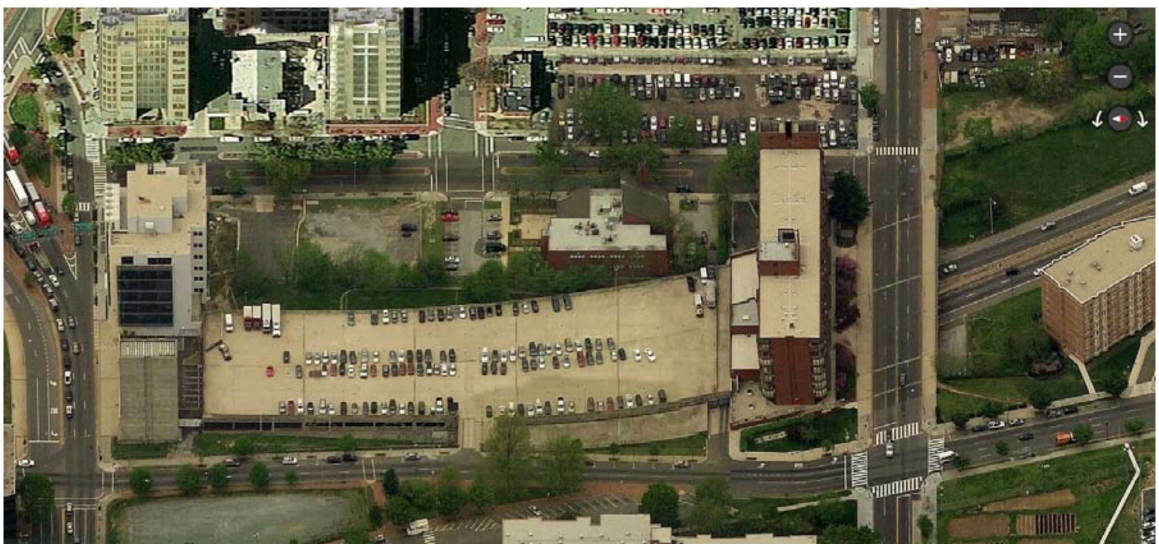
McMullan & Associates, Inc.

DMPED PARKING DECK
STRUCTURAL INVESTIGATION



Prepared for: **GeoCapitol Engineering, LLC**
4545 42nd Street, N.W.
Suite 307
Washington, D.C. 20016

Prepared by: **McMullan & Associates, Inc.**
M&A Project No. 3767

DRAFT SUBMISSION
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Consulting Structural Engineers
1861 Wiehle Avenue, Suite 125 • Reston, Virginia 20190
Telephone: (703) 556- 0651 • Facsimile: (703) 481- 0378 • Email: Mcma@mcmse.com

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Executive Summary

The District of Columbia Office of the Deputy Mayor for Planning and Economic Development (DMPED) retained GeoCapital Engineering and McMullan and Associates (MA) to perform a load analysis and structural condition assessment for the parking structure located at 200 K Street NW in Washington, DC 20001.

The existing structure consists of a two level structure that spans above the Highway I-395 tunnel. An overall plan view is shown in Figure 1 below. The structure has two levels, the “plaza” (upper) level and the “garage” (lower) level (Figure 2). The plaza level is a parking deck owned by the District of Columbia. The garage level is used as a parking garage by the adjacent mid-rise apartment building, and also houses a fan room for the tunnel exhaust at the south end. The DMPED requested that MA determine the maximum load that can be supported at this level by the existing structure.

Figure 1. Plan view of existing structure.

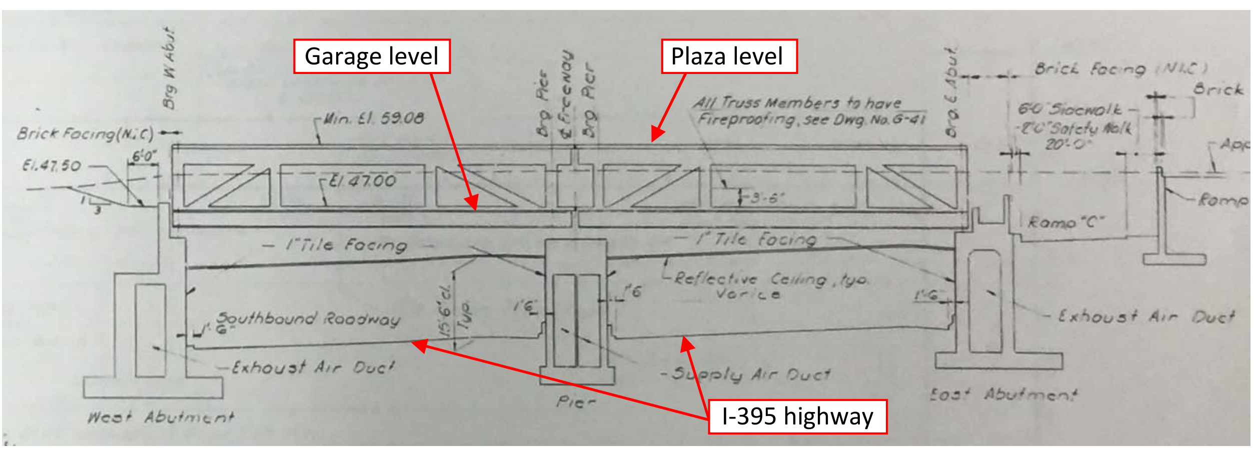


Figure 2. Typical section through existing structure.

The condition assessment was performed by investigation and testing of several areas of the concrete slab and steel structure. Corrosion of the steel reinforcing in the slab and corrosion of some of the steel trusses was discovered. Repairs of the slab and steel trusses are therefore recommended prior to the application of additional loads. The underside of the garage slab has not yet been observed because access to the structure has not yet been granted. Once we investigate its conditions, we will update this report.

The load analysis was performed by calculating the load capacity of the slab, beams, and trusses in several locations. The maximum superimposed load that can be placed on top of the plaza deck is 120 psf. Repairs to the structure should be completed prior to the application of additional loading.

Scope and Methodology

This report contains a condition assessment of the garage, as well a load analysis to determine the maximum load that can be built on top of the existing parking deck. The scope of our work includes:

- searching for, obtaining, and reviewing any existing design drawings and as built records
- performing a condition assessment
- performing a load analysis of the existing structure.

The Condition Assessment was performed in two phases. Phase 1 of the field investigation consisted of a visual survey to spot check the configuration of the structure, compare it to the framing drawings, and identify the locations where field testing is to be performed. During Phase 2 of the investigation, limited field tests were conducted at the locations identified in Phase 1 to evaluate the condition of the structure, including concrete material testing at potential problem areas. We surveyed a limited number of locations to see if there were any conditions requiring repair. If items requiring repair were found, the quantities of repair would be determined as part of construction document preparation.

The load analysis was performed by calculating the load capacity of the plaza slab, plaza beams, and trusses. We determined the load capacity by increasing the superimposed load on the member until the material strength (obtained from the original construction drawings) was reached. The plaza slab thickness and reinforcement was typical for the entire level, and therefore the longest span was chosen for analysis. Several different plaza level beams of varying lengths and member sizes were analyzed and the lowest allowable superimposed load was reported. There are three types of trusses: Types A, B, and C. One truss of each type was chosen for analysis and the maximum superimposed load for each truss was reported. We chose the individual truss within a type that appeared to be the controlling element based on deflections on the drawings. Lastly, we calculated the soil bearing stress under the abutments and piers for the existing condition and compared it to the condition under maximum superimposed load for geotechnical adequacy.

Using the results from the investigation and analysis, structural concerns associated with additional construction or development on the site were identified. If a deficiency was found for any element, the deficiency would be documented in this report and recommendations of strengthening those elements would be included as part of construction document preparation.

Existing Drawings

McMullan and Associates photographed and reviewed the construction documents by Tippetts-Abbett-McCarthy-Stratton dated 1974 as part of our work. We photographed the drawings because they were not allowed to be temporarily borrowed for scanning. There are a total of 257 sheets in the full set. The drawings are available at the DDOT office in the Frances Perkins (US Department of Labor) building.

Per the drawings, the design loading for the existing structure is:

- Plaza Area
    - Live Load 250 psf for emergency vehicle area
    - Live Load 100 psf for all other areas
    - 2-story future townhouse loading
- Garage Area
    - Live Load 40 psf for supply fan room
    - Live Load 75 psf for all other areas

Although the original construction documents indicated that the structure is designed for the loads from 2 levels of future townhouses built atop the parking garage, the anticipated dead load and live load from this expansion was not documented. In addition, it is not clear in the drawings where the emergency vehicle area was located.

Construction and Historical Background

As shown in Figure 2, the existing structure consists of steel trusses that are approximately 15' tall and span the I-395 tunnel. The top chords of these trusses support the plaza level and the bottom chords support the garage level. Concrete abutments support the trusses at the exterior (east and west) sides. A central pier separates the highway below into northbound and southbound traffic, and provides the interior support for the trusses. Diagonal members between the trusses at the exterior (abutment) sides transfer diaphragm forces to the abutments.

Both levels are constructed as a one-way reinforced concrete slab supported on steel beams that span to the truss chords. Concrete encasement for fire protection was specified for the steel beams at the plaza level, the top chord of the trusses, and the web members of the trusses. In the original drawings, the truss web members are shown to be encased in solid concrete from the garage level to 3'-6" above the garage level slab (Section 2-2 in Figure 3). Above that, a hollow concrete shell created by wrapping the steel in wire lath (Section 1-1 in Figure 3) is specified for the truss web members, truss top chords, and also the garage level beams. This solid encasement at the lower portion of the truss web members was likely meant to protect them against damage from vehicle impacts. Concrete encasement was <u>not</u> specified for the steel members corresponding to the garage level, including the bottom chords of the trusses and the garage beams.

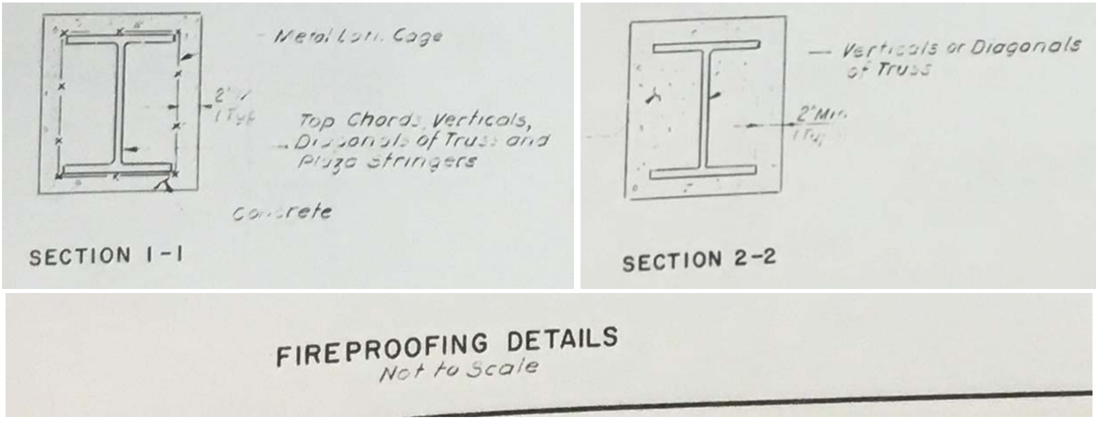
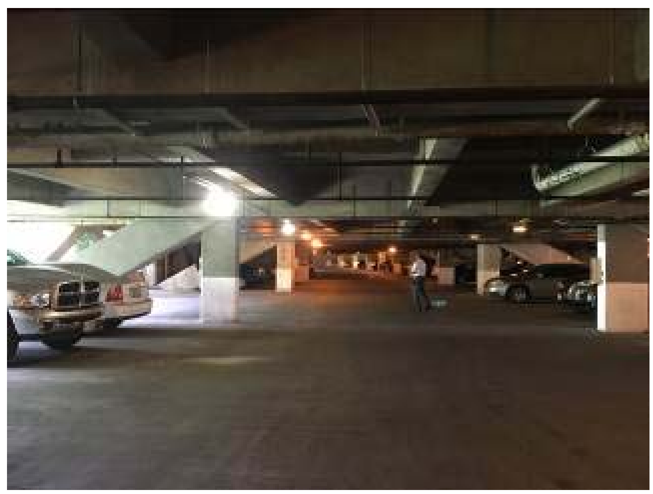


Figure 3. Concrete encasement details.

Condition Assessment, Phase 1 - Initial Visual Survey

McMullan & Associates, Inc. performed a visual survey of the garage on July 13, 2016 as Phase 1 of the field investigation. During the visual survey, we documented areas of concern where field testing was to be performed. The following photographs summarize this initial survey:

1. Overall view of the garage supported on the lower slab. Steel truss configuration can be seen.



2. There are several cracks in the exterior columns along the east and west faces.

3. Steel straps have been placed around several of the concrete diagonals and columns in the garage.

4. There are previous concrete repair patches in many locations.

5. There are cracks in the concrete slab under the plaza level.

6. In addition to concrete cracking the vertical truss members, there are dovetail slots and existing shelf angles for future brick facing.

7. There is cracking in the concrete fireproofing cover of the interior vertical truss members.

8. There are cracks in the concrete fireproofing cover of the underside of the diagonal members. In addition, there is staining from water on the perimeter beam.

9. There are several locations in the garage where loose concrete fireproofing cover has been removed and replaced with fireproofing

10. There are pieces of corroded steel on the floor of the garage under an area where the concrete cover was replaced with spray on fireproofing.

Condition Assessment, Phase 2 - Field Testing

Based on our observations in Phase 1 of the field investigation, plans for the Phase 2 field testing were developed to evaluate the condition of the structure. Appendix A contains the location plans where field testing occurred during Phase 2.

McMullan & Associates, Inc. performed field testing of the garage on August 24, 2016 and August 25, 2016. The testing conducted during this Phase included:

- Delamination surveys were performed, including:
  - Three (3) bays of the plaza slab
  - Three (3) bays of the garage slab
  - Several columns and beams
- Chloride tests were taken at eleven (11) locations to check the chloride concentrations in the concrete
- Concrete encasement was removed to expose structural steel at seven (7) locations. Visual condition assessment of concrete and steel, and thickness measurement of existing steel performed
- Ground penetrating radar (GPR) testing was performed at three (3) locations of the slab for identification of rebar spacing and cover
- Additional visual observations at the upper and lower deck.

We have attempted to coordinate a visit to make visual observation of the underside of the garage level from the I-395 tunnel for several weeks, but have been unable to obtain clearance from DDOT as of this date. Further findings and recommendations will be added to the report once this task is completed.

Delamination Survey

Areas of both the upper and lower decks were sounded by chain drag (Figure 4) to determine the approximate size and extent of concrete delaminations. These occur when the steel reinforcing bars corrode and expand, forcing the concrete above to separate. A "hollow" sound is created when the chain is dragged across the concrete surface. As the corrosion gets worse, delaminations progress to visible spalls which eventually break off from the slab (Figure 5).

Figure 4. Chain drag sounding on the concrete slab.

Figure 5. Visible concrete delamination on the slab.

The extent of our delamination survey and approximate size and locations of observed delaminations are shown in Appendix A. Parked cars obstructing access to the slab caused the irregular shape of the survey. In general, the plaza level has fewer and smaller delaminations than the garage level, and no visible spalling. The garage level delaminations are generally larger and most are concentrated in the drive lane south of the entrance/exit ramp. Several of the delaminations have progressed to visible spalls in the garage level slab.

Chloride Tests

Concrete powder samples for chloride testing (Figure 6) were taken at eleven (11) locations to represent the full range of structural conditions (Appendix A). Descriptions of the sample locations are provided below:

- Vertical truss member, upper (hollow) fireproofing surround
    - Location C1-U
- Vertical truss member, lower (solid) fireproofing surround
    - Locations C2-L and C4-L
- Garage level slab
    - Locations C3, C5, C6 and C7
- Plaza level slab
    - Locations C8, C9, C10 and C11

Most locations have samples taken at two depths to determine the extent of penetration into the concrete. In these cases, the location name is followed by "-1" or "-2". For example, garage level slab Location C3 was tested at two different depths, and therefore has sample results for "C3-1" and "C3-2".

Samples were delivered to Concorr, Inc. for laboratory analysis, the results of which are presented in Appendix B. Overall, results range from 15 ppm to 4994 ppm, but breaking the result out by area proved to be informative. At the plaza level slab, results range from 20 to 123 ppm. At the garage level slab, they vary from 34 to 2219 ppm. The highest results are in a sample taken from the lower (solid) section of a vertical truss member that was spalling (4994 ppm). All of the high concentration results are at shallow depths in the concrete, while chloride penetration deeper into the concrete (2.5" to 3.5" depth) is generally much lower, ranging only from 20 to 85 ppm.

Several conclusions can be made from these results:

- The upper deck has little if any salt applied to it during the winter, or has had a well maintained clear sealer.
- The lower deck has a significant amount of salt either directly applied to it or brought in with vehicles which then drips onto the slab. This is a common cause of deterioration in garages.
- The existing concrete is fairly dense and of good quality.
- The mix of very high and low chloride concentrations at the truss verticals could be a result of ponding which would create both wet conditions and locations that are prone to have ice and therefore salt applied more that the rest of the slab.

Figure 6. Concrete powder sample for chloride testing.

Concrete Encasement Removal

Patches of the hollow and solid concrete fireproofing encasement were removed using a saw cut and hand tools to expose the existing steel of the truss and diaphragm members at seven (7) locations as documented in Appendix A. Visual observations were recorded and flange thicknesses were measured at each opening.

In general, two (2) different types of concrete were used for fire encasement:

- Coarse, rounded aggregate mix for solid concrete encasement
- Fine aggregate mix for hollow concrete/steel lath.

Regardless of the type of encasement (solid versus hollow), the 2” concrete cover that was specified in the construction drawings was confirmed.

In the following sections, the visual observations for each opening location is presented along with a corresponding photograph. A summarized table documenting the specified and measured flange thicknesses is subsequent to the visual observations. Finally, a discussion of the truss conditions is presented.

Visual Observations

Location A - Diagonal Member:

- Concrete removal description: 12"x12" patch of concrete encasement removed, and additional spalled off during demolition.
- Observations:
  - Bottom flange edge facing exterior corroded along length of diagonal through hollow fire encasement.
  - Original paint (yellow) visible.
  - Rust accumulated at boundary between solid concrete fire encasement and hollow concrete/steel lathe. Rust expansion was measured as 1/8"-1/4".

Location B - Vertical Member:

- Concrete removal description: 12"x12" patch of concrete encasement removed at region showing significant rust staining.
- Observations:
  - Efflorescence present at verticals along truss line, indicating water infiltration present at locations. Likely through nearby expansion joint.

Location C - Vertical Member:

- Concrete removal description: 12"x12" patch of solid concrete encasement removed. Taken as a "control" since encasement did not appear cracked/corroded.
- Observations:
  - No paint observed.
  - Thin rust patina apparent, but no signs of corrosion or rust expansion.

Location D - Vertical Member:

- Concrete removal description: 12”x18” patch of hollow concrete encasement removed. Encasement cracking vertically at corners
- Observations:
  - Thin layer of concrete covers web and inside faces of flanges. Paint visible on column web; no corrosion visible.
  - Wire was tack-welded to column flange at one spot.
  - Looking inside hollow cavity up to the bottom flange of the top chord, some paint and minimal rust is visible. No spalling observed on truss member (no rust flakes accumulated at bottom of hollow cavity).
  - 1/8” rust buildup/expansion on outside flange face.

Location E - Diagonal Member:

- Concrete removal description: 12”x12” patch of concrete encasement removed. Encasement cracking along bottom face of diagonal. Cut taken at boundary between solid and hollow encasement (again, site of most cracking).
- Observations:
  - Some efflorescence visible in existing cracks.
  - Rust prevalent in cut section, but not causing major section loss. Mostly pitting. Some rust lines in concrete coming from cracks.
  - Water likely coming from adjacent expansion joint.

Location F – Top Chord Member:

- Concrete removal description: 12”x12” patch of fireproofing material removed. Section was previously covered in hollow concrete/steel lathe, but was removed and replaced with spray fireproofing material.
- Observations:
  - Rust is prevalent, but not causing major section loss. Mostly pitting. Steel mesh for fireproofing. No rust discoloration in fireproofing.

Location G – Vertical Member:

- Concrete removal description: 8”x8” patch of solid concrete encasement. Two (2) bottom corners of vertical encasement could be easily removed by hand (opposite corners, not on same side).
- Observations:
  - Concrete noticeably powdery and white, indicating a chemical reaction deteriorating cement. Paint peeling off easily at base of vertical.
  - Vertical to north exhibits similar degradation at base of column. Slab in proximity to vertical is significantly delaminated.
  - 1/2” thick layer of expanded rust up to 9” tall from finish floor of garage deck.

Specified and Measured Flange Thicknesses

The specified and measured flange thicknesses for all locations is presented in the table below.

Location	Specified member size	Flange thickness		Percent reduction
		Specified (in)	Measured (in)	
A	W14x43	0.528	0.438	17%
B	W24x61	0.591	0.563	5%
C	W27x84	0.636	0.563	12%
D	W36x150	0.940	0.813	14%
E	W30x108	0.750	0.719	4%
F	W36x230	1.260	1.250	1%
G	W36x230	1.260	1.063	16%

Discussion of Truss Conditions

Corrosion is occurring at the steel trusses, some of it is severe with up to 17% reduction in thickness. While not at a critical location, it raises concerns about other hidden conditions. Note that some fairly large portions of the truss concrete fireproofing have been removed and replaced with spray fire proofing. This occurs most frequently at expansion joints where the trusses were exposed to leaks in the past. It is not known whether the joints are currently leaking. It is also not known whether the removal of concrete and replacement with spray applied fire proofing was accompanied by any strengthening of the steel truss members. We did not find any drawings or field evidence that this occurred.

There appear to be three main sources of water associated with the corrosion:

1. At exterior locations, the trusses are exposed to driven rain from the open sides of the garage.
2. At expansion joints, leaks occur from expansion joints above.
3. At the lower portions of the truss, water accumulates on the lower level slab and is splashed or wicked up into the concrete.

There were two typical locations where corrosion of the truss is likely to begin and to be the most pronounced:

1. At the transition from the hollow to the solid concrete fireproofing section where water that has penetrated the hollow section is trapped on top of the solid portion.
2. At the bottom of the members where they are most exposed to chlorides from the deck. This condition presented the most severe damaged that we observed (location G).

Ground Penetrating Radar (GPR) Testing

Ground Penetrating Radar was used to locate reinforcing steel within the concrete slab (Figure 7). The testing showed that the reinforcing layout was generally as indicated in the drawings.

Figure 7. Markings of reinforcement in slab found using GPR.

Additional Visual Observations

Location I – Expansion Joint:

Observations:

- Severe rusting of embedded steel angles in joint.
- Section loss is visible in the joint.
- Pieces of expansion joint metal are dropping onto garage level deck.
- Top surface of expansion joint angle is partially painted and in good condition.

Location J – Repaired Deck Patches:

Observations:

- Rusting of embedded steel angles in joint.
- Mild section loss is visible in the joint.
- Pieces of expansion joint metal are dropping onto garage level deck.
- Top surface of expansion joint angle is partially painted and in good condition.

Location K – Crack in Soffit Near Expansion Joint**:**

Observations:

- Crack is approximately 5' long.
- Efflorescence visible at crack.
- Crack in proximity to rusting expansion joint.

Load Analysis

The plaza slab, plaza beams, trusses, piers, and abutments were analyzed for the existing loads and to determine the maximum superimposed load that each element could support. Since both the plaza and garage levels are used for parking, the uniformly distributed design live load in the existing configuration is 40 psf per ASCE 7-10, which is referenced by the latest edition of the IBC code. We determined the load capacity by increasing the superimposed load on the plaza members until the material strength (obtained from the original construction drawings) was reached.

Per the design documents, the bar reinforcement for concrete is either ASTM A615 Grade 40 (fy = 40 ksi) or ASTM A615 Grade 60 (fy = 60 ksi). However, the grade of the reinforcement in the plaza reinforced concrete slabs was not defined. In addition, the concrete 28-day strength was not specified. Based on the time period of the construction, Grade 40 rebar and 3,000 psi concrete with a unit weight of 145 pcf was assumed for the slab analysis. At the plaza level, the one-way reinforced concrete slab is 7" thick with reinforcement at the top and bottom that was typical for the entire level, and therefore the longest span was chosen for analysis.

The plaza beams are standard steel sections of varying sizes and lengths with a concrete fireproofing surround. Per the design documents, the plaza beam steel is AASHO M183 (fy = 36 ksi). Shear studs make the plaza beams composite with the 7” concrete slab. However, in some locations, knockout pans were provided along the centerline of the beam. These knockout pans could interrupt the shear transfer at the steel to concrete interface and therefore the beams act non-compositely. Several different plaza level beams of varying lengths and member sizes were analyzed, because it was not clear which individual element would control.

Analysis software (Enercalc) was used to determine the maximum superimposed load for the plaza concrete slab and plaza beams. The maximum superimposed loads for the slab and the controlling element of the beams are reported in Table 1.

Two different grades of structural steel were specified in the design documents for the truss members. The standard sections used in the trusses are AASHO M222 Grade 50 (fy = 50 ksi), and the built-up sections are ASTM A514 (fy = 100 ksi). There are three types of trusses: Types A, B, and C. We analyzed one truss of each type that appeared to be the controlling element. The existing drawings contain a table that specifies the overall geometry (member lengths), member sections, and dead load joint deflections for each truss. It is not initially clear which individual truss within a type has the maximum stresses under existing loads—and will therefore likely result in the lowest allowable superimposed load—because although a given truss may have a longer span, it also has bigger members. To determine which truss could be the controlling element within a type, the deflection ratio was calculated for each truss by dividing the specified deflection by the overall length. Comparing the deflection ratios of the trusses indicates how stressed the trusses are relative to one another and their relative stiffness. The one truss within each type with the largest deflection ratio was therefore chosen for the analysis, for a total of three trusses analyzed.

For the analysis, the trusses were modeled using finite element analysis (FEA) software RISA 3D (Figure 8). The existing dead loads from the reinforced concrete slab, plaza beams with fireproofing, and garage beams were applied as uniform line loads to the respective top and bottom chords of the truss. The weight of the ceiling, hung from the bottom chord of the trusses, was applied as a uniform existing dead load to the bottom chord. The self-weight of the truss members and the fireproofing of the trusses were also included as an existing dead load. A uniform live load of 40 psf was applied to the lower chord (garage level) of the truss. To determine the superimposed load capacity of each truss, we increased the superimposed load applied to the top chord (at the plaza level) until the material strength (obtained from the original construction drawings) was reached. The maximum allowable superimposed load for each truss analyzed are presented in Table 1.

Truss Type A and Truss Type C control the design, and therefore a maximum superimposed (dead plus live) load of 120 psf may be applied to the plaza level for future construction.

Table 1. Maximum superimposed load for each structural component.

Structural Component	**Maximum Superimposed Load (psf)**
Plaza concrete slab	400
Plaza beam	130
Truss Type A	120
Truss Type B	220
Truss Type C	120

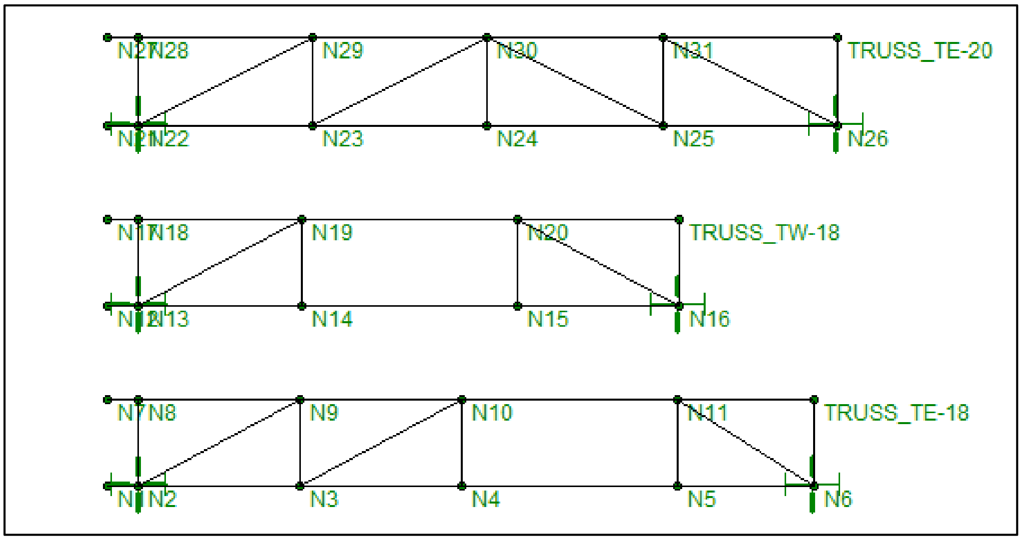


Figure 8. Truss analysis in RISA 3D.

Lastly, we calculated the soil bearing stress under the abutments and piers for the existing condition and compared it to the condition under maximum superimposed load of 120 psf for geotechnical adequacy (Figure 9). The loads listed below were used to calculated the soil bearing stress for each condition:

1. Existing conditions
    a. Existing dead loads of superstructure (concrete slabs, concrete encasement, steel beams, hung ceiling, and trusses)
    b. Self-weight of the concrete making up the abutments, piers, and their footings
    c. At the abutments, the weight of the soil above the footing.
    d. At the abutments, a uniform design lane load of 64 psf was applied where the highway is located, per AASHTO LRFD Bridge Specifications, Section 3.6.1.2.4.
    e. Live load of 40 psf applied to the garage level
    f. Live load of 40 psf applied to the plaza level
2. Condition under maximum superimposed load
    a. Loads (a) through (e) listed above
    b. Superimposed load of 120 psf applied to the plaza level

The reactions at the trusses were distributed at a 1:1 vertical to horizontal ratio down to the footing, with a maximum distributed width equal to the tributary area of the truss. These results of the bearing stress analysis are presented in Table 2. The maximum bearing stress due to existing and superimposed loading was calculated as 5.9 ksf at the pier, with an 11% increase from existing loads. GeoCapitol confirmed that this is acceptable.

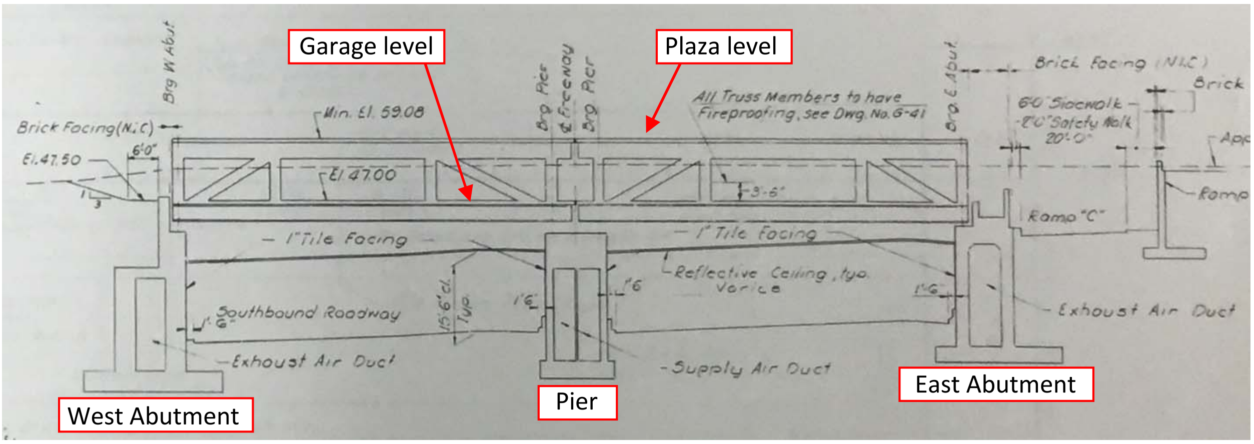


Figure 9. Abutments and pier shown in typical section.

Table 2. Soil bearing stresses for each structural component.

	Structural Component		
	East Abutment	**West Abutment**	**Pier**
Existing Bearing Stress (ksf)	4.4	3.8	5.3
Existing + Superimposed Bearing Stress (ksf)	4.9	4.0	5.9
Percent Increase	11%	6%	11%

We did not evaluate the lateral capacity of the structure due to the maximum superimposed load of 120 psf for this report. Currently, diagonal bracing spanning north-south between at the ends of the trusses transfers the lateral loads to the piers and abutments. The large cantilevered walls of the piers and abutments transfer the lateral loads to the soil in both the north-south and east-west directions. However, it is not known what addition will be built or what the dead load from this new structure will be. Once the additional loading configuration is determined, the lateral capacity of the existing structure may need to be checked.

Conclusions and Recommendations

Load Analysis

According to our calculations, the maximum allowable superimposed load that may be applied to the plaza level is 120 psf. This includes dead and live load to be applied through future expansions such as two levels of townhouses. Once the additional loading configuration is determined, the lateral capacity of the existing structure may need to be checked before construction. We do not recommend applying this load until repairs of the trusses are completed.

Repair Recommendations

There are repairs that should be made prior to building townhouses on top of the existing structure. The same repairs should be considered for the current use of the garage, although there may be adjustments in the specific approach.

Truss Repairs:

The corrosion of the existing truss members must be addressed. Best practice prior to adding to the structure would be to remove all of the existing concrete and spray fire proofing so that all damage is exposed and appropriate design solutions can then be applied.

The bottom chord members on the underside of the slab must also be thoroughly inspected and repaired if necessary, which will require work from the 395 tunnel. We have as of today not been able to access this area and will update this report once access is obtained.

All existing corrosion must be removed and the remaining steel sections measured. Where there is significant section loss, repairs would likely involve welding on of additional steel plates or sections. At the same time, the existing connections should be inspected with any damaged welds or bolts removed and remediated. From our spot observations, there are likely several locations that need repair.

Garage Level Deck:

The top deck concrete is relatively in good condition. Minor concrete cut and patch repairs of delaminations should be made prior to building on top since these will become hidden in the future. The expansion joints should be replaced with a better system. These have been ongoing problems and will be much more difficult to repair once any future construction is in place above.

Depending on the future use of the structure, waterproofing of some sort should be applied over the entire deck.

- For the current use with apparently little salt application, a penetrating sealer may be appropriate.
- For a more active use as public parking, a traffic bearing membrane would be more appropriate.
- For a scenario with new construction above, a hot applied waterproofing membrane should be considered as the best long term protection.

Plaza Level Deck:

The lower deck concrete is a greater concern than the upper deck due to more widespread delaminations and the current high levels of chloride contamination. While spalling is not widespread, it likely will become so in the future. Chloride contamination is also a major concern for future deterioration of the truss. Installing a traffic membrane at this point would slow down the penetration of chlorides into the concrete, but would not remove the existing chlorides and they would eventually severely impact the trusses.

Repair options to address this condition need to be evaluated and compared but would include:

- Electro-chemical chloride extraction.
- Cathodic protection in conjunction with other repairs.

- Removal of the top two inches of the concrete deck over the entire area and replacement with a bonded overlay. Better slopes for improved drainage could also be included with this method.
- Installation of a traffic bearing membrane to prevent future chloride intrusion is recommended for all methods.
- All repair solutions would require a phased approach to minimize the impact on use of the garage and on the tunnel below.

Note that the condition of the bottom chord of the truss has not yet been observed due to getting access approval. While their condition would not change the need for repairs, if in very poor condition could become a recommendation for immediate repairs.

Miscellaneous Items:

The original construction included dove tail slots and brick shelves for support of a future brick veneer around the exterior of the structure. These were not used and have been directly exposed to the elements. The dove tail slots are corroding, so likely cannot be used. The brick shelves appear be in better condition and may be re-useable. Note that if the concrete fire protection is completely removed, this would also remove both of these items. Installation of new exterior façade supports would need to be coordinated with the future use of the structure.

Similarly, exterior non-structural elements such as gratings and security fences are deteriorating. We did not specifically inspect these as part of our work, but noted that they have been exposed to the elements for 40 plus years with normal aging and deterioration.

The top deck was not constructed with a railing or parapet around the edge. Jersey barriers and chain link fencing have been installed in different locations around the perimeter. This study did not include evaluation of these items, but they may not be code compliant and we recommend further evaluation.

Drainage systems appeared to be in fair shape, although the slopes at the lower deck are not adequate for good drainage. The plumbing appears aged, but generally serviceable. If new construction occurs, it would be best to replace the drains and pipes to bring them up to date.

McMullan & Associates, Inc.

Appendix A

Phase 2 Field Testing Plan

Consulting Structural Engineers
1861 Wiehle Avenue, Suite 125 • Reston, Virginia 20190
Telephone: (703) 556- 0651 • Facsimile: (703) 481- 0378 • Email: Mcma@mcmse.com

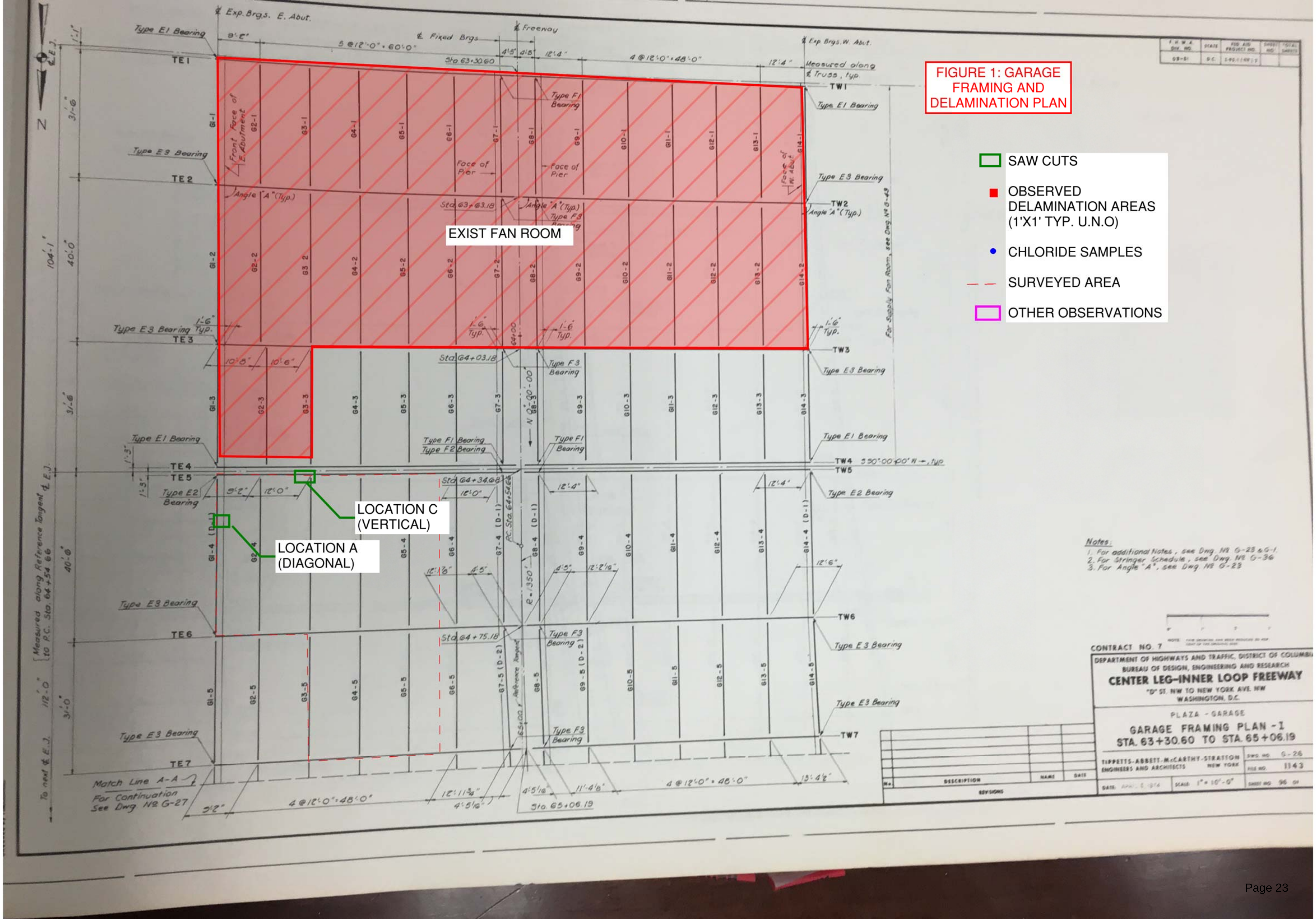


FIGURE 1: GARAGE FRAMING AND DELAMINATION PLAN
SAW CUTS
OBSERVED DELAMINATION AREAS (1'X1' TYP. U.N.O)
CHLORIDE SAMPLES
SURVEYED AREA
OTHER OBSERVATIONS
EXIST FAN ROOM
LOCATION C (VERTICAL)
LOCATION A (DIAGONAL)
CENTER LEG–INNER LOOP FREEWAY
GARAGE FRAMING PLAN - I
STA. 63+30.60 TO STA. 65+06.19

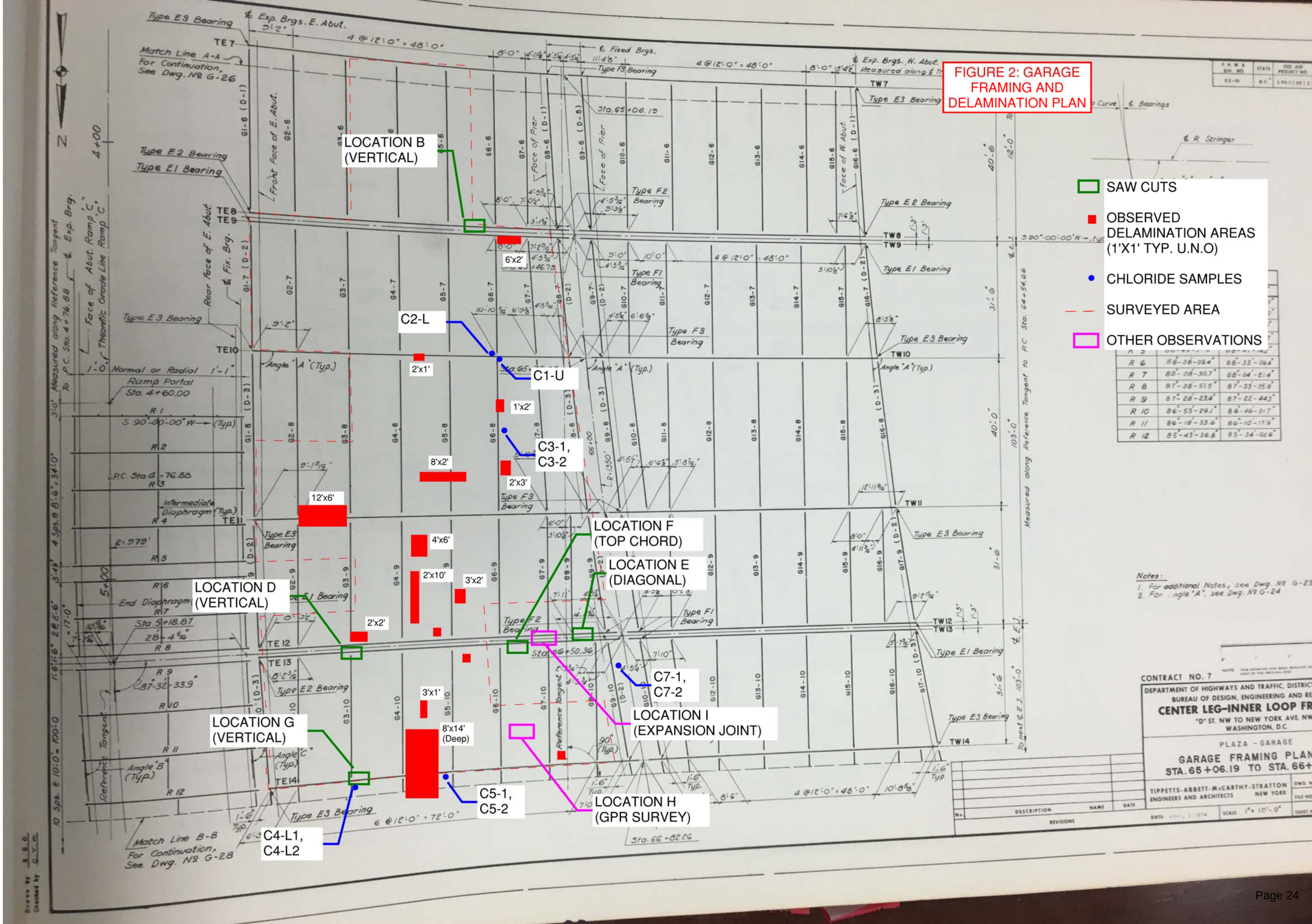


FIGURE 2: GARAGE FRAMING AND DELAMINATION PLAN
SAW CUTS
OBSERVED DELAMINATION AREAS (1'X1' TYP. U.N.O)
CHLORIDE SAMPLES
SURVEYED AREA
OTHER OBSERVATIONS
LOCATION B (VERTICAL)
LOCATION D (VERTICAL)
LOCATION E (DIAGONAL)
LOCATION F (TOP CHORD)
LOCATION G (VERTICAL)
LOCATION H (GPR SURVEY)
LOCATION I (EXPANSION JOINT)
C1-U
C2-L
C3-1, C3-2
C4-L1, C4-L2
C5-1, C5-2
C7-1, C7-2
6'x2'
2'x1'
1'x2'
8'x2'
2'x3'
12'x6'
4'x6'
2'x10'
3'x2'
2'x2'
3'x1'
8'x14' (Deep)
CONTRACT NO. 7
CENTER LEG-INNER LOOP FREE
PLAZA - GARAGE
GARAGE FRAMING PLAN
STA. 65+06.19 TO STA. 66+82
TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS AND ARCHITECTS NEW YORK
Match Line A-A For Continuation, See Dwg. № G-26
Match Line B-B For Continuation, See Dwg. № G-28

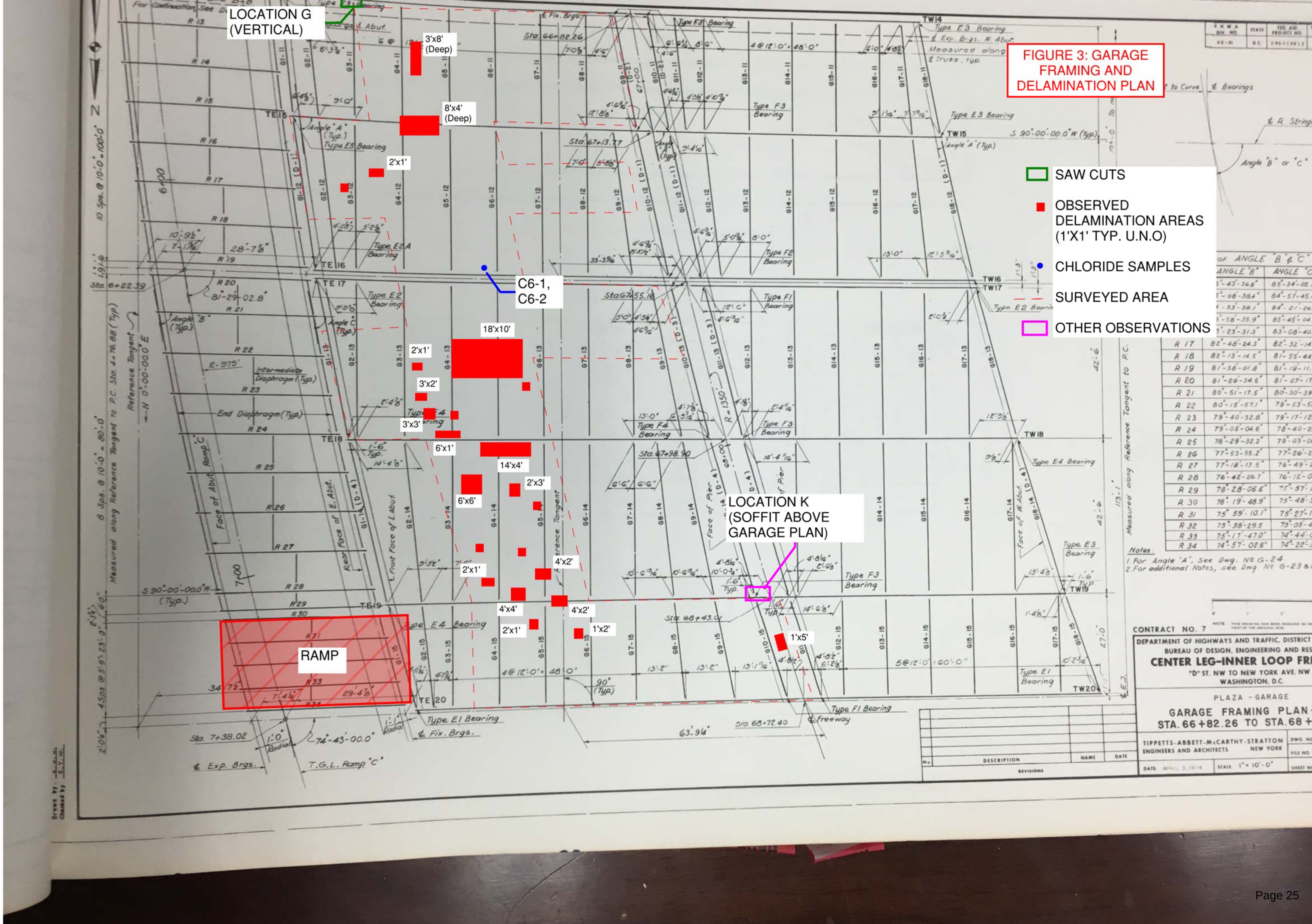
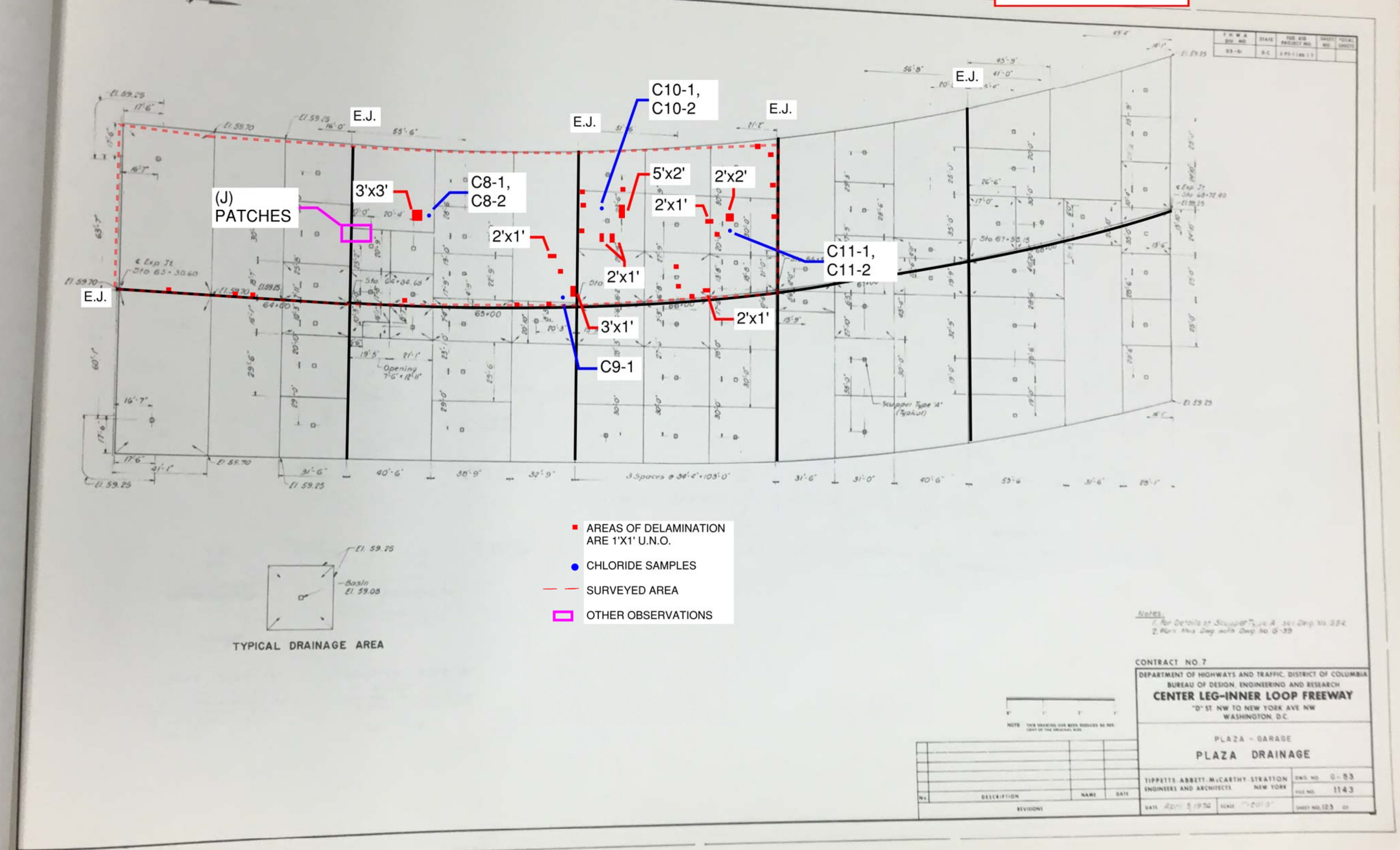


FIGURE 3: GARAGE FRAMING AND DELAMINATION PLAN
SAW CUTS
OBSERVED DELAMINATION AREAS (1'X1' TYP. U.N.O)
CHLORIDE SAMPLES
SURVEYED AREA
OTHER OBSERVATIONS
LOCATION G (VERTICAL)
3'x8' (Deep)
8'x4' (Deep)
2'x1'
C6-1, C6-2
18'x10'
2'x1'
3'x2'
3'x3'
6'x1'
14'x4'
2'x3'
6'x6'
4'x2'
2'x1'
4'x4'
4'x2'
2'x1'
1'x2'
1'x5'
LOCATION K (SOFFIT ABOVE GARAGE PLAN)
RAMP
CONTRACT NO. 7
DEPARTMENT OF HIGHWAYS AND TRAFFIC, DISTRICT OF
BUREAU OF DESIGN, ENGINEERING AND RESEAR
CENTER LEG-INNER LOOP FREE
"D" ST. NW TO NEW YORK AVE. NW
WASHINGTON, D.C.
PLAZA - GARAGE
GARAGE FRAMING PLAN-II
STA. 66+82.26 TO STA. 68+7
TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS
NEW YORK

FIGURE 4: PLAZA DRAINAGE AND DELAMINATION PLAN



McMullan & Associates, Inc.

Appendix B

Chloride Test Results

CONCORR, Inc.
LABORATORY ANALYSIS
TOTAL CHLORIDE ION CONTENT OF CONCRETE SAMPLES

www.concorr.com

Client: McMullan & Associates, Inc.
1861 Wiehle Ave., Ste. 125
Reston, VA 20190

Project: #3767-L 200 K St. Garage

Total Chloride Ion Analysis Performed in Accordance with AASHTO T260

Total Chloride: ✓
Water Soluble: ☐

Date Test Performed: 8/30/2016
Date Results Reported: 8/30/2016

Note: pound per cubic yard calculated using unit weight of concrete as 3915 pcy.

Sample	Location	Depth	% by Wt. of Concrete	parts per million (ppm)	pounds per cubic yard (pcy)
1	C1-U	.5"-1.5"	0.002	15	0.06
2	C2-L	.5"-1.5"	0.002	22	0.09
3	C3-1	.5"-1.5"	0.211	2105	8.24
4	C3-2	2.5"-3.5"	0.004	41	0.16
5	C4-L1	.5"-1.5"	0.499	4994	19.55
6	C4-L2	2.5"-3.5"	0.009	85	0.33
7	C5-1	.5"-2.25"	0.043	427	1.67
8	C5-2	3"-4"	0.003	34	0.13
9	C6-1	.5"-1.5"	0.222	2219	8.69
10	C6-2	2.5"-3.5"	0.006	59	0.23
11	C7-1	.5"-1.5"	0.003	34	0.13
12	C7-2	2.5"-3.5"	0.008	80	0.31
13	C8-1	.5"-1.5"	0.010	103	0.40
14	C8-2	2.5"-3.5"	0.004	43	0.17
15	C9-1	.5"-1.5"	0.004	37	0.14
16	C10-1	.5"-1.5"	0.003	29	0.11
	cont.				

CONCORR, Inc.
LABORATORY ANALYSIS
TOTAL CHLORIDE ION CONTENT OF CONCRETE SAMPLES

www.concorr.com

Client: McMullan & Associates, Inc.
1861 Wiehle Ave., Ste. 125
Reston, VA 20190

Project: #3767-L 200 K St. Garage

Total Chloride Ion Analysis Performed in Accordance with AASHTO T260

Total Chloride: ✓

Water Soluble: ☐

Date Test Performed: 8/30/2016

Date Results Reported: 8/30/2016

Note: pound per cubic yard calculated using unit weight of concrete as 3915 pcy.

Sample	Location	Depth	% by Wt. of Concrete	parts per million (ppm)	pounds per cubic yard (pcy)
17	C10-2	2.5"-3.5"	0.002	24	0.09
18	C11-1	.5"-1.5"	0.012	123	0.48
19	C11-2	2.5"-3.5"	0.002	20	0.08

CONCORR, Inc. 45710 Oakbrook Court, Suite 160, Sterling, VA 20166-7223 Ph: 571-434-1852 fax:571-434-1851