reduction of many of the longitudinal dynamic DOF at panel points on the bridge deck, as assumed in the bridge model II, would not lead to a serious error in the computed responses of the bridge.

It is interesting to note that critical seismic responses in bridge model II were found in the members near the ends, middle, and quarter points of the bridge span. For example, the arch rib minor
axis stress under seismic load shows its largest value near the ends of the arch rib. The major axial and minor axis bending stress at panel point U of arch rib TU were calculated as 5.3 ksi (36.5 MN/m²) and 21.1 ksi (145.5 MN/m²), respectively. When adding the dead load stress to the above seismic stresses, the total stress exceeded nominal yield stress by 22%. Under dead load, the minor axis stress at panel point U in the member TU (see Fig. 1) was calculated to be 13.8 ksi (95.2 MN/m²). In addition, the dead load compressive stresses transferred through the arch rib hinges were exceeded by the calculated minor axis bending stress at the joints under the seismic loads. Thus, partial separation of the arch pin and one of the arch rib hinge shoes, as in the deck arch bridges studied by Dusseau and Wen (1982), might occur during the period of seismic excitation.

Modal Contributions

As shown in Table 1, since there are many modes in a low frequency range and the frequencies of many modes are closely spaced, it is questionable how many modes should be derived and used to accurately predict the structure’s responses. Therefore, it is necessary to check and derive the modal contributions in order to obtain a reasonable representation of the response and reduce computational time. From the results of this analysis, the dynamic characteristics of each mode, as listed in Table 1, could also be clearly identified.

The modal contributions were investigated on bridge model II. By changing the total number of modes included in the computation, the maximum nodal displacements at critical sections were computed under the orthogonal ground motions of the El Centro 1940 earthquake. Each plot of Fig. 10 depicts the maximum displacement at a critical section as a function of the total number of modes involved in the calculation. Note that the critical sections under the seismic loads were selected from Fig. 6.

The following observations were made from the plots in Fig. 10 showing the modal contributions on the displacements of the bridge members under seismic loads. The longitudinal responses on the bridge deck members were dominated by the rigid body motion (mode 2), from which the mode contribution reached about 92% of the total responses. The arch rib longitudinal responses were dominated by the first six modes.

The vertical responses on the arch rib and the tension-tie members were dominated by the first torsional mode (mode 3) and the second vertical mode (mode 4). The modal contributions for the vertical responses of the bridge members were observed to be 67 and 18% from modes 3 and 4, respectively.

In the lateral responses of the bridge members, the arch rib members were dominated by mode 3, while the bridge deck members were dominated by the first lateral mode (mode 6). Note that mode 3 contains a strong lateral motion in the arch rib members, even though a torsional motion is dominated in this mode. Thus, about 99% of the total lateral responses on the arch rib members were contributed by mode 3. In addition, the modal contributions on the lateral responses of the bridge deck members were provided about 25% by mode 3 and about 75% by mode 6.

Based on the above observation, the modal responses for the bridge under seismic loads could be predicted to a certain level of accuracy by considering only the first six modes in the modal solutions. However, in order to accurately predict the results for the arch bridge, at least 10 modes should be considered in the calculations of the response analysis.

Conclusions

In order to assess the potentially damaging effects of earthquakes on tension-tied arch bridges, the Birmingham Bridge was chosen for study. Conclusions based on the findings of seismic responses made during the study are:

1. It was found that the lateral simple beam model (bridge model I) was not adequate to represent the bridge deck system. Thus, the lateral as well as the longitudinal rigidities of the deck system should be considered in the seismic response analysis of the bridge in order to accurately predict the structure’s responses.

2. The computed dynamic characteristics of bridge model II revealed many modes of closely spaced translational and torsional vibrations in the lower frequency range. However, the modal responses for the bridge under seismic motions could be predicted accurately by considering only the first 10 modes in the modal solutions.

3. To reduce the number of dynamic DOF, the mass of the bridge deck was lumped at panel points along the bridge and then properly redistributed to specified panel points. The results of this study validated the assumption that was made in the deck modeling.

4. Critical seismic responses were observed in the members near the ends, middle, and quarter points of the bridge. When the bridge member stresses due to dead load and live load were added to the seismic stresses under the El Centro 1940 earthquake, some of the members would be stressed beyond their yield points.

Based on our results, it seems that steel deck tension-tied arch bridges may be subjected to severe damage and potential failure under high intensity seismic motions. As previously indicated, the maximum stresses in some bridge members exceeded their yield stresses under the El Centro 1940 earthquake. A more comprehensive analysis of arch bridges would need to consider maximum resistance and, therefore, would require an examination of nonlinear behavior. Furthermore, it would be a worthwhile study for future research to conduct field testing on the bridge deck. By obtaining the dynamic characteristics of the bridge from the ambient vibration test, the validity of the assumptions made in the bridge deck modeling could be identified.

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