Part 7: Accelerated pavement testing to evaluate functional performance
Accelerated traffic load testing of seismic expansion joints for the new San Francisco–Oakland Bay Bridge

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ABSTRACT: A relatively unique opportunity was recently identified for accelerated traffic load testing of a bridge expansion joint designed to withstand severe seismic activity. This study was part of the construction of the new East Span of the San Francisco–Oakland Bay Bridge and assessed whether the expansion joints (which were designed to function in harmony with the bridge decks in the event of a high magnitude earthquake) linking the Self-anchored Span with the Transition and Skyway spans would withstand traffic loading. A test structure incorporating one of the joints was constructed close to the actual bridge and tested with the California Department of Transportation/University of California Pavement Research Center Heavy Vehicle Simulator in a series of phases. On completion of three months of testing, no structural damage was recorded by any of the LVDTs or strain gauges installed on the steel plates, steel frames, bolts, or washers. There was also no visible damage on any of these components. Excessive overloading caused some damage to the Trelleborg unit towards the end of the test. Based on the results of this limited testing, it was concluded that the expansion joint would perform adequately under typical Bay Bridge traffic. The distress observed to the Trelleborg unit under the high loads in the last phase of testing is unlikely to occur under normal traffic. The findings from this study indicate that the Caltrans seismic expansion joint tested will be appropriate for typical Bay Bridge traffic. These joints will be used in the new bridge, due to be opened in 2013. The study also concluded that APT can be effectively used for testing bridge deck components to provide rapid answers for design and construction teams.

1 INTRODUCTION

The 13.5 km (8.4 mile) San Francisco–Oakland Bay Bridge connects the city of San Francisco with the East Bay cities of Oakland, Emeryville and Berkeley and is the start point of the Interstate 80 (I-80) corridor. The bridge carries approximately 280,000 vehicles per day (compared to the 100,000 carried by the Golden Gate Bridge). It currently consists of two separate bridges linked by a short tunnel on Yerba Buena Island. The existing East Span, a steel box girder design constructed in 1936, was damaged by the 7.1 magnitude Loma Prieta earthquake in 1989, during which a section of the top span, carrying the five west bound lanes, collapsed onto the lower east bound lanes. Although repairs were made and the bridge reopened approximately one month after the earthquake, a complete seismic retrofit of the East Span to withstand future similar or more severe earthquakes was not considered viable and construction of a new bridge was approved. The West Span, which consists of two suspension bridge spans connected at a center anchorage, was easier to retrofit to accommodate higher magnitude earthquakes. Retrofit work on this part of the bridge was completed in 2004 and retrofit work on the West Approach was completed in 2009.

The new East Span consists of four separate parts (Figure 1):

- The Oakland Approach and Touchdown, linking the new bridge to the existing I-80 infrastructure.
- The Skyway, two side-by-side 1.9 km (1.2 mile) long concrete spans (completed in 2008).
- The Main Span, a self-anchored suspension structure with two side-by-side 470 m (1,540 ft) long spans supported by a single tower, which was still under construction at the time this study was undertaken. It will be the longest bridge of its kind in the world. The span’s single 160 m (525 ft) tall tower will match the height of the West Span’s towers. Its placement closer to the west end of the structure creates a distinctive asymmetrical design, with the single 1.6 km (1.0 mile) long main cable presenting a sharper angle on the west side and a more sloping appearance on the east.
- The Yerba Buena Island Transition Structure (YBITS), still under construction, will connect the Self-anchored Suspension Span to Yerba Buena Island (YBI), and will transition the new East Span’s side-by-side road decks to the upper and lower decks of the YBI tunnel and West Span.
The three radically different structures also require a new expansion joint design to link the three main parts (Skyway, SAS, and YBITS) while integrating with the seismic functioning of the entire bridge system. This expansion joint was subsequently designed and incorporates a Trelleborg Transflex 2400 expansion joint, a steel connector plate, and fastening systems. The main focus of the design was to ensure that the joint acted in harmony with the three structures during seismic activity. A secondary focus was the requirement for separate joints for each lane, which would facilitate maintenance without major disruption to traffic. During review of the joint design, questions were raised with regard to how it would perform under traffic loading, given the focus on seismic and maintenance requirements. An accelerated loading test, using the California Department of Transportation/University of California Pavement Research Center Heavy Vehicle Simulator (HVS) was therefore undertaken to provide a quick indication of how the joint would perform under truck traffic.

This paper describes the accelerated traffic load study, summarizes the results, and relates performance in the accelerated test with expected performance on the actual bridge structure.

2 STUDY OBJECTIVE AND WORKPLAN

Two objectives were identified for this accelerated load study:
- Identify any fatal flaws in the design related to vehicle trafficking.
- Determine how the joint will fail under vehicle trafficking.

A project was consequently designed to meet these objectives (Jones and Wu, 2011). This included design and construction of a test structure, instrumentation of the test section, accelerated loading with a Heavy Vehicle Simulator (HVS), data analysis, and recommendations. The design of the test structure is provided in Figure 2. A prototype of the joint was manufactured and a replica of the joint support structures constructed to accommodate the testing.

A review of the literature found no published reference to any similar studies and given a testing period limitation of three months, best use of this time was taken into consideration in preparing a test plan to meet the study objectives. A phased approach was followed, starting with normal truck loads in the center of the joint to identify any fatal flaws, followed by incremental changes in loading and wheel position. A summary of the test plan is provided in Table 1.

In the first phase (Phase 1.1), testing at standard wheel loads for four weeks (i.e., equivalent to an 80 kN [18,000 lb] axle load), was included to identify any potential major flaws in the design. The following phases would then evaluate the joint response under wandering traffic, increasing wheel load, and different wheel path (specifically along the edge of the joint). Assuming that no damage was caused in the first two phases, the final phase would investigate impact loads and very high wheel loads and tire pressures with a view to identifying the weakest point of the design.

Load variations on a single day were included in the study to establish relationships between wheel load and structural response, and to identify any nonlinearity that might lead to structural damage. Most trafficking was applied in a channelized, bidirectional mode using dual wheel truck tires (Goodyear G159 – 11R22.5 – steel belt radial inflated to 720 kPa [104 psi]) with these exceptions:
- Phase 1.3, which assessed the effects of bidirectional traffic wander using the dual tires.
Table 1. Loading Program for HVS Testing.

<table>
<thead>
<tr>
<th>Phase No.</th>
<th>Test Location</th>
<th>Duration (days)</th>
<th>Half-Axle Wheel Loads (kN)</th>
<th>Repetitions Applied</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1 Center</td>
<td>30</td>
<td>1 day at 25, then 29 days at 40</td>
<td>5,18,000</td>
<td></td>
</tr>
<tr>
<td>1.2 Center</td>
<td>6</td>
<td>1 day each at 25, 40, 60, 80, and 100, then 1 day at 40</td>
<td>120,000</td>
<td></td>
</tr>
<tr>
<td>1.3 Center + Edge</td>
<td>7</td>
<td>1 day each at 40, 80, 100, then 4 days at 60</td>
<td>1,20,000</td>
<td></td>
</tr>
<tr>
<td>2.1 Edge</td>
<td>11</td>
<td>2 days at 40, 1 day each at 60, 80 and 100, then 6 days at 80</td>
<td>1,89,000</td>
<td></td>
</tr>
<tr>
<td>3.1 Edge</td>
<td>3</td>
<td>60, impact load*</td>
<td>23,000</td>
<td></td>
</tr>
<tr>
<td>3.2 Edge</td>
<td>15</td>
<td>5 days at 60, 5 days at 80, and 5 days at 100, all with impact load</td>
<td>2,40,000</td>
<td></td>
</tr>
<tr>
<td>3.3 Edge</td>
<td>15</td>
<td>1 day at 100, then 14 days at 150</td>
<td>1,50,000</td>
<td></td>
</tr>
<tr>
<td>– – – – –</td>
<td>–</td>
<td>No test days</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>–</td>
<td>90</td>
<td>13,60,000</td>
<td></td>
</tr>
</tbody>
</table>

*Impact load was applied by forcing the HVS wheel over a step in the wheelpath created by either a 13 mm (1/2 in.) neoprene pad or 19 mm (3/4 in.) hardwood board.

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3 CONSTRUCTION

A suitable test site was identified on a temporarily vacant area close to the bridge construction offices at the Port of Oakland. Construction was started in March 2011 and completed in July 2011. Photographs of the construction are provided in Figure 3 and Figure 4 and completed project with the HVS in place in Figure 5 and Figure 6.
4 INSTRUMENTATION

The expansion joint was comprehensively instrumented to monitor status and responses under HVS trafficking (Jones and Wu, 2011). Parameters monitored included: ambient and steel plate temperatures, vertical deflections at various locations, and longitudinal strain at the bottom of the steel plate. A layout of the instrumentation is shown in Figure 7. Instruments #1 through #9 are Linear Variable Differential Transducers (LVDTs), Instruments #11 to #13 are strain gauges, and Instruments #14 to #18 are thermocouples.

Figure 8 and Figure 9 show general views of the instruments on top and underneath the plate, respectively. Figure 10 and Figure 11 respectively show closer views of two LVDTs positioned on bolts on the surface of the steel plate, and how the movement of washers underneath the steel plate was monitored.

Permanent deformation of the Trelleborg unit was monitored daily using a laser profilometer that recorded surface profiles in a longitudinal direction (i.e., the trafficking direction) at 200 mm (8.0 in.) intervals in the transverse direction (Figure 12).

5 HVS TRAFFICKING

HVS trafficking commenced on August 08, 2011 and followed the test plan described earlier. The impact loads used in Phase 3.1 and Phase 3.2 were applied by creating a step in the wheel path using either a 12.5 mm (1/2 in.) neoprene pad or a 19 mm (3/4 in.) wood panel (Figure 13).

6 TEST RESULT SUMMARY

6.1 Phase 1: Normal load and pavement response at center location

This section covers the test results for Phases 1.1, 1.2 and 1.3 as listed in Table 1. The main objectives of these phases were identification of any major flaws in the joint design and evaluation of the strain and deflections caused by increasing wheel loads (Jones and Wu, 2011).

6.1.1 Phase 1: Fatal flaw assessment

No apparent damage was observed at the end of Phase 1.1. The permanent vertical settlement of the structure after testing was 0.2 mm, which was considered minimal and unlikely to influence joint performance. No permanent deformation in the steel plate occurred during this phase, based on the strain data recorded.

Example vertical deflection bowls and strain bowls measured at the mid-span of the steel plate are shown in Figure 14 and Figure 15, respectively. These figures show that the deflections and longitudinal strains induced by the 80 kN standard axle load (40 kN half-axle) at mid-span of the steel plate were approximately 0.9 mm and 60 microstrain, respectively, and
remained constant throughout the phase (i.e., deflections and strains did not increase with increasing load repetitions).

The vertical deflections at the bolts and washers were less than 0.1 mm, with washers deflecting a little more than the bolts. There was no distinct correlation between temperature and elastic response in the steel plate; however, very small changes in peak strain between the coldest and warmest periods each day were observed on the data plots on most days. Minor fluctuations in strain and deflection measurements were most likely caused by very small fluctuations in the actual load applied by the HVS. No fatal flaws in the deck joint design were identified.

6.1.2 Phase 1.2: Load response on the center of the steel plate

No damage was observed at the end of Phase 1.2. No permanent deformation in the steel plate occurred during this phase, based on the strain data recorded. Increases in peak deflection and peak strain showed a linear relationship with increasing load. The maximum
deflection and maximum strain recorded was 2.3 mm and 135 microstrain respectively, both at the mid-point of the steel plate, under the 100 kN wheel load. Examples of the data recorded are shown in Figure 16 (peak vertical deflection against number of load repetitions) and Figure 17 (peak vertical deflection against load) for the LVDT corresponding to instrument #5 in Figure 7. Changes in deflection and strain with increasing wheel load showed similar trends. Very small daily variations in peak deflection and peak strain were consistent with daily temperature change on the data plots. Minor fluctuations in strain and deflection measurements were again likely caused by very small fluctuations in the actual load applied by the HVS.
6.1.3 Phase 1.3: Load response comparison at center and edge of the steel plate

Phase 1.3 checked the joint response under wandering traffic. No damage was observed at the end of this phase and based on the deflection and strain data recorded, no permanent deformation in the steel plate occurred. Peak strain and deflection at any time was influenced by the position of the wheels in the wander pattern, as expected. Sensors furthest away from the wheels (i.e., on the edge of the steel plate) had larger differences between the lowest and highest deflection and strain (ratio of $\sim 2.5$) compared to the sensors inside the wheelpath (i.e., at the mid-point of the steel plate), which had highest strain to lowest strain ratios of about 1.5. Increases in peak deflection and peak strain continued to show a linear relationship with increasing load. Very small daily variations in peak deflection and peak strain were consistent with daily temperature change on the data plots. Minor fluctuations in strain and deflection measurements were again likely caused by very small fluctuations in the actual load applied by the HVS. Based on the results and observations in this phase, it was concluded that there was no significant difference in the measurements recorded during traffic wander compared to those recorded during channelized traffic and that wander had very little effect on the behavior of the deck joint. Consequently all further testing was carried out in a channelized mode as this was considered more likely to induce damage given the concentrated nature of the loading.

6.2 Phase 2: Higher loads at edge location

During Phase 2, testing was carried out on the edge of one of the expansion joints as shown in Figure 7. The term “edge” was used because wheelpaths for this location were closer to the edge of one of the steel plates even though it was closer to the center of the entire lane width. Normal trafficking on the actual bridge would not occur in this way except when vehicles change lanes. The objective in this phase was to assess whether trafficking at higher loads on the edge of the plate would cause any damage to the expansion joint.

No damage was observed at the end of the phase and based on the deflection and strain data recorded, no permanent deformation in the steel plate occurred. Responses were similar to those recorded in earlier phases during loading on the center of the deck joint. Increases in peak deflection and peak strain continued to show a linear relationship with increasing load. Figure 18 shows the histories of longitudinal strains measured at the mid-span of the steel plate. The figure indicates some slight daily variation (attributed to temperature), but no significant increasing trend. Figure 19 shows the permanent vertical deformation of the Trelleborg unit, measured with the laser profilometer. Approximately 4.0 mm of downward deformation, caused by the HVS trafficking, was measured at the end of this phase of testing.
Based on the results and observations in this phase, it was concluded that there was no significant difference in the trends of measurements recorded during trafficking on the edge compared to those recorded during trafficking on the center. However, since higher deflections and strains were measured in this phase for the same loads, it was decided to undertake all further testing on the edge of the bridge deck joint as this was considered more likely to induce damage.

6.3 Phase 3: Impact load and overloading

After reviewing the Phase 2 results, it was concluded that continued trafficking at 80 kN and 100 kN was unlikely to cause any significant structural damage to the seismic joint in the time available. The study therefore proceeded to the third phase of the test plan, which required significantly heavier wheel loads (using an aircraft tire) and impact loading (caused by including a step in the wheel path).

6.3.1 Phase 3.1: Edge test with impact load and unidirectional traffic

A 60 kN impact load did not appear to influence response in the deck joint at the location of the sensors, and no damage was observed on completion of this short phase. Responses were similar to those recorded in earlier phases. There was also no difference observed between unidirectional and bidirectional trafficking and consequently all further testing was carried out in a bidirectional mode, which applies more wheel loads than unidirectional trafficking in a given period of time.

6.3.2 Phase 3.2: Load response with impact load

No damage was observed at the end of Phase 3.2 and based on the deflection and strain data recorded, no permanent deformation in the steel plate occurred. Responses continued to be the same as those recorded in earlier phases and increases in peak deflection and peak strain continued to show a linear relationship with increasing load.

Figure 20 shows an example of the deflections on the bolt washers (LVDT3 and LVDT4) and bolts (JDMD1 and JDMD2) caused by impact loading with a 100 kN half-axle wheel load. The impact load had some influence on the deflection bowls (influence lines) for each instrument, but did not result in an increase in the peak deflections. It was therefore concluded that this level of impact (the maximum possible without influencing the hydraulic load controls of the HVS) would not cause additional damage and consequently impact loads were not applied in Phase 3.3.

6.3.3 Phase 3.3: Edge test with high load

The objective of Phase 3.3 of the testing was to cause as much damage to the seismic joint as possible to identify the weakest part of the design. Loading was carried...
out with an aircraft tire with a half-axle load of 150 kN. On completion of this testing, the measured data from the LVDTs (Figure 21) and strain gauges (Figure 22) indicated that there was still no structural damage on any steel parts of the expansion joint. Observed damage was limited to the wheelpath over the Trelleborg unit only and consisted of significant wear and deformation on the rubber sections of the Trelleborg unit.
and deformation and shearing in one of the steel ribs supporting these rubber sections. Some cracking was also observed in the concrete approach slab to the expansion joint, but this was not considered relevant to the study. Figure 23 shows the permanent vertical deformation of the Trelleborg unit, measured with the laser profilometer. Approximately 4.8 mm of downward deformation, caused by the HVS trafficking, was measured at the end of this phase of testing.

Photographs of the structure after completion of testing and a close up of damage to the Trelleborg unit are shown in Figure 24 and Figure 25, respectively.

The distress observed to the Trelleborg unit under the very high loads (almost four times the legal limit) applied in this last phase of testing is unlikely to occur under normal traffic on the Bay Bridge.

7 CONCLUSIONS AND IMPLEMENTATION

A relatively unique opportunity was recently identified for accelerated traffic load testing of a new bridge expansion joint designed to withstand severe seismic activity. This study was part of the construction of the new East Span of the San Francisco–Oakland Bay Bridge and assessed whether the expansion joints (which were designed to function in harmony with the bridge decks in the event of a high-magnitude earthquake) linking the Self-anchored Span with the Transition and Skyway spans would withstand traffic loading. A test structure incorporating one of the full-scale joints was constructed close to the actual bridge and tested with the California Department of Transportation/University of California Pavement Research Center Heavy Vehicle Simulator in a series of phases.

On completion of seven phases of testing over a three-month period, no structural damage was recorded by any of the LVDTs or strain gauges installed on the steel plates, steel frames, bolts, or washers. There was also no visible damage on any of these components. Excessive overloading with a 150 kN half-axle load (four times the standard axle load) on an aircraft tire in the last phase of the test caused some damage to the Trelleborg unit in the joint. The damage included abrasion, tearing, shoving and permanent deformation of the rubber inserts, and deformation and shearing of one of the steel supports directly under the wheel load.

Although no seismic or structural testing was undertaken and no recommendations towards its seismic or structural performance are made, and no vehicle suspension dynamics (i.e., vehicle bounce) or speed effects were considered, based on the results of this limited testing, it was concluded that the Caltrans seismic expansion joint would perform adequately under typical Bay Bridge traffic. The distresses observed on the Trelleborg unit under high loads in the last phase of testing are unlikely to occur under normal Bay Bridge traffic. However, the Trelleborg unit was found to be the weakest point of the expansion joint, as expected. On the actual bridge structure, these units will need to be checked periodically to confirm the findings of this study, and to assess any effects of higher speeds and vehicle dynamics that were not identified. The joints will require periodic maintenance and replacement in line with manufacturer’s specifications.

This study also demonstrated and concluded that mobile accelerated pavement testing equipment can be effectively used for testing bridge deck components to provide rapid indications of how these will perform/behave under traffic loading. The results can be used by design, fabrication, manufacturing and construction teams to ensure safe and efficient performance of the bridge.

Based on the findings of this study and other studies carried out to assess seismic performance and skid resistance, the expansion joint design as tested in this study will be used in the new Bay Bridge, due to be opened to traffic in 2013.

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REFERENCES