FANNY APPLETON
PEDESTRIAN
BRIDGE DETAILING
AND VIBRATION
PERFORMANCE

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SUMMARY
The Fanny Appleton Bridge is a pedestrian bridge that consists of a slender vierendeel arch that was part of the Longfellow Bridge Design-Build project. During the preliminary design phase, the structure was identified as being potentially sensitive to pedestrian induced vibrations. As a result, a vibration analysis was performed at the 75% design level. The results were provided to the owner and the acceptable levels of acceleration due to vibrations for final design was agreed upon. Alterations to the steel structure were made to improve the vibration response of the bridge instead of incorporating tuned mass dampers to meet the agreed criteria. The vibration analysis performed for the final design was later confirmed by testing and simulations with groups of up to 50 pedestrians.
FANNY APPLETON PEDESTRIAN BRIDGE VIBRATION ANALYSIS AND TESTING

Introduction

The Fanny Appleton Bridge is a pedestrian bridge that was included as part of a larger design-build project for the rehabilitation of the Longfellow Bridge that connects the cities of Boston and Cambridge in Massachusetts. The Fanny Appleton Bridge provides direct access for pedestrians to the Longfellow Bridge, along with Charles Circle, and the Esplanade park areas along the Charles River. The bridge replaced an existing structure that was narrow and could not support mixed use, it did not meet the requirements of the Americans with Disabilities Act (ADA), and was the cause of regular vibration complaints.

The Fanny Appleton Bridge is 750 ft long and consists of 200 ft of concrete ramp structure, and 550 ft of continuous steel superstructure. The main span is 220 ft and consists of an arched vierendeel truss system with diverging tubular arches and tubular columns. The main span was required to be very slender design as seen in Figure 1 to meet ADA grade requirements while tying into existing infrastructure on either end of the bridge and to meet clearance requirements over Storrow Drive. The bridge provides a deck width that varies between 12ft and 14ft for a mixed use of bicycles and pedestrians. The bridge curves in and out of the existing trees of the park and was intended to have a ribbon-like appearance. An aerial view of the bridge, surrounding infrastructure and parkland can be seen in Figure 2.

Initial Design

The main span of the bridge consists of two 18 inch diameter pipes with 1 3/8 inch walls. The spandrel columns vary in size based on the load demand and vary from HSS12x0.500 to HSS14x0.625. Spandrel columns are connected to the arch pipes by CJP welds. The tops of the spandrel columns are connected to continuous, 16 inch deep, built-up tub girders that run the full length of the bridge. The spandrel columns are welded to a cap plate with CJP welds. The cap plates were fabricated with alignment studs to allow the columns to be bolted to the tub section prior to fillet welding the cap plate to the tub girders. The remaining framing consisted of built-up I-shapes for cross members and infill framing in the plaza areas. All primary connections were required to be moment connected by contract requirements and were required to be unobtrusive. For this reason, most of visible connections on the bridge are welded.

The Fanny Appleton bridge was designed for 120 psf live loading. This exceeds the typical 90 psf required by AASHTO, but the 90 psf is expected to be exceeded yearly on this bridge during the Boston 4th of July fireworks events. This pedestrian bridge
was also designed and fabricated to the requirements of AWS D1.5 where possible. The tubular details and connections required the use of AWS D1.1 for design, fabrication, and inspection when not covered by the AWS D1.5 code.

As a contract requirement, the bridge was required to consider fatigue resulting from the design pedestrian loading that is expected to occur repeatedly when events on the Esplanade are held and loadings could approach the design live load. A conservative number of events was determined and it was assumed that the bridge would be fully loaded prior to and after the event, to account for the crowd arriving and leaving. Additionally, infinite wind gusts and truck gusts were considered for the fatigue design. As a result, tubular connections were specified to have improved weld profiles. This was achieved by the fabricator by grinding the welds to a smooth profile after the connection had been completed. Some connections, such as the arch pipe-base plate connection were evaluated using fatigue categories represented in the AASHTO signs and luminaires guide. These tubular base plate connections are not common for typical bridges, and do not necessarily fit the defined categories provided by AASHTO and AWS D1.5. Fatigue categories considered as part of the design include the following:

- Category D for tubular splices with backer left in place
- K1 for arch pipe at spandrel connections
- Category DT for spandrel member connections
- Category E for welded connections to tub girders with a radius less than 2 inches
- Category D for weld access holes in tub girders
- Category C’ for fillet welded stiffeners
- Category C for welded shear studs

Spandrel column connections were designed to transmit the full plastic capacity of the column. The column connections to the arch pipe and tub girder were designed using the CIDECT guides for circular and rectangular tubular connections. The connection to the tub girders required stiffening of the section to accommodate the anticipated loads in many locations due to the width of the bottom flange. A stiffened connection was provided at all tubular connections to the tub girders to provide uniformity. In addition to the tubular connection, cross beams were typically located at all spandrel column and pier locations, which required an additional full height stiffener to provide the necessary restraint for the crossbeam loads.

The crossbeams and fascia plate were an integral part of the design. All of the cross beam cantilevers were shop welded to the fascia. For transportation reasons, the cantilever portions of the crossbeams were field bolted, while the portion between the tub girders was shop welded. The fascia girder acted both as a pour stop and was utilized to support stay in place (SIP) forms. These sections would be preassembled to the extent possible, including the placement of SIP forms prior to erection of larger assemblies. Figure 3 shows the preassembled main span girder assembly during erection.

Figure 3: Preassembled Deck Section

**Initial Vibration Evaluation**

The preliminary design provided low vertical and longitudinal frequencies. The first transverse frequency was 1.80 Hz. The combined vertical and longitudinal frequency of the structure was 2.34 Hz. The AASHTO Guide Specifications for Pedestrian Bridges provides minimum frequency limits of 1.3 Hz for horizontal modes of vibrations and 3Hz for vertical modes. When bridge frequencies are above these minimum values, vibrations due to pedestrian loading is typically acceptable. Additionally, AASHTO allows for the use of an alternate criteria of a vertical frequency of \( f \geq 2.86 \times \ln(180/W) \) where \( W \) is the weight of the span under consideration in kips. This equation returns a negative frequency due to the main span length and weight. The bridge did not satisfy the 3Hz requirement but did satisfy the...
weight-based equation. Due to the significance of this structure, a more detailed analysis of the structure was deemed necessary. The SETRA guide, as referenced by AASHTO was used as the basis for this additional vibration analysis.

A 3d finite element model had been created using LARSA 4D for the design of the bridge, including all steel elements, shell elements for the concrete deck, and spring supports at the foundations. The vibration analysis included applying time history loads to the bridge deck in the design model, and were patterned to match the mode shapes of the structure. A damping of 0.6% was used for design as recommended by SETRA for steel structures. Figure 4 shows the combined vertical and longitudinal mode shape. The load pattern for this mode had the vertical time history load applied downward where the mode shape of the main span is deflected below its original position, and had the time history load applied upward where the mode shape deflects above the original position of the structure.

Figure 4: Combined Vertical and Longitudinal Mode Shape

The preliminary design returned accelerations that ranged from maximum comfort as defined by SETRA (up to 5%g in the vertical direction and up 1.5%g in the horizontal direction) to minimum comfort (up to 25%g in the vertical direction and up 8%g in the horizontal direction). None of the accelerations fell outside of the acceptable range, although the accelerations that fell into the minimum comfort range would provide vibrations that would be perceptible but would not be uncomfortable or cause concern for bridge users. Discussions with the owner determined that the structure based on its usage should be designed to meet the Maximum Comfort thresholds defined in SETRA as this would provide vibrations that are “practically imperceptible” to users. To achieve Maximum comfort, changes to the structure would be required, or tuned mass dampers would need to be included in the design. Based on the cost to install tuned mass dampers and the ongoing costs associated with maintenance, structural alterations were the preferred method of improving the bridge’s dynamic response.

Final Design

The final design evaluated the mode shapes that were of greatest concern for possible methods to increase these frequencies to limit the influence of pedestrian induced vibrations. Numerous alternatives were evaluated to determine which would have the greatest benefits for the mode shapes that were of a concern. The concrete deck was changed from a 4000 psi normal-weight deck to a 5000 psi light-weight deck with foam filled SIP forms. This provided an increase to all bridge frequencies due to the reduced mass.

The foundations and pier elements were increased in stiffness to stiffen the longitudinal response of the structure. This included providing select foundations with a battered 3-pile configuration instead of the typical 2-pile configuration used throughout the project for the pier foundations. The piers were also provided with internal stiffeners and increased wall thicknesses. In addition to increasing the longitudinal frequency, it also eliminated the contribution of the transverse movement of the west approach spans from main spans mode of vibration that included vertical and longitudinal displacements of the main span.

Alternatives to steel member sizes and geometries were also evaluated. The size of the main span columns could not be increased due to the architectural desire to keep the columns at a smaller scale than the main arch pipe. The arch pipe was limited to what is commercially available in the United States and could be obtained within the project schedule. Evaluations of the tub girders found that there was no appreciable gain in stiffness that could be achieved within the slender envelope of the bridge.

Ultimately, to increase the stiffness of this span, the
angles of the spandrel columns from vertical were increased. The preliminary design had the columns set 7 degrees from vertical. This was increased to 15 degrees from vertical. The difference in spandrel column angles can be seen in Figure 5. This allowed the structure to perform more similar to a typical truss with the columns being able to develop tension and compression loads as opposed to a pure vierendeel truss that relies on moment transfer through the web members. The change in spandrel column angle increased the vertical frequencies of the main span, but also eliminated one of the longitudinal modes of vibration that was identified in the preliminary design.

Figure 5: 7 Degree Vs 15 Degree Spandrel Column Comparison

The modeling effort for the final design was intentionally conservative to account for unknowns. The soil springs were evaluated and both an upper and lower bound to make sure the actual field conditions would be represented. The deck stiffness used for analysis was reduced based on the anticipated stress range due to the full thermal range of the structure, and AASHTO equations for the effective stiffness for cracked sections. Due to the curved nature of the bridge and the fact that there are no deck joints except at the ramp abutments, it was expected that cracking of the deck would likely occur. This was anticipated to occur due to shrinkage and thermal expansion and contraction despite additional reinforcing that was provided to limit crack size.

As a result of the modifications that were made to the structure, the first horizontal mode of the bridge increased from 1.80 Hz to 1.95 Hz, and the first vertical mode of the bridge increased from 2.34 Hz to 2.54 Hz. The increase in vertical frequency neared the 2.6 Hz limit that SETRA uses to determine if a structure would be susceptible to pedestrian vibrations. The resulting SETRA analysis improved vertical accelerations from 11%g in the preliminary design to 5%g in the final design to meet maximum comfort criteria outlined in SETRA.

As-Built Testing

Prior to opening the bridge to the public, testing was performed on the bridge to determine the anticipated performance. Testing consisted of exciting the bridge to determine the natural frequencies of modes susceptible to pedestrian, the damping associated with each mode, having groups of pedestrians simulate actual service conditions. Testing was performed prior to the installation of the wearing surface.

Testing equipment consisted of 6 triaxial accelerometers. They were placed on the main span in an arrangement to detect vertical, longitudinal, and torsional. Additional arrangements were used to measure the longest approach spans for vertical and torsional modes of vibration. During the simulated pedestrian tests, the accelerometers were placed on both the main span and the approach span 9 and 10.

Testing returned higher frequencies than were assumed during design. The first horizontal mode of the bridge increased from 1.95 Hz to 2.47 Hz with 0.8% damping, and the first vertical mode of the bridge increased from 2.54 Hz to 2.86 Hz with 2.6% damping. The data from the accelerometers showed that the transverse movement of the main span also caused a torsional movement of the main span, causing vertical displacements at either curb line.
Pedestrian simulations consisted of 48 participants. The two shortest spans, span 1 and 2, were not accessible during the testing due to minimal overhead clearance underneath the existing pedestrian bridge, but this area was not a concern for pedestrian induced vibrations. The participants were allowed to walk freely across the bridge to represent the typical in-service condition. Additionally, participants walked across the bridge to a metronome set to the frequencies of the field measured mode shapes. Participants crossed the bridge in both groups of 2 and 4 to change the density of the pedestrians actively exciting each span. In addition to the walking tests, groups of 5, 10, and 15 runners ran across the bridge, and groups of 5 and 10 participants jumped on the bridge to simulate a vandalism scenario.

The case of participants freely walking returned maximum accelerations of 0.2%g longitudinally and 2.1%g vertically on span 9, with the maximum transverse acceleration of 0.5%g measured at the main span. This would most closely represent the service case that would be represented by the SETRA vibration analysis. A few things should be noted that separate the simulation from the assumptions used by SETRA. The average weight of the participants used was 171 lbs and is greater than the weight assumed by SETRA of 154 lbs. If all of the participants were tightly grouped over the main span, it would produce a density of 0.02 pedestrians/ft². When participants were spaced along the entire length of the bridge being tested, the density was 0.01 pedestrians/ft². The dense crowd considered by SETRA consists of 0.09 pedestrians/ft². All of the participants were able bodied and could match the pace required to match the structure’s frequencies that were being targeted. No children, older participants, or anyone with a walking impairment participated in the testing.

<table>
<thead>
<tr>
<th>Test Group</th>
<th>Longitudinal Accel. %g</th>
<th>Transverse Accel. %g</th>
<th>Vertical Accel. %g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Random Walking</td>
<td>0.20%</td>
<td>0.50%</td>
<td>2.10%</td>
</tr>
<tr>
<td>Organized Walking</td>
<td>0.30%</td>
<td>0.70%</td>
<td>3.70%</td>
</tr>
<tr>
<td>Grouped Runners</td>
<td>0.60%</td>
<td>0.80%</td>
<td>4.50%</td>
</tr>
<tr>
<td>Vandalism (Jumping)</td>
<td>0.50%</td>
<td>1.70%</td>
<td>8.90%</td>
</tr>
</tbody>
</table>

Table 1: Measured Accelerations from Tests

In general, synchronization of pedestrians becomes less likely as the density of crowd increases. To compensate for this, tests were also performed with the participants walking to a metronome, which would apply the synchronized load of a much larger crowd that was walking freely across the bridge. Table 1 summarizes the accelerations measured in all of the 9 tests that were performed.

All of the tests returned accelerations that fell within the maximum comfort range of acceleration (1.5%g horizontal and 5%g vertical) except for the vandalism case. In these tests groups of 5 and 10 participants jumped at the center of the main span to a metronome to excite the bridge. Only the accelerations from the group of 10 participants exceeded the maximum comfort range. It would be difficult for a group of people without knowledge of the bridge frequency to determine it and to jump accordingly to excite it in this vandalism case. It would also be obvious to other users what the cause of those vibrations were and would be less concerning than vibrations caused by normal walking across the bridge.
The running tests, which also targeted bridge frequencies with the use of a metronome, produced accelerations that approached the maximum comfort limit in the vertical direction. The bridge was not analyzed for running scenarios but was tested to check the response of the structure. The duration of the excitations from running generally lasted less than 30 seconds as can be seen in Figure 7. In addition to the running tests that were performed, during one of the walking tests, two participants stopped walking and then ran to catch up to their previous locations. A similar spike is seen in Figure 8 during this time frame before accelerations return to normal levels. It was also observed that the response of the structure decreased with the size of the group of runners crossing the bridge. Accelerations decreased from 4.5%g for 15 runners to 3.6%g for 10 runners, 2.6%g for 5 runners, and 2.2%g for the case where two participants ran during the walking test.

Original design assumptions that proved to be conservative included concrete strength, the assumption of a reduced concrete stiffness for cracking, and the use of lower bound foundation stiffnesses. The concrete deck was specified to have a minimum compressive strength of 5000 psi. Testing of concrete cylinders returned average strengths exceeding 7000 psi at 28 days. At the time of vibration testing, the concrete deck cured from 41 to 93 days depending on location and likely gained additional strength although no 56 day cylinders were tested to confirm what additional strength gains may have occurred.

It was also recognized during construction that the SIP forms sat approximately ½ inch high due to the type of straps used to support the SIP forms. The structural model was adjusted for this revised deck thickness which decreased both the mass and stiffness of the concrete deck.

The concrete deck underwent minimal cracking due to shrinkage and thermal movements. The reduction in stiffness used in design to represent excessive cracking was overly conservative and gross

Design and Constructed Dynamic Properties Comparison

The final design and as-built dynamics of the bridge varied as would be expected. As a result, the previous design assumptions were reviewed and structural models revised to replicate the field measured properties. Since the bridge was tested prior to installation of the wearing surface, it was important to rerun models to replicate the as-built condition, so that they could then be modified to incorporate this additional mass. This would allow the SETRA analysis to be rerun for the final condition and to verify that accelerations would still remain within the maximum comfort range after installation of wearing surface although physical testing of the bridge wouldn’t be performed for this condition.

No major walking or running events, such as 5ks, or marathons are planned to use the bridge as part of the route. Since the bridge empties into Charles Circle, it would be impractical to have that volume of pedestrians use the route without closing Charles Circle, which would have major traffic impacts for the area. The tested running groups of up to 15 people would represent the typical usage of the bridge by runners.
properties were considered to match the field results. Additionally, AASHTO C4.5.2.2 states that “tests indicate that in the elastic range of structural behavior, cracking of concrete seems to have little effect on the global behavior of bridge structures,” and seems to be confirmed by the test results. The deck was heavily reinforced with longitudinal reinforcing consisting of #6 bars top and bottom at 6 inches on center for a 7½ inch deck. This area of steel was not considered during the final design to be conservative, but was considered when replicating the field results.

The foundation springs were taken as a lower bound stiffness during final design. This assumption proved too conservative since measured frequencies of the bridge were consistent with higher bound foundation stiffnesses that were previously calculated.

No adjustments to the steel structure modeling were made since variations in steel sections would be negligible given the standard material thickness and dimensional tolerances of the members used. Tubular connections from the spandrel columns to the girders utilized stiffened connections that perform close to the idealized fixed member connections used in the model. The structural model also included secondary members and non-structural members such as the fascia plates, to ensure that additional stiffness from these components would be accounted for and would minimize the difference between the modeled steel stiffness and the as-built stiffness.

To precisely match the field measured frequencies, the density of concrete used in the models were adjusted from the 120 pcf specified down to 116 pcf. Testing of the concrete to confirm the density used was not performed. The modification of density may have also helped to account for areas where stay in place forms were set higher than expected and could compensate for a minor reduction in mass.

After the model adjustments were made to replicate the field measured frequencies, the weight of the wearing surface was added into the model to represent the final condition. No additional stiffness from the epoxy wearing surface was considered. The first horizontal and torsional frequency decreased from 2.47 Hz to 2.44 Hz, and the first vertical mode decreased from 2.86 Hz to 2.82 Hz. The vertical mode no longer falls within the range of susceptibility as defined by SETRA. Since the torsional movement associated with the horizontal mode shape was detected during testing, this case was analyzed with the vertical load patterning so that upward loading would be applied on one half of the deck and downward loading would be applied on the remaining half.

SETRA Results Validation

After adjustments were made and the analysis was rerun, the SETRA analysis provided a maximum transverse acceleration of 0.9%g and a maximum vertical acceleration of 0.6%g. The horizontal acceleration provided by analysis is slightly higher than that measured in the field (0.7% for organized walking). Given the differences between the tests performed and the analysis assumptions, the SETRA analysis appears to be accurate with the tendency to be on the conservative side. This level of conservatism would be desirable when being used for design.

The vertical accelerations measured exceeded those determined by the SETRA analysis. Since the only mode shape that fell into the range of susceptibility was primarily a transverse mode, the higher frequency mode shapes that were primarily vertical were not analyzed for the as-built condition with time history loadings. During final design when those mode shapes were still within the range of susceptibility, SETRA predicted accelerations of 5%g, and fell within the maximum comfort range. As the dynamic properties improved with the as-built condition, a measured acceleration of 3.7%g for the organized walking tests was recorded. This would suggest that the original SETRA analysis may have been somewhat conservative, but overall is fairly accurate.

Even though the vertical mode shapes were no longer in the range of susceptibility, they were still able to be excited by pedestrians during both walking and running conditions. The measured results show that these vibrations would still fall in the maximum comfort range, which supports the analysis of mode shapes with vertical frequencies below 2.6 Hz, and acceptance of frequencies above 2.6 Hz without analysis.

Through the use of the SETRA guide and analysis methods, the design of the Fanny Appleton Bridge
was able to be improved for vibrational performance. Since the bridge did not meet AASHTO criteria for a 3 Hz vertical frequency, but met the alternate criteria based on span weight, it was unclear how the bridge would perform. Increasing the natural frequency during design to exceed 3 Hz was not feasible. Although the as-built dynamic properties exceeded those during design, using a higher than specified concrete strength for stiffness or using the upper bound of foundation stiffnesses could have provided an unconservative design that would not have performed as well. The SETRA guide provided a reliable and simple dynamic analysis method that allowed the bridge to achieve the goal of having service vibrations that fell within their defined maximum comfort range.

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**Engineer of Record:** STV Incorporated

**Vibration Testing:** RWDI

References


