STRUCTURAL FORENSIC INVESTIGATION REPORT

Partial Failure of Ramp AC
Dunn Memorial Bridge Interchange
BIN 109299A
City of Albany, Albany County, New York
July 27, 2005

Prepared by:
NYSDOT
Albany, New York
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Appendix A Failure Sequence Cells

Appendix B Historical Information for Pier 11 Rocker Bearings of BIN 109299A from Bridge Inspection Reports

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Executive Summary

As a result of a partial failure of the I-787 Ramp Northbound to the South Mall Expressway Westbound [BIN 109299A] on July 27, 2005, the New York State Department of Transportation conducted an investigation into the cause of the failure. The results of this investigation are summarized here.

The failure resulted from the loss of support provided by Pier 11, causing the two spans carried by it to displace vertically. This can be attributed to the unique combination of all of the following factors:

- Development of improper bearing alignment of the Span 12 bearings due to shifting of the Span 12 superstructure combined with some initial lateral deformation of Pier 11,
- Eventual loss of proper function of the Span 12 bearings,
- The high degree of flexibility and flexural properties of Pier 11 in the direction longitudinal to the bridge.
- An initiating event that was likely induced by thermal movements or possibly traffic loads.

While it is known that high rocker bearings can be susceptible to loss of function leading to unintended forces on substructures; the effects of such forces on structural support and stability depends on the entire structural system’s behavior. This includes the superstructure spans supported by the pier, the supporting and adjacent substructure units, and the overall bridge and bearing geometry. This failure resulted when conditions created unintended forces of moderate magnitude combined with a substructure having a relatively low resistance to horizontal deflection and low elastic ductility, which resulted in the behavior of the bridge system that was unanticipated.

A. Failure Factors

The underlying cause of the partial failure was the loss of proper function of the Span 12 rocker bearings, which became over-rotated and restrained from proper movement. This resulted in horizontal forces and displacements on pier 11 sufficient to cause the rocker bearings to tip over. The sequence of the bearings tipping caused the pier to continue to deflect laterally to where Span 12 girders fell onto and sheared the pier pedestals. The high degree of flexibility and lack of elastic ductility of Pier 11 in the longitudinal direction contributed to this chain of events. The findings in this regard are as follows:

1. **Bearings**: The bearings became misaligned at some time prior to 1985. This was likely the result of movement of the superstructure caused by vehicular braking forces and centrifugal forces that pushed the structure northward. This exposed the bottom of the bearing rockers and pin assemblies to the elements. This situation resulted in corrosion to the rocker seat areas and pin contact surfaces, which restrained the bearings from moving freely as intended by design, thus allowing for the transfer of horizontal forces to the pier. These unintended forces deflected the top of the pier to the south.
2. **Pier Column**: The Pier 11 column concrete was in very good condition up to the time of the failure.
   a. A review of the original pier design indicated that the design stress requirements of the 1965 AASHO Design Code used at the time were met. The review did reveal that a code provision to provide a minimum amount of reinforcement to ensure ductile behavior in bending was not satisfied. While this probably did not have a direct effect on the failure, it contributed to the extent of the failure.
   b. A detailed computer model analysis of the in-situ pier was performed to better understand the actual behavior of the column and identify the forces that were required to deflect the pier. This analysis revealed that the pier column would crack at a deflection of 2.5 inches from a 141 kip (1 kip = 1000 pounds) horizontal load. It further revealed that, after the column cracked, the steel reinforcing in the column would yield at a deflection of 5.4 inches with a horizontal load of 108 kips. This means that the load needed to yield the column once the column cracked was less than the load required to crack the column. In other words, once the column cracked, the pier had no additional capacity to resist flexural yielding failure, assuming that the horizontal load is sustained.

B. **Failure Mechanism**

Evidence indicates that the failure was due to the column failing in flexure and the Span 12 bearings becoming over-rotated to the extent that they became unstable and tipped over. For the bearings to over-rotate, the pier column had to deflect in the longitudinal direction of the bridge. Since the investigation revealed that the pier foundation did not move, the only plausible alternative is that the top of the pier was deflected horizontally by forces caused by loads and thermal movements from the superstructure that were transferred through the bearings. This is reasonable based on the geometrics of the failed condition, and the findings of laboratory inspections of the bearings. Observed levels of rust build-up and dishing of the masonry plates created conditions which inhibited their free movement and allowed for the transfer of horizontal forces. Over an extended period of time, the bearings continued to transfer horizontal forces to the pier, gradually increasing the column deflection. This increased deflection caused the bearings to tip further to the north to a point where the Span 12 bearings became unstable and tipped over. At about this point the pier column cracked, which enabled the column to deflect further from reduced horizontal loads. With the Span 12 bearings tipping, enough column deflection was produced to result in the Span 11 bearings also tipping over, which in turn produced even more column deflection. The Span 12 girders fell onto the edges of the pier pedestals, shearing the pedestal concrete and ultimately rested on edge of the pier cap. The Span 11 girders fell onto the tipped over Span 11 bearings.
**Introduction**

On July 27, 2005 Spans 11 and 12 of the I-787 Ramp Northbound to the South Mall Expressway Westbound (BIN 109299A) fell off their supporting bearings at pier 11, with span 12 partially falling off the pier. The New York State Department of Transportation conducted an investigation into the cause of this failure and this report provides the findings.

![Figure 1: Partial Failure Pier 11, East Fascia](image)

**Structure Description**

This ramp structure is a 24 span single lane “flyover” ramp comprised of a series of single span and two span continuous steel multi-girder span units. Most spans are supported on single column ‘hammerhead’ reinforced concrete piers, with a few spans supported on multi-column frame piers that also support adjacent ramps. The partial failure occurred at Pier 11 of the structure, an 82 ft. high single column hammerhead pier. The adjacent piers (Piers 9, 10, 12 and 13) are similar in type and size. Pier 11 supports the ends of two span continuous superstructure units: Spans 10 and 11 (188 ft.-188 ft.) and Spans 12 and 13 (116 ft-117 ft.). Spans 10 and 11 are on a tangent, as is the first 50 ft. of Span 12. The remainder of Span 12 and all of Span 13 are on a horizontal curve to the west having a centerline radius of 344 ft. See Figure 2 for an elevation view of bridge Spans 10 through 13.

The substructure element designations used in this report follow current NYSDOT Bridge Data Management System (BDMS) nomenclature. The reader is cautioned that different designations were used in the original contract plans namely, Pier 11 in this report was designated Pier AC 12 in the original contract documents.
Description of the Failure
The failure occurred when the top of the pier was pushed, or deflected, laterally to the south, out from under the ends of the Span 12 girders causing them to drop 25 inches onto the edge of the pier cap. The force to push the pier was initially imparted by the Span 12 high rocker bearings after becoming overextended to where they became unstable and tipped. As the pier cap was pushed laterally, the Span 11 bearings also became unstable and then tipped which further pushed the pier cap to the south. These combined displacements were sufficient to cause the ends of the Span 12 girders to fall onto the ribs of the tipped rocker bearings and then onto the edges of the north pedestals. The impact from the girder sheared off the edges of the pedestals until the girder ends became wedged on the edge of the pier cap. The span 11 girders fell 5 inches onto their tipped bearings. (See Figure 3)
The underlying causes of the failure were the loss of proper function of the Span 12 bearings from being in an inclined position for an extended period of time, the flexibility in the longitudinal direction of the 82 foot tall Pier 11, as well as insufficient flexural capacity of the pier stem to elastically resist loads beyond the anticipated design loads. All of these factors were required for this failure to occur.

**Behavior of the High Rocker Bearings**

The high rocker bearings used at Pier 11 were designed to accommodate thermal movements of the supported spans by rolling on a masonry plate at the bearing base and pivoting on a semi-circular pin at the top of the bearings. The rotational centers of the rocker and the pin are concentric and act as a wheel or roller. The leverage provided by the height of the rocker is effective in overcoming any sliding friction in the upper pin assembly. Thus, when functioning properly, high rocker bearings are very efficient in minimizing longitudinal forces being transmitted from the superstructure or from friction.

The Span 12 bearings on Pier 11 had a rocker radius of 9 inches, a rocker width of 8 inches, and were intended to accommodate design lateral eccentricities (displacements) of up to 2.5 inches. The design thermal movement in one direction for Span 12 (117 ft. span) is 0.9 inches. The Span 11 bearings had a rocker radius of 13 inches, a rocker width of 12 inches, to accommodate up to a 3.25 inches of eccentricity. The design thermal movement for Span 11 (188 ft. span) is 1.45 inches.
Observed Conditions: 1985-2003

When the first detailed biennial inspection was done in 1985 it was noted that the bearings on Pier 11 for both Spans 11 and 12 were out of plumb, all tilted to the north. A photo from the 1987 inspection report shows the Span 12 and 11 lines of bearings displaced in the northern direction about 1.7 and 0.9 inches respectively at a temperature of 45° F. Ideally at this temperature the bearings should be effectively vertical, or both slightly in the contracted direction (Span 11 tilted south and Span 12 tilted north). During any temperature variations they should move in opposing directions, not the same direction as was the case here. Subsequent inspections show that the bearings remained tilted to the north through the 2003 inspection, although the amount of tilt of the span 12 bearings had gradually increased in the more recent inspections. In 2003 the Span 12 bearings were tilted 3.4 inches or 0.9 inches overextended beyond the design limit. The Span 11 bearings were tilted north about 1 inch, still well within their design range. At 3.4 inches of eccentricity, the Span 12 bearings were still geometrically stable, since the eccentricity is within the bounds of the rocker width.

Figure 5: Pier 11 Bearings, 1989 Bridge Inspection Report

In addition to the biennial inspection records, a 1990 survey by the Region 1 Bridge Maintenance Engineer [See Appendix D] recorded that the Span 12 bearings were tilted north by 2.56 inches and the Span 11 bearings were tilted north 1.75 inches with the air temperature at approximately 60° F. The bearings should have been near vertical at this temperature.
Initial Misalignment of the Pier 11 Bearings

Bridge bearings can get misaligned due to shifting of the superstructure spans, or movement of the supporting substructure, either of its own accord or by being moved by the superstructure. Available evidence suggests that some shifting of the superstructure spans occurred early in the life of the bridge, when the bearings should have been in good functional condition. Shifting of the spans requires that the piers connected to them by fixed bearings, in this case Piers 10 and 12, would have to had shifted with the spans. Surveys taken after the failure and again after the spans were lifted off Pier 11 indicate a prior lateral shift to the north of both spans that is consistent with the early inspection data on the bearing tilting on Pier 11. (See Table 1)

<table>
<thead>
<tr>
<th>Pier No.</th>
<th>Fascia Location</th>
<th>Distance Top of Pier from Plumb*</th>
<th>Measured Movement from Release**</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>West West</td>
<td>1.73” (44 mm)</td>
<td>0.55” (14 mm) north</td>
</tr>
<tr>
<td>10</td>
<td>East West</td>
<td>0.63” (16 mm)</td>
<td>0.63” (16 mm) north</td>
</tr>
<tr>
<td>12</td>
<td>West East</td>
<td>1.38” (35 mm)</td>
<td>0.67” (17 mm) south</td>
</tr>
<tr>
<td>12</td>
<td>East West</td>
<td>2.72” (69 mm)</td>
<td>0.59” (15 mm) south</td>
</tr>
<tr>
<td>13</td>
<td>West East</td>
<td>1.81” (46 mm)</td>
<td>(not measured)</td>
</tr>
<tr>
<td>13</td>
<td>East East</td>
<td>2.32” (59 mm)</td>
<td>(not measured)</td>
</tr>
</tbody>
</table>

* Positive values indicate movement to the north.
** Measured change in position of a target on the side of the pier cap before and after release, where “release” is when the failed spans were lifted off Pier 11.

Table 1: Survey of longitudinal position of tops of piers-- Piers 10, 12 and 13.
(Survey done in Metric Units, English conversion)

A detailed survey by Region 1 Bridge Maintenance in June, 1990 also noted that the expansion finger joint in the deck at Pier 13 was at 0.3 inches from being fully closed at 60o F. The survey memo implies that these measurements were taken sometime after this joint was reset vertically to match a new concrete overlay being placed under a deck repair contract. The repair contract records give no indication of the joint opening also being reset, and this subsequent survey information suggests that this was not done. The 1991 biennial inspection notes that the joints had been “repaired” but provided no further details or photos, and it gives the joint an inspection rating of 3—the same value given in 1989. Since the 1991 inspection was compiled from site visits in August, October and November, the temperature at the time of the joint inspection is not known. The 1993 inspection also notes that the joint was “repaired”, without further documentation, and gives a rating value of 6 suggesting that the joint was somewhat opened at the time. Records indicate that the 1993 inspection was performed between November 29 and December 3 when temperatures ranged from 28 - 44o F. The 1995 inspections reported that this joint was nearly closed (within ¼ inch at 20o F) and subsequent inspections consistently reported the joint as being closed.

The same 1990 survey recorded the Pier 13 curb plates having an average opening of 1.3 inches versus its original theoretical opening of 3 inches at 68o F. Measurements taken after the failure and after Span 12 was lifted back to its proper profile show the curb plate openings remaining between 1 and 1.5 inches and the Pier 13 joint still effectively closed.

This sequence of joint measurements from 1990 to the present further documents the persistent shifting of Spans 12-13 to the north. The extent of the shifting appeared to be constant during this period and constrained by the closure of the Pier 13 joint. This joint was originally installed with a theoretical opening of $\frac{13}{16}$ inches at 68o F.
The most likely cause of such shifting is repeated longitudinal braking forces and, to some extent, centrifugal forces from vehicular loads (trucks) entering the roadway curve on Spans 12 and 13. These effects would tend to push Spans 12 and 13 northward away from Pier 11. A contributing factor is that the bridge is on a nearly 4 percent uphill grade at Pier 11. Rocker bearings that support continuous spans on such grades have a slight tendency to resist any movement back to their plumb position once they are rotated “upgrade” as was the case here.

There is no evidence that Pier 11 moved on its own accord to cause the bearing misalignment. Such a case would imply that the pier foundation shifted or tilted, and surveys of the footing and a geotechnical analysis indicates that the footing is level and the foundation is adequate in meeting design requirements for capacity. The possibility that the pier cap was moved southward by lateral forces from the superstructure early in the bridge’s life is also unlikely. This would have required the rocker bearings to transmit forces well beyond their frictional capacity if functioning, or somehow become non-functional while relatively new.

The original positioning of the bearings combined with the difference in dead load reactions between Span 11 and Span 12 resulted in a slight net eccentricity in the combined vertical load on the pier. An analysis determined that this would cause the top of the pier to deflect 0.05 inches to the south. This displacement would be insignificant to the pier behavior and would have been negated once the bearings began leaning toward the north.

**Condition of the Pier 11 Rocker Bearings**

As indicated above, the Pier 11 rocker bearings were in a tilted position for at least the past 20 years which increased the potential for them to become non-functional. The tilted position made them more susceptible to corrosion and rust build up under the exposed rocker surface, as well as to corrosion of the bearing pintles that keep the rockers positioned on the masonry plates. The corrosion formed a ‘wedge’ under the rocker that prevented it from rocking back toward the vertical.

The bearings from Spans 11 and 12 were removed from the bridge and taken to the DOT labs for inspection. (Figure 5) Visual inspection confirmed the development of corrosion and rust accumulation that would prevent the free movement of the rocker back toward the vertical. The rocker bearing surfaces in contact were also found to have flattened or dished so that when free to stand, the bearings assume tilted positions. The contact surfaces of the semi-circular pins at the tops of all the Span 12 bearings were found to be in a rusted condition. The bases of the Span 12 rockers also have paint intact over much of their surfaces. These conditions indicate that these bearings had not moved in some time. In contrast, the pin surfaces at the tops of the Span 11 bearings exhibited shiny metal, an indication that these bearings had been moving. The pintles on both the Spans 11 and 12 bearing masonry plates were found to be heavily corroded.

Once the rocker bearing is restrained from movement, or “frozen”, longitudinal forces from the superstructure can be transmitted through the bearing. Restraint of thermal movement generates very large forces on the bearings which may overcome any bearing resistance. The pier, because it was tall and slender, was less resistant to movement than the restrained bearings and any thermal movement was then accommodated by displacing the top of the pier instead of by the bearings. A rocker bearing in a tilted position and “bound up” by corroded pintles would also be capable of transmitting a horizontal force component of the vertical dead and live loads on the bearings.
Figure 6a: Pier 11 Rocker Bearings in Lab

Figure 6b: 4N Masonry Plate & Rocker

Figure 6c: 4N Sole Plate

Figure 6d: 4S Masonry Plate & Rocker

Figure 6e: 4S Sole Plate

Figure 6: Pier 11 Rocker Bearings in Lab
**Evaluation of Pier 11**

Pier 11 is a solid stem “hammerhead” pier, 82 ft. tall from the top of footing to top of pier cap, and with stem plan dimensions of 13.88’ x 6.44’ at the base and tapering to 9.0’ x 4.0’ at the base of the pier cap. This pier is similar in height and dimensions to the adjacent Pier 10 to the south and Piers 12 and 13 directly to the north. According to the contract record plans, Pier 11 had significantly less primary reinforcement compared to these other piers. (Table 2).

<table>
<thead>
<tr>
<th>Pier No</th>
<th>Height</th>
<th>Base Dimensions</th>
<th>Reinforcing Steel</th>
<th>Bearings (Fix or Exp.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>67.47’</td>
<td>131.4” x 71.2”</td>
<td>42 - #8</td>
<td>33.18 Fix - Exp</td>
</tr>
<tr>
<td>10</td>
<td>72.79’</td>
<td>132.6” x 72.6”</td>
<td>36 - #11</td>
<td>56.16 Fix</td>
</tr>
<tr>
<td>11</td>
<td>82.31’</td>
<td>166.6” x 77.3”</td>
<td>46 - #8</td>
<td>36.34 Exp – Exp</td>
</tr>
<tr>
<td>12</td>
<td>83.38’</td>
<td>155.3” x 77.6”</td>
<td>42 - #11</td>
<td>65.52 Fix</td>
</tr>
<tr>
<td>13</td>
<td>84.75</td>
<td>156.4” x 84.2”</td>
<td>42 - #11</td>
<td>65.52 Exp – Exp</td>
</tr>
</tbody>
</table>

*Table 2: Cross-sectional properties of pier stems, Piers 9 - 13*

Piers 10 and 12 would be expected to have more reinforcement since they support fixed bearings and thus would have to resist design longitudinal forces from the superstructure. Pier 13 and Pier 11 would be expected to resist similar horizontal and vertical loads since they both support only expansion bearings; however Pier 13 contains 1.8 times the stem reinforcement of Pier 11.

The 2003 and prior biennial inspection reports show that the Pier 11 concrete stem was consistently in very good condition (rated 6 on a scale of 7 in 2003) with no deficiencies reported. A concrete core sample taken on August 13, 2005 was tested and showed a concrete strength of 9,210 psi, well above the design requirement of 3,000 psi.

Pier 11 was originally designed in accordance with the provisions of the 1965 edition of the AASHO Design Specifications for Highway Bridges. [Original design calculations are not available] A design-level analysis of Pier 11 shows that the pier stem reinforcement was adequate to satisfy working stress requirements for loadings imposed in accordance with design specification assumptions, but with no reserve capacity. The capacity-to-demand ratio was calculated as 0.98, acceptable compared to the 1.00 target value. These design load assumptions consider the rocker bearings to be functional, thus limiting the amount of longitudinal forces assumed to be transferred to the pier. The 1965 specifications contained design provisions for pier columns with small load eccentricities, i.e., where vertical loads are large compared to overturning moments. There were no provisions for large eccentricity conditions other than referring to using accepted cracked section concrete theory. For most bridge pier stems or columns, especially for tall piers, the design is controlled by such large eccentricity load cases.

Both the 1965 and current design codes require pier columns to contain a minimum of 1 percent reinforcement by area. The Pier 11 reinforcement ratio at the stem base is 0.28 percent. The codes allow the percentage of steel to be reduced if it is not less than that required by the minimum column designed with one percent longitudinal steel. Thus for pier 11 a minimum column of 3634 sq. in. gross area would have to be used for design to satisfy this provision. A design level analysis indicates that the capacity-to-demand ratio is 0.35 for this ‘design’ minimum column size and thus this specification provision is not satisfied.
Detailed Pier Analysis

A detailed non-linear finite element analysis (FEA) of the pier was made to determine its flexural characteristics. The analysis loads the pier with its vertical loads from its self-weight plus the superstructure loads and then the top of pier is deflected laterally in increments. A lateral force applied at the pier top to cause such displacements is then calculated, and plotted as a force vs. displacement graph as shown in Figure 7.

![Graph: Load-Deflection graph for Pier 11](image)

The FEA shows that the pier reinforcement begins to yield when the lateral displacement reaches 5.4 inches, thus reaching its maximum flexural resistance at this point. The lateral force needed to reach this point is about 108 kips. Once yielded, the pier behaves plastically and the continued application of this 108 kip force will cause continued deflection of the pier up to and beyond the 18 inch limit of this analysis. When the restraining loads were jacked off the pier during the repair operations, the pier rebounded about 5 1/2 inches which validates the 5.4 inch displacement as the elastic range.

The analysis shows that concrete cracking of the pier stem occurs when the top of pier displacement reaches 2.5 inches. While the cracking displacement is mostly a function of the pier dimensions and to a lesser extent a function of the concrete strength, the force to cause cracking is greater with increased concrete strength. If the concrete strength is at the design level of 3000 psi, a 92 kip force is needed to reach the cracking displacement. If the 9210 psi core sample value is used, a 141 kip load is required for cracking. Cracking capacity is generally not considered in determining the design strength of a reinforced concrete flexural member and the member is designed as a cracked section, as was most likely the case here.

This analysis also shows that Pier 11 had very little elastic ductility as well as having the concrete cracking and steel yielding limits undesirably close, with the cracking limit seemingly greater than the yield strength. Once the pier concrete cracked there was little or no reserve capacity to resist yielding. The 108 kip or even a 141 kip longitudinal force needed to cause these displacements are easily generated if the high rocker bearings are tilted and not functioning, as was likely the case.
If it indeed took 141 kips, or over 108 kips, to crack the pier then the pier ‘fails’ once it cracks as the force, if sustained, will drive it right to yielding. Once yielding is reached sustaining the yielding force will cause the pier displacement to continue virtually unimpeded.

The flexibility of the pier is mostly a function of its geometry vs. its reinforcement, at least for lower deflection values. Load-deflection analysis comparisons of Pier 11 and its more heavily reinforced replacement design demonstrate this by having similar load-deflection curves up to approximately 8 inches of displacements (See Figure 8). However the new design remains elastic up to an 18 inch displacement and the load required to impart a large deflection does increase noticeably.

![Figure 8: Load-deflection graphs for Pier 11 original and replacement designs.](image-url)
Failure Hypothesis

Based on the evidence, the partial failure at Pier 11 can be attributed to the over-rotated Span 12 bearings becoming unstable combined with the flexural failure of the Pier 11 column.

What initiated the failure may never be known for certain, but the evidence strongly suggests that it was a combination of the Span 12 bearings becoming so overextended that they reached their limit of stability and eventually tipped over, combined with the pier failing in flexure. It is known that these bearings were overextended to within ¾ of an inch of their geometric stability limit in 2003, and that the failure itself involved the pier cap being moved laterally out from under the bearings and out from the south end of the Span 12 girders. It is also known that the Span 11 bearings were consistently tilted northward between 1 to 1.75 inches since 1985.

Two failure initiation scenarios are plausible: (1) If the bearing overextension was due in large part to the pier being displaced, the failure displacement of the pier could have been reached before the bearings tipped. In other words, the pier failed first. Once failed, the pier cap continues to displace laterally out from under the bearings. (2) If much of the initial bearing overextension was not due to pier displacement then the bearings could have become unstable and tipped first. The tipping of the bearings then pushed over the pier to its failure point.

Since Spans 12 & 13 were physically constrained by the deck joint at Pier 13 from moving northward more than about 1.8 inches, this is the maximum plausible displacement of the Span 12 bearings that could have occurred without Pier 11 being displaced southward. This movement would have been taken up in Pier 12 being displaced; however, the survey suggests that Pier 12 was permanently displaced approximately 1.5 inches northward. The history of different displacements between the 2 lines of bearings at Pier 11 also suggests that some combination of superstructure movements and displacements of Pier 11 occurred before the failure. At some point in time when the bearings ceased to function properly they began imparting forces on the pier and began displacing it southward. Displacement of the pier was necessary to bring the Span 12 bearings to the verge of tipping over. The onset of failure could have occurred when the pier was displaced to the concrete cracking limit, but still before the bearings tipped. Once the pier cracked it had to readjust its internal equilibrium, whereby all the tensile stress resisted by the concrete had to be taken up by the steel. To do this, the steel had to become highly stressed which required a sudden increase in displacements (steel strain) to regain equilibrium. This “jump” in the pier displacement would have been sufficient to bring the Pier 12 bearings past their tipping threshold.

The stability limit of the Span 12 bearings is just over 4 inches, or half the width of the bearing rockers. In 2003 these bearings were overextended to 0.7 inches of their stability limit. At the same time the pier stem was reported in very good condition, with no signs of cracking or distress, indicating that the cracking threshold displacement was not yet reached. The 4+ inch stability limit of the Span 12 bearings was most likely reached by the Span 12 superstructure being shifted north between 1 and 2 inches, and the top of Pier 11 being pushed south between 2 and 3 inches. The northward movement of the Span 12 superstructure is based on the survey of Pier 12 as well as the history of closure of the joint above Pier 13. The limits of accuracy of the survey data and historical joint data result in the 1 inch range in the estimated superstructure movement. These values suggest that the bearing stability threshold and the pier cracking threshold were close to being concurrent.

The most likely scenario is that the Span 12 bearings continued to become overextended after 2003. This would have occurred by their inability to return toward vertical during times of expansion of
Span 12, thus pushing Pier 11 southward to accommodate the expansion. During periods of Span 12 contraction, the bearings could tip further north thus adding to their overextension. Repeated cycles of warm and cold weather would cause the overextension these bearing to 'ratchet' further northward. July 2005 had an extended period of 90°F daytime temperatures followed by a 20°F cooling the night of the failure. This final push on the pier by the span expansion and the cooling pulling the bearings back north would have brought the pier-bearing system to the brink of instability.

The Failure Sequence
While there is a more than one possible exact sequence leading up to and initiating the final failure, they all center on the pier bearings, more notably the Span 12 bearings, becoming overextended to the point where they were unstable and tipped over. The tipping of the bearing pushed the top of the pier southward causing the Span 11 bearings to become overextended and also tip over. Movement of the pier was enough for the end of Span 12 to land on the north outer edges of the pier pedestals. The weight of the Span 12 bearings on these pedestals was enough to shear off the pedestal concrete and drop Span 12 further until it reached the bottom of the pedestal and wedged into the top edge of the pier cap.

Appendix A depicts the sequence of events beginning with the 1970 as-built condition [Cell 0] and concluding with the failed position on July 27, 2005 [Cell 13]. Cells 0-4 indicate the position of the bearings as measured from bridge biennial inspection reports from 1985 to 2003. Cells 5-7 indicate the first phase of the failure where the Span 12 bearings became unstable [Cell 5]; the column concrete cracked [Cell 6]; and the bearing ribs contacted the bearing sole plates [Cell 7]. Cells 8-13 depict the final sequence where the girders dropped and came to rest on the pier cap. The Span 12 bearings tipped and the concrete column reinforcing steel began to yield [Cell 8]; the Span 12 girders impacted the sides of the rocker bearings [Cell 9]; the Span 12 girders slid down the bearing ribs and the Span 11 bearings became unstable [Cell 10]; the Span 11 bearings impacted the pedestals [Cell 11]; the Span 12 girders crushed the pedestal concrete and the Span 11 bearings began sliding across the masonry plates [Cell 12]; and the Span 12 girders became wedged on the pier cap and the Span 11 girders came to rest on the sides of the Span 11 rocker bearings [Cell 13].
Conclusion
The partial failure of the I-787 Ramp resulted from the loss of support provided by Pier 11 and resulted in vertical displacements of the two spans carried by it. This can be attributed to the simultaneous presence of all of the following factors:

- Development of improper bearing alignment of the Span 12 bearings due to shifting of the Span 12 superstructure combined with some initial lateral deformation of Pier 11,
- Eventual loss of proper function of the Span 12 bearings,
- The high degree of flexibility of Pier 11 in the direction longitudinal to the bridge.

Contributing to the failure was the limited elastic range of Pier 11 due to its being lightly reinforced, and the actual bearing dimensions, including the different height bearings used for Span 11 and 12.

Persistently misaligned or tilted high rocker bearings are prone to corrosion, flattening and rust accumulation at their contact surfaces. This causes them to become resistant to free movement, especially inhibiting the bearings’ ability to move back to the vertical position. Such bearing resistance was developed in the Span 12 bearings, which enabled them to transfer horizontal forces to the pier. The high degree of flexibility of the pier made it susceptible to be moved or pushed by thermal expansion of Span 12, while the Span 11 bearings remained functional to accommodate Span 11 thermal movements. An analysis of the pier showed that the magnitude of bearing resistance did not have to be very high to overcome the resistance of the pier.

While it is known that high rocker bearings can be susceptible to loss of function leading to unintended forces on substructures; the effects of such forces on structural support and stability depend on the entire structural system behavior. This includes the superstructure spans supported by the pier, the supporting and adjacent substructure units, and overall bridge and bearing geometry. This failure resulted from a case where conditions created unintended forces of moderate magnitude that combined with a substructure having relatively low resistance to horizontal deflection. This occurred without the bearings becoming completely frozen, as they continued to move in the tipping direction during periods of contraction of Span 12.

While bearing behavior can be monitored through inspections, the load vs. deflection characteristics of a pier such as this one is not apparent without further detailed analysis. For smaller deflections in the range sufficient to overextend and possibly tip over a rocker bearing, these characteristics are primarily a function of the pier’s shape and overall dimensions. The pier reinforcement and concrete properties also influence this behavior. In general, tall slender piers are more susceptible to such deflections. A shorter, stiffer pier in combination with these bearing conditions would have resisted the excessive pier deflection that contributed to initiating this failure sequence. The relatively light reinforcement in the pier stem resulted in the pier’s inability to elastically resist the increased deflections and prevent movement large enough for the girder end to fall off the pier pedestals. This also resulted in a yielding failure of the pier reinforcement, severe cracking of the pier stem and a permanent set in the deflected position. This combination of distress to the pier made it unsalvageable.