CONNECTION FAILURES IN CONCRETE CONSTRUCTION

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Summary

The behavior of non-standard connections in concrete construction, particularly cast-in-place concrete construction, is often not as well-defined and sometimes not as well understood as other components of concrete construction. In situations such as these, a complete understanding of the connection mechanics is essential to analyze and design the connection. The lack of understanding can lead to underestimation of demand and/or overestimation of the capacity of a connection resulting in poor performance and/or failure. This paper examines the failure of two types of connections at expansion joints in concrete construction. In both cases, the failures of the connections compromised the structural performance.

The subject of the first case study is the failure of expansion joints on an automated people mover system, which occurred during commissioning of the system. The basic design flaw was that the statical behavior of the connection was misunderstood. This resulted in the incorrect resolution of forces and the underestimation of demand. In addition, two repairs were initially proposed after the failure based on the same misunderstanding. The analysis of the connection forces and the revised design of this connection will be discussed.

The subject of the second case study is the failure of a vertical expansion joint in a water treatment tank. The failure of this connection occurred after the tank was in-service for about five years. In this case, the underestimation of demand was the result of incorrect modeling of the joint behavior and the overestimation of capacity was the result of inappropriate application of design code provisions. The analysis and repair of the existing structure, as well as design recommendations will be discussed.

Keywords: Concrete construction, expansion joints, transportation structure, guideway, ride plate, anchorage to concrete, dowel behavior, connected elements in shear, concrete bearing, water treatment tank.

1. Failure in a Transportation Structure

1.1 Background

The subject of the first case study is the failure of expansion joints on an automated people mover (APM) system. The actual failure corresponded to the connection of the steel ride plate to the reinforced concrete running surface at the expansion joint. These failures occurred during commissioning of the system immediately prior to the system going into service.

The APM vehicles are rubber tired, electrically powered and automatically controlled. A general view of the guideway is shown in Figure 1. The running surface for the guideway consists of two 20 in. wide concrete plinths offset 39 3/4 in. on either side of the vehicle centerline to the centerlines of the plinths. The concrete running surfaces vary in nominal minimum depth from 8 in. to 17 in. The plinths are anchored to the guideway superstructure by hairpin reinforcement using post-installed adhesive anchors.

Expansion joints in the running surface correspond to expansion joints in the underlying superstructure. Two types of expansion joints were specified. Gaps normal to the centerline of the running surface (Type 1) were specified not to exceed 1 in. in width in all environmental conditions. When gaps larger than 1 in. were required, the joint face was specified at a 45 degree angle to the
vehicle’s tire path (Type 2). The maximum specified longitudinal opening was 5 in. for Type 2 joints. Steel ride plates were embedded in the concrete at Type 2 joints to effectively bridge the underlying joint in the guideway deck. The components of the joint are illustrated in Figure 2.

The steel ride plate consists of a horizontal plate on the running surface and a connected bent vertical plate that extends down the face of the plinth to the guideway deck. Both the horizontal and vertical legs of the ride plate are anchored to the concrete plinth with headed stud anchors (HSA). The end of the horizontal ride plate that cantilevers over the expansion joint is stiffened with a vertical plate.

During commissioning of the system, failures occurred at five of about 300 ride plate locations. Based upon our observations of the failed joints, the failure of the ride plates was due to concrete breakout in shear. The failure plane originated at the back group of headed stud anchors and projected diagonally downward toward the extended vertical leg of the ride plate. An exposed plinth and ride plate at one of the failure locations is shown in Figures 3 and 4, respectively. Cracking on the side surfaces of the plinth at the location of the back group of headed stud anchors was also observed at other locations.

1.2 Analysis

The guideway structure was reportedly designed in accordance with the latest “Standard Specifications for Highway Bridges”, adopted by the American Association of State Highway and Transportation Officials. The maximum total vehicle weight is 63,800 lbs (28,940 kg). The guideway is designed for centrifugal forces at all sections of curved alignment and a longitudinal force equal to 12.5 percent of the live load due to vehicle acceleration and deceleration.
A simplified static model was used in our analysis of the ride plate connection. A free body diagram of the assumed forces of the ride plate is illustrated in Figure 5. Our analysis assumed rigid body rotation of the ride plate assembly, with the pivot point located at the corner of the plinth; no plan eccentricity of the assembly; the vertical steel plate is rigid relative to the concrete; the minimum plinth depth is 7 1/2 in.; one half the tire load is applied at a distance of 2/3 the cantilever length from the pivot point; an assumed impact factor equal to 30 percent of the live load; and an acceleration/deceleration load 12.5 percent of the live load.

The behavior of the simplified ride plate model is such that the tire load \( W_{\text{VERT}} \) produces a rigid body rotation of the ride plate assembly about point O, the assumed pivot point. The rigid body rotation is resisted by tensile forces in the vertical HSA \( P_T \) and compressive forces between the vertical concrete plinth face and the vertical steel plate of the ride plate assembly \( P_{\text{BH}} \). In addition, a vertical compressive bearing force \( P_{\text{BW}} \) is produced to resist the sum of the tensile forces in the HSA and the applied tire load, and a horizontal shear force in the vertical HSA \( P_V \) is produced to equilibrate the horizontal bearing force \( P_{\text{BH}} \). As such, the vertical HSA experience both tensile and shear forces.

The capacity of the ride plate connection is controlled by the capacity of the HSAs to resist the interaction of shear and tensile demands. The concrete shear and tensile breakout capacities, along with the tensile pullout strengths, of the HSAs were evaluated based upon the Building Code Requirements for Structural Concrete (ACI 318-11), Appendix D. The nominal concrete tensile breakout strength and nominal tensile pullout strength of the headed stud anchors were evaluated based upon similar equations in ACI 318. Based upon the estimated demand and capacities as described above, the calculated design demand - capacity ratio (DCR) was determined to be 2.0 for concrete shear breakout alone, and 2.2 for combined concrete breakout in shear and tension.

A review of the original design calculations found that the overturning moment was assumed to be resisted by the vertical headed stud anchors in tension only, and the controlling failure mode was assumed to be tension pull-out. Although the tensile failure of the headed stud anchors was recognized as a potential failure mode, the concrete breakout strength in tension was not checked. In addition, concrete breakout in shear was determined to be the most significant contributor to the shear demand on the headed stud anchor, which was apparently not considered at all.

### 1.3 Repair

Given the timing of the failures immediately prior to the system going into service, the analysis of the cause of the failures and the development of an acceptable repair solution were time critical. Two different repair solutions were initially proposed that were intended to minimize the time and effort in implementing the repairs, and the disruption of service for the people mover system.

The first solution proposed by the original design team was to essentially “pin” the back of the ride plate with No. 6 reinforcement embedded vertically into the guideway deck and welded to the horizontal plate of the ride plate assembly behind the back line of the HSA. While this proposed solution would have moved the failure plane further back, it would not have increased the area of the failure plane as the depth of the failure plane is limited by the depth of the plinth. As a result, this proposed solution would have had no effect on the capacity of the ride plate assembly.

The second solution proposed by another consultant was to effectively eliminate the stiffener supporting the cantilevered horizontal plate of the ride plate assembly in-situ by drilling and flame cutting the stiffener, in an effort to reduce shear demand on the HSA. However, the shear demand on the HSA would only have been reduced if the concrete bearing force on the vertical plate of the assembly were reduced. As shown in Figure 4, the vertical plate in the assembly was bent, not
straight as assumed in the simplified free body diagram. In addition, the plate was attached to the horizontal plate with welds on both faces of the vertical plate. As a result, the rigid body behavior of the assembly, including the vertical plate and consequently the concrete bearing force, would have been effectively unchanged by the “removal” of the stiffener.

The authors concluded that if the existing steel ride plate embeddings were to be re-used, the shear demand on the headed stud anchors could not be reduced. The only method to increase the shear capacity was to reconstruct the plinths in the vicinity of the ride plates and add supplemental reinforcement. The ACI Code allows the use of anchor reinforcement, illustrated in Figure 6, to resist shear demand instead of concrete breakout strength where the anchor reinforcement encloses the anchors and is developed beyond the breakout surface. This detail allowed for the re-use of the existing ride plate fabrications and minimized the time necessary to repair the connections. All of the existing ride plate connections were repaired in this manner and the repairs were completed within 60 days.

1.4 Summary

The basic flaw in the design of the ride plate connection was that the behavior of the connection was misunderstood. This resulted in the incorrect resolution of forces and the underestimation of demand. Without an effective method to reduce demand, the final repair solution utilized hairpin anchor reinforcement to preclude concrete shear breakout of the embedded headed stud anchors and provide sufficient capacity to resist the design demand. The design of this reinforcement was based upon the applicable provisions for anchoring to concrete in ACI 318-11.

2. Failure in a Water Treatment Structure

2.1 Background

The subject of the second case study is the failure of vertical expansion joints in a reinforced-concrete water treatment aeration tank, which occurred after the tank was in-service for about five years. In this case, the underestimation of demand was the result of incorrect modeling of the joint behavior and the overestimation of capacity was the result of inappropriate interpretation and application of design code provisions.
The tanks are approximately 383 ft. by 530 ft. in plan and consist of six basins. The structure is subdivided into three sections along the 383 ft. (east-west) length with vertical expansion joints in the exterior walls of Basin Nos. 3 and 5. The structure is sub-divided in half in the north-south direction. A perspective view of part of the south half of the structure is shown in Figure 7. A typical wall section at the expansion joint is shown in Figure 8. The joint is specified to include 3/4 in. diameter stainless steel bars spaced at 18 in. in the basin wall and basin slab.

2.2 Analysis

The authors performed a linear finite element analysis of the aeration tanks. The analysis model, shown in Figure 7, was based on a model provided by the design engineer. In the provided model, the tank walls and slab were modeled with 12 in. by 12 in. shell elements. A physical gap was not provided at the expansion joint locations. Instead, moment releases in the shell element joints were provided at each expansion joint. The dowel design was reported based on a quantity referred to as “differential shear”, which was calculated as the shear difference between two horizontally adjacent shell element joints. Based on the design dowel shear demands provided to the authors, it was determined that the shear forces used to calculate the differential shear were out-of-plane shear forces on the vertical face of the shell element, shown as $V_{13}$ in Figure 9. Also shown in Figure 9 is $V_{23}$, the out-of-plane shear force on the horizontal face of the shell element.

After reviewing the provided model, modifications were made to perform the investigative analysis. A gap of 1 in. between adjacent shell elements was provided at each expansion joint location. Frame elements were used to model the dowels. The mesh was revised as necessary to provide shell element joints at the actual dowel locations, as shown in Figure 9. The resulting model more closely reflected the construction of the joint, which provided a good baseline to review the sensitivity of the results to different modeling assumptions.

Four primary analysis scenarios were investigated. Scenarios 1 and 2 involved modeling the expansion joint considering the as-designed and as-built dowel locations, respectively. Scenario 3 assumed that the dowels had yielded by introducing moment releases at both ends of each dowel frame element, which was intended to model the observed differential top-of-wall displacement. Although it was unlikely that every dowel yielded, this provided a simplified, upper-bound estimate on the differential displacement. Scenario 4 involved investigating the approach of the design engineer in calculating the design dowel shear demand, reportedly based on the differential shear quantity. The analysis model for Scenario 4 was identical to Scenario 1.

In general, our analysis considered three primary load cases: LC 1. Dead + Hydrostatic Fluid (Unfactored), LC 2. Dead + Live + Hydrostatic Fluid + Seismic (Unfactored), and LC 3. Dead + Live + Hydrostatic Fluid + Seismic (Factored). LC 1 was not a design load case, but represented a reasonable estimate of the daily loading on the tank. LC 2 and LC 3 were typically the most critical unfactored and factored load cases with respect to dowel shear demand, which were used to evaluate the stainless steel dowels and concrete dowel bearing design, based on allowable stress design and limit state design respectively.

The analysis results for Scenario 1, with as-designed dowel placement, for all three load cases are shown in Table 1. The maximum unfactored dowel shear demand (LC 2) was 24.1 kips, which occurs at the upper-most dowel and represented the design shear demand for the dowels. The shear demand in Dowel 12 was approximately 23 kips. The high shear demand at this location was likely a result of the change in wall thickness from 18 in. to 28 in. The locations of maximum shear demand for LCs 1 and 3 were consistent with LC 2. The maximum factored shear demand (LC 3) was 42.7 kips at Dowel 1, which represented the design demand for concrete bearing. The maximum unfactored shear demand due to dead and hydrostatic fluid loading only was
approximately 75 percent of the maximum unfactored shear demand in LC 2, or the design shear.

**Table 1. Analysis results for Scenario 1**

<table>
<thead>
<tr>
<th>Load Case</th>
<th>LC 1 - Dead + Fluid (Unfactored)</th>
<th>LC 2 - Dead + Live + Fluid + Seismic (Unfactored)</th>
<th>LC 3 - Dead + Live + Fluid + Seismic (Factored)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dowel</td>
<td>Distance from Top (Dowel Demand (k)</td>
<td>Failure DCR Shear</td>
<td>Dowel Demand (k)</td>
</tr>
<tr>
<td>1</td>
<td>0'-9&quot;</td>
<td>18.0</td>
<td>2.1</td>
</tr>
<tr>
<td>2</td>
<td>2'-3&quot;</td>
<td>5.4</td>
<td>0.6</td>
</tr>
<tr>
<td>3</td>
<td>3'-9&quot;</td>
<td>3.4</td>
<td>0.4</td>
</tr>
<tr>
<td>4</td>
<td>5'-3&quot;</td>
<td>3.8</td>
<td>0.4</td>
</tr>
<tr>
<td>5</td>
<td>6'-9&quot;</td>
<td>4.4</td>
<td>0.5</td>
</tr>
<tr>
<td>6</td>
<td>8'-3&quot;</td>
<td>4.5</td>
<td>0.5</td>
</tr>
<tr>
<td>7</td>
<td>9'-9&quot;</td>
<td>4.4</td>
<td>0.5</td>
</tr>
<tr>
<td>8</td>
<td>11'-3&quot;</td>
<td>4.0</td>
<td>0.5</td>
</tr>
<tr>
<td>9</td>
<td>12'-9&quot;</td>
<td>3.7</td>
<td>0.4</td>
</tr>
<tr>
<td>10</td>
<td>14'-3&quot;</td>
<td>4.0</td>
<td>0.5</td>
</tr>
<tr>
<td>11</td>
<td>15'-9&quot;</td>
<td>8.2</td>
<td>0.9</td>
</tr>
<tr>
<td>12</td>
<td>17'-3&quot;</td>
<td>18.0</td>
<td>2.1</td>
</tr>
<tr>
<td>13</td>
<td>18'-9&quot;</td>
<td>14.1</td>
<td>1.6</td>
</tr>
<tr>
<td>14</td>
<td>20'-3&quot;</td>
<td>6.0</td>
<td>0.7</td>
</tr>
<tr>
<td>15</td>
<td>21'-9&quot;</td>
<td>2.9</td>
<td>0.3</td>
</tr>
<tr>
<td>16</td>
<td>23'-3&quot;</td>
<td>2.0</td>
<td>0.2</td>
</tr>
</tbody>
</table>

The shear demand for Scenario 4 was represented by the differential shear quantity used in the original design, and was calculated based on the difference in $V_{13}$ out-of-plane shear at shell element joints 12 in. apart. The maximum unfactored $V_{13}$ dowel shear demand at the expansion joint was approximately 12 kips, at Dowel 1. The maximum unfactored differential shear was approximately 5 kips, at Dowel 12. As such, the dowel shear demand used in the original design was about 21 percent of the design shear demand, 24.1 kips, calculated in the investigative analysis.

Our analysis indicated peak demands at the uppermost dowel and at one of the dowels near the increase in wall thickness for all scenarios. Although the shear demands discussed here have been slightly greater for the uppermost dowel than for the dowel near the change in thickness, these results are likely dependent on the height of the basin wall, relative change in wall thickness, location of wall thickness increase, and possibly other factors. Although the shear demands are reduced significantly for Dowels 2-10, these results are only applicable if the uppermost dowel remains elastic. Analyses have shown that if the uppermost dowel yields or fails, the shear demand on Dowel 2 may increase significantly, and so on. The potential for this type of load redistribution is not accounted for in a linear elastic analysis. Due to these factors, a conservative approach would be to design all expansion joint dowels for the peak calculated demand in shear and bearing.

The dowels were 300-series stainless steel, with $F_Y = 30$ ksi and $F_U = 75$ ksi. The authors evaluated the shear capacity according to AISC 360 Section J4.2, which addresses connecting elements in shear by considering both the limit states of shear yielding and shear rupture. The nominal and allowable dowel shear capacities were calculated as 8.7 kips and 5.8 kips, respectively. The design engineer evaluated the shear capacities according to AISC 360 Section J3.6, which addresses the capacity of high-strength bolts in shear based on the limit state of shear rupture only. The nominal and allowable dowel shear capacities calculated by the design engineer were 16.6 kips and 8.3 kips, respectively. Only the limit state of shear rupture is considered for high-strength bolts because, due to the heat treatment required to provide increased strengths, they do not have a well-defined yield point or exhibit significant post-yield behavior. The 300-series stainless steel dowels do have a well-defined yield point and do exhibit significant post-yield behavior.

ACI 318 Section 10.17 addresses the bearing strength of concrete supports and specifies an allowable bearing stress of 0.85f'c. ACI 318 does not directly address concrete bearing capacity for a dowel-concrete interaction surface. The bearing stress distribution along the length of a dowel is nonlinear with large stress concentrations at the face of the joint. Some research has been conducted on the behavior of dowels in pavement expansion and construction joints. The research typically
models the dowel-concrete behavior using Timoshenko’s linear elastic theory for a bar on an elastic foundation. The displaced shape of a dowel bar, which is proportional to the concrete bearing stress, as predicted by Timoshenko, is shown in Figure 10. ACI 325 – Design of Jointed Concrete Pavements for Streets and Local Roads does specifically address expansion and construction joint dowels, but only provides dowel size recommendations based on pavement thickness. In the absence of a specific code requirement, the concrete bearing capacity was evaluated by the authors based on ACI 318 Section 10.17 assuming a constant bearing stress distributed over a length of 3 times the dowel diameter. In the ultimate state, a flexural hinge typically forms over a length of 2-3 times the dowel diameter and therefore the bearing stress would be primarily distributed in this region. Based on this approach, the nominal and factored concrete bearing capacities at the dowel interface were calculated to be 5.7 kips and 3.7 kips, respectively. The bearing capacity was calculated by the design engineer based on a linearly increasing stress distribution from the embedded end of the dowel to the joint face. The nominal and factored concrete bearing capacities at the dowel interface were calculated by the design engineer to be 14.7 kips and 9.5 kips, over 2.5 times greater than the capacity calculated by the authors.

For the expansion joint discussed here, due to the 1-inch width of the expansion joint, the formation of flexural hinges within the joint would likely be precluded. Flexural hinges typically form over a length of 2-3 diameters, which in this case would be within the confinement provided by the basin wall concrete. Preventing, or limiting, the formation of flexural hinges is dependent on providing adequate concrete bearing capacity. The bearing stress concentrations that occur at the joint face can result in local concrete crushing, which can cause the dowel sockets to elongate and allow for greater dowel deformation. This may result in the formation of flexural hinges and a reduction of the allowable dowel shear capacity. As yielding begins to occur, the dowel stiffness is reduced which will reduce the local dowel shear demand. Therefore, the effects of local concrete crushing and the resulting inelastic dowel behavior may be self-limiting. However, the joint movement at the final equilibrium state may exceed the movement capabilities of the joint seal. Due to the effect of the confining concrete in preventing or limiting the formation of flexural hinges, providing adequate concrete bearing capacity is a critical factor in the behavior and performance of the dowels. As such, we recommend a minimum concrete bearing strength based on the assumption of a bearing length equal to 3 times the dowel diameter. Due to the large bearing stress concentrations at the face of the joint, local concrete crushing may still occur under ultimate loads. If this is determined to be unacceptable, concrete bearing design may be more conservatively evaluated based on another approach such as the Timoshenko elastic foundation theory.

The failure demand-to-capacity ratios (Failure DCR) for dowel shear and bearing were calculated based on the ratio of unfactored service demand to nominal capacity, and are shown in Table 1. The Failure DCR for the uppermost dowel, at the top of the basin wall, was calculated to be about 2.1 based on the shear capacity of the dowel itself and about 3.1 based on the concrete bearing capacity. In addition, because the Failure DCR for bearing is significantly greater than one, it is likely that significant concrete crushing occurred. As described above, this may have resulted in a reduction in the dowel shear capacity which would result in an increase in the Failure DCR in shear.

The design demand-to-capacity ratios (Design DCR) for dowel shear and bearing were calculated based on a ratio of the design demand for each dowel and the design capacity. The design capacity is the calculated capacity modified by the reduction factor(s) prescribed by AISC 360 or ACI 318. If the ratio is less than one, the dowel was in compliance with AISC 360/ACI 318. The Design DCR is over 4 for the uppermost dowel in shear and over 11 for the uppermost dowel in bearing.

Although an inelastic analysis was not performed to determine the shear redistribution behavior, Scenario 3 was performed assuming that all dowels had failed. The calculated relative displacement at the top of the basin wall due to in-service loading was 1.2 inches. A relative displacement across the joint of 0.75 inches was measured at Basin No. 5 while full. This indicated that, although some dowels may have yielded, the expansion joint still had some shear transfer capability.
2.3 Repair

In order to re-establish the shear capacity of the expansion joint, repairs were implemented which provided twenty new 1 3/4-inch diameter stainless steel dowels at variable spacing from 8 inches to 18 inches, based on shear demand. The new stainless steel dowels were set into full-height built-out concrete sections, approximately 55 inches wide and 20 inches thick, on the dry side of the existing basin walls as shown in Figure 11. Similar sections were also constructed on the wet side of the existing walls. High-strength post-tensioned 1 inch-diameter threaded rods were used to connect the dry side and wet side sections and effectively clamp the repair to the existing walls. In order to re-establish the water-tightness of the expansion joints, supplemental joint seals were provided on both the wet and dry sides of the existing waterstop.

2.4 Summary

The failure of the expansion joint was due to both an underestimation of demand and overestimation of capacity. The underestimation of demand was due to the use of the differential shear quantity. It was unclear why this approach was taken. In addition, it appeared that calculating the difference in shear values 12 inches apart was an arbitrary choice based on the size of the shell elements used in the analysis model. Had the shell element sizes been different, the calculated demand would have been different because the shear demand in the basin wall varies horizontally. Compounding this problem was an incomplete understanding of the in-plane and out-of-plane shear components. A simpler, more intuitive approach would have been to model the dowels as frame elements which would have correctly captured the shear demand. The overestimation of bearing capacity and dowel shear capacity was the result of the improper application of code provisions. Neither AISC 360 nor ACI 318 explicitly outlines how to apply the code to this type of doweled configuration. Unfortunately, the code interpretation by the design engineer in this case resulted in a significant overestimation of capacity.

While the expansion joints in Basin Nos. 3 and 5 were the only joints to fail, there were other expansion joints in the tank structure which were constructed based on the same detail. As shown in Figure 7, the expansion joints that failed were both located close to the side of the basin. The basin partition walls effectively restrained the exterior basin wall laterally. The resulting behaviour of the exterior wall is similar, particularly at the top of the wall, to a continuous beam. The remaining expansion joints in the tank were typically located near or at “midspan” between the walls separating the basins. As a result, the overall shear demand was significantly reduced.

3. References

[2] American Concrete Institute, Committee 318, “Building Code Requirements for Structural Concrete and Commentary (ACI 318-11)”, American Concrete Institute, Detroit, MI, 2012.