17TH STREET BRIDGE
Canopy Failure Investigation

Atlanta, Georgia

Final Report
January 20, 2012
WJE No. 2011.3732.0

Prepared for:
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WJE ENGINEERS ARCHITECTS MATERIALS SCIENTISTS

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EXECUTIVE SUMMARY

At the request of the Georgia Department of Transportation (GDOT), Wiss, Janney, Elstner Associates, Inc. (WJE) has conducted an investigation of the partial collapse of the canopy structure on the 17th Street Bridge in Atlanta, Georgia. The collapse occurred on the evening of August 13, 2011 and affected approximately 190 feet of the 700 foot long canopy-fence structure along the south parapet of the bridge.

The supporting frames for the canopy-fence were comprised of built-up steel column and cantilever assemblies, attached to the outside face of the south concrete parapet using post-installed adhesive anchors. During the investigation, anchors within the collapse area were found to have pulled out from the concrete parapet. Vacated anchor holes were visually inspected, and epoxy samples from inside the holes were collected for materials testing. Epoxy was observed to vary in color and consistency at different locations, and at some locations, uncured and wet adhesive components were found. In addition to the epoxy samples, concrete core samples were taken from the south parapet for materials testing.

During the investigation, anchors away from the collapse area were observed to have measurable withdrawal from the parapet, indicating that long-term creep of the epoxy was a factor in the failure. In-situ load tests consisting of direct tension tests of existing anchor rods were performed on five anchors away from the collapse area in order to evaluate the performance of the adhesive anchors under short term tensile loading. Maximum applied loads ranged from 10,436 to 21,094 pounds, significantly exceeding the design tension service load of 4,000 pounds.

Structural analysis was performed to estimate the actual demands on the anchors during service. Due to the eccentricity of the cantilever arm supporting the overhead canopy relative to the anchor group, the adhesive anchors connecting the canopy support frames to the south parapet were subjected to sustained tensile loading. The sustained in-service tensile load at a typical lower anchor was calculated to be 1,053 pounds.

Based on the investigation, it was concluded that the canopy failure was the result of material and installation deficiencies. The primary cause of the collapse was the use of an epoxy anchor adhesive with poor resistance to long-term creep under sustained tensile loading. Other factors contributing to the collapse included disproportionate and incomplete mixing of the adhesive components and incomplete encapsulation of the anchors in the epoxy as indicated by voids and air pockets. Over time, the anchors connecting the supporting frames of the canopy to the south parapet of the bridge withdrew as the epoxy adhesive deformed and failed under a sustained tensile load that was substantially lower than the design service load.

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INTRODUCTION

At the request of the Georgia Department of Transportation (GDOT), Wiss, Janney, Elstner Associates, Inc. (WJE) has completed an investigation of the partial collapse of the canopy structure on the 17th Street Bridge in Atlanta, Georgia. The purpose of the investigation was to determine the probable cause(s) and mechanism of the collapse. The scope of the investigation included document review, field investigation, materials sampling and testing, and structural analysis. This report contains the findings of the investigation, a discussion of the findings, and conclusions regarding the cause(s) and mechanism of the collapse.

BACKGROUND

The 17th Street Bridge was opened for service in 2004, connecting Midtown Atlanta and the Atlantic Station development by spanning over Interstate Highway 75/85 in Atlanta, Georgia. The bridge carries 6 lanes of traffic and features 30 and 22 foot wide pedestrian sidewalks on the north and south sides of the bridge, respectively. High architectural fencing on the bridge parapets protects interstate traffic from debris and loose objects from the bridge deck and sidewalks. An architectural sun-shade canopy is integral to the fence along the southern parapet.

At 11:20 PM on August 13, 2011 approximately 190 feet of the 700 foot long canopy-fence structure detached from the south parapet wall and fell onto the roadway below. There were no injuries reported and no known damaged vehicles. Daytime temperatures on August 13 had reached 94 degrees Fahrenheit, but there had been no significant winds or precipitation reported. Immediately following the failure, GDOT removed the collapsed portion of the canopy-fence from the roadway and removed the remaining canopy-fence structure along the south side of the bridge.

DESCRIPTION OF THE STRUCTURE

The 17th Street Bridge is 830 feet long, 137 feet wide, and spans over the 20 traffic lanes that comprise the Interstate Highway 75/85 “Downtown Connector” in Atlanta. The structure consists of a series of three-span trapezoidal, variable depth, steel box girders with a cast-in-place reinforced concrete deck. The Engineer of Record (EOR) for the bridge was URS Corporation (URS), and the general contractor for the construction of the bridge was C.W. Matthews Contracting Co., Inc. (C.W. Matthews). Construction of the bridge was completed in 2004.

Along the south side of the bridge is a 22 foot wide pedestrian walkway which is sheltered by a canopy-fence structure that is attached to the south parapet wall. The parapet wall is approximately 16 inches thick and is constructed of reinforced concrete. The structural support frames of the canopy-fence are comprised of built-up column and cantilever beam assemblies of galvanized steel, spaced at 10 feet, 6 inches on-center. A typical detail of a column-cantilever assembly is shown in Figure 1. The architectural shade canopies which span between the canopy frames consist of stainless steel plates, stainless steel 2-1/2 inch by 2-1/2 inch tube shapes, and perforated 14 gage stainless steel sheets. Architectural

components are attached to structural components with stainless steel bolts. Structural frames are attached to the south face of the parapet wall using two pairs of post-installed epoxy-grouted anchor rods.

In total, 64 column-cantilever support frames were included as part of the south canopy-fence structure. Five additional column assemblies (two on the east and three on the west end of the bridge) without cantilever arms were also included. In order to achieve an undulating appearance for the overhead canopy, the pitch of the cantilever arms of adjacent support frames was varied. Four distinct frame geometries, designated as Types A, B, C, and D were used. These geometries are presented in Figures 2 and 3. Between the various frame types, only the pitch of the cantilever arms and the position of the architectural canopy components along the lengths of the arms were varied. The overall size and geometry of the individual structural elements were the same for each configuration.

At the time of the failure, a total of 19 canopy support frames near the east end of the bridge detached from the south parapet, with the majority falling onto the interstate roadway below. The remaining south canopy structure was immediately removed from the bridge by GDOT due to safety concerns. A total of 74 galvanized steel column assemblies, similar to those used on the south canopy structure, are used to support the fencing along the north parapet of the bridge. The north column assemblies were not removed from the structure following the canopy failure.

DOCUMENT REVIEW

Among the documents made available to WJE for review during the course of the investigation, the following contained information which may be pertinent to the collapse:

- Canopy and parapet fencing design drawings by URS, sheets 67 through 72, dated September 2001. Sheets titled Miscellaneous Structure Details (1 of 6) through (6 of 6). Sheets signed and sealed by Georgia registered Professional Engineer Steven L. Stroh.
- Fabrication shop drawings of shade canopy and parapet fencing by Bowers Fabrication Service, Inc. (Bowers), sheets E-01 and S-01 through S-17, dated October 4, 2002, bearing stamps by URS and GDOT.
- GDOT Specification Section 643, *Fence, Ornamental, Special Design* (4 pages), revised November 2, 2001.
- GDOT Standard Specification Section 886, *Epoxy Resin Adhesives* (3 pages)
- GDOT Qualified Products List 15 Epoxy Resin Adhesives (6 pages), revised April 18, 2003.
- GDOT Qualified Products List 15 Epoxy Resin Adhesives (5 pages), revised February 24, 2011.

The review of available documents focused primarily on those related to the elements comprising the canopy fence and the connections to the main bridge structure. A review of the design of the bridge itself was not performed as part of this investigation. The Bowers fabrication drawings were used to determine the sizes and weights of the various components of the built-up structural column and cantilever assemblies and architectural framing and screening. The URS design drawings were used to determine the variations in assembly geometry between canopy support frame types A through D and for design information related to the anchors used to connect the canopy-fence to the bridge parapet.

The available design and fabrication documents contain relatively little information on the anchors used to connect the canopy-fence to the bridge. Section G-G on Bridge Sheet 71 of 86 of the URS design drawings is shown in Figure 4. This detail specifies that four 7/8 inch diameter epoxy adhesive anchors are to be used to connect each support frame to the parapet. In a special note below the detail, it is stated

that each adhesive anchor shall have sufficient embedment to develop a tension service load of 4 kips (4,000 pounds). The hole diameter and embedment depth are not specified. In addition, no specifications for anchor material or adhesive system are given. Similarly, GDOT Specification Section 643.12 for the Installation and Workmanship related to the fence states that "Anchoring devices shall be fabricated and spaced to provide adequate support for the intended use." No reference to the adhesive anchors is made in the Bowers fabrication drawings.

The GDOT Qualified Products List 15 (QPL 15), revised April 18, 2003, contains a list of epoxy resin adhesives evaluated and approved by the GDOT Office of Materials and Research to meet the requirements of the Standard Specification and satisfy the requirements of SOP-17, "Acceptance of Miscellaneous Construction Items." Approved adhesives are grouped and categorized by type of use. Type VIII materials include epoxy adhesives used for anchors and dowel bar implants. A list of Type VIII adhesives from QPL 15 is presented in Table 1 below.

Table 1. Epoxy Resin Adhesives in GDOT QPL 15 (revised April 18, 2003)

Manufacturer	Trade Name	Mix Ratio (A:B)
Five Star Products, Inc.	RS Anchor Gel	1:1
Futura Companies, Ltd.	Futura Bond 566 R	1:1
ITW Ramset/Red Head	Epcon C6	Cartridge
Prime Resins, Inc.	Prime Rez 1100 High Mod LV	2:1
	Speed Bond #1	1:1
Sika Corporation	Sikadur DOT-SP3	1:1
Superior Epoxies	Dowel Bar Adhesive - 5	Cartridge or 1:1
Symons Corporation	Res-Con 304	Cartridge
Tamms Industries	Duralcrete Fast Set Epoxy Gel	Cartridge or 1:1
Unitex	Pro-Poxy 300 Fast	Cartridge
U.S. Anchor Corp.	Ultrabond 1300	1:1
Weg-It Fastening Systems	Inject-Tite Fast Set	Cartridge
W.R. Meadows of Georgia	Rezi-Weld Gel Paste	Cartridge

FIELD INVESTIGATION

WJE conducted an on-site investigation of the failed canopy structure at the bridge from Wednesday, August 17 through Saturday, August 27, 2011. The field investigation included a visual examination of the concrete deck, parapet, anchor holes, and canopy framing, the extraction of material samples for subsequent laboratory study, and in-situ load testing of anchor rods.

Supporting frames of the canopy-fence and their locations were numbered sequentially from west to east (1 through 69), as shown on the Bowers shop drawings (Figure 5). Anchor rods at each location were assigned identification letters by WJE according to their relative position. Rods were lettered 'A' through 'D' in a clockwise orientation (as viewed looking north at the south face of the parapet) starting with 'A' as the upper left rod, as shown in Figure 6. Throughout this report, the nomenclature of using 1 through 69 to identify the location of the element being discussed and A through D to identify the anchor or anchor hole will be used.

Visual Inspection

When WJE arrived at the site, the failed portion of the canopy fence (canopy support frames 49 through 67) had been moved to the shoulders of the interstate. Twelve anchor rods from the failed portion of the

canopy fence had been removed from the debris by GDOT and were made available to WJE. An additional anchor rod was recovered from news channel WXIA. The original location of these individual anchors was unknown. Anchor rods were observed to be 7/8 inches in diameter, 11 inches long, and partially covered in epoxy. Some of the anchor rods were all-thread rods, and others consisted of 6 inches of threaded rod with 5 inches of smooth unthreaded bar. Unthreaded sections of rods were typically observed to be relatively clean and free of epoxy.

Prior to WJE's investigation, the elements of the canopy-fence that did not detach from the bridge during the canopy failure (column and canopy support frames 1 through 48, 68 and 69) were removed from the bridge and transported to an off-site GDOT storage yard. These elements of the canopy-fence were removed from the south parapet by cutting the embedded anchors using a torch. Torch marks on the remains of the existing anchor rods and on the surface of the parapet left a clear indication of the extent of the previously attached portion of the canopy-fence. Many of the torch-cut rods were observed to have partially withdrawn from the wall (Figure 7). The lower anchor rods typically exhibited more withdrawal (pull-out) than the upper rods. Of the 168 torch-cut rods inspected, 112 were observed to have measurable withdrawal measuring from approximately 1/16 inch up to a maximum withdrawal of 1-3/4 inches. Many of the anchor rods were observed to have been torch cut very close to the parapet surface. These locations were typically characterized by heavy scorching on the surface of the concrete, as shown in Figure 8. Due to the geometry of the column assemblies, it would have been difficult to make such a cut unless the column had already rotated away from the parapet face due to rod pull-out, allowing the torch to be passed between the column flange and parapet to make the cut.

Inspection of Vacated Anchor Holes

In the area of the failure, the anchor rods were observed to have completely pulled out from the parapet wall, with the exception of the lower two rods at Location 67 (rods 67C and 67D) which remained in place but had failed in shear (Figure 9). The vacated anchor holes were inspected and documented using a Hawkeye optical borescope and an Olympus videoscope. The average depth of anchor embedment was measured to be 8 inches. Epoxy residue was observed in each of the vacated holes in the collapse area and generally appeared to be bonded to the substrate concrete surface of the hole. Voids or air pockets were frequently observed in the epoxy matrix and almost always observed in the back 1.5 inches of the hole. Epoxy material was observed to vary in color from opaque dark gray, to opaque light gray, to translucent brown. In three locations, wet epoxy components were extracted from the back of the holes using a small spatula. A photograph of a sample of wet epoxy extracted from hole 56B is shown in Figure 10. Opaque gray and translucent brown components were extracted from different locations.

A photographic summary of the inspected anchor holes is provided in Appendix A. Included in the summary are overall photographs of the exteriors of the holes, showing features such as concrete spalls and excess epoxy around the holes, and individual captures of the interiors of the holes taken from the videoscope inspection. Many of these images focus on notable characteristics such as voids or air pockets in the epoxy and variations in epoxy color and consistency. Appendix B contains a disk with video files recorded during the vacated anchor hole inspections.

Bridge Deck Survey

A visual assessment of the bridge deck in the area of the failure identified a number of distress marks and scrapes on the south sidewalk indicating that the cantilever support frames impacted the sidewalk during the failure. Examples of this distress are shown in Figures 11 through 13. As shown in the photographs, the distress typically consisted of one or more impact marks near the north edge of the sidewalk with a

scrape mark extending towards the anchorage at the south parapet. Distress marks were also noted at the steel pedestrian perch rail adjacent to canopy support frames 63 and 64, as shown in Figure 14. A land surveyor from Boundary Zone, Inc. was contracted to survey the locations and extents of the distress on the bridge deck. Figure 15 presents the survey data on a partial bridge plan. Survey data points are numbered corresponding to the support frame associated with the distress.

The dashed lines on the partial plan in Figure 15 connect the surveyed original centerlines of the support frame bolt groups to the initial points of impact on the sidewalk, indicating the general direction that the frames fell or rotated during the failure. The other distress marks give an indication of the direction of frame movement after the initial impact. As shown in the figure, there is a distinct change in direction between support frames 55 and 56. Canopy support frames 49 through 55 generally fell in the direction of the west end of the bridge, while frames 56 through 67 generally fell toward the east end of the bridge.

Below the partial plan is a set of diagrams for selected frames showing variations between the actual points of initial impact and predicted points of impact assuming pure rotation about the south corner of the parapet. The predicted points assume no lateral translation (north-south) or rotation (east-west). The variations between the actual and predicted points of impact suggest that lateral translation and rotation of the frames occurred during the failure. Of all frames within the failure area, frame 56 was found to have the closest actual and predicted points of impact, indicating that it experienced the smallest amount of combined lateral translation and rotation as it rotated towards the bridge deck. Moving east and west away from frame 56, the variations between the actual and predicted points of impact for adjacent frames increase.

Off-site Observations

Following the canopy failure, GDOT removed the various elements of the south canopy from the site and transferred them to an off-site storage yard. WJE visited the off-site storage yard to view the structural elements from the failure area on August 31, 2011. The 19 canopy support frames that detached near the east end of the bridge during the failure were transferred in their post-failure condition and were laid out individually in the storage yard for viewing. (The canopy shades and fencing were removed and piled separately. In addition, the support frames west of the failure area were partially disassembled and piled in a separate area of the storage yard.)

Common types of distress observed on the canopy support frames from the failure area included:

- Bent and missing connection plates between the cantilever tube assemblies and stainless steel square tube canopy supports (Figures 16 through 18)
- Bent cantilever tube assemblies (Figure 19)
- Failed column connections (Figure 20)
- Bent/twisted plates in column assemblies (Figure 21)
- Bent anchor rods (Figure 22)
- Miscellaneous paint scrapes

No tearing, cracking, or distortion at anchor rod holes in the frames was observed.

Note that because many of the elements were removed from the roadway as part of the emergency response immediately following the failure, some of the observed distress may have occurred during the initial clean-up effort.

Reports following the failure indicated that a portion of the canopy structure hit the adjacent high mast light post, immediately south of support frame location 52. Inspection of support frame 52 showed black paint marks, matching the paint of the high mast light post, on the south face of the column assembly, suggesting that the frame hit the light post as it fell from the bridge. Photographs of the paint marks are shown in Figures 23 and 24. In addition, yellow paint marks were observed on the bottoms of cantilever assemblies for the support frames that impacted the pedestrian perch rail in the failure area, as shown in Figure 25.

In addition to the distress that occurred at the time of the failure, WJE noted several holes for anchor rods in support frames that had been modified. The modifications included widening and slotting the holes as shown in Figure 26. It is likely that these field modifications were made to address problems with fit-up and alignment during installation of the canopy elements. Also, the frame that was installed at location 51 was modified with two welded plates on the west side of the inner flange to accommodate eccentric anchor placement as shown in Figure 27. The eccentric anchor placement was likely necessary due to the frame's close proximity to a joint in the parapet. As shown in the photograph, the welded plates were bent inward following the canopy failure, suggesting that the frame rotated to the west during the failure.

At the storage yard WJE identified 33 anchor rods located with the elements from the collapse area. Some of these anchor rods were in place in the holes in the frame with nuts and washers attached. Others were located on the ground scattered about the frames.

Materials Sampling

As noted above, small amounts of wet epoxy material were removed from vacated anchor rod holes in the failure area. Samples of this material were collected in glass vials for laboratory analysis.

WJE utilized local contractor Penhall to remove concrete samples from the south parapet wall for laboratory examination. A photograph of the coring in progress is shown in Figure 28. Samples consisted of 4 inch diameter cores. Six cores were removed from the south face of the parapet (designated 40D, 50D, 55C, 55D, 56C and 56D), and four were removed from the north face (designated P54, P57, P59, P62). Of the six cores removed from the south face, five were removed directly over vacated anchor hole locations within the failure area to a depth of 12 inches. During the retrieval of core 56C, the core broke off near the back of the anchor rod hole at a depth of approximately 8 inches. At this location, the back of the original anchor rod hole was exposed, and the ring from the core barrel that was used during the original installation could be clearly seen (Figure 29). The cored hole was measured to be approximately 1-1/8 inches in diameter. The last core removed from the south face of the parapet (40D) was located over an existing torch-cut anchor rod west of the failure area. The cores removed from the north face of the parapet were taken to a depth of 10 inches at intervals of approximately 20 feet throughout the failure area. Cores were removed in general accordance with ASTM C42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. Material samples were transported to WJE's Janney Technical Center in Northbrook, Illinois or the NOVA Engineering and Environmental, Inc. (NOVA) test lab in Buford, Georgia for evaluation.

In-Situ Load Testing

Five existing anchor rods away from the failure area were selected for in-situ load testing. Rods were selected for testing which showed no evidence of previous withdrawal and which had sufficient exposed threaded length to attach the load testing equipment. Direct tension testing was performed in general accordance with ASTM E488-96, *Standard Test Methods for Strength of Anchors in Concrete and*

Masonry Elements by ASTM. Photographs of the load testing equipment are shown in Figures 30 and 31. Only loads, and not anchor displacements, were measured during the testing. While the intent was to load all tested anchors to failure, only a single tested rod failed below the capacity of the test equipment. This anchor is shown in Figure 32. As can be seen in the figure, the failure occurred within the epoxy and at the interface between the epoxy and surrounding concrete and not in the rod itself or the bulk concrete material. The remaining rods were tested to a load near the capacity of the test equipment, between 15,000 and 20,000 pounds. While a small amount of withdrawal occurred for each of these rods, the rods continued to resist additional load after displacing. An informal creep test was performed at a single anchor (33A) which was loaded to approximately 4,000 pounds for a period of one hour. The anchor was permitted to displace during the test, gradually reducing the load applied by the hydraulic ram. After one hour, the applied load had reduced to approximately 2,800 pounds. No leaks were detected in the hydraulic system of the test apparatus.

The anchor locations tested with the respective maximum applied loads are listed in Table 2 below.

Table 2. Results of In-Situ Load Testing

Anchor Number	Maximum Applied Load (lbs)
13A	10,436†
33A	17,634‡
33B	21,094‡
34A	20,374‡
41A	15,619‡

† Anchor tested to failure.
‡ Load test stopped prior to failure.

LABORATORY INVESTIGATION

The following material samples recovered from the 17th Street Bridge during the field investigation were sent to WJE's Janney Technical Center in Northbrook, Illinois for testing and evaluation:

- Wet epoxy material taken from anchor holes 55C, 55D, and 56B.
- Hardened epoxy material recovered from various anchor holes within the failure area.
- Concrete cores from south face of parapet containing anchor holes 50D, 55C, 55D, 56C, and 56D.
- Concrete core from south face of parapet containing anchor rod 40D.
- Concrete core from the north face of the parapet containing bulk concrete from a location near canopy frame 59 (core P59).
- Thirteen selected 7/8 inch diameter anchor rods from the debris pile (original location unknown).
- One anchor rod extracted from the parapet during in-situ load testing at location 13A.

In addition to these selected material samples, three concrete cores taken from the north face of the parapet were delivered to NOVA for compressive strength testing. The cores contained only bulk concrete and were extracted near canopy frames 54, 57, and 62 (cores P54, P57, and P62 respectively).

Laboratory Observations of Concrete Material

Petrographic Studies

Petrographic studies of Core P59 were conducted to characterize the concrete substrate. The studies were conducted in accordance with the methods and procedures outlined in ASTM C 856, *Standard Practice*

for Petrographic Examination of Hardened Concrete. The core was visually examined, photographed, and then cut in half longitudinally. One of the resulting halves was lapped using successively finer grits to achieve a fine, matte finish suitable for examination at stereomicroscope magnifications ranging from approximately 5 to 50X. A thin section was prepared from a block cut from the outer 2 inches of the core and then examined at magnifications of up to 500X using a petrographic (polarized-light) microscope.

The concrete was well consolidated and consisted of crushed siliceous coarse aggregate and fine aggregate that were fairly uniformly dispersed in an air-entrained portland cement paste. Total air content was estimated at 5 to 6 percent. The aggregate was generally well graded. The top size was 3/4 inch. Paste characteristics were consistent with a moderately low water-cement ratio, estimated at 0.45 +/- 0.02. Paste in the outer 0.1 inch of the concrete was carbonated, consistent with the reported age of the concrete and the estimated water-cement ratio. The concrete did not contain significant cracks and did not exhibit evidence of distress. Inconsequential amounts of secondary ettringite were observed in the smallest air voids in the concrete.

Paste-aggregate bond was judged to be moderately weak, based on fracture characteristics observed in the laboratory and fractures produced in the field during removal of the cores. Crushed coarse aggregate usually results in tight paste-aggregate bond. The aggregate appeared to be clean, and no explanation for the moderately weak bond is apparent. Weak paste-aggregate bond could contribute to lower than expected tensile strength of concrete. The crushed coarse aggregate consists of metamorphic rock types that contain optically strained quartz, which is susceptible to alkali-silica reaction (ASR). ASR gel reaction product was not observed, and evidence of incipient ASR reaction also was not observed in the thin section. Core P59 and several of the other cores exhibited dark rims around some coarse aggregate particles. The rims disappeared after several days of air drying in the laboratory suggesting that they were caused by expulsion of water that had entered the concrete at paste-aggregate boundaries during coring.

Compressive Strength Testing

Compressive strength testing of concrete cores by NOVA was performed in general accordance with ASTM C39, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. Results of the testing are summarized in Table 3 below.

Table 3. Results of Compressive Strength Testing by NOVA

Core Number	Compressive Strength (pounds per square inch)	Fracture Type (per ASTM C39)
P54	6,190	3
P57	6,800	4
P62	7,730	4
Average	**6,787**	

Laboratory Observations of Adhesive Material

Samples were examined visually, and selected samples were examined microscopically and analyzed using Fourier transform infrared spectroscopy, scanning electron microscopy with energy dispersive x-ray spectroscopy, differential scanning calorimetry, and thermal gravimetric analysis. Selected samples were also analyzed for nitrogen content.

Spectroscopical Analyses

Several samples of adhesive, including material of different color and texture, were analyzed using Fourier transform infrared spectroscopy with an attenuated total reflectance attachment (FTIR-ATR). The analyses were conducted to determine the composition of the adhesive and to evaluate composition differences in adhesive with different color or texture. FTIR-ATR analysis indicates that the adhesive is an epoxy with calcium carbonate and amorphous silica fillers. The epoxy spectrum is consistent with a Bisphenol A/glycidyl ether-based epoxy, which forms the basis of most epoxy resins.

Thermal Analysis

Selected samples of adhesive were analyzed using differential scanning calorimetry (DSC) and thermal gravimetric analysis (TGA). Samples were selected based on their dramatically different visual appearances as compared to each other. The selected samples were as follows: (1) gray adhesive from 40D, selected to serve as a control, unfailed adhesive; (2) gray pliable adhesive from 55C, selected as a candidate poorly proportioned material; (3) amber transparent adhesive from 55D, selected as a strong candidate for poorly proportioned material; and (4) gray, rough adhesive from 56C, selected because of the rough texture, but similar color to the gray adhesive from 40D. Results from the TGA testing are provided in Table 4.

Table 4. Results of TGA testing.

Sample	Description	Polymer content, percent	Calcium carbonate content, percent	Other filler content, percent
40D	Gray, non-failed adhesive	54.5	35.4	9.9
55C	Darker gray, pliable adhesive	32.8	58.0	10.3
55D	Amber, transparent adhesive	86.7	9.9	3.5
56C	Gray, rough textured adhesive	53.8	37.7	8.6

The data obtained by differential scanning calorimetry showed substantially different curves, further indicating that the materials are of different composition. Similar materials are expected to produce similar curves in DSC analysis. Therefore, the differences in the curves indicate differences in composition.

Nitrogen Content Analysis

A portion of the same samples selected for thermal analysis were analyzed for nitrogen content. It is very likely that the adhesives are amine-cured epoxies, and therefore, the hardener component is expected to have significantly greater nitrogen content (from the amine functional groups) than the resin component. By correlating the nitrogen content with the filler content determined by TGA, the degree of hardener versus resin, and therefore, proportion, can be deduced. The nitrogen content of each sample, calculated as an average of duplicate readings, is presented in Table 5. The nitrogen content by weight of polymer is also calculated by taking the nitrogen content and relating it to the polymer content found by thermal analysis.

Based on these analyses, the amber portion of the adhesive sample has a greater nitrogen content compared to the gray adhesive, and may represent hardener-rich areas. The darker gray portion has a lower nitrogen content compared to the gray adhesive, and may represent resin-rich areas.

Table 5. Nitrogen content analysis.

Sample	Description	Nitrogen content of sample, percent	Nitrogen content of polymer, percent
40D	Gray, non-failed adhesive	3.64	2.04
55C	Darker gray, pliable adhesive	2.62	0.86
55D	Amber, transparent adhesive	6.70	5.81
56C	Gray, rough textured adhesive	3.43	1.85

STRUCTURAL ANALYSIS

Due to the eccentricity of the cantilever arm supporting the overhead canopy relative to the anchor group, the adhesive anchors connecting the canopy support frames to the south parapet of the bridge were subjected to sustained tensile loading when subjected to gravity loads alone. In order to estimate the magnitude of the gravity-induced forces applied to these anchors, a structural analysis was performed. This analysis consisted of determining the self-weight and tributary loads of a typical column-cantilever support frame, calculating the center of gravity of the frame, and calculating the reactions at the anchors due to the eccentric dead loads.

The combined self-weight of the primary structural elements of a typical frame, including the built-up column assembly and cantilever tube assembly, was determined to be approximately 2,075 pounds. In addition to this self-weight, the weight of the shade canopies (including the stainless steel tube supports) tributary to a single assembly was determined to be approximately 550 pounds. The center of gravity of the typical Type A frame, including the tributary shade canopy, is located approximately 2 feet, 6 inches inboard of the outside face of the parapet. Assuming a 2,625 pound downward load applied 2 feet, 6 inches from the point of support, the sustained tensile force at each of the two bottom anchors was calculated to be 1,053 pounds. This exercise was repeated to determine how the tensile load on the bottom anchors increases in response to a rotation of the overall support frame assuming a partial withdrawal of the bottom anchors. The results of this analysis are presented in Table 6 below.

Table 6. Increase of tensile reaction due to anchor pull-out.

Service load, P (lbs)	Anchor pull-out (in)	Tensile reaction at bottom anchor, T_{bot} (lbs)
2,625	0	1,053
2,625	0.5	1,107
2,625	1.0	1,161
2,625	1.5	1,216
2,625	2.0	1,268
2,625	3.0	1,376
2,625	4.0	1,483

Wind and other lateral loads on the frames were not considered in the analysis because no significant wind was reported on the night of the collapse.

DISCUSSION

Field and laboratory observations indicate that the failure of the canopy was not related to a deficiency in the concrete parapet or structural steel. Relatively little spalling of the concrete was observed at the

vacated anchor rod holes in the collapse area. Concrete compressive strength was sufficient for this application. While petrographic studies found moderately weak cement paste-aggregate bond, no other deleterious qualities of the concrete were observed. In the absence of a significant number of anchor rod spalls, WJE considers the weak paste-aggregate bond incidental. Structural steel frames had relatively little distortion and no cracking, tearing, or distortion of any kind at anchor rod holes on the base plates. Although only 48 of the 76 anchor rods involved in the failure were available to WJE for inspection, no fractured anchor rods were observed among those which pulled out from the parapet.

While it is unknown what brand of adhesive material was used to install the anchor rods at the 17th Street Bridge, some observations can be made using properties of common construction epoxies and common industry practices.

Anchor rods at the 17th Street Bridge were installed using a 1-1/8 inch diameter cored hole. While some epoxy manufacturers permit the use of either cored or drilled holes, hole diameter is commonly restricted to be from 1/16 inch to 1/8 inch larger than the rod diameter. Cored holes at the 17th Street Bridge were nearly 1/4 inch larger than the embedded rods. The effect of the oversized holes cannot be quantified without testing, but elastic and creep deflections will increase in proportion to epoxy thickness. In addition, it can be expected that the oversize holes would be problematic to fill and would decrease tensile load capacity of the anchor. WJE observed no evidence of oil, grease, debris or any other contaminant in the cored holes examined in the laboratory.

A number of the anchor rods were observed to be unthreaded for the majority of their embedment. Unthreaded portions of these rods were typically found to be relatively clean and free of epoxy. While most construction epoxy manufacturers have no provisions for their products when used with unthreaded rods, it is not known what was permitted by the manufacturer of the material used on this project. In general, an unthreaded rod can be expected to provide lower bond capacities than threaded rods of the same diameter.

The presence of air voids in the epoxy matrix indicates that the holes were incompletely filled with epoxy when the rods were inserted. Almost all anchor rod holes in the collapse area exhibited a void between 1 and 1.5 inches in length at the back of the hole which reduces the effective embedded length of the anchors. Air voids are known to cause a reduction in bond area and a corresponding reduction in tensile load capacity.

The presence of wet epoxy material in the holes 7 years after placement and the variety in observed epoxy colorations indicate that the epoxy components were not thoroughly mixed or entirely unmixed in some locations. This conclusion was verified by laboratory analysis. Improperly mixed or unmixed epoxy results in substantially diminished load capacities. In addition, the laboratory testing indicates that there were variations in the proportioning of the epoxy components.

Despite the variations in as-built conditions related to hole diameter, anchor rod material, and the mixing and installation of the epoxy materials, it is possible that the anchors were capable of sustaining (in the short term) the 4,000 pound tensile service load force required by the design. Many epoxy manufacturers would predict an expected short-term tensile failure load of around 40,000 pounds for a properly installed anchor of 7/8 inch diameter and 8 inch embedment. While the variations in as-built conditions may have resulted in substantial reductions in the ultimate capacity of the anchors, many of the anchors may have been sufficient to each sustain a short-term 4,000 pound service load in their as-installed condition. Indeed, when WJE performed in-situ load testing of anchors away from the collapse area, all of the

anchors sustained tensile loads well in excess of 4,000 pounds. It should be noted here that none of the anchors tested were loaded into the range of this 40,000 pound expected failure load, although one anchor did fail at a load of approximately 10,000 pounds and this failure was an anchor pull-out with the adhesive failing within the adhesive material.

Structural analysis by WJE showed that the long-term service load acting on the anchor rods was nearer to 1,000 pounds per anchor rod than to 4,000 pounds, even assuming some initial displacement of the rod. An analysis using code recommended short-term wind loadings indicates the design specification of 4,000 pounds of tensile capacity is sufficient to encompass the additional demands from wind. No shear capacity requirement for the anchors was specified in the design. Because the failure mechanism was one of tensile withdrawal of the anchor rods (and not of shear) and because the shear capacity of the installed adhesive system was likely sufficient for the applied shear loads, the omission of a shear specification for the anchors has no relevance to the collapse.

Displacement of a structural element when subjected to sustained, constant loading over a long period of time is a phenomenon known as "creep". In an investigation[1] of the July 10, 2006 collapse of a portion of the ceiling of the Interstate 90 tunnel in Boston, Massachusetts, the National Transportation Safety Board (NTSB) concluded that some formulations of construction epoxies are highly susceptible to creep and will creep to failure if appropriately loaded. Further, the report found that "the maximum load capacity of an adhesive anchor, which relates to short-term loading, does not indicate that the anchor will be able to support even lighter loads over time, and thus a larger design safety factor cannot compensate for an adhesive material that is susceptible to creep."[2]

While it is unknown what brand of adhesive material was used to install the anchor rods at the 17th Street Bridge, some of the formulations listed on the 2003 GDOT QPL 15 have been shown to be susceptible to creep. Laboratory work by WJE was not able to determine the manufacturer of the adhesive but was able to identify that the adhesive could be one of the epoxy formulations on the list. WJE performed an in-situ informal creep test on a single anchor away from the failure area of the bridge. Although the testing was not performed in rigorous adherence to a recognized national standard creep test, it did indicate that the adhesive anchors were susceptible to creep. In addition, the failure mode observed in creep test specimens developed by the NTSB (failure within the adhesive material and not at the bond interface) matches the failure mode observed at most anchor rods from the collapse portion of the 17th Street Bridge. Taken together, these factors suggest that the adhesive installed at the 17th Street Bridge was creep susceptible and that a creep-to-failure occurred in at least some locations. In accordance with the NTSB findings, a maximum short-term load capacity of 4,000 pounds (or greater) does not indicate that the anchors would support a sustained load of 1,000 pounds over time if a creep susceptible adhesive was used.

The tendency of adhesive anchors to creep, and the rate of creep are influenced by a number of factors including the magnitude of the sustained load, the presence of voids in the adhesive, and the service temperature. In the case of the 17th Street Bridge, the magnitude of the sustained load was relatively low, but voids were prevalent and services temperatures were elevated, particularly during the weeks before the collapse. Different epoxies have different thermal performance, but all epoxies transition to a softer, less creep resistant material at high temperatures. The failed canopy anchorage was located on the south face of the bridge parapet where it would be exposed to direct sunlight during all hours of the day and all

[1] National Transportation Safety Board, *Ceiling Collapse in the Interstate 90 Connector Tunnel, Boston, Massachusetts, July 10, 2006, Highway Accident Report NTSB/HAR-07/02*, (Washington, DC: 2007)
[2] Ibid., 106

seasons of the year. The summer of 2011 was particularly hot in Atlanta and concrete temperatures could have easily exceeded 100 degrees Fahrenheit for several hours a day during much of this period, a temperature which approaches the transition temperature for many epoxies. Creep rates in the adhesive would have been elevated during each period of high temperature. Thus, the timing of the collapse (in regards to season of the year) was likely influenced by its recent thermal history.

Possible Collapse Sequence

Prior to the night of the failure, evidence shows that many of the canopy support frames within the failure area had experienced significant inward rotation over time as the lower anchors, subjected to sustained tensile loading, underwent creep, and partially withdrew from the parapet. This is supported by the high number of torch-cut anchor rods west of the failure area observed with partial withdrawal. Because of the undulating nature of the canopy and the limited visual access to the support anchorages on the south face of the parapet, these rotations would have been difficult to detect visually while the canopy structure was in place.

The bridge deck survey data collected during the failure investigation suggests that at the time of the failure, the lower anchors of frames 55 and 56 were the first to fully release. At these locations, the epoxy materials in the bottom of the anchor holes were found to not be thoroughly mixed. In addition, measurement made after the collapse reveal relatively small amount of lateral translation were experienced by frames 55 and 56 prior to impact.

The initial points of impact for frames 55 and 56 were measured to be approximately 15 feet, 7 inches apart in plan. This is considerably greater than the initial 10 foot, 6 inch spacing between the frames, suggesting that elements of the shade canopy connecting the two frames (such as the stainless steel bolts used to connect the square tubes spanning between the frames) fractured as the frames rotated towards the deck surface. The separation between frames 55 and 56 was likely related to the eccentric anchorage of frame 51. Unlike other frames, frame 51 is anchored to the west of the column centerline. Because of the eccentric anchorage, the cantilever arm has a tendency to twist to the west when loaded. As frames 55 and 56 rotated downward, the tendency of frame 51 to twist to the west provided sufficient resistance for frames 55 and 56 to separate from one another and fall/rotate in opposite directions. Frames to the west of the separation rotated with frame 51, and frames to the east rebounded slightly in the opposite direction.

As frames 55 and 56 rotated downward after the extraction of the bottom anchors, adjacent frames to the east and to the west were also pulled down towards the bridge deck. These adjacent frames had likely experienced long term rotation also, though possibly of a lesser degree than frames 55 and 56. As the frames rotated about the outside edge of the parapet, the top anchors were subjected to bending and increased tensile demands, causing them to bend and eventually withdraw from the parapet. No longer anchored to the bridge, the rotating frames also began to translate (or slide) to the south, back over the parapet. With the anchors lost and the column portions of the frames sloping down over the outside edge of the parapet, the only force resisting the southward translation was friction between the concrete bridge deck and parapet, and the frame itself. The amount of translation experienced prior to impact for a given frame was directly related to the proximity of that frame to the point of origin for the collapse. As shown in Figure 15, frame 56 experienced the lowest amount of lateral translation and rotation of all the frames within the failure area. Frame 55 also experienced a relatively low amount of lateral translation, although its westward rotation was increased due to the rotation of frame 51 as previously discussed. Moving west from frame 55 and east from frame 56, the lateral translations and rotations of individual frames steadily increase, suggesting that these frames were pulled to the south by the initiating frames as they fell towards

the sidewalk. Formed after the initial impact, the scrape marks on the sidewalk also indicate that frames 55 and 56 were the first to begin translating back over the parapet. All frames including, and west of, frame 55 translated in a south-southeast direction, while all frames including, and east of, frame 56 translated in a south-southwest direction. Based on the patterns of the scrape marks, the translations indicate that the frames were being pulled south, back over the parapet, by a force originating at frames 55 and 56. After sufficient outward translation, the cantilever ends of the frames struck the inside face of the parapet, and the frames rotated outward over the parapet and fell onto the roadway and median below.

On the west end, the canopy failure arrested at support frame 48. Post-incident measurements of the torch-cut anchors at this location indicated that each of the bottom anchors had withdrawn approximately 1-3/4 inches but did not fully release. Two primary factors are believed to be related to the arrest of the failure. These include the rate of loading and the quality of mixing of the epoxy adhesive. The in-situ load testing performed as part of the failure investigation showed that under short term loading the epoxy anchors were able to resist loads well in excess of the tensile loads predicted by the structural analysis, even after partial withdrawal from the parapet. With sufficient embedment remaining at the time of the failure, the anchors at frame 48 were able to resist the forces due to overturning, and the frame remained in place. Preceding the failure, the amount of remaining anchor embedment for a given frame was partially related to the quality of mixing of the anchor adhesive at that frame and the degree of rotation at adjacent frames.

As discussed previously, improper mixing of the adhesive components during installation increased the susceptibility of the adhesive to long-term creep effects and deformation under sustained tensile loading. Visual inspection and laboratory analysis showed that there was substantial variability in the quality of adhesive mixing for the anchors that failed within the failure area, and although the adhesive was not evaluated for the anchors west of the failure area, it is possible that the overall quality was better, resulting in higher resistance to long-term creep effects, lower levels of anchor withdrawal, and higher resistance to short term loading at the time of the failure. As the collapse progressed away from frames 55 and 56, anchors were subjected to more and more rapidly applied loads, thereby mobilizing their significant short-term load resistance to a greater and greater degree. Therefore the sequence of the collapse caused it to be inherently self-arresting. This finding is supported by the greater frequency of concrete spalling (indicating greater resistance offered to extraction) at vacated anchor rod holes at the extreme east and west ends of the collapse area. Indeed, at the extreme east end of the collapse area at canopy frame 67, the lower anchor rods did not fail by extraction from the parapet, but by shear of the rods. At this location the lower rods fractured and the upper rods withdrew from the parapet but the canopy frame remained on the bridge deck and did not fall to the interstate below the bridge.

CONCLUSIONS

The canopy failure at the 17th Street Bridge was the result of material and installation deficiencies. Listed in estimated order of importance, the factors contributing to the collapse are as follows:

1. Use of epoxy anchor adhesive with poor resistance to long-term creep under sustained loading.
2. Disproportionate and incomplete mixing of adhesive components.
3. Incomplete encapsulation of the anchors in the epoxy as indicated by voids and air pockets.

Over time, the anchors connecting the supporting frames of the canopy to the south parapet of the bridge withdrew as the epoxy adhesive deformed and failed under a sustained tensile load that was substantially lower than the design service load.

This finding is similar to the findings of the NTSB in regards to the July 10, 2006 collapse of a portion of the ceiling of the Interstate 90 tunnel in Boston, Massachusetts. As a result of the NTSB investigation, the design community became more conscious of the long term performance of adhesive anchors when subjected to sustained load. Because the design of the 17th Street Bridge was completed prior to the 2006 collapse in Boston, it is likely that the epoxy selected for anchorage of the 17th Street Bridge canopy was not evaluated by the epoxy manufacturer or specifier for sustained loads and susceptibility of the epoxy material to creep.

RECOMMENDATIONS

Based on the findings of this investigation, two recommendations are made:

1. If GDOT elects to re-install the canopy structure on the south parapet of the 17th Street Bridge, the use of a positive connection is recommended to secure the frames to the parapet wall.
2. If GDOT elects to leave the fence system on the north parapet in place, the existing adhesive anchors should be modified to use a positive connection to the parapet.

FIGURES

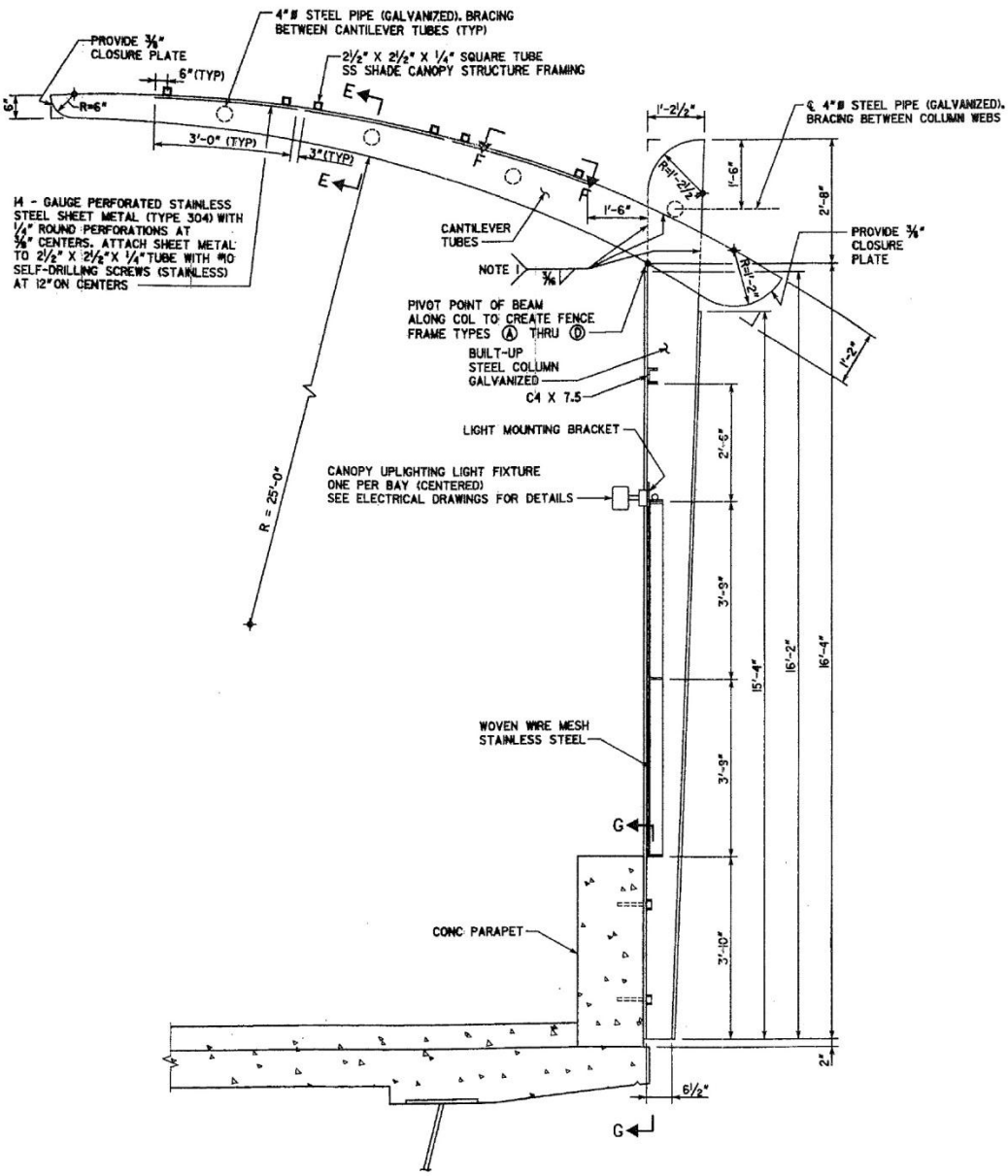


Figure 1. Typical column-cantilever assembly detail (from URS Sheet 69)

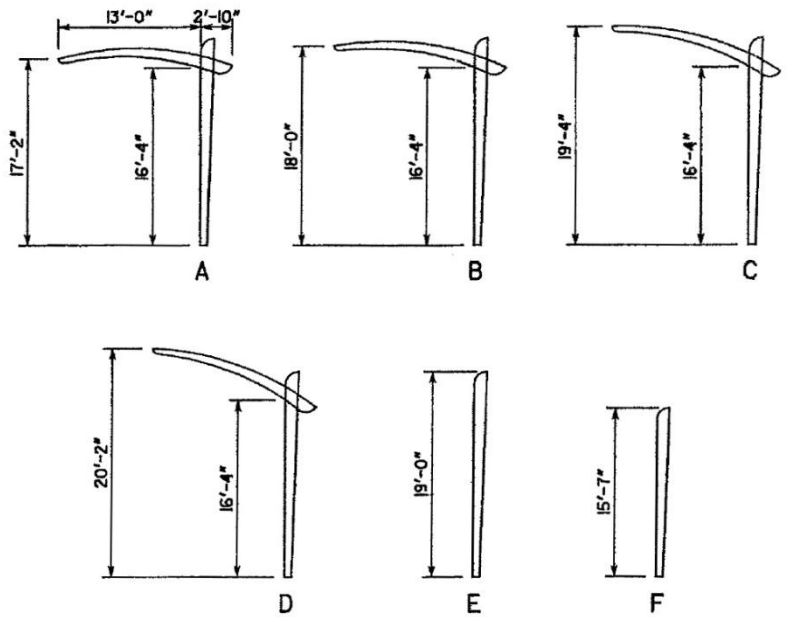


Figure 2. Geometries for various canopy support frame types (from URS Sheet 69)

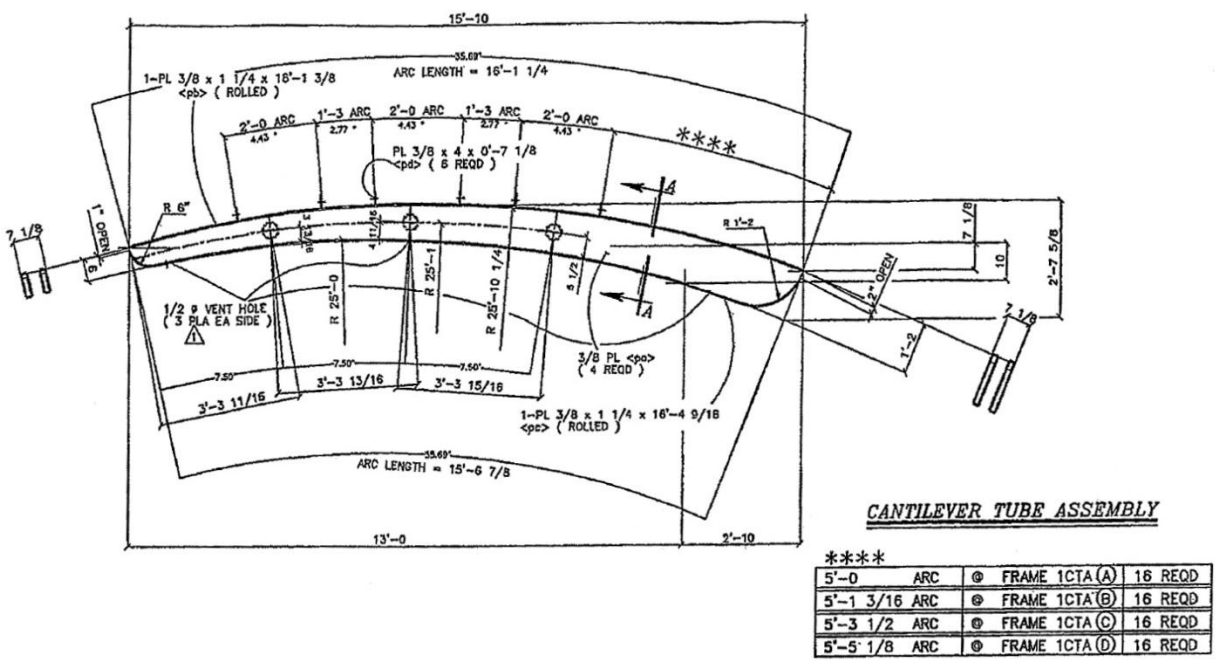


Figure 3. Position of architectural canopy components for various canopy support frame types (from Bowers Sheet S-01)

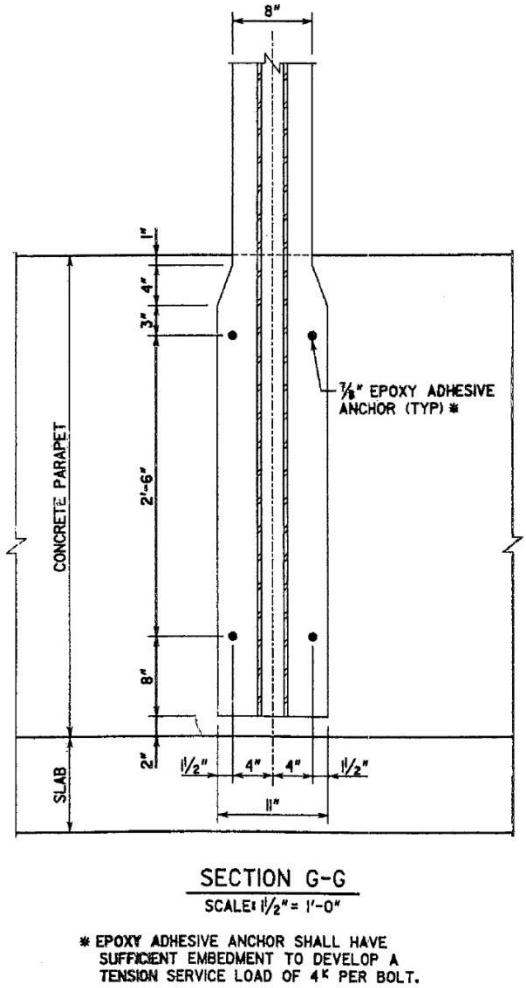


Figure 4. Anchor detail (from URS Sheet 71)

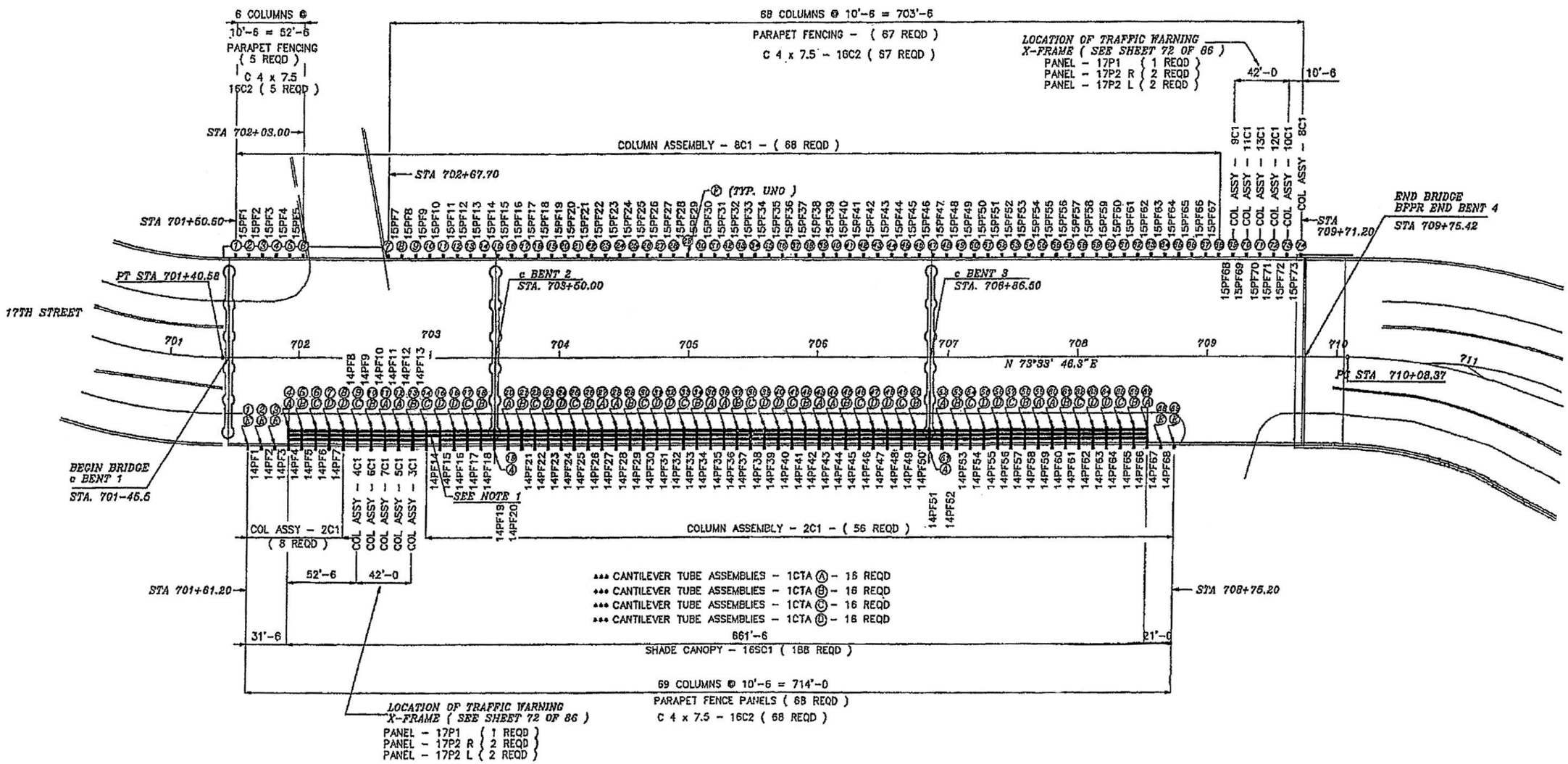


Figure 5. Bridge plan showing shade canopy support frame layout (from Bowers Sheet E-01)

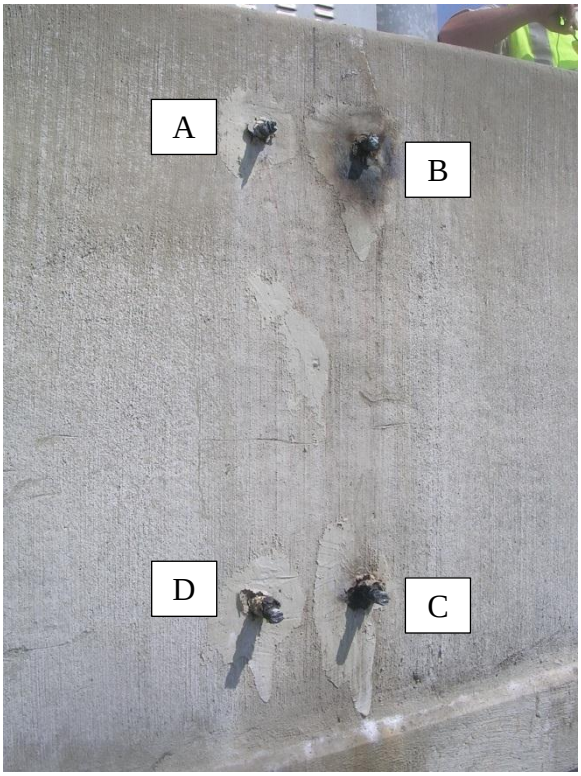


Figure 6. Typical nomenclature for anchors

Figure 7. Torch-cut anchor rod partially withdrawn from parapet

Figure 8. Heavy scorching on parapet surface at torch-cut anchor rods

Figure 9. Failed lower anchor rods at Location 67

Figure 10. Wet epoxy extracted from hole 56B

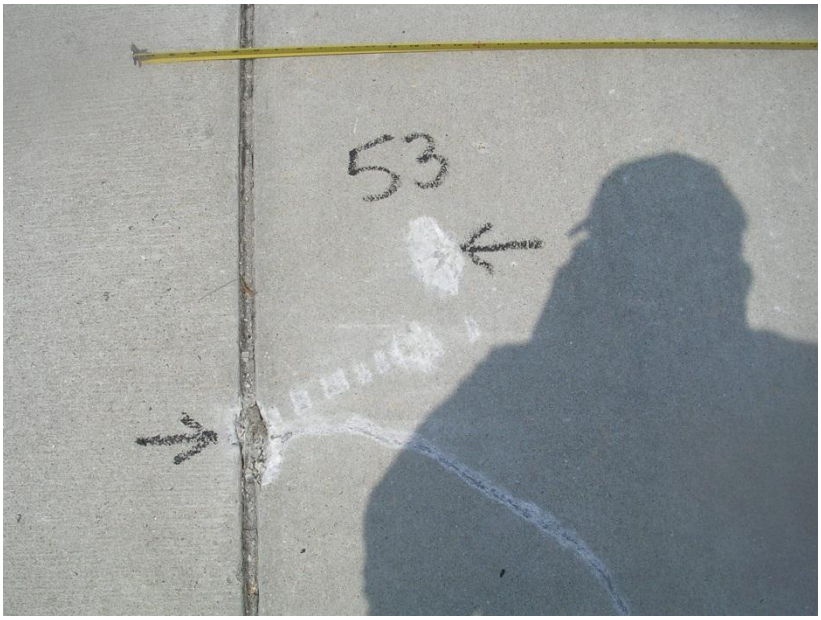


Figure 11. Close-up view of impact damage and scrape marks on sidewalk at location 53

Figure 12. Overall view of distress at sidewalk at location 55 due to canopy failure

Figure 13. Overall view of distress at sidewalk at location 56 due to canopy failure

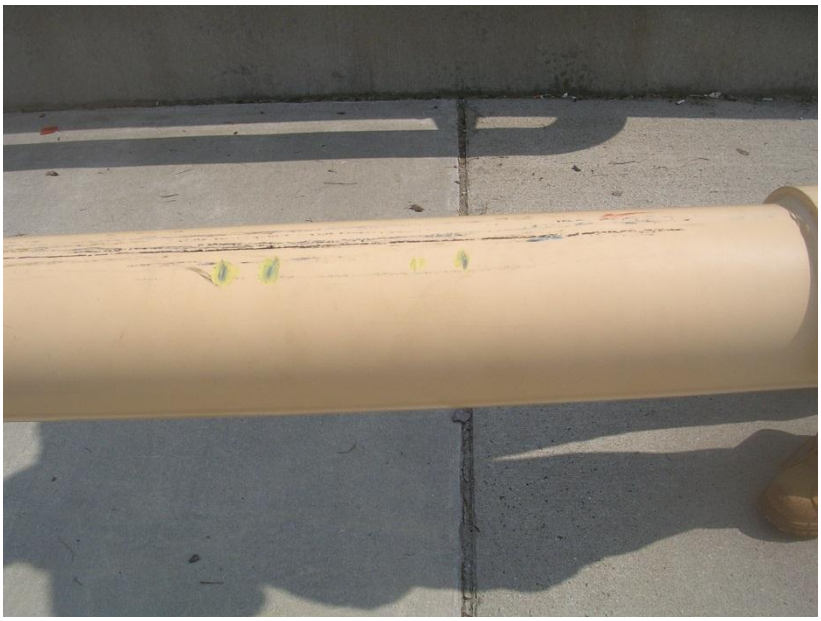


Figure 14. Marks on pedestrian perch rail due to impact from canopy support frame

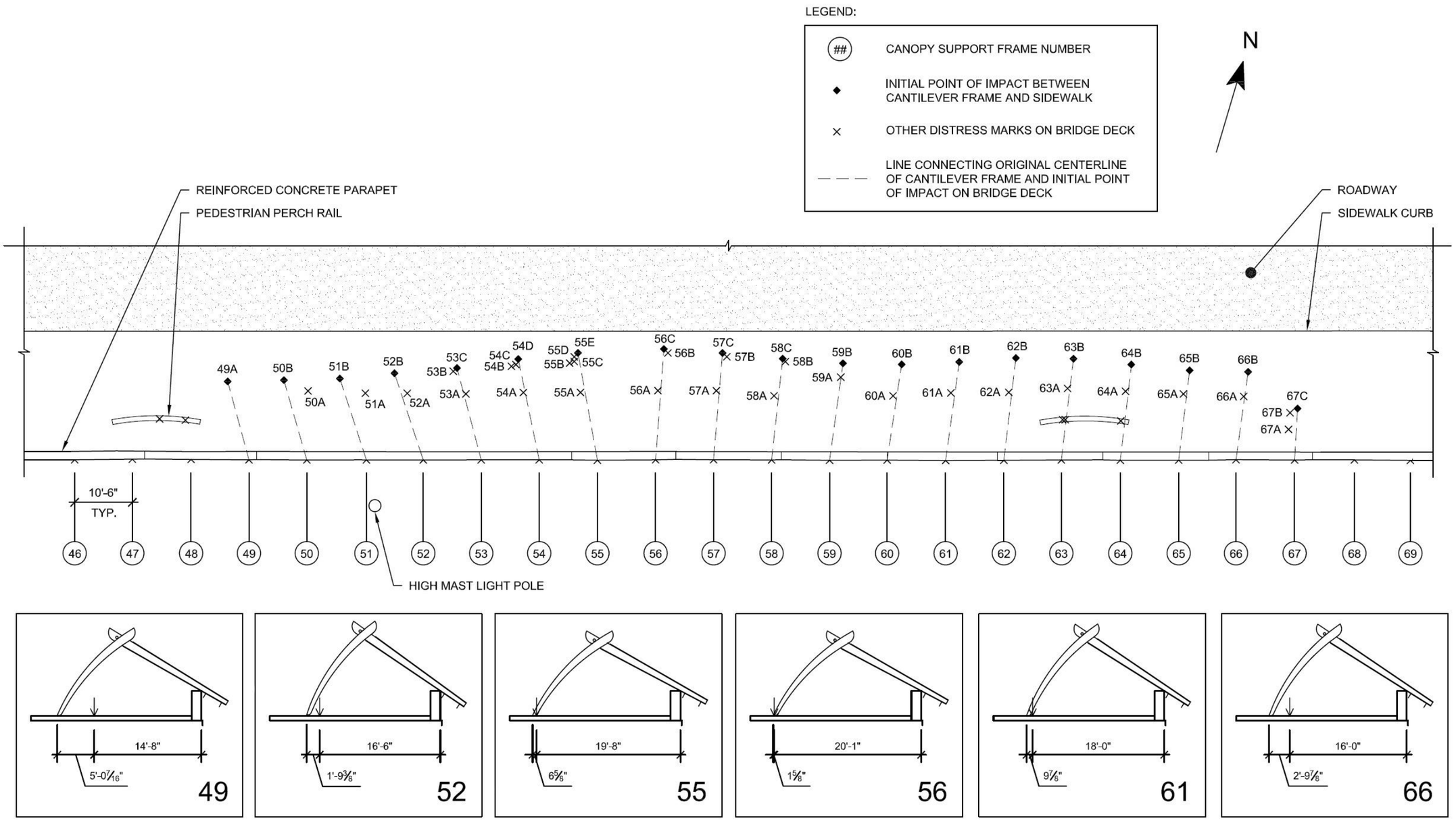


Figure 15. Top: Partial bridge plan showing surveyed locations of support frames at parapet and distress points on sidewalk
Bottom: Diagrams showing variation between actual points of initial impact (arrows) and predicted points of impact assuming rotation about outside corner of parapet for selected frames

Figure 16. Bent connection plates at cantilever tube assembly

Figure 17. Missing connection plates at cantilever tube assembly

Figure 18. Close shot of missing connection plate at cantilever tube assembly

Figure 19. Bent cantilever tube assembly

Figure 20. Failed plate at column assembly

Figure 21. Twisting of column-cantilever assembly

Figure 22. Bent anchor rods

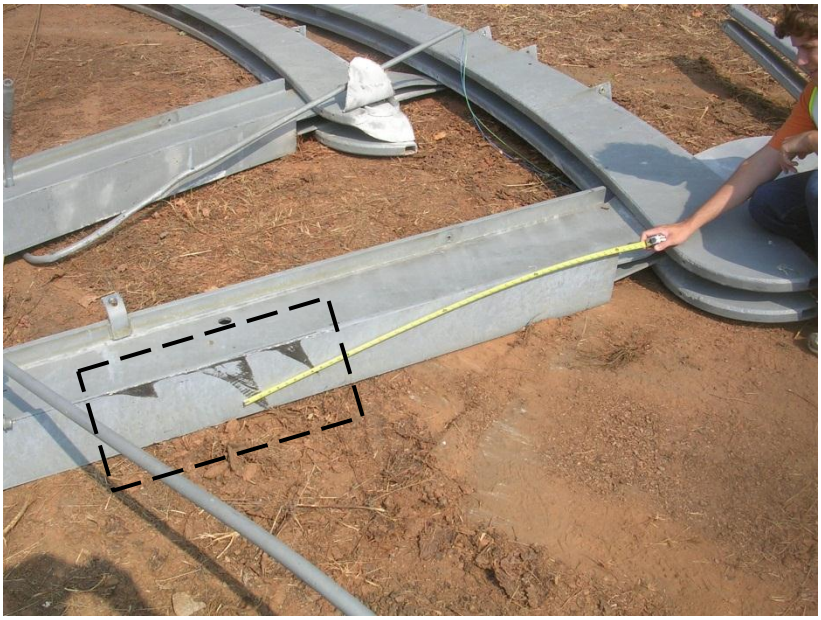


Figure 23. Black paint marks on south side of support frame 52

Figure 24. Close-up shot of black paint marks on support frame 52

Figure 25. Yellow paint on bottom of cantilever tube assembly

Figure 26. Field modified hole for anchor rod in column assembly

Figure 27. Welded plates for eccentric anchorage on frame 51

Figure 28. Coring in progress on south face of parapet

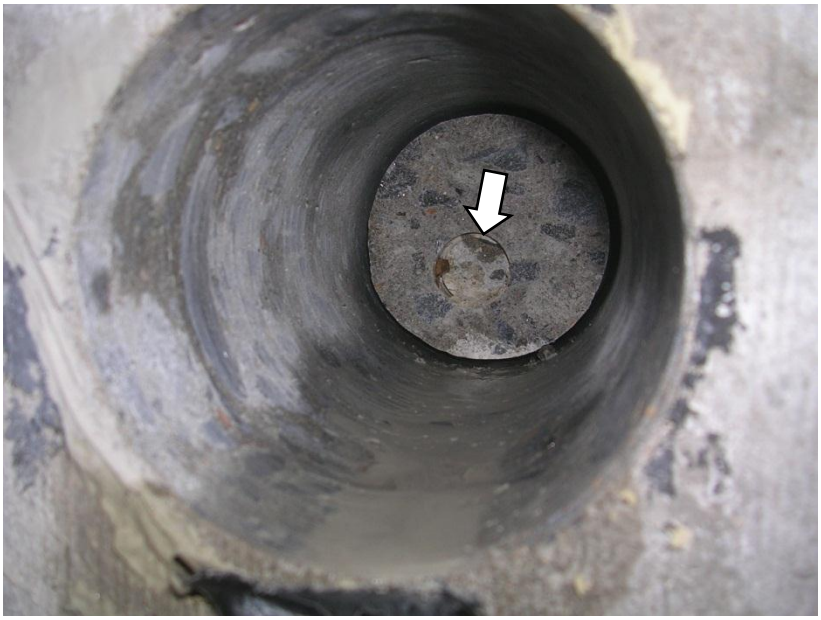


Figure 29. Back of core 56C showing original core barrel impression for anchor rod hole

Figure 30. Load testing of in-situ anchors

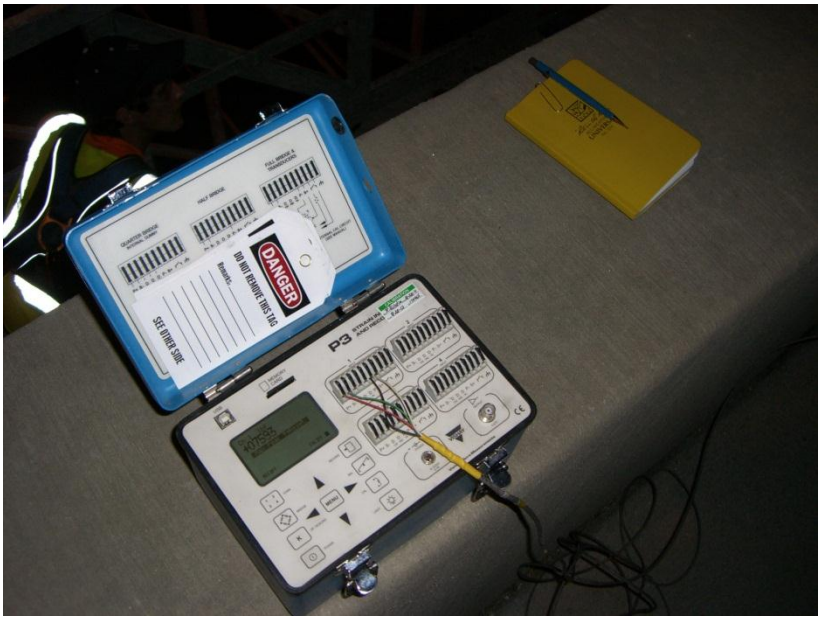


Figure 31. Data collection unit for in-situ load testing

Figure 32. Anchor tested to failure during in-situ load testing

APPENDIX A

Location 49

A & B

D & C

Location 49

A

B

D

C

Location 50

A & B

D & C

Location 50

A

B

D

C

Location 51

A & B

D & C

Location 51

A

B

D

C

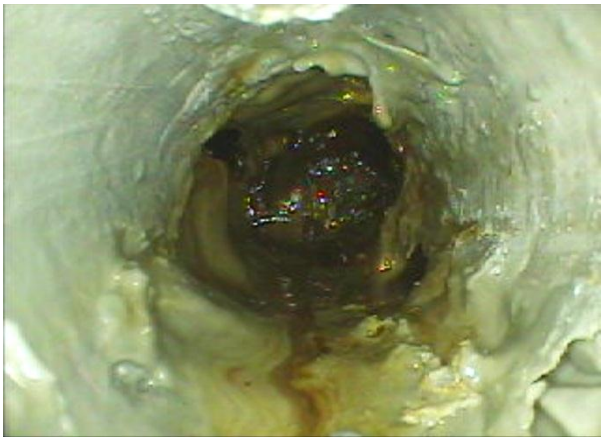
Location 52

A & B

D & C

Location 52

A



B

D

C

Location 53

A & B

D & C

Location 53

A

B

D

C

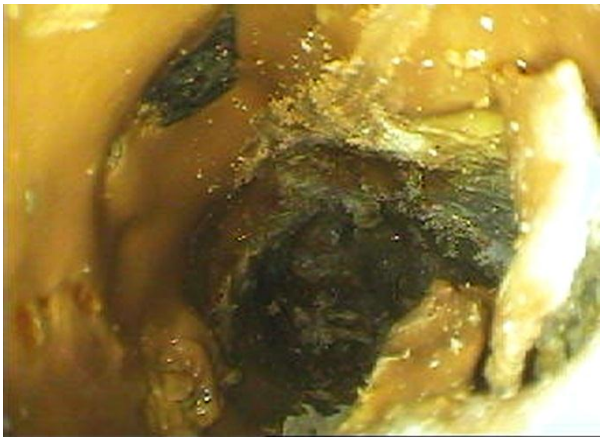
Location 54

A & B

D & C

Location 54

A



B

D

C

Location 55

A & B

D & C

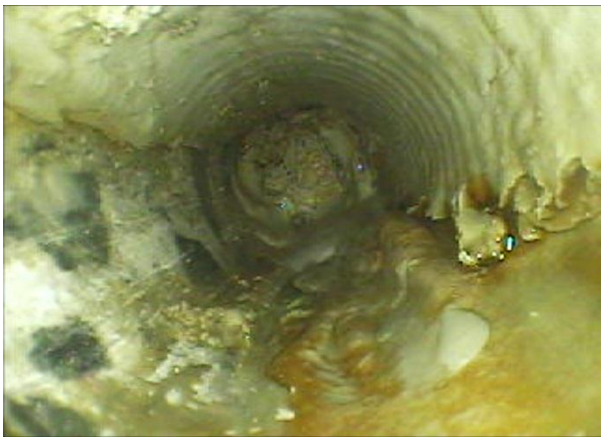
Location 55

A

B

D

C



Location 56

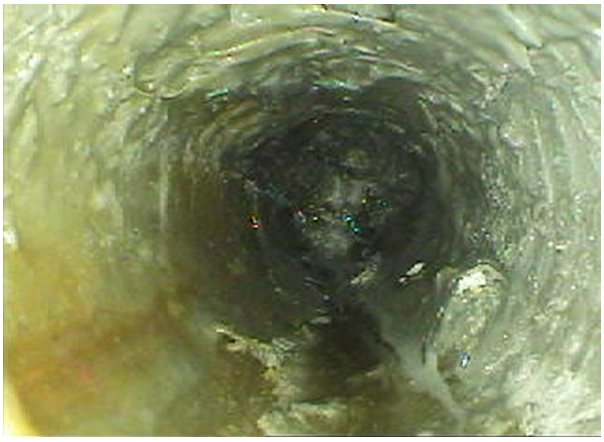
A & B

D & C

Location 56

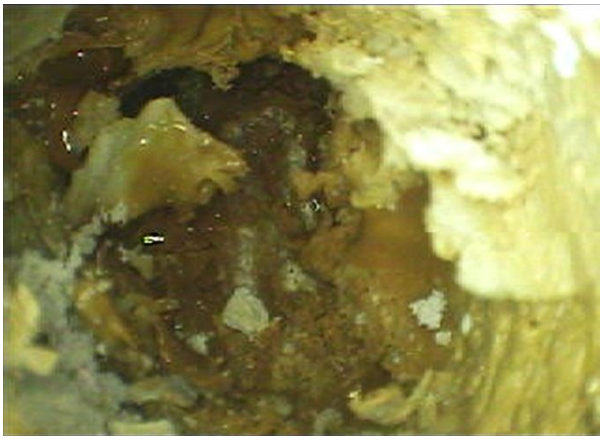
A

B



D

C



Location 57

A & B

D & C

Location 57

A

B

D

C

Location 58

A & B

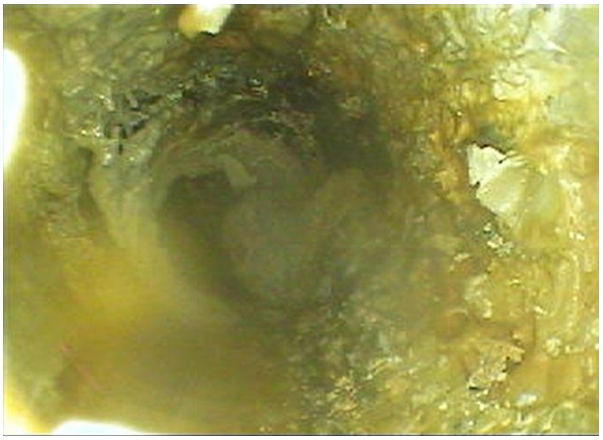
D & C

Location 58

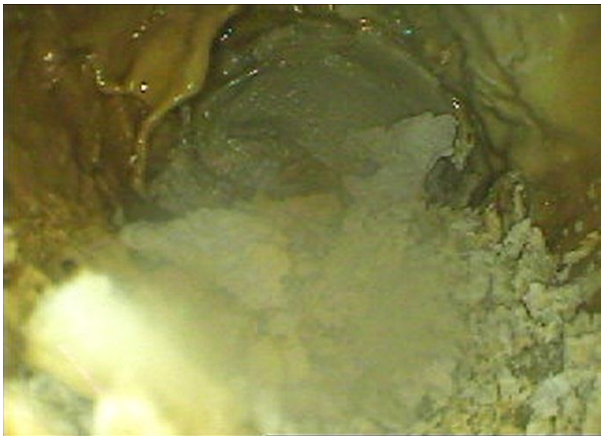
A

B

D



C



Location 59

A & B

D & C

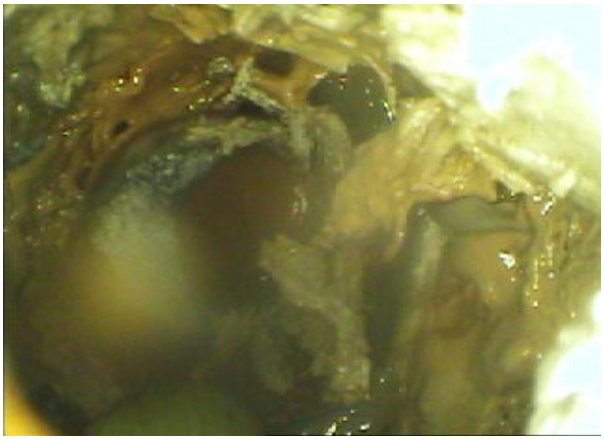
Location 59

A

B

D

C



Location 60

A & B

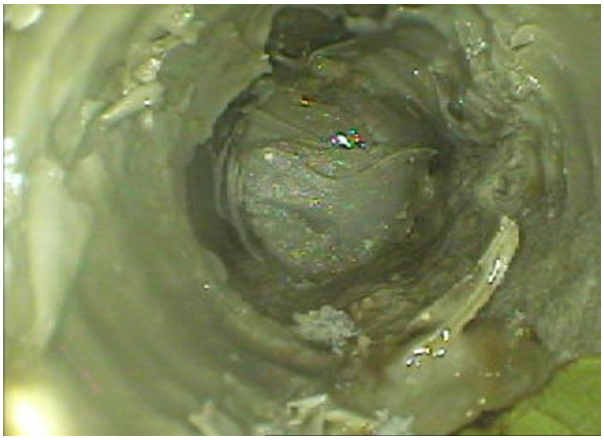
D & C

Location 60

A

B

D



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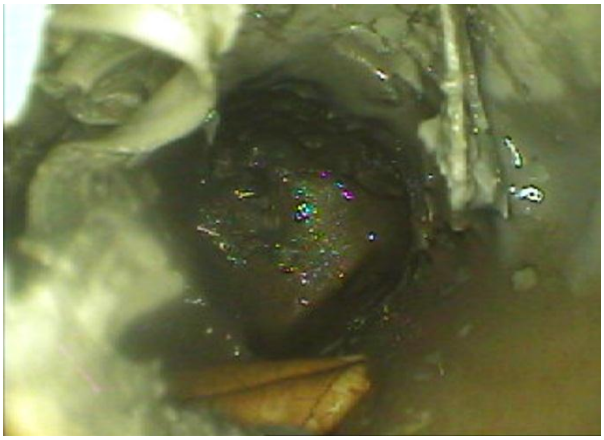


Location 61

A & B

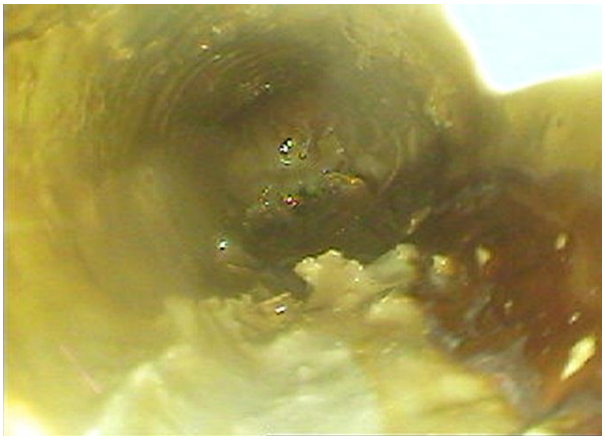
D & C

Location 61

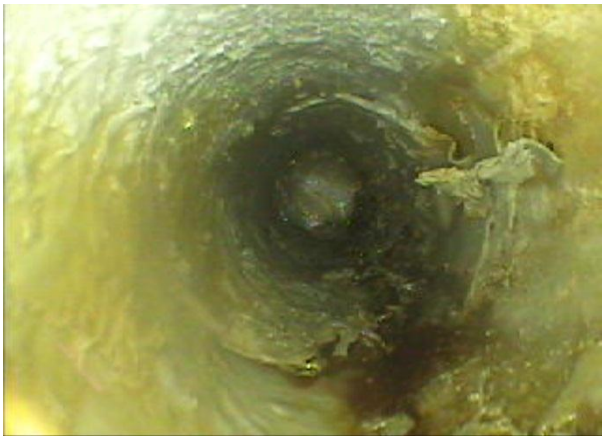
A

B

D



C



Location 62

A & B

D & C

Location 62

A

B

D

C

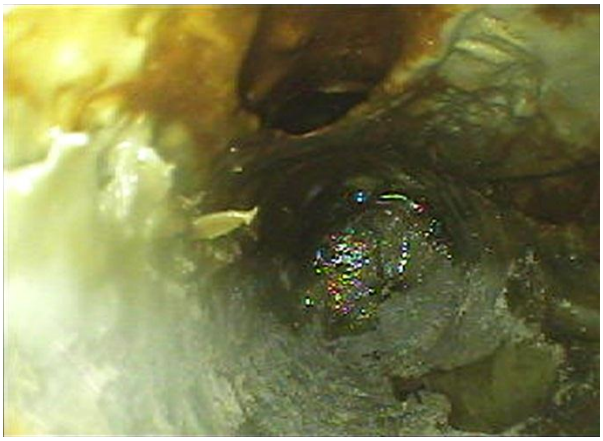
Location 63

A & B

D & C

Location 63

A



B

D

C

Location 64

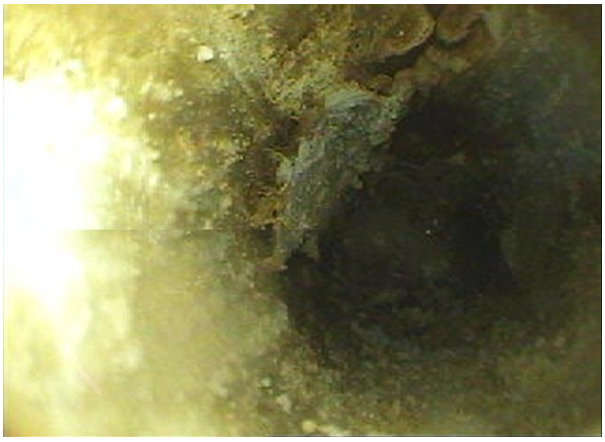
A & B

D & C

Location 64

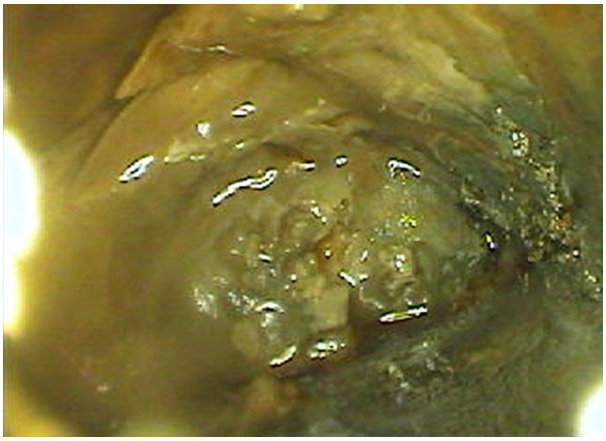
A

B



D

C



Location 65

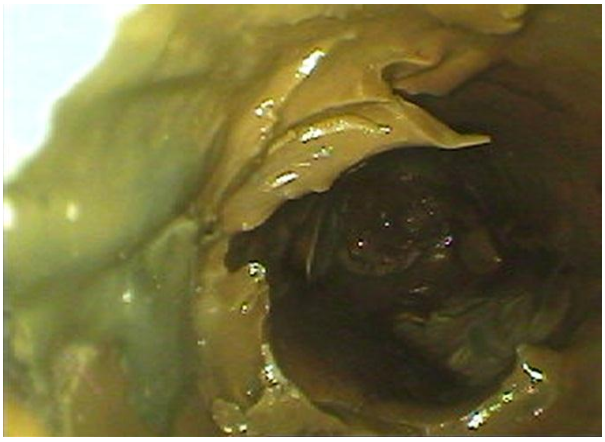
A & B

D & C

Location 65

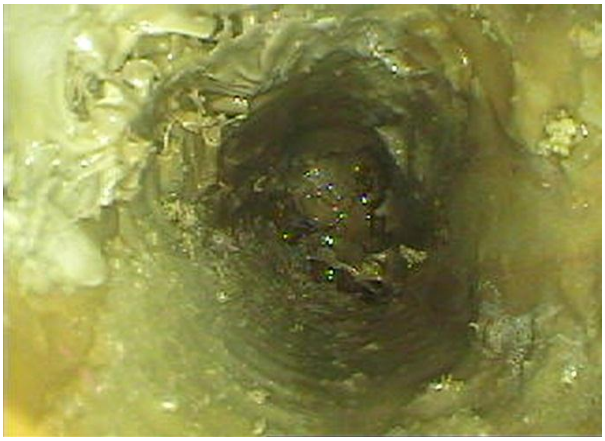
A

B



D

C



Location 66

A & B

D & C

Location 66

A

B

D

C

Location 67

A & B

D & C

Location 67

A

B