Structural Investigations & Geotech Report

The following City of Austin documents are included in this Appendix:

All Stations

1. **AFD 3 and AFD 22 / EMS 12 Summary Report - Ph. 1**
2. **AFD 3 and AFD 22 / EMS 12 Summary Report - Ph. 2**
3. **City Engineer letter from Forensic Study**
4. **AFD 3 and AFD 22 / EMS 12 Geotech Report**

CTLGROUP

Austin, TX Office:

3737 Executive Center Drive, Suite 255
Austin, TX 78731-1633
P: 512-219-4075 F: 512-219-4077

May 12, 2017

Mr. Karim Helmi
City of Austin – Public Works Department
Quality Management Division
105 Riverside Drive, Suite 100
Austin, TX 78704

Phone: 512-974-6539
Email: Karim.Helmi@austintexas.gov

Summary Report
Structural Floor System Capacity Assessment – Austin Fire Department
Fire Station No. 3 – 201 W. 30th St., Austin, TX
Fire Station No. 22 – 5309 E. Riverside Dr., Austin, TX
CTLGroup Project No. 231701

Dear Mr. Helmi:

As authorized by the City of Austin (COA), CTLGroup has completed an as-built assessment of the garage-area structural floor systems at Austin Fire Department Stations No. 3 and No. 22. The Austin Fire Department plans to operate certain vehicles in and out of each fire station garage and is concerned about the weight of the vehicles. The COA requested that the floor systems be evaluated to determine if their structural capacities are adequate to support the loads from these vehicles.

As part of this work, CTLGroup performed an on-site evaluation of each fire station that consisted of the following:

- Documenting the layouts and dimensions of the garage areas, including the structural elements of the floor systems
- Performing ground penetrating radar (GPR) scans to evaluate member/slab thicknesses, the general layout of embedded reinforcing steel, and to estimate the concrete cover over embedded reinforcing steel in a limited number of representative areas
- Performing localized concrete removal in a limited number of representative areas to verify GPR findings and evaluate reinforcing steel bar dimensions through direct measurements
- Extracting concrete core samples in a limited number of representative areas

In addition to the on-site evaluations, compressive strength tests were performed by CTLGroup's laboratory on the extracted core samples. Based on the results of our on-site evaluations and laboratory testing, structural analyses were performed on the garage-area structural floor systems. This report summarizes CTLGroup's findings.

ONSITE EVALUATIONS

CTLGroup's site visits were conducted on February 22-24, 2017 (Fire Station No. 22) and March 6-7, 2017 (Fire Station No. 3). The following CTLGroup staff members were present during the site visits: Bradley East, P.E. and Jon Poole, Ph.D., P.E. Various Austin Fire Department personnel were present during CTLGroup's site visits.

NON-DESTRUCTIVE TESTING AND LOCALIZED DESTRUCTIVE TESTING

Ground Penetrating Radar (GPR) is a nondestructive test method that employs high-frequency electromagnetic energy that can assess a variety of characteristics when applied to concrete structures. GPR surveys performed on concrete elements allow for the detection of embedded objects (steel reinforcement, prestressing/post-tensioning strand, conduit, and other embedded items), material interfaces, and internal discontinuities such as voiding. The technique involves the use of a high-frequency radar antenna which transmits electromagnetic radar pulses along a longitudinal scan at the surface of a structural element. Electromagnetic signals are optically reflected from material interfaces of varying dielectric constant along the propagation path of the wave. The reflected signals are collected by the antenna, amplified and displayed for subsequent interpretation.

GPR is commonly used to determine the location and depth of reinforcing bars in concrete structures. The contrast between the electromagnetic properties of embedded steel and that of cured concrete provides a distinct direct reflection from the reinforcing bars. The magnitude and phase of these reflections are analyzed to determine the location of the reinforcement.

During CTLGroup's site visit, GPR scans were performed on the structural floor systems at each fire station. The depth and distribution of the lateral, longitudinal, and/or vertical reinforcement were noted. A StructureScan Mini HR with a 2600 MHz antenna manufactured by Geophysical Survey Systems, Inc. (GSSI) was used on this project.

Localized destructive testing was also performed on the structural floor systems at each fire station at representative locations. Concrete was removed using a hammer drill with a chipping bit to confirm cover depths and determine steel reinforcing bar sizes. Following completion of the localized destructive testing, the chipped concrete was patched by CTLGroup using a non-shrink grout material.

The as-built reinforcing details are discussed further in the **General Observations and As-Built Findings** section of this report.

CONCRETE CORING

Core samples were extracted from the structural floor systems at each fire station by Texas Cutting and Coring, L.P. Cores were obtained for compressive strength testing. Several cores were also purposely taken through steel reinforcing bars to further confirm cover depths and bar sizes.

The cores were extracted in general accordance with ASTM C42[1]. The core locations are shown on the plan views of each garage in **Appendix A**. At Fire Station No. 22, cores were taken vertically (top down) through the topping slab and joists from the interior of the garage. Cores were also taken horizontally through the beams from within the crawlspace. The joist/slab cores measured nominal 2.0 in. or 1.5 in. diameter. The beam cores measured nominal 4.0 in. diameter. At Fire Station No. 3, all cores were taken vertically (top down) through the slab and beams from the interior of the garage. The slab/beam cores measured nominal 3.0 in. diameter. The core holes were patched by CTLGroup using a non-shrink grout material after coring operations were completed.

The concrete core samples were placed in plastic bags and packaged for shipment. The cores were shipped to CTLGroup's Skokie, IL Laboratory at the completion of each on-site assessment. Note that the time from when the cores were extracted to the time of testing varied from the requirements in ASTM C42 (testing no more than 7 days after extraction). It is CTLGroup's opinion that this variance does not significantly affect the findings in this report.

GENERAL OBSERVATIONS AND AS-BUILT FINDINGS

The interiors and exteriors of each Fire Station were visually evaluated. The undersides of the structural floor systems were also evaluated from within the crawlspaces beneath each Fire Station. The crawlspaces were accessible through entry at the exterior of each building.

Fire Station No. 22

CTLGroup was provided with Structural Sheets S1 and S2 for Fire Station No. 22. Included on the sheets were structural notes, plan views, and cross-section details of various elements. It was found that the layout and subsequent plan dimensions of Fire Station No. 22 generally matched the information on the plans. For reference purposes, Structural Sheets S1 and S2 can be found in **Appendix A** at the end of this report.

For directional references in this report, the front elevation of Fire Station No. 22 faced **east**. The garage structural floor system consisted of precast reinforced concrete joists that spanned between pier-supported reinforced concrete beams. The joists were oriented north-south and were bearing into the sides of the beams. The beams at the garage area were situated along the north and south ends of the garage and along the middle of the garage. The finished floor of the garage consisted of a concrete topping slab, which had been placed over the joists. General views of the garage from both the interior and crawlspace can be seen in Figures B1 and B2 in **Appendix B**.

The beams were generally rectangular in shape and it was found that the beam dimensions and steel reinforcing details generally matched the measurements/information given on Structural Sheets S1 and S2. Refer to **Appendix A** for cross-section details of the beams and the beam schedule.

---

[1] ASTM C42 "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete"

To summarize, the north beam had cross-section dimensions of 14 in. by 36 in. The reinforcement in the north beam consisted of four (4) #9 bars along both the top and bottom of the member. The middle beam had cross-section dimensions of 48 in. by 23 in. The reinforcement in the middle beam consisted of 10 #8 bars along both the top and bottom of the member. The south beam had cross-section dimensions of 36 in. by 23 in. The reinforcement in the south beam consisted of eight (8) #8 bars along both the top and bottom of the member.

The joists were inverted U-shaped. The overall cross-section dimensions of the joists measured approximately 13.75 in. tall by 31.5 in. wide. The top flange thickness measured approximately 1.5 in. The side webs tapered in thickness. The top of the webs (directly beneath the flange) measured approximately 4.25 in. thick. The bottom of the webs measured approximately 3.5 in. thick. The topping slab over the joists measured approximately 3.5 in. thick. A cross-section drawing of the joists can be found in **Appendix A**.

The reinforcement in the joists consisted of a 10 gauge 6x6 welded wire reinforcement (WWR) which was located at mid-height of the top flange. Steel reinforcing bars were also detected in the webs. This includes one (1) #4 bar in each web approximately 2 in. from the top of the member and one (1) #6 bar in each web approximately 0.75 in. from the bottom of the member.

Cracks were observed in the topping slab, as viewed from the interior of the garage (Figures B3 and B4). Cracks were generally evenly spaced transverse to the beams in the north-south direction at approximately 5-6-ft. on-center. Longitudinal cracks were also observed in the topping slab. The longitudinal cracks were situated at mid-span between the beams and approximately above both vertical edges of the middle beam. The largest crack widths measured approximately 0.05 in. The approximate locations of the cracking in the topping slab are shown on the plan view of the garage in **Appendix A**.

Visible portions of the beams and joists were in good condition, as viewed from within the crawlspace. No cracks were found in the beams or joists.

Fire Station No. 3

For directional references in this report, the front elevation of Fire Station No. 3 faced **north**. As of this writing, no construction drawings were found for this fire station. As a result, the plan layout and dimensions of the garage area, including the locations of the structural floor system elements, were identified during CTLGroup's site visit. The plan view of the garage area at Fire Station No. 3 can be found in **Appendix A**. General views of the garage from both the interior and crawlspace can be seen in Figures B5 and B6.

The garage structural floor system consisted of a monolithic cast-in-place reinforced concrete slab over pier-supported reinforced concrete beams. Beams were oriented north-south approximately beneath the middle of each garage bay, and along the west end of the garage area. The cross-section dimensions of the middle beams measured approximately 5 in. deep (as measured from the underside of the slab) by 36 in. wide. The cross-section dimensions of the west beam measured approximately 14.5 in. deep (as measured from the underside of the slab) by 12 in. wide.

Based on CTLGroup's GPR scans and localized destructive testing, the longitudinal reinforcing steel in the middle beam consisted of five (5) #8 bars in the bottom of the member approximately 1.5 in. from the bottom surface. Six (6) #8 bars were also detected in the top of the member approximately 2.5 in. from the top surface. The longitudinal reinforcing steel in the west beam consisted of three (3) #6 bars in the bottom of the member approximately 1.5 in. from the bottom surface. The west perimeter wall of the garage was situated directly on top of the west beam. As a result, CTLGroup was unable to confirm the reinforcing layout and size in the top of the west beam.

The garage floor consisted of a steel reinforced concrete slab. The slab thickness throughout the garage area measured approximately 5.5 in. thick. There was no topping slab placed over the structural slab system.

Based on CTLGroup's GPR scans and localized destructive testing, longitudinal and transverse steel were detected in the concrete slab. The longitudinal steel (i.e. north-south steel) consisted of #3 bars spaced approximately 12 in. on-center. The longitudinal steel was located at a depth of approximately 4 in. from the top surface of the slab. Transverse steel consisted of a bottom layer of continuous #4 steel bars spaced approximately 8 in. on-center. The bottom layer of transverse steel was located a depth of approximately 4.5 to 4.75 in. from the top surface of the slab. A top transverse layer of reinforcement was also detected in the slab over the beams. The top reinforcement in the slab was not continuous across the full width of the garage. The top layer of transverse steel was located approximately 1.5 in. from the top surface and consisted of #4 bars spaced approximately 8 in. on-center.

The cross-section details of the beams and slab can be found in **Appendix A**.

Cracks were observed in the top of the slab, as viewed from the interior of the garage (Figure B7). Cracks were oriented transverse across the floor slab. The spacing of the transverse cracks varied throughout. Longitudinal cracking was also observed. The longitudinal cracks were generally aligned above the sides/edges of both beams. The largest crack widths measured approximately 0.06 in.

Transverse cracks were also observed in the underside of the slab, as viewed from within the crawlspace (Figure B8). The cracks appeared to originate adjacent to the column-to-beam supports. The visible portions of the beams were in good condition, as viewed from within the crawlspace. A diagonal crack was observed in the west beam above the first column from the north end of the garage (Figure B9). No other cracks were found in the beams.

Horizontal cracks were observed in the columns (Figure B10). The cracks were generally located in the top portions of the columns, beneath the underside of the beams.

Horizontal cracks were found in the foundation walls directly beneath the middle beams at both the north and south ends of the west garage bay and at the south end of the east garage bay (Figure B11). Diagonal cracks were also observed in the walls at these locations adjacent to the beams (refer to B11). A similar diagonal crack was found in the foundation wall at the northwest corner of the garage. Additional cracks were observed in the foundation walls. This includes the following: vertical cracks in the east wall directly above the first and second piers from the northeast corner, vertical crack at the northwest corner directly below the north end of the west beam, and a vertical crack at the southwest corner.

The approximate locations of the above discussed cracking in the slab, columns and foundation walls can be found on the plan view of the garage in **Appendix A**.

The underside of the slab was spalled at several locations (Figure B12). At several spalled areas, the reinforcing steel was exposed and was visibly corroded/rusted.

SUMMARY OF LABORATORY TESTING

Compressive strength testing was performed on select core samples from each fire station in general accordance with ASTM C42. At Fire Station No. 22, compressive strength tests were performed on samples C1 (topping slab), C2 (topping slab), C4 (topping slab and joist), C4A (topping slab and joist), C5 (beam), C6 (beam), and C7 (beam). At Fire Station No. 3, compressive strength tests were performed on samples C1 (slab/beam), C2 (slab/beam), and C3 (slab/beam). Prior to testing, each core sample was capped with a sulfur capping compound (meeting the requirements of ASTM C617[2]), and placed in bags for 5 days.

The average concrete core compressive strengths of the structural floor system elements at each fire station are summarized below in Table 1. The average core strengths in Table 1 were used in the structural analyses of the fire station floor systems (unless noted otherwise in the **Structural Analyses** section of this report).

Table 1 – Summary of average core compressive strengths

Element		Average Core Compressive Strength (psi)
Fire Station No. 22	Topping Slab	5188
	Joists	6850
	Beams	5300
Fire Station No. 3	Slab/Beams	3767

CTLGroup's compressive strength testing reports can be found in **Appendix C** at the end of this report.

STRUCTURAL ANALYSES

Structural analyses were performed on the floor systems at both fire station garages. The purpose of the structural analyses was to determine the capacities of the structural floor system elements and to determine if the floors are adequate to support the anticipated vehicular loads. The capacities of the structural floor system elements were calculated in general accordance with ACI 318[3] ultimate strength design method (USD).

The loads considered in the structural analyses included the self-weight of the concrete elements and the gross weight of the fire trucks. Two (2) different fire trucks were considered for the analysis of each fire station floor system. The fire trucks considered for the analysis of Fire Station No. 3 included a Pierce Impel Pumper (Job No. 25403) and a Pierce 105' Heavy Duty Aerial Ladder with water tank (Job No. 27566). The fire trucks considered for the analysis of Fire Station No. 22 included a Pierce Impel Pumper (Job No. 25403) and a Pierce 105' Heavy Duty Aerial Ladder without water tank (Job No. 13122). Provided specifications for these vehicles, including dimensions and weights that were used in our analyses can be found in **Appendix D**.

---

[2] ASTM C617 "Standard Practice for Capping Cylindrical Concrete Specimens"
[3] ACI 318 "Building Code Requirements for Structural Concrete"

Several liberal assumptions were made during the analyses. It should be noted that these liberal assumptions were purposely made to study whether the floor systems could support the new vehicular loads in a favorable condition. These liberal assumptions generally decreased the calculated demand on the elements and increased the calculated capacity of the elements. These liberal assumptions will be replaced by conservative assumptions during retrofit design, if needed. Some of the liberal assumptions are as follows:

- The average concrete core strengths (as determined by CTLGroup laboratory testing) for the joists and slab at Fire Stations Nos. 22 and 3, respectively, were used in the analyses. No statistical adjustment, such as that in accordance with ACI 214.4R[4], was performed on the core strength values. Considering the scatter of test data, the usable concrete strength for structural analysis is smaller than the average value. While concrete strength does not affect the moment capacity of a flexural member considerably, it affects the shear capacity of the flexural member more significantly. Therefore, using average concrete strength is a liberal assumption.

- Although vehicular loading was considered, an impact factor was not included in our analysis. Considering that even a gentle brake could result in some impact, this is a liberal assumption.

- At Fire Station No. 22, composite elastic behavior was assumed between the topping slab and joists. As a result, the vehicular loading was distributed amongst several adjacent joists, depending on the wheel locations. The load distribution was determined using elastic finite element analysis. Considering that concrete cracking would prevent as much distribution as calculated by elastic analysis, this is a liberal assumption.

- At Fire Station No. 22, adequate shear transfer was assumed between the topping slab and joists. As a result, the joists and slab were assumed to provide composite action. The composite member was favorably assumed to have a concrete compressive strength equal to the average of the core strengths of the joists (approximately 6.85 ksi). For comparison purposes, the average of the core strengths of the topping slab was 5.19 ksi.

- The rear axle of the Aerial Ladder truck is a tandem axle and therefore consists of two (2) sets of parallel tires. During our analysis of the slab at Fire Station No. 3, loading from only one (1) set of the tandem rear axle tires was considered. This is a liberal assumption that reduces the demand on the slab. It should be noted that the per tire weight of the rear wheels of both the Aerial Ladder and Impel Pumper vehicles are equal. The rear axle of the Impel Pumper vehicle is not a tandem axle and because only one (1) set of the tandem rear axle tires was considered for the Aerial Ladder vehicle, the calculated demands placed on the slab at Fire Station No. 3 were identical for both vehicles.

Some additional assumptions are as follows:

- As previously discussed, the joists at Fire Station No. 22 were bearing into the sides of the beams. CTLGroup found no evidence to indicate that there were any tie-bars (or similar) connecting the joists to the beams. As a result, the joists were assumed to be simply supported.

---

[4] ACI 318 "Building Code Requirements for Structural Concrete"

- A vertical WWR was found in the webs of the joists at Fire Station No. 22. CTLGroup found no evidence to indicate that the vertical mesh was tied to the longitudinal steel. As a result, it was assumed that the vertical mesh in the joists did not provide shear resistance.
- Using AASHTO Standard Specifications for Highway Bridges as a guide, it was assumed that the effective slab width at Fire Station No. 3 was approximately 8-ft.

It was determined during our analyses that the precast joists at Fire Station No. 22 and the floor slab at Fire Station No. 3 do not have the necessary capacity to support the appropriate Aerial Ladder truck or the Impel Pumper truck. The joists and slab were analyzed for both shear and moment/flexure. Tables 2 to 4 below summarize the capacities of the joists and slab, the load demands placed on these elements, and the Demand Capacity Ratios (DCR). It should be noted again that these DCRs are for a set of favorable assumptions and the actual DCRs would be higher.

Table 2 – Joist and Slab Capacities at Fire Station Nos. 22 and 3

Element		Capacities		
		Shear (kips)	Positive Moment (k-ft)	Negative Moment (k-ft)
Fire Station No. 22	Joists	13.6	52.4	N/A
Fire Station No. 3	Slab	34.2	34.2	26.9

Table 3 – Joist and Slab Demands at Fire Station Nos. 22 and 3

Element		Demands					
		Impel Pumper			Aerial Ladder		
		Shear (kips)	Positive Moment (k-ft)	Negative Moment (k-ft)	Shear (kips)	Positive Moment (k-ft)	Negative Moment (k-ft)
Fire Station No. 22	Joists	14.0	47.3	N/A	19.8	66.8	N/A
Fire Station No. 3	Slab	32.0	54.6	52.0	32.0	54.6	52.0

Table 3 – Joist and Slab Demands Capacity Ratios (DCR) at Fire Station Nos. 22 and 3

Element		Demands					
		Impel Pumper			Aerial Ladder		
		Shear (kips)	Positive Moment (k-ft)	Negative Moment (k-ft)	Shear (kips)	Positive Moment (k-ft)	Negative Moment (k-ft)
Fire Station No. 22	Joists	1.03	0.90	N/A	1.46	1.27	N/A
Fire Station No. 3	Slab	0.94	1.60	1.93	0.94	1.60	1.93

The City of Austin has requested a remedial design repair be developed to strengthen the existing floor systems, because of the analyses discussed above showing that the joists at Fire Station No. 22 and the floor slab at Fire Station No. 3 lack adequate strength. This work will be performed as part of Phase 2 of this project. As of the date of this report, the Phase 2 proposal is being prepared and it is anticipated that it will be submitted to the City of Austin for review by the week of May 19, 2017. Because analyses indicated that the joists and slab are inadequate, a hold was placed on further analyses. As a result, the beams at each fire station have not yet been evaluated. Since additional analyses will be required as part of Phase 2, analysis of the beams will be included as part of this work.

DISCUSSION AND RECOMMENDATIONS

Based on CTLGroup's as-built assessment and subsequent structural analyses, the joists and slab at Fire Station Nos. 22 and 3, respectively, lack adequate strength for the anticipated vehicular loads. The City of Austin was notified of these findings via phone after the completion of each analysis. An email was also sent to the City of Austin regarding the deficiency of the slab at Fire Station No. 3 on April 26, 2017. It is strongly recommended that the aerial ladder trucks not be operated in and out of the garages at Fire Station Nos. 22 and 3. It is also strongly recommended that the impel pumper not be operated in and out of the garage at Fire Station No. 3.

It is our understanding that the aerial ladder truck was in use at Fire Station No. 3 (prior to our findings), so risk of catastrophic failure (i.e. collapse) is likely minimal. It is plausible that the truck alignment when entering the bay of the garage generally forces the alignment of the truck tires directly over the supporting beams. This likely lessens the loading on the slab. However, misalignment of the truck could cause a failure of the slab. As a safety precaution, this vehicle should no longer be parked in this fire station.

The longitudinal cracking in the top of the slab at Fire Station No. 3 may have been caused by negative moment flexural cracking at the slab-to-beam interface, which is further evidence that the slab at this garage may have been overloaded. However, restraint shrinkage may have also caused/contributed to the longitudinal cracking. Restrained shrinkage of the slab is also likely the cause of the observed transverse cracking

The cracking at Fire Station No. 22 was limited to the topping slab. Considering the generally even spacing of the cracks, this cracking has most likely been caused by drying shrinkage and/or thermal movement. This cracking is not a structural concern, especially since no cracks were found in the joists or beams when viewed from within the crawlspace.

Based on CTLGroup's structural analyses and on-site observations, we propose the following recommendations:

- Prior to preparing a remedial repair design, petrographic examination should be performed on the cores that were taken during our initial site visits to ensure that the existing concrete is of adequate quality. This includes determining the carbonation depths. The spalled and corroded reinforcement on the underside of the slab at Fire Station No. 3 is indicative of damage due to carbonation. Based on the petrographic findings, concrete repairs may also be needed in addition to the strengthening of each garage floor system.

- To reduce the scatter of core strength data which would allow for the use of a higher compressive strength for the concrete during retrofit design, a few additional core samples should be taken and tested from each fire station.
- The floor systems at each garage needs to be strengthened so that the aerial ladder and impel pumper vehicles could be safely operated in and out of the garages. Some potential repair options at each fire station are discussed below. It should be noted that these are preliminary options that are subject to change prior to issuance of our Phase 2 proposal. It should also be noted that the below repairs take into consideration that there is minimal clearance in each garage crawlspace and that it might be difficult to get large materials/equipment into the crawlspace at Fire Station No. 22.
  - **Fire Station No. 22:**

    Installation of fiber reinforced polymer (FRP) wrap at shear and positive moment regions of the joists may be considered. FRP is a common fabric material (typically glass or carbon) that is impregnated with epoxy and bonded to the concrete member to provide additional strength.

    Additional options such as external steel plate bonding to joists, concrete/steel jacketing of joists, casting an additional concrete overlay will be studied prior to issuance of our Phase 2 proposal.

    Depending on the repair option, anchors might also be driven into both the joists and slab layers to ensure composite action.

    Additionally, the cracks in the topping slab should be filled by epoxy injection and/or gravity feed to enhance durability.
  - **Fire Station No. 3:**

    Installation of FRP wraps at positive moment regions and FRP bars at negative moment regions of the slab may be considered.

    Additional options such as external steel plate bonding to beams, concrete/steel jacketing of beams, casting an additional concrete overlay will be studied prior to issuance of our Phase 2 proposal.

    Conventional steel reinforcement could also be added to the slab in the transverse direction at the negative moment regions. This will require that strips of concrete be removed from the top of the slab so that the reinforcement can be placed. The additional imbedded reinforcement would then be grouted or epoxied into the slab.
- The observed cracking in the foundation walls adjacent to the north and south ends of the beams at Fire Station No. 3 are concerning. These cracks should be further evaluated and the wall supports beneath the ends of the beams should be analyzed to ensure that they are adequate to support the anticipated loads. The foundation walls were not included as part of CTLGroup's Phase 1 scope.

CLOSING

Thank you for the opportunity to assist you on this project. Please do not hesitate to let me know if you have any questions or concerns, or need any additional information.

STATE OF TEXAS
BRADLEY L. EAST
107268
LICENSED
PROFESSIONAL ENGINEER
5.12.17

Bradley L. East, M.S., P.E.
Senior Engineer
BLEast@CTLGroup.com
P. 512-220-2137
M. 512-971-3911

cc: Hamid Lotfi, Ph.D., P.E. (CA), CTLGroup
Eric Vanduyne, P.E., S.E. (IL), CTLGroup
Jon Poole, Ph.D., P.E. (TX), CTLGroup

Appendix A

Plan Views and Cross-Section Details

1 GARAGE FLOOR PLAN
3/16" = 1'-0"

LEGEND:

- SLAB TOPSIDE CRACKING
- SLAB UNDERSIDE CRACKING
- COLUMN/FOUNDATION WALL CRACKING (U.N.O.)
- C# CORE LOCATION

CTLGROUP
CONSTRUCTION TECHNOLOGY LABORATORIES
ENGINEERS & CONSTRUCTION TECHNOLOGY CONSULTANTS

3737 Executive Center Dr., Suite 255
Austin, TX 78731-1633
Phone: 512-219-4057 Fax: 512-219-4077
www.CTLGroup.com

CTLGroup COA No.: F-3849

Austin Fire Department - Station #3

201 W. 30th Street
Austin, TX 78701

Structural Floor System Capacity Assessment

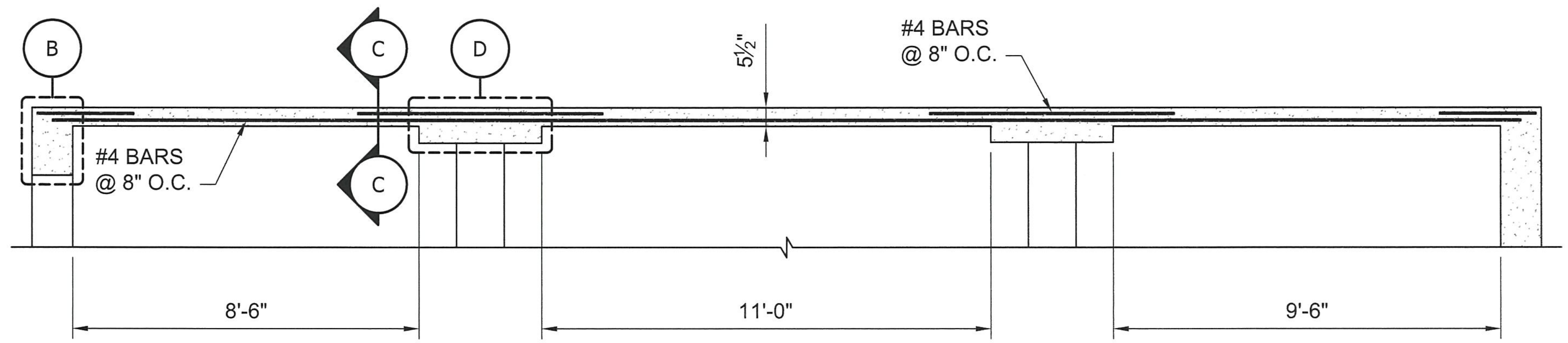
ISSUE

No.	Description	Date

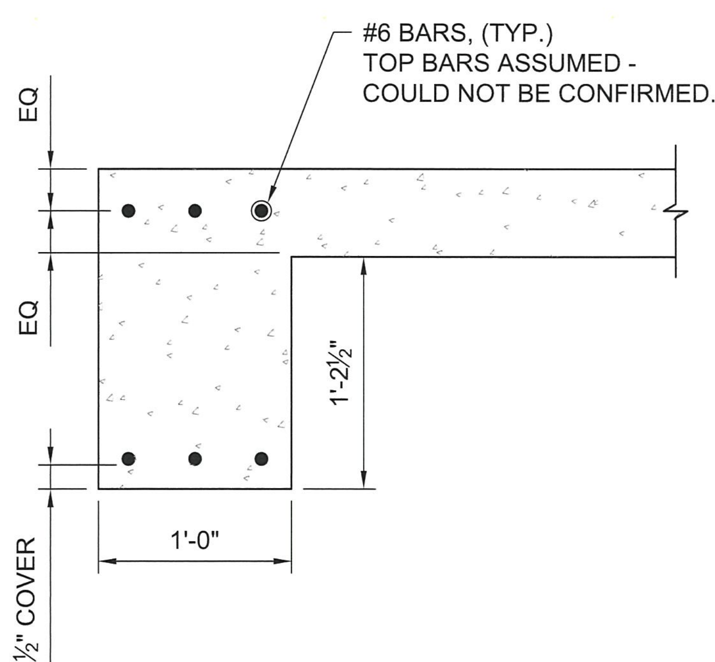
CTLGroup No.: **231701**
Drawn: **SEH**
Checked: **BLE**
Date: **12MAY17**
Scale: **AS SHOWN**

Title:
Garage Floor Plan

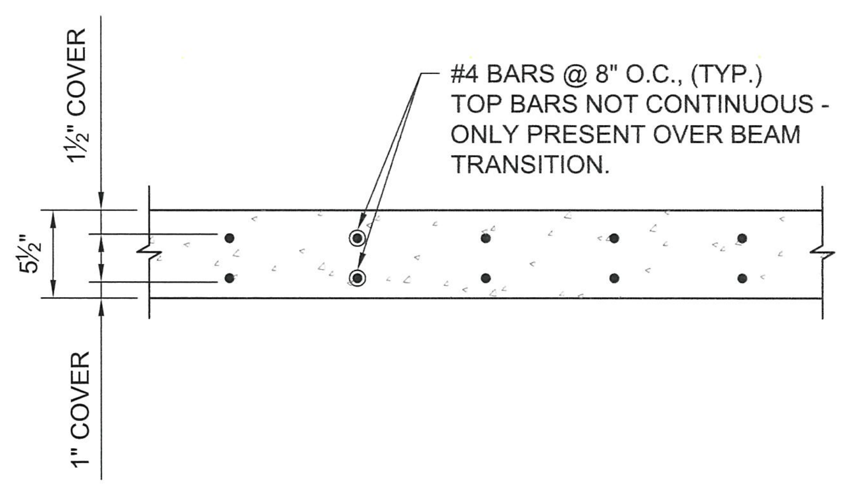
Sheet No.:
1



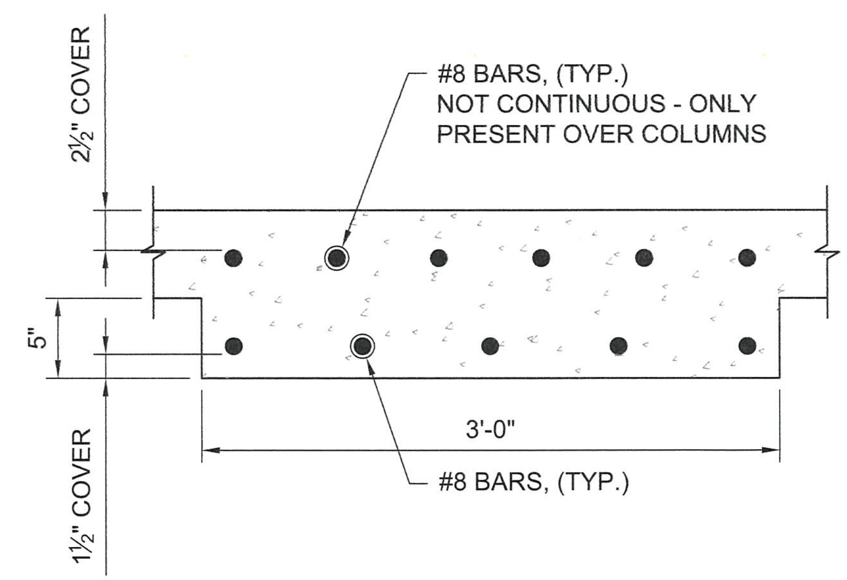
1 SECTION A-A
1/4" = 1'-0"



2 DETAIL B
1" = 1'-0"



3 SECTION C-C
1" = 1'-0"



3 DETAIL D
1" = 1'-0"

CTLGroup

CONSTRUCTION TECHNOLOGY
LABORATORIES
ENGINEERS & CONSTRUCTION
TECHNOLOGY CONSULTANTS

3737 Executive Center Dr., Suite 255
Austin, TX 78731-1633
Phone: 512-219-4057 Fax: 512-219-4077
www.CTLGroup.com

CTLGroup COA No.: F-3849

Austin Fire Department - Station #3

201 W. 30th Street
Austin, TX 78701

Structural Floor System Capacity Assessment

ISSUE

No.	Description	Date

CTLGroup No.: **231701**
Drawn: **SEH**
Checked: **BLE**
Date: **12MAY17**
Scale: **AS SHOWN**

Title:
Garage Floor Plan

Sheet No.:
2

Appendix B

CTLGroup Photographs

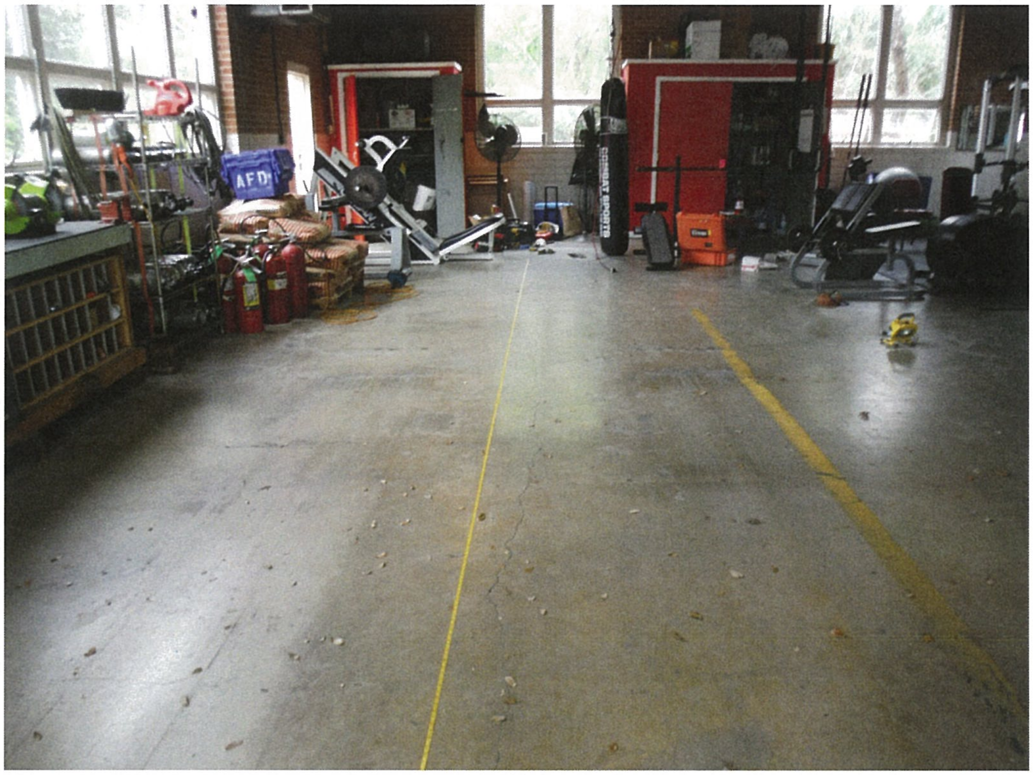


Figure B5 – General view at interior of Fire Station No. 3 garage

Figure B6 – General view of crawlspace beneath Fire Station No. 3 garage

Figure B7 – Typical cracking in top of slab at Fire Station No. 3 garage

Figure B8 – Typical cracking in underside of slab at Fire Station No. 3 garage

Figure B9 – Crack in west beam at north end of garage at Fire Station No. 3

Figure B10 – Typical horizontal crack in column at Fire Station No. 3

Figure B11 – Typical cracks in foundation wall at ends of middle beams at Fire Station No. 3

Figure B12 – Spalled concrete and corroded reinforcement at underside of slab at Fire Station No. 3

Appendix C

CTLGroup Compression Test Results

CTLGroup

Client:	**City of Austin**	CTLGroup Project No.:	**231701**
Project Name:	**Austin Fire Department Stations 3 & 22**	CTLGroup Project Mgr.:	**Bradley East**
	Structural Capacity Assessment	Analyst:	**WD, CA**
Contact:	**Karim Helmi**	Approved by:	**Anthony Bentivegna**
Submitter:	**Bradley East**	Date Analyzed:	**March 21, 2017**
Date Received:	**March 3, 2017**	Date Reported:	**March 28, 2017**

ASTM C42 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
Section 7: Cores for Compressive Strength

Specimen Identification

CTLGroup Identification	4402610-01	4402611-01	4402612-01
Client Identification	No. 3-C1	No. 3-C2	No. 3-C3
Date Core Obtained from the Field	2/23/17	2/23/17	2/23/17
Date end preparation was completed and core was placed in sealed bag	3/16/17	3/16/17	3/16/17
Date Core was Tested	3/21/17	3/21/17	3/21/17

Concrete Description

Nominal Maximum Aggregate Size, in.	3/4	3/4	3/4
Concrete Age at Test	~65 years	~65 years	~65 years
Moisture Condition at Test	Per Standard	Per Standard	Per Standard
Length of Core, As Drilled, in.	6	6 1/4	8 1/2
Orientation of Core Axis in Structure	Vertical	Vertical	Vertical
Cylinder End Preparation	Capped	Capped	Capped

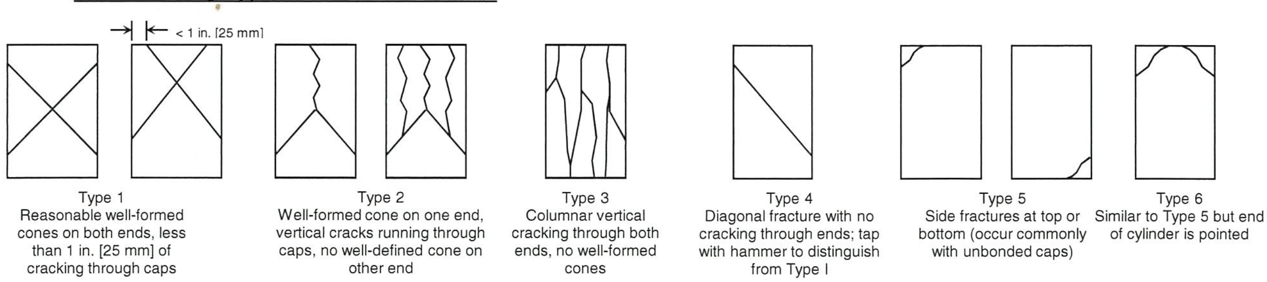
Concrete Dimensions

Diameter 1, in.	2.74	2.74	2.75
Diameter 2, in.	2.74	2.75	2.74
Average Diameter, in.	2.74	2.75	2.75
Cross-Sectional Area, in^2	5.90	5.94	5.94
Length Trimmed, in.	3.4	5.4	5.6
Length Capped, in.	3.5	5.5	5.7
Density, pcf	139	138	140

Compressive Strength and Fracture Pattern

Maximum Load, lb	22,748	29,779	16,066
Uncorrected compressive Strength, psi	3,860	5,010	2,700
Ratio of Capped Length to Diameter	1.29	1.99	2.08
Corrected Compressive Strength, psi	3,590	5,010	2,700
Fracture Pattern	Type 1	Type 1	Type 1

Schematic of Typical Fracture Patterns

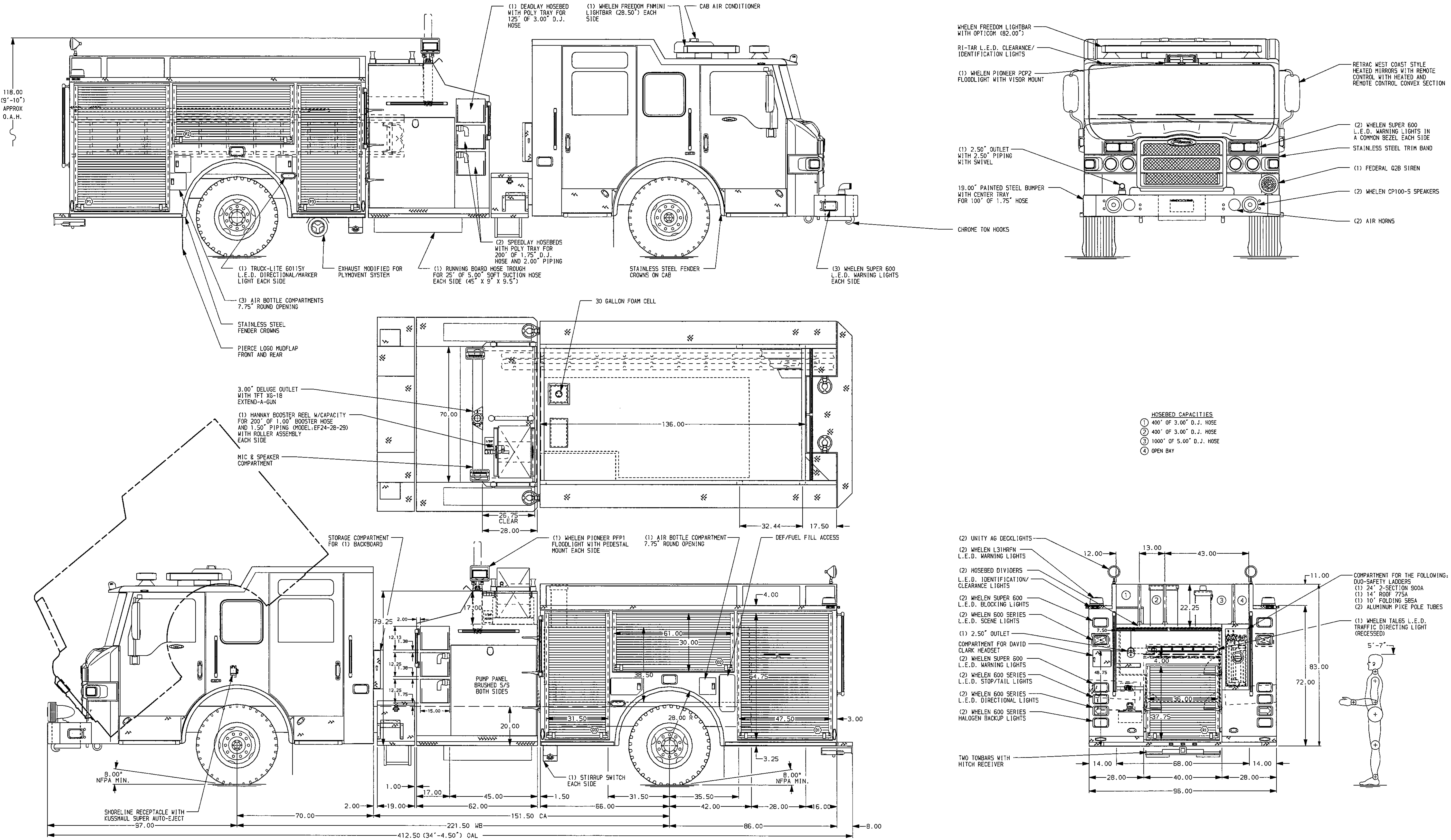


Notes:

1. This report may not be reproduced except in its entirety.

Appendix D

Fire Station Vehicle Drawings and Specifications



(STAINLESS STEEL BODY) (SIDE ROLL & FRONTAL IMPACT PROTECTION)

NOTE

DIMENSIONS SHOWN ARE APPROXIMATE AND ARE SUBJECT TO MINOR DEVIATIONS AS MAY OCCUR OR BE NECESSARY IN CONSTRUCTION.
MINOR DETAILS NOT SHOWN.

1. AIR INLET/OUTLET WITH DISCONNECT FITTING LOCATED ON DRIVER SIDE PUMP PANEL
2. AIR COMPRESSOR LOCATED UNDER CREW AREA SEAT
3. TWO ZIAMATIC AC-44 WHEEL CHOCKS SHIPPED LOOSE
4. MOUNTING BASES FOR ANTENNAS LOCATED PER SHOP ORDER
5. KUSSMAUL BATTERY CHARGER LOCATED IN CREW CAB SEAT RISER
6. BATTERY CHARGE INDICATOR LOCATED ON DRIVER SEAT WITH BRACKET
7. ONE FLOOR MOUNTED SLIDE-OUT TRAY IN COMPARTMENTS D1 AND R1
8. ONE ADJUSTABLE SLIDE-OUT TRAY IN COMPARTMENT D3
9. 4-WAY ALUMINUM ON THE FLOOR OF COMPARTMENT D3
10. ALUMINUM PEGBOARD WITH TRACKS IN COMPARTMENT D1
11. ONE ADJUSTABLE FULL WIDTH/DEPTH SHELF IN COMPARTMENTS P2 AND R1; TWO IN D1, D2, D3 AND P1; THREE IN P3
12. PERIMETER SCENE LIGHTS LOCATED PER SHOP ORDER
13. HUSKY FOAM SYSTEM RESERVOIR MOUNTED IN DRIVER SIDE OF PUMP HOUSE

REV	DATE	BY	CH
1	12JAN12	SHA	-

CHASSIS DATA	TITLE	1500-D-600P W/30 GAL. FOAM CELL, 152" BODY, DS FR FD (2)SPEEDLAYS, (1)DEADLAY, LADDERS BTWN TANK & S.SHT	
MAKE PIERCE	FOR	AUSTIN FIRE DEPARTMENT AUSTIN, TEXAS	
MODEL IMP7010	DWG NO.	25403AD	

Pierce MANUFACTURING INC.

JOB NO.	25403 01-02
SCALE 1:24	DATE
DRAWN BY SHI	03JAN12
CHECKED BY GRM	06JAN12
SHEET SIZE D	SHEET NO. 1 OF 1

PERFORMANCE BOND, NOT REQUESTED
A performance bond will not be included. If requested at a later date, one will be provided to you for an additional cost and the following will apply:

The successful bidder will furnish a Performance and Payment bond (Bond) equal to 100 percent of the total contract amount within 30 days of the notice of award. Such Bond will be in a form acceptable to the Owner and issued by a surety company included within the Department of Treasury's Listing of Approved Sureties (Department Circular 570) with a minimum A.M. Best Financial Strength Rating of A and Size Category of XV. In the event of a bond issued by a surety of a lesser Size Category, a minimum Financial Strength rating of A+ is required.

Bidder and Bidder's surety agree that the Bond issued hereunder, whether expressly stated or not, also includes the surety's guarantee of the vehicle manufacturer's Bumper to Bumper warranty period included within this proposal. Owner agrees that the penal amount of this bond will be simultaneously amended to 25 percent of the total contract amount upon satisfactory acceptance and delivery of the vehicle(s) included herein. Notwithstanding anything contained within this contract to the contrary, the surety's liability for any warranties of any type will not exceed three (3) years from the date of such satisfactory acceptance and delivery, or the actual Bumper to Bumper warranty period, whichever is shorter.

APPROVAL DRAWING
A drawing of the proposed apparatus will be prepared and provided to the purchaser for approval before construction begins. The Pierce sales representative will also be provided with a copy of the same drawing. The finalized and approved drawing will become part of the contract documents. This drawing will indicate the chassis make and model, location of the lights, siren, horns, compartments, major components, etc.

A "revised" approval drawing of the apparatus will be prepared and submitted by Pierce to the purchaser showing any changes made to the approval drawing.

ELECTRICAL WIRING DIAGRAMS
Two (2) electrical wiring diagrams, prepared for the model of chassis and body, will be provided.

IMPEL™ CHASSIS
The Pierce Impel is the custom chassis developed exclusively for the fire service. Chassis provided will be a new, tilt type custom fire apparatus. The chassis will be manufactured in the apparatus body builder's facility eliminating any split responsibility. The chassis will be designed and manufactured for heavy duty service, with adequate strength, capacity for the intended load to be sustained, and the type of service required. The chassis will be the manufacturer's first line tilt cab.

WHEELBASE
The wheelbase of the vehicle will be 221.5.

GVW RATING
The gross vehicle weight rating will be 46,500.

FRAME

The chassis frame will be built with two (2) steel channels bolted to five (5) cross members or more, depending on other options of the apparatus. The side rails will have a 13.38" tall web over the front and mid sections of the chassis, with a continuous smooth taper to 10.75" over the rear axle. Each rail will have a section modulus of 25.992 cubic inches and a resisting bending moment (rbm) of 3,119,040 in-lb over the critical regions of the frame assembly, with a section modulus of 18.96 cubic inches with an rbm of 2,275,200 in-lb over the rear axle. The frame rails will be constructed of 120,000 psi yield strength heat-treated .38" thick steel, with 3.50" wide flanges.

FRAME REINFORCEMENT

In addition, a mainframe inverted "L" liner will be provided. It will be heat-treated steel measuring 12.00" x 3.00" x .25". Each liner will have a section modulus of 7.795 cubic inches, yield strength of 110,000 psi, and rbm of 857,462 in-lb. Total rbm at wheelbase center will be 3,976,502 pounds per rail.

The frame liner will be mounted inside of the chassis frame rail and extend the full length of the frame.

FRONT NON DRIVE AXLE

The Oshkosh TAK-4® front axle will be of the independent suspension design with a ground rating of 19,500 lb.

Upper and lower control arms will be used on each side of the axle. Upper control arm castings will be made of 100,000-psi yield strength 8630 steel and the lower control arm casting will be made of 55,000-psi yield ductile iron.

The center cross members and side plates will be constructed out of 80,000-psi yield strength steel.

Each control arm will be mounted to the center section using elastomer bushings. These rubber bushings will rotate on low friction plain bearings and be lubricated for life. Each bushing will also have a flange end to absorb longitudinal impact loads, reducing noise and vibrations.

There will be nine (9) grease fittings supplied, one (1) on each control arm pivot and one (1) on the steering gear extension.

The upper control arm will be shorter than the lower arm so that wheel end geometry provides positive camber when deflected below rated load and negative camber above rated load.

Camber at load will be zero degrees for optimum tire life.

The ball joint bearing will be of low friction design and be maintenance free.

Toe links that are adjustable for alignment of the wheel to the center of the chassis will be provided.

The wheel ends must have little to no bump steer when the chassis encounters a hole or obstacle.

The steering linkage will provide proper steering angles for the inside and outside wheel, based on the vehicle wheelbase.

The axle will have a third party certified turning angle of 45 degrees. Front discharge, front suction, or aluminum wheels will not infringe on this cramp angle.

FRONT SUSPENSION
Front Oshkosh TAK-4™ independent suspension will be provided with a minimum ground rating of 19,500 lb.

The independent suspension system will be designed to provide maximum ride comfort. The design will allow the vehicle to travel at highway speeds over improved road surfaces and at moderate speeds over rough terrain with minimal transfer of road shock and vibration to the vehicle's crew compartment.

Each wheel will have torsion bar type spring. In addition, each front wheel end will also have energy absorbing jounce bumpers to prevent bottoming of the suspension.

The suspension design will be such that there is at least 10.00" of total wheel travel and a minimum of 3.75" before suspension bottoms.

The torsion bar anchor lock system allows for simple lean adjustments, without the use of shims. One can adjust for a lean within fifteen minutes per side. Anchor adjustment design is such that it allows for ride height adjustment on each side.

The independent suspension was put through a durability test that simulated 140,000 miles of inner city driving.

SHOCK ABSORBERS
Heavy-duty telescoping shock absorbers (KONI) will be provided on the front suspension.

OIL SEALS
Oil seals with viewing window will be provided on the front axle.

FRONT TIRES
The front tires will be Michelin 385/65R22.50 radials, 18 ply XFE wide base tread, rated for 19,840 lb maximum axle load and 75 mph maximum speed.

The tires will be mounted on 22.50" x 12.25" steel disc-type wheels with a ten (10)-stud, 11.25" bolt circle.

REAR AXLE
The rear axle will be a Meritor™, Model RS-26-185, with a capacity of 27,000 lb.

TOP SPEED OF VEHICLE
A rear axle ratio will be furnished to allow the vehicle to reach a top speed of 68 MPH.

REAR SUSPENSION
The rear suspension will be Standens, semi-elliptical, 3.00" wide x 53.00" long, 12-leaf pack with a ground rating of 27,000 lbs. The spring hangers will be castings.

The two (2) top leaves will wrap the forward spring hanger pin, and the rear of the spring will be a slipper style end that will ride in a rear slipper hanger. To reduce bending stress due to acceleration and braking, the front eye will be a berlin eye that will place the front spring pin in the horizontal plane within the main leaf.

A steel encased rubber bushing will be used in the spring eye. The steel encased rubber bushing will be maintenance free and require no lubrication.

OIL SEALS
Oil seals will be provided on the rear axle.

REAR TIRES
Rear tires will be four (4) Michelin 12R22.50 radials, 16 ply "all position" XZY 3 tread, rated for 27,120 lb maximum axle load and 75 mph maximum speed.

The tires will be mounted on 22.50" x 8.25" steel disc-type wheels with a ten (10)-stud 11.25" bolt circle.

TIRE BALANCE
All tires will be balanced with Counteract balancing beads. The beads will be inserted into the tire and eliminate the need for wheel weights.

TIRE PRESSURE MANAGEMENT
There will be a VECSAFE LED tire alert pressure management system provided that will monitor each tire's pressure. A chrome plated brass sensor will be provided on the valve stem of each tire for a total of six (6) tires.

The sensor will calibrate to the tire pressure when installed on the valve stem for pressures between 20 and 120 psi. The sensor will activate an integral battery operated LED when the pressure of that tire drops eight (8) psi.

Removing the cap from the sensor will indicate the functionality of the sensor and battery. If the sensor and battery are in working condition, the LED will immediately start blinking.

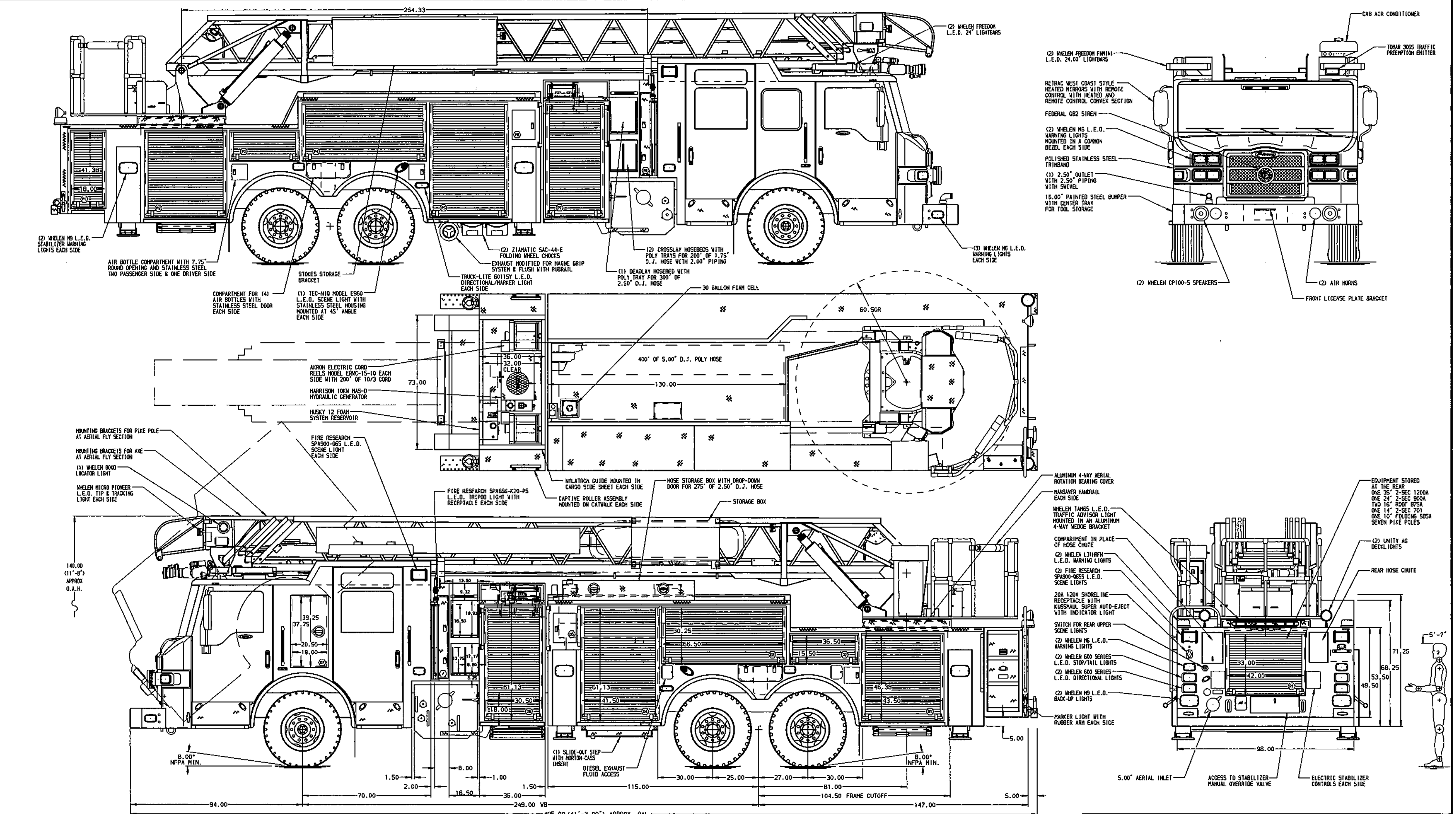
HUB COVERS (front)
Stainless steel hub covers will be provided on the front axle. An oil level viewing window will be provided.

HUB COVERS (rear)
A pair of stainless steel high hat hub covers will be provided on rear axle hubs.

COVERS, LUG NUT, CHROME
Chrome lug nut covers will be supplied on front and rear wheels.

MUD FLAPS
Mud flaps with a Pierce logo will be installed behind the front and rear wheels.

WHEEL CHOCKS
There will be one (1) pair of Ziamatic AC-44, aluminum alloy wheel blocks provided.



ALUMINUM BODY

SIDE ROLL AND FRONTAL IMPACT PROTECTION

PIERCE MANUFACTURING INC.

NOTE

DIMENSIONS SHOWN ARE APPROXIMATE AND ARE SUBJECT TO MINOR DEVIATIONS AS MAY OCCUR OR BE NECESSARY IN CONSTRUCTION.
MINOR DETAILS NOT SHOWN.

1. PERIMETER SCENE LIGHTS LOCATED PER SHOP ORDER
2. ANTENNA MOUNTS LOCATED PER SHOP ORDER
3. AMDOR AY9220 DUAL LIGHT STRIPS IN COMPARTMENT D1,D2,D3,D4,D5,D6,P1,P2,P3,P4,P5,P6,P7
4. SHELF TRACKS LOCATED IN COMPARTMENT D1,D2,D3,D4,P1,P2,P3,P4,P5
5. ONE FULL WIDTH/DEPTH BRUSHED ALUMINUM ADJUSTABLE SHELF IN COMPARTMENT D3,P1,TWO IN D1,P2,P4,P5 & THREE IN P7
6. ONE FLOOR MOUNTED BRUSHED ALUMINUM SLIDE-OUT TRAY IN COMPARTMENT D1,D4,P2,P5
7. ALUMINUM PEGBOARD WITH TRACKS ON THE SIDE WALLS AND REAR WALL OF COMPARTMENT D4
8. CIRCUIT BREAKER PANEL IN COMPARTMENT D5
9. CAST AIR COMPRESSOR IN COMPARTMENT D5
10. KUSSMAUL 1200 BATTERY CHARGER IN COMPARTMENT D5
11. BATTERY CHARGE INDICATOR MOUNTED ON DRIVER SIDE CREW CAB SEAT RISER
12. HUSKY 12 SINGLE AGENT FOAM SYSTEM
13. AIR INLET/OUTLET WITH DISCONNECT FITTING LOCATED ON THE DRIVER SIDE PUMP PANEL
14. PERIMETER SCENE LIGHTS LOCATED PER SHOP ORDER
15. TRIDENT FOLDING STEPS MOUNTED ON AERIAL DEVICE PER SHOP ORDER
16. 120V POWER TO AERIAL TIP
17. REMOTE LADDER CONTROLS AT AERIAL TIP
18. L.E.D. LIGHTING ON AERIAL LADDER SECTIONS
19. 2-WAY INTERCOM SYSTEM BETWEEN AERIAL TIP AND TURNTABLE
20. LIFTING EYE RESCUE ATTACHMENT AT AERIAL TIP
21. REMOTE MONITOR CONTROLS AT PUMP PANEL
22. NON-RAISED AERIAL PEDESTAL
23. HOSE STORAGE BOX WITH LIFT-UP DOOR SHIPPED LOOSE

REV	DATE	BY	CH
D	28MAR14	TKA	KBO
C	28FEB14	BST	KBO
B	26FEB14	TKA	KBO
A	20DEC13	TKA	GRM

CHASSIS DATA	
MAKE	PIERCE
MODEL	VEL7010
TITLE	105 AERIAL LADDER & BODY ASSEMBLY (105 PAL) 300 GALLON TANK, 270 GAL. WATER / 30 GAL. FOAM
FOR	AUSTIN FIRE DEPARTMENT AUSTIN, TEXAS
DWG NO.	27566AD
JOB NO.	27566
SCALE	1:24
DATE	
DRAWN BY	TKA — 04DEC13
CHECKED BY	GRM — 06DEC13
SHEET SIZE	D
SHEET NO.	1 OF 1

View File Created 28-MAR-2014 11:53:35

the Owner and issued by a surety company included within the Department of Treasury's Listing of Approved Sureties (Department Circular 570) with a minimum A.M. Best Financial Strength Rating of A and Size Category of XV. In the event of a bond issued by a surety of a lesser Size Category, a minimum Financial Strength rating of A+ is required.

Bidder and Bidder's surety agree that the Bond issued hereunder, whether expressly stated or not, also includes the surety's guarantee of the vehicle manufacturer's Bumper to Bumper warranty period included within this proposal. Owner agrees that the penal amount of this bond will be simultaneously amended to 25 percent of the total contract amount upon satisfactory acceptance and delivery of the vehicle(s) included herein. Notwithstanding anything contained within this contract to the contrary, the surety's liability for any warranties of any type will not exceed three (3) years from the date of such satisfactory acceptance and delivery, or the actual Bumper to Bumper warranty period, whichever is shorter.

APPROVAL DRAWING

A drawing of the proposed apparatus will be prepared and provided to the purchaser for approval before construction begins. The Pierce sales representative will also be provided with a copy of the same drawing. The finalized and approved drawing will become part of the contract documents. This drawing will indicate the chassis make and model, location of the lights, siren, horns, compartments, major components, etc.

A "revised" approval drawing of the apparatus will be prepared and submitted by Pierce to the purchaser showing any changes made to the approval drawing.

ELECTRICAL WIRING DIAGRAMS

Two (2) electrical wiring diagrams, prepared for the model of chassis and body, will be provided.

VELOCITY™ CHASSIS

The Pierce Velocity is the custom chassis developed exclusively for the fire service. Chassis provided will be a new, tilt type custom fire apparatus. The chassis will be manufactured in the apparatus body builder's facility eliminating any split responsibility. The chassis will be designed and manufactured for heavy duty service, with adequate strength and capacity for the intended load to be sustained and the type of service required. The chassis will be the manufacturer's first line tilt cab.

WHEELBASE

The wheelbase of the vehicle will be 249.0.

GVW RATING

The gross vehicle weight rating will be 76,800 .

FRAME

The chassis frame will be built with two (2) steel channels bolted to five (5) cross members or more, depending on other options of the apparatus. The side rails will have a 13.38" tall web over the front and mid sections of the chassis, with a continuous smooth taper to 10.75" over the rear axle. Each rail will have a section modulus of 25.992 cubic inches and a resisting bending moment (rbm) of 3,119,040 in-lb over the critical regions of the frame assembly, with a section modulus of 18.96 cubic inches with an rbm of 2,275,200 in-lb over the rear axle. The frame rails will be constructed of 120,000 psi yield strength heat-treated 0.38" thick steel with 3.50" wide flanges.

FRAME REINFORCEMENT

In addition, a mainframe inverted "L" liner will be provided. It will be heat-treated steel measuring 12.00" x 3.00" x 0.25". Each liner will have a section modulus of 7.795 cubic inches, yield strength of 110,000 psi, and rbm of 857,462 in-lb. Total rbm at wheelbase center will be 3,976,502 pounds per rail.

The frame liner will be mounted inside of the chassis frame rail and extend the full length of the frame.

FRONT NON DRIVE AXLE

The Oshkosh TAK-4® front axle will be of the independent suspension design with a ground rating of 22,800 lb.

Upper and lower control arms will be used on each side of the axle. Upper control arm castings will be made of 100,000-psi yield strength 8630 steel and the lower control arm casting will be made of 55,000-psi yield ductile iron.

The center cross members and side plates will be constructed out of 80,000-psi yield strength steel.

Each control arm will be mounted to the center section using elastomer bushings. These rubber bushings will rotate on low friction plain bearings and be lubricated for life. Each bushing will also have a flange end to absorb longitudinal impact loads, reducing noise and vibrations.

There will be nine (9) grease fittings supplied, one (1) on each control arm pivot and one (1) on the steering gear extension.

The upper control arm will be shorter than the lower arm so that wheel end geometry provides positive camber when deflected below rated load and negative camber above rated load.

Camber at load will be zero degrees for optimum tire life.

The ball joint bearing will be of low friction design and be maintenance free.

Toe links that are adjustable for alignment of the wheel to the center of the chassis will be provided.

The wheel ends will have little to no bump steer when the chassis encounters a hole or obstacle.

The steering linkage will provide proper steering angles for the inside and outside wheel, based on the vehicle wheelbase.

The axle will have a third party certified turning angle of 45 degrees. Front discharge, front suction, or aluminum wheels will not infringe on this cramp angle.

FRONT SUSPENSION
Front Oshkosh TAK-4™ independent suspension will be provided with a minimum ground rating of 22,800 lb.

The independent suspension system will be designed to provide maximum ride comfort. The design will allow the vehicle to travel at highway speeds over improved road surfaces and at moderate speeds over rough terrain with minimal transfer of road shock and vibration to the vehicle's crew compartment.

Each wheel will have torsion bar type spring. In addition, each front wheel end will also have energy absorbing jounce bumpers to prevent bottoming of the suspension.

The suspension design will be such that there is at least 10.00" of total wheel travel and a minimum of 3.75" before suspension bottoms.

The torsion bar anchor lock system allows for simple lean adjustments, without the use of shims. One can adjust for a lean within 15 minutes per side. Anchor adjustment design is such that it allows for ride height adjustment on each side.

The independent suspension was put through a durability test that simulated 140,000 miles of inner city driving.

FRONT SHOCK ABSORBERS
KONI heavy-duty telescoping shock absorbers will be provided on the front suspension.

FRONT OIL SEALS
Oil seals with viewing window will be provided on the front axle.

FRONT TIRES
Front tires will be Michelin 425/65R22.50 radials, 20 ply XFE wide base tread, rated for 22,800 lb maximum axle load and 65 mph maximum speed.

The tires will be mounted on 22.50" x 12.25" steel disc type wheels with a ten (10)-stud, 11.25" bolt circle.

REAR AXLE
The rear axle will be a Meritor™, Model RT-52-185, tandem axle assembly with a capacity of 54,000 lb.

An inter-axle differential, which divides torque evenly between axles, will be provided with an indicator light mounted on the cab instrument panel.

TOP SPEED OF VEHICLE
A rear axle ratio will be furnished to allow the vehicle to reach a top speed of 60 mph.

REAR SUSPENSION
Rear suspension will be a Ridewell Dynalastic Model 202S with a ground rating of 54,000 lbs. The suspension will have the following features:

- Individually articulating torque beams pivoted to a compensator providing independent axle movement and steady load distribution

- Utilizes Ultra Torque Rod Plus torque rods

REAR OIL SEALS
Oil seals will be provided on the rear axle.

REAR TIRES
Rear tires will be eight (8) Michelin 12R22.50 radials, 16 ply all position XZE* tread, rated for 54,240 lb maximum axle load and 75 mph maximum speed.

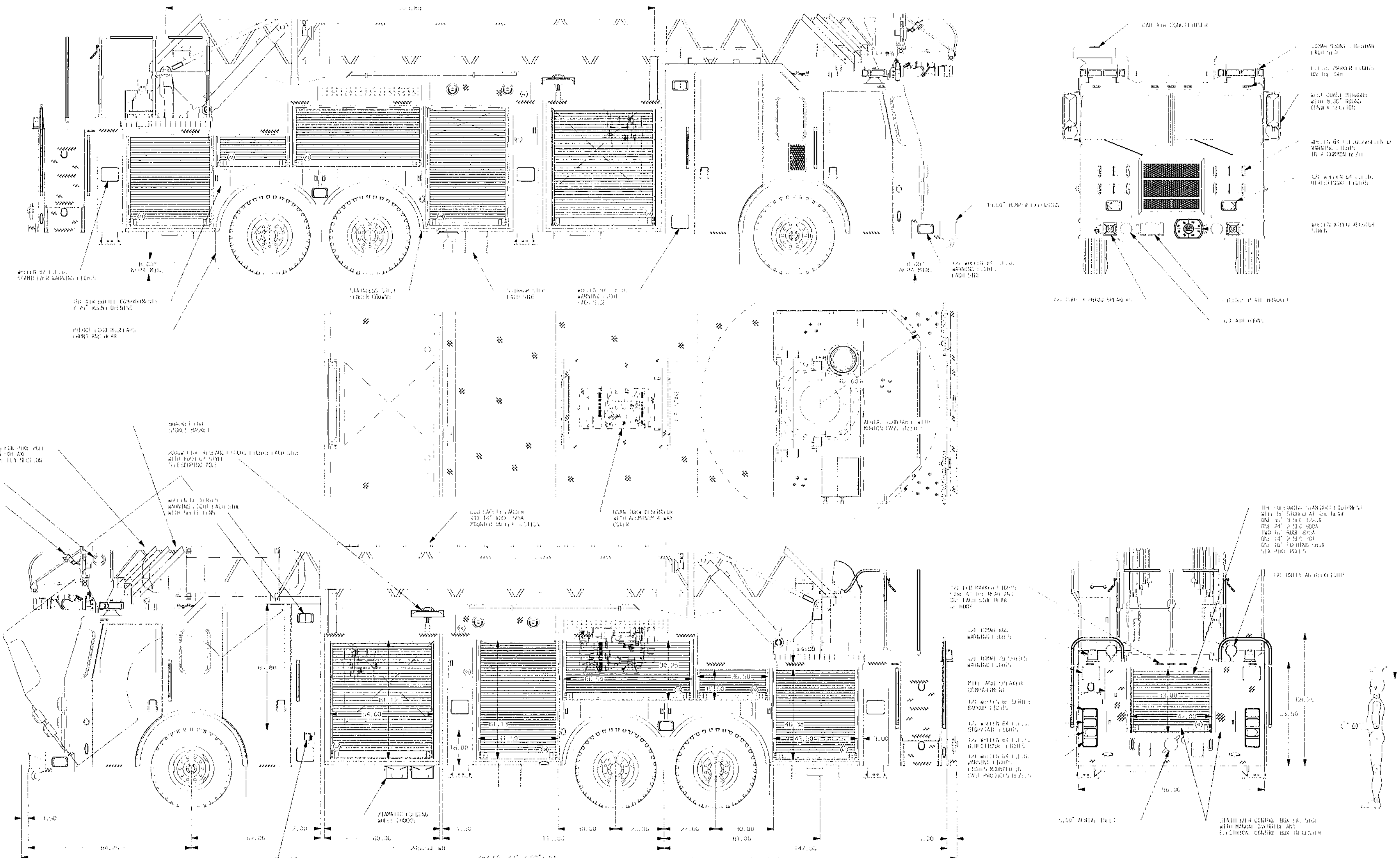
The tires will be mounted on 22.50" x 8.25" steel disc type wheels with a ten (10) stud, 11.25" bolt circle.

TIRE BALANCE
All tires will be balanced with Counteract balancing beads. The beads will be inserted into the tire and eliminate the need for wheel weights.

TIRE PRESSURE MANAGEMENT
There will be a VECSAFE LED tire alert pressure management system provided that will monitor each tire's pressure. A chrome plated brass sensor will be provided on the valve stem of each tire for a total of 10 tires.

The sensor will calibrate to the tire pressure when installed on the valve stem for pressures between 20 and 120 psi. The sensor will activate an integral battery operated LED when the pressure of that tire drops 8 psi.

Removing the cap from the sensor will indicate the functionality of the sensor and battery. If the sensor and battery are in working condition, the LED will immediately start blinking.



(ALUMINUM BODY)

NOTE
DIMENSIONS SHOWN ARE APPROXIMATE AND ARE SUBJECT TO MINOR DEVIATIONS AS MAY OCCUR OR BE NECESSARY IN CONSTRUCTION.
MINOR DETAILS NOT SHOWN.

1. ALUMINUM TREADPLATE ON CAB ROOF
2. ONE FLOOR MOUNTED SLIDE OUT TRAY IN COMPARTMENTS D1, D4 & P1
3. ONE ADJUSTABLE SHELF IN COMPARTMENTS D1, D3, D4, P1, P3 & P4
4. PERIMETER SCENE LIGHTS LOCATED PER SHOP ORDER
5. CIRCUIT BREAKER PANEL IN COMPARTMENT D2
6. HANNAY ELECTRIC REWIND CORD REEL MODEL ... IN COMPARTMENTS D2 & P2 WITH 200' OF 12/3 ... POWER CABLE, CIRCUIT JUNCTION BOX AND ALUMINUM ...
7. AIR INLET/OUTLET LOCATED ON THE DRIVER SIDE STEP WELL
8. TWO ... 2 WAY ... OUT ALUMINUM TOOL BOARDS IN COMPARTMENT D2 & P2
9. ONE ANTENNA MOUNTED ON THE CAB ROOF BEHIND THE LIGHTBAR ON EACH SIDE
10. QUICK LOCK WATERWAY ... SYSTEM
11. 110V POWER TO AERIAL TIP WITH (2) FIRE POWER RECEPTACLES AT TIP
12. REMOTE LADDER CONTROLS AT AERIAL TIP
13. ... RESCUE ATTACHMENT AT AERIAL TIP
14. KUSSMAUL BATTERY CHARGER LOCATED IN COMPARTMENT ...

				CHANGES DATA	TITLE	105' ... PIERCE AERIAL LADDER & BODY ASSEMBLY (...105A) NO WATER TANK / NO PUMP
				CHASSIS	FOR	AUSTIN FIRE DEPARTMENT AUSTIN, TEXAS
...	...	TKA	JHR	PIERCE		
...	...	TKA	JHR	MODEL	DRWG.	
REV	DATE	BY	CHK'D	DASH 2 ...	NO.	

Pierce MANUFACTURING INC
APPLETON, WISCONSIN

JOB NO.	...
DATE	SCALE 1:24
ORDERED	DRAWN BY TKA
ISSUED	CHK'D BY JHR
SHEET NO.	SHEET SIZE D

Pierce

Manufactured Exclusively For AUSTIN CITY OF JOB #: 13122

DATE OF MANUFACTURE: 03/02

GVWR: 69,500 LBS.

GAWR FRONT: 21,500 LBS. with 425/65R22.50 L tires, 22.50X12.25 rims, @ 125 psi cold SINGLE

GAWR REAR: 48,000 LBS. with 11R22.50 H tires, 22.50X8.25 rims, @ 120 psi cold DUAL

CONFORMITY OF THE CHASSIS-CAB TO UNITED STATES FEDERAL MOTOR VEHICLE SAFETY STANDARDS, WHICH HAVE BEEN PREVIOUSLY FULLY CERTIFIED BY THE INCOMPLETE VEHICLE MANUFACTURER OR BY THE INTERMEDIATE VEHICLE MANUFACTURER, HAS NOT BEEN AFFECTED BY FINAL-STAGE MANUFACTURE. THE VEHICLE HAS BEEN COMPLETED IN ACCORDANCE WITH PRIOR MANUFACTURER'S INSTRUCTIONS, WHERE APPLICABLE. THIS VEHICLE CONFORMS TO ALL OTHER APPLICABLE FEDERAL MOTOR VEHICLE STANDARDS IN EFFECT IN 03/02 MO/YR.

CUSTOM HIGH GRADE PAINT FINISH

COLOR: RED

PAINT # 80 SIKKENS AUTOCOAT LV

COLOR: WHITE

PAINT # 10 SIKKENS AUTOCRYL

COLOR: MS-90 #907049

MULTICOLOR SPECIALTIES

VEHICLE IDENTIFICATION NO: 4P1CT02S22A002162

VEHICLE TYPE: FIREFIGHTING TRUCK

IMPORTANT MAINTENANCE INSTRUCTIONS

PERFORM ALL SUGGESTED MAINTENANCE ITEMS OUTLINED IN THE CHASSIS OPERATION MANUAL AT THE RECOMMENDED TIME INTERVALS.

DEVICE	QTY	FLUID
ENGINE: SERIES 60	41 QTS	15W40 CG-4
TRANSMISSION: HD 4060P	[illegible] QTS	DEXRON IIE
COOLANT	[illegible] QTS	50% ANTIFREEZE (ETHYLENE GLYCOL)
POWER STEERING	[illegible] QTS	DEXRON II
REAR AXLE	[illegible] PINTS	80W90 GEAR LUBE
FRONT AXLE (NON DRIVE)	[illegible]	[illegible] GEAR LUBE
FRONT AXLE (DRIVE)	[illegible] QTS	80W90 GEAR LUBE
CAB TILT	1 GAL	ATF
GENERATOR (Crankcase oil capacity) (Refer to owners manual for temperature ranges)	5.5 QTS	10W40
TRANSFER CASE		
EQUIPMENT RACK - PER RESERVOIR	N/A	
WATEROUS PUMP TRANSMISSION	N/A	ATF DEXRON III
HALE PUMP PRIMER	N/A	PRIME SAFE

Austin, TX Office:

3737 Executive Center Drive, Suite 255
Austin, TX 78731-1633
P: 512-219-4075 F: 512-219-4077

August 31, 2017

Karim Helmi, P.E.
City of Austin – Public Works Department
105 Riverside Drive, Suite 100
Austin, TX 78704
Phone: 512-974-6539
Email: Karim.Helmi@austintexas.gov

Alejandro Wolniewitz
Austin Fire Department
4201 Ed Bluestein Boulevard
Austin, TX 78721
Phone: 512-974-1286
Email: Alejandro.Wolniewitz@austintexas.gov

Feasibility Study Report – Austin Fire Department Fire Stations Nos. 3 and 22
Fire Station No. 3 – 201 W. 30th St., Austin, TX
Fire Station No. 22 – 5309 E. Riverside Dr., Austin, TX
CTLGroup Project No. 231701, Phase 2

Dear Mr. Helmi and Mr. Wolniewitz:

Based on the work performed during Phase 1 of this project, it was determined that the garage floor systems at both fire stations lack adequate strength to support the anticipated vehicular loads. The City of Austin (COA) requested that a repair design be developed to strengthen the existing floor systems. In order to properly identify repair requirements and a strengthening solution, a feasibility study was performed on the floor systems at each fire station garage (Phase 2). The following tasks were performed as part of Phase 2 for this project:

- CTLGroup obtained additional core samples for compressive strength testing and carbonation depth testing. As discussed in our Phase 1 report, the purpose of the additional core sampling and subsequent compressive strength testing was to reduce the scatter of core strength data. This allows for more representative compressive strength values to be used in the structural analysis and subsequent repair design. Also as discussed in our Phase 1 report, carbonation could be an issue at Fire Station No. 3. The extent of carbonation will influence our repair recommendations and carbonation depth testing was performed to evaluate this condition.
- The preliminary structural analysis performed during Phase 1 of this project was updated to include the revised compressive strength values. It should also be noted that liberal assumptions were purposely made in our analysis during Phase 1 to study whether the floor systems could support the anticipated vehicular loads in a favorable condition. For Phase 2, these liberal assumptions were replaced by more conservative assumptions (where applicable/appropriate per code requirements). The capacity and demand values for the beams at both fire stations were also calculated as part of Phase 2.
- Based on the results of our testing and structural analysis, the feasibility of various repair options was evaluated.

CTLGroup's Phase 1 report was issued on May 12, 2017. Please refer to this report for additional information regarding this project, including background information. The following report summarizes the findings from our feasibility study.

SITE WORK

CTLGroup re-visited the fire stations on June 14 and 15, 2017 (Fire Station No. 22), and June 16 and 19, 2017 and July 28, 2017 (Fire Station No. 3). The following CTLGroup staff members were present during the site visits: Bradley East, P.E. and Jonathan Poole, Ph.D., P.E. (June 15, 2017 only). Various Austin Fire Department personnel were present during CTLGroup's site visits.

During the site visits, additional cores were taken through the slab/joists and beams at Fire Station No. 22, and slab/beam at Fire Station No. 3. As previously discussed, the additional core samples were obtained for compressive strength testing and carbonation depth testing. The core samples were extracted by Texas Cutting and Coring, L.P. The cores were extracted in general accordance with ASTM C42[1]. Following the removal of the cores, all core holes were patched by CTLGroup using a non-shrink grout material.

Additional Ground Penetrating Radar (GPR) scans were also performed to confirm shear reinforcing and to more accurately identify the locations and lengths of negative moment reinforcing (Fire Station No. 3). One particular item of note was that no discernible shear reinforcing was detected in the middle beams at Fire Station No. 3. The middle beams were scanned from both the sides and underside.

The core locations and reinforcing details are included on the drawings in **Appendix A**. These drawings have been updated/revised since issuance of our Phase 1 report.

SUMMARY OF LABORATORY TESTING

COMPRESSIVE STRENGTH TESTING

Compressive strength tests in general accordance with ASTM C42 were performed on the additional core samples obtained during Phase 2. At fire station No. 22, compressive strength tests were performed on core samples C10 and C11 (joist), and C13 through C18 (beam). At Fire Station No. 3, compressive strength tests were performed on core samples C6 through C14 (slab/beam). CTLGroup's compressive strength testing reports for Phase 2 can be found in **Appendix B** at the end of this report.

A statistical adjustment was applied to the core strength data (from Phases 1 and 2) in general accordance with ACI 214.4R[2]. The purpose of this adjustment was to convert the core strength data to an equivalent design compressive strength value. The equivalent compressive strength used for design purposes "is the lower tenth percentile of the in-place strength and is consistent with the statistical description of the specified compressive strength of concrete". Two methods are presented in ACI 214.4R for estimating the equivalent strength. For reference purposes, the Tolerance Factor Method with a 75% confidence level was used during our analysis. It should also be noted that during the statistical analysis the core data was evaluated for outliers in general accordance with ASTM E178[3]. One (1) outlier was discarded from the slab/beam core strength data sample from Fire Station No. 3.

---

[1] ASTM C42 "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete"

[2] ACI 214.4R-10 "Guide for Obtaining Cores and Interpreting Compressive Strength Results"

[3] ASTM E178 "Standard Practice for Dealing with Outlying Observations"

The equivalent strengths of the floor system elements at each fire station are summarized below in Table 1. As previously discussed, the preliminary structural analyses performed during Phase 1 of this project were updated based on these values.

Table 1 – Summary of equivalent design compressive strengths

Element		Equivalent Compressive Strengths, f'c (psi)
Fire Station No. 22	Joists	4823
	Beams	4572
Fire Station No. 3	Slab/Beams	2639

CARBONATION DEPTH TESTING

Carbonation is the reaction between CO_2 in the air and the hydrated cement paste, generally the calcium hydroxide ($Ca(OH)_2$, or CH in cement chemistry notation). In dense, well consolidated and properly cured concrete, carbonation is a slow reaction that generally occurs over many years. This reaction converts the CH to calcium carbonate ($CaCO_3$), which reduces the pH of the concrete and can lead to the depassivation of the steel. Depassivation of the steel allows corrosion to occur.

Carbonation depth tests in general accordance with ASTM C 856[4] were performed on core samples obtained/collected during Phases 1 and 2. This includes Cores C5, C14, C15 and C16 from Fire Station No. 3, and Core C12 from Fire Station No. 22. The following are items of note regarding the tested samples:

- The tested sample for Core C14 at Fire Station No. 3 was a partial sample. Only the bottom approximately 0.9 in. of the original core underwent testing.
- Core C5 at Fire Station No. 3 was collected during Phase 1. This core had been drilled by others prior to CTLGroup's involvement with this project (likely for plumbing/mechanical purposes) and had been left onsite. This core had not been extracted from garage floor framing, but rather from concrete framing in another area of the fire station.
- Core C12 at Fire Station No. 22 was originally a core taken through both the topping slab and precast joist flange at the garage area. The bottom approximately 0.3 in. of the sample had been removed prior to carbonation depth testing. Only the carbonation depth of the precast concrete was tested.

The carbonation depth test reports can be found in **Appendix B** at the end of this report. To summarize, carbonation depth testing indicates that the bottom portion of the slab concrete in the garage area at Fire Station No. 3 is significantly carbonated. The carbonation depths exceed the bottom concrete cover in the slab (i.e. distance from the underside of the slab to the surface of the bottom layer of reinforcing steel). The extent of carbonation in the slab/beam concrete in the garage area at Fire Station No. 3 is not known; however, carbonation was

[4] ASTM C856 "Standard Practice for Petrographic Examination of Hardened Concrete"; an abbreviated version of this test standard was performed pertaining to paste carbonation; a full petrographic examination was not performed on these core samples.

detected in all three (3) samples tested from this area. Minimal to no carbonation was detected in samples C5 from Fire Station No. 3 and C12 from Fire Station No. 22.

STRUCTURAL ANALYSES

As previously discussed, the preliminary structural analyses performed during Phase 1 of this project were updated/revised to include equivalent design compressive strength values for the concrete. Additionally, our assumptions were modified to reflect typical design standards rather than favorable conditions. The results/calculations from our analyses can be found in **Appendices C and D** at the end of this report. The methodology used to calculate the capacities of the various structural elements and the demands placed on these elements was similar for both fire stations, which includes the following:

- The flexural capacities of the various structural elements were computed using StructurePoint[5] software in accordance with ACI 318-14[6].
- The shear capacities of the various structural elements were calculated in general accordance with ACI 318-14.
- Analyses were performed using SAP2000[7] software on both the slab and joists at Fire Station Nos. 3 and 22, respectively. The shear, flexure and end reaction envelopes for these elements were determined based on this analysis. Trucks were assumed to occupy either centered as well as left-of-center or right-of-center positions within each bay.
- Based on the end reaction envelopes of the slab and joists at Fire Station Nos. 3 and 22, respectively, a load distribution ratio was determined for the beams at both fire stations (i.e. percent of axle load distributed to the beams).
- Taking into consideration the load distribution ratio, a moving wheel load analysis in the longitudinal direction was performed on the beams at each fire station using SAP2000 software. The shear and moment envelopes for the beams were determined based on this analysis.
- From the shear and moment envelopes, the maximum moment and shear demands on the various structural elements were determined. The demand capacity ratios were then calculated.

In addition to the above methodology, the following conditions and assumptions were included in our analyses:

General

- Since issuance of our Phase 1 report, CTLGroup received clarification on the anticipated vehicles that will operate from each fire station. At Fire Station No. 22 this includes a Pierce 105' Heavy Duty Aerial Ladder with water tank (Job No. 27566) and a Pierce Impel Pumper (Job No. 25403). At Fire Station No. 3 this includes a Pierce 105' Heavy

[5] StructurePoint, LLC, https://www.structurepoint.org/; computer software for the analysis and design of reinforced concrete structures.

[6] ACI 318-14 "Building Code Requirements for Structural Concrete"

[7] Computers and Structures, Inc., SAP200 software

Duty Aerial Ladder with water tank (Job No. 27566) and a Pierce Velocity Pumper (Job No. 29905). The dimensions and weights associated with these vehicles were used in our analyses. These specifications can be found in the structural analyses packet included in **Appendices C and D**.

- The loads considered in the structural analyses included the self-weight of the concrete elements and the axle weights of the above vehicles.

Fire Station No. 3

- As previously discussed, no discernible shear reinforcing was detected in the middle beams at Fire Station No. 3. As a result, it was assumed in our analyses that there was no shear reinforcement in the middle beams at Fire Station No. 3.

Fire Station No. 22

- Additional non-destructive testing (NDT) would need to be performed on the slab at Fire Station No. 22 to adequately evaluate the extent of composite action between the existing topping slab and joists. However, of all the cores taken through both the topping slab and joists at this fire station, approximately half were de-bonded. Additionally, visual evaluation of the joist cores indicates that there was minimal roughening of the top surface of the joists. Therefore, it was conservatively assumed that there was no composite action between the existing topping slab and joists in our analyses.
- Extensive cracking was observed in the topping slab at Fire Station No. 22. Therefore, the non-composite cracked topping was considered incapable of distributing wheel loads between adjacent joists.
- The joists at Fire Station No. 22 frame into the sides of the beams. CTLGroup found no evidence to indicate that there were any tie-bars (or similar) connecting the joists to the beams. As a result, the joists were assumed to be simply supported.
- Welded wire reinforcement (WWR) was found in the stems/webs of the joists at Fire Station No. 22. Since code requires multiple cross-wires of WWR in order to provide full development, at best only partial development of WWR would be effective in joists.

Based on our analyses, various elements of both fire stations lack the necessary capacity to support the anticipated vehicular loads. Tables 2 to 4 below summarize the capacities of the various structural elements, the load demands placed on these elements, and the Demand Capacity Ratios (DCR). A DCR greater than 1.0 indicates a strength deficiency.

Table 2 –Capacities at Fire Station Nos. 22 and 3

Element		Capacities		
		Shear (kips)	Positive Moment (k-ft)	Negative Moment (k-ft)
Fire Station No. 22	Joists	7.6	35.4	N/A
	North Beam	60.0	385.5	385.5
	Middle Beam	115.5	470.2	470.2
	South Beam	82.9	374.4	374.4
Fire Station No. 3	Slab	49.5	45.6	45.3
	West Beam	40.9	67.9	64.4
	Middle Beams	23.6	89.1	90.3

Table 3 –Demands at Fire Station Nos. 22 and 3

Element		Demands		
		Shear (kips)	Positive Moment (k-ft)	Negative Moment (k-ft)
Fire Station No. 22	Joists	37.0	118.6	N/A
	North Beam	60.8	202.2	173.3
	Middle Beam	130.5	320.9	401.2
	South Beam	74.6	192.5	158.9
Fire Station No. 3	Slab	66.3	98.0	112.0
	West Beam	44.4	48.3	58.3
	Middle Beams	91.1	184.7	189.7

Table 4 –Demand Capacity Ratios (DCR) at Fire Station Nos. 22 and 3. Values in red have a strength deficiency

Element		DCR		
		Shear	Positive Moment	Negative Moment
Fire Station No. 22	Joists	4.83	3.35	N/A
	North Beam	1.01	0.52	0.45
	Middle Beam	1.13	0.68	0.85
	South Beam	0.90	0.51	0.42
Fire Station No. 3	Slab	1.34	2.15	2.47
	West Beam	1.09	0.71	0.91
	Middle Beams	3.86	2.07	2.10

DISCUSSION OF REPAIR OPTIONS

FIRE STATION NO. 3

The underside of the slab was spalled at several locations. At several spalled areas, the reinforcing steel was exposed and visibly corroded/rusted, likely indicative of carbonation-induced corrosion. Carbonation depth testing performed by CTLGroup further confirms that carbonation is an issue of concern in the garage area at Fire Station No. 3. Due to the depth of carbonation, the future service life of the garage floor system could be limited. However, additional testing and service life modeling would be needed to more accurately estimate the functional lifespan of the garage floor system.

Considering the slab thickness, it would be difficult to repair existing areas of corroded reinforcing without the repair extending through the full depth of the slab. Additional NDT work would also be needed to determine the full extent of existing corroded reinforcing. Additionally, preventing future carbonation-induced corrosion (such as with cathodic protection) would add considerable cost to any repair/strengthening program.

The slab and middle beams at Fire Station No. 3 are considerably deficient with respect to supporting the anticipated vehicular loads (see Table 4). The slab is overloaded by nearly 150% in flexure. The middle beams are overloaded by nearly 300% in shear and nearly 100% in flexure. Due to the degree to which the slab and middle beams are overloaded in conjunction with the presence of carbonation-induced corrosion, we do not believe that repair/strengthening of the garage floor system at Fire Station No. 3 can be accomplished in a cost-effective manner without substantial replacement of framing elements.

CTLGroup proposes two (2) options to address the strength deficiency and carbonation issue, which includes the following:

1. Remove and replace large portions of the existing floor system, or
2. Fill the crawlspace beneath the garage area with a cementitious flowable fill material.

With regard to removal and replacement, this will require the removal of the slab and middle beams in the garage area. The west beam, perimeter foundation walls, and columns can likely remain in place. A new monolithic slab/beam system would be designed and constructed such that it would tie into these existing elements. In lieu of a cast-in-place monolithic slab/beam system, structural precast members could also be considered. If the City of Austin decides to replace the garage floor system, CTLGroup is available to design its replacement and provide details and drawings for construction phase services. This work would be performed as part of Phase 3 of this project. Some geotechnical investigation may be necessary to demonstrate adequacy of existing foundations. As an alternative to this repair option, the City may also consider replacement of the entire bay area of the fire station. This would allow other upgrades including increasing overhead clearance.

With regard to Option 2, the existing garage floor system at Fire Station No. 3 would remain in place and the crawlspace area beneath the garage would be filled with a cementitious flowable fill material. In this scenario, the garage floor system would generally function as a slab-on-grade type system. The slab and middle beams would no longer be suspended, and as a result the strength deficiencies in these elements would no longer be a concern. This is likely the fastest and least disruptive remedy. However, depending on the soil characteristics at the subject site, this option may not be possible. Specifically, expansive soil is common in the Austin area. The void underneath the slab systems provides protection against differential soil movement due to moisture variations in the soil. Filling the void beneath the slab could compromise this protection.

Based on a preliminary review of the soils at the subject site, the structure is situated on "Urban land" according to the United States Department of Agriculture (USDA) Web Soil Survey[8]. No additional information is provided for this soil. This includes the plasticity index which generally governs a soil's shrink/swell potential. Geotechnical borings and a soil evaluation would be needed to determine the precise characteristics of the foundation subgrade. If the City of Austin desires to explore this option further, CTLGroup can arrange for a geotechnical evaluation as part of Phase 3 of this project.

FIRE STATION NO. 22

CTLGroup considered multiple repair/retrofit options as repair strengthening solutions for the floor framing at Fire Station No. 22. However, the extent of deficiencies present in the existing floor system results in a relatively complex and expensive repair/strengthening program. Repair/strengthening requirements included the following:

- Replacement of the existing, poorly bonded topping slab,
- Shear strengthening of existing joists, and
- Flexural strengthening of existing joists.

The current 3.5 in. topping slab is not a reliable composite overlay. To achieve a sound composite overlay system, the current topping would need to be removed, the top of the existing joist flanges would need to be roughened to an approximately ¼ in. amplitude, and a new composite topping slab would need to be installed. However, the existing joist flange thickness is only 1½ in. Removing the topping and roughening the top of joist flange would likely involve damaging the existing joist flange. Repairing damaged joist flanges would be difficult and would increase the cost and duration of the retrofit.

The joists are potentially overloaded in shear by over 400%. Shear strengthening of existing joists could potentially be accomplished by use of FRP reinforcing, or installation of external threaded rod reinforcement. FRP is a composite material composed of a polymer matrix that is reinforced with high strength fibers. As a repair material for concrete, the fibers typically consist of carbon or glass. FRP can be installed by laying dry fabric into uncured epoxy resin or by adhering FRP laminates to existing concrete framing. However, there are limits to the extent of strengthening that can be accomplished with FRP. ACI 440.2R[9] that governs the use of FRP as an externally applied repair material for concrete structures requires that "the unstrengthened structural member, without FRP reinforcement, should have sufficient strength to resist a certain level of load". More specifically, the standard generally requires that the concrete member be able to support 75% of the service live load (i.e. the vehicular wheel loads) in addition to the

[8] USDA, "Web Soil Survey," http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx (accessed August 17, 2017).

[9] ACI 440.2R "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures"

dead load (i.e. self-weight of the concrete). The extent of strength deficiencies in the floor framing is greater than this threshold.

Shear strengthening by use of external threaded rods would involve installing threaded rods through the 1 in. space between adjacent joists. The rods would be secured to the joists with steel plates and nuts at both the tops and bottoms of the joists. The top plates would be embedded/encased in the topping composite slab concrete. While this is a viable repair methodology, the extent of strengthening required in some areas compromises the practicality of this repair.

Flexural strengthening of the joists would require a strengthening level that also prohibits use of FRP reinforcing alone. The most practical method of strengthening appeared to be thickening the concrete overlay. This, however, would reduce overhead clearance, thereby requiring retrofit of overhead doors to accommodate the thickened overlay. Transitions would also be necessary where the garage meets other portions of the fire station.

Thus, addressing each deficiency would result in a complex and expensive retrofit program. Therefore, similar to Fire Station No. 3, CTLGroup proposes two (2) options to address the strength deficiency in the floor framing at Fire Station No. 22, which include the following:

1. Remove and replace the existing topping slab and joists in the garage area, or
2. Fill the crawlspace beneath the garage area with a cementitious flowable fill material

Removal and replacement would be limited to the topping slab and joists. The beams can remain in place with limited strengthening. A new joist/slab system would be designed and constructed such that it would tie into the existing beams. It would likely be most practical to replace the joists with custom precast members. If the City of Austin decides to replace the joists and slab at the garage area, CTLGroup is available to design its replacement and provide details and drawings for construction phase services. This work would be performed as part of Phase 3 of this project. Some geotechnical investigation may be necessary to demonstrate adequacy of existing foundations. As an alternative to this repair option, the City may also consider replacement of the entire bay area of the fire station. This would allow other upgrades including increasing overhead clearance.

With regard to Option 2, the existing garage floor system at Fire Station No. 22 could remain in place and the crawlspace area beneath the garage would be filled with a cementitious flowable fill material. As discussed above, expansive clay could make this option not feasible. Geotechnical borings and a soil evaluation would be needed to determine the precise characteristics of the existing subgrade. If the City of Austin desires to explore this option further, CTLGroup can arrange for a geotechnical evaluation as part of Phase 3 of this project.

CLOSING

Thank you for the opportunity to assist you on this project. Please do not hesitate to let me know if you have any questions or concerns, or need any additional information.

Peter R. Kolf

Peter R. Kolf
Principal Structural Engineer
PKolf@CTLGroup.com
Phone: (847) 972-3214

Hamid R Lotfi

Hamid R. Lotfi
Senior Engineer
HLotfi@CTLGroup.com
Phone: (847) 972-3206

STATE OF TEXAS
JONATHAN L. POOLE
105120
LICENSED
PROFESSIONAL ENGINEER

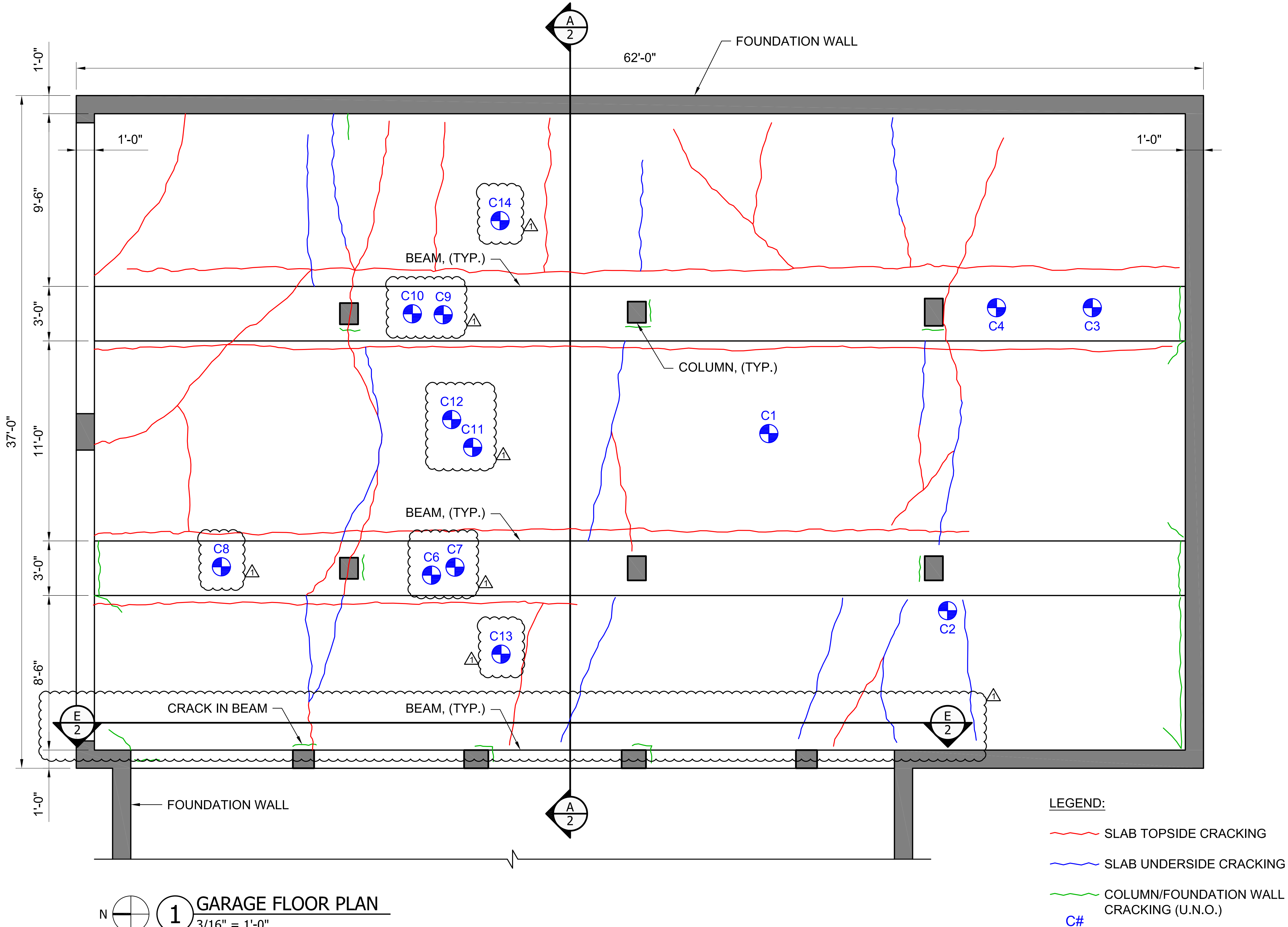
August 31, 2017

Jonathan L. Poole, Ph.D., P.E.
Principal Engineer
JPoole@CTLGroup.com
P. 512-219-4075

COA# F3849

Appendix A

Plan Views and Cross-Section Details



1 GARAGE FLOOR PLAN
3/16" = 1'-0"

LEGEND:

SLAB TOPSIDE CRACKING

SLAB UNDERSIDE CRACKING

COLUMN/FOUNDATION WALL CRACKING (U.N.O.)

C#

CORE LOCATION

CTLGroup

CONSTRUCTION TECHNOLOGY LABORATORIES
ENGINEERS & CONSTRUCTION TECHNOLOGY CONSULTANTS

3737 Executive Center Dr., Suite 255
Austin, TX 78731-1633
Phone: 512-219-4057 Fax: 512-219-4077
www.CTLGroup.com

CTLGroup COA No.: F-3849

Austin Fire Department - Station #3

201 W. 30th Street
Austin, TX 78701

Structural Floor System Capacity Assessment

ISSUE

No.	Description	Date
1	Revision No. 1	25AUG17

CTLGroup No.: **231701**

Drawn: **SEH**

Checked: **BLE**

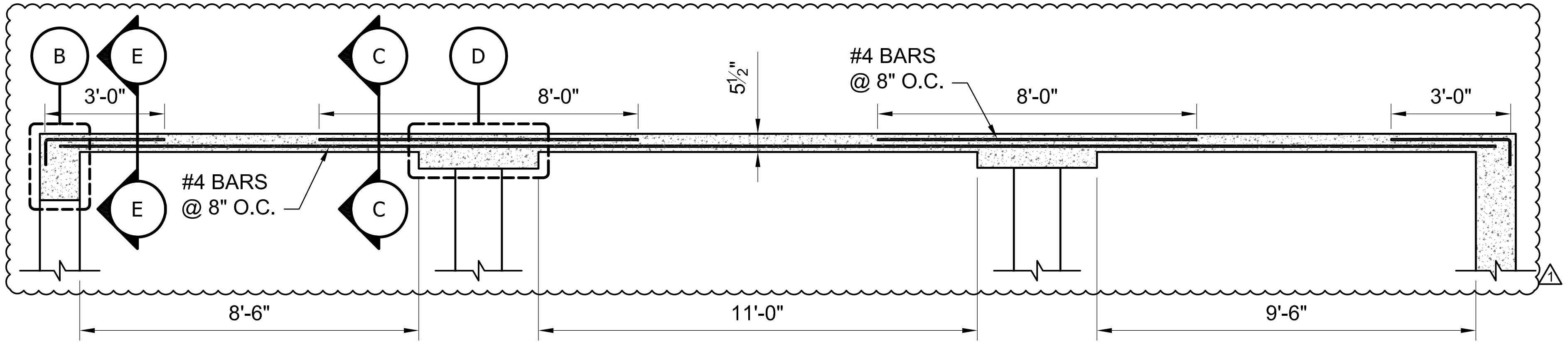
Date: **12MAY17**

Scale: **AS SHOWN**

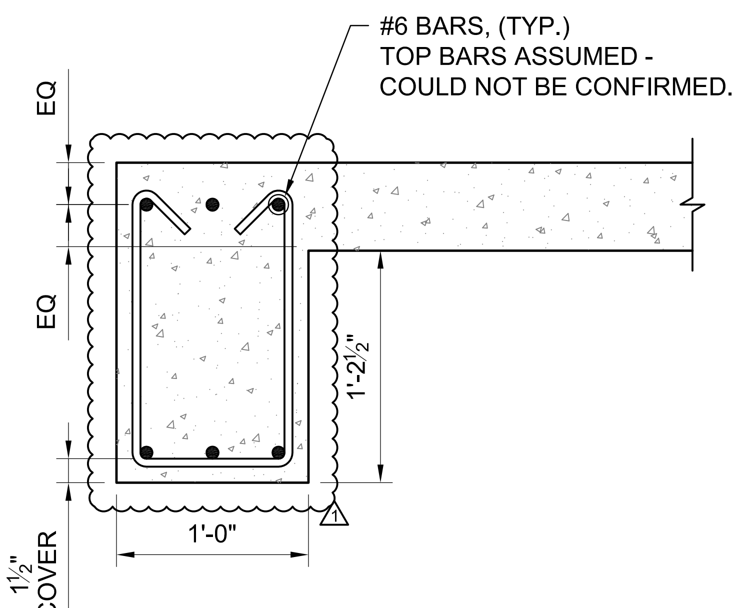
Title:
Garage Floor Plan

Sheet No.:

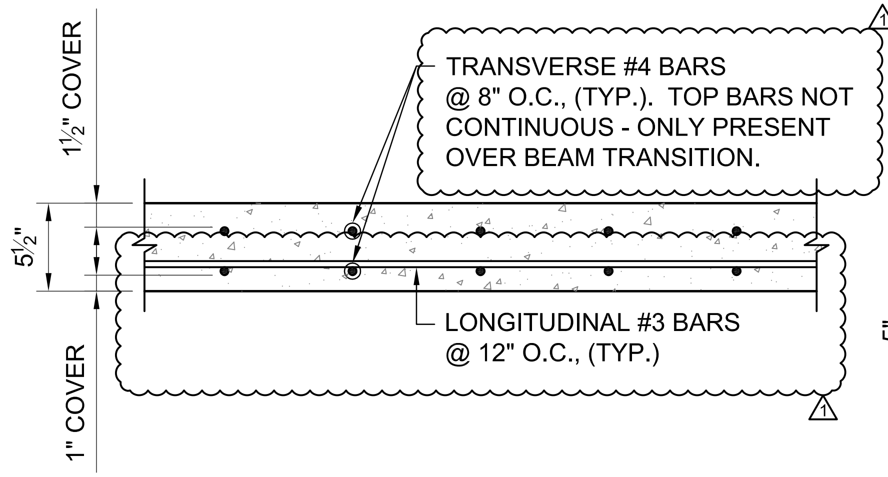
1



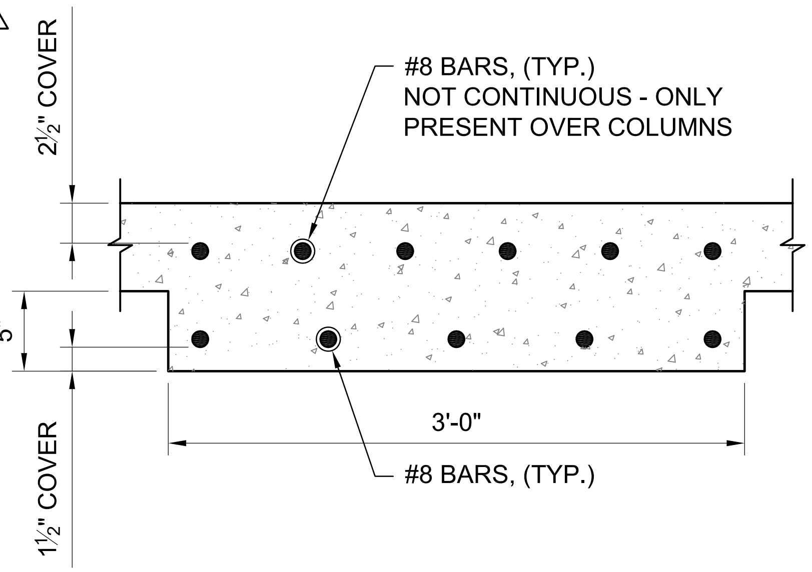
1 SECTION A-A
1/4" = 1'-0"



2 DETAIL B
1" = 1'-0"

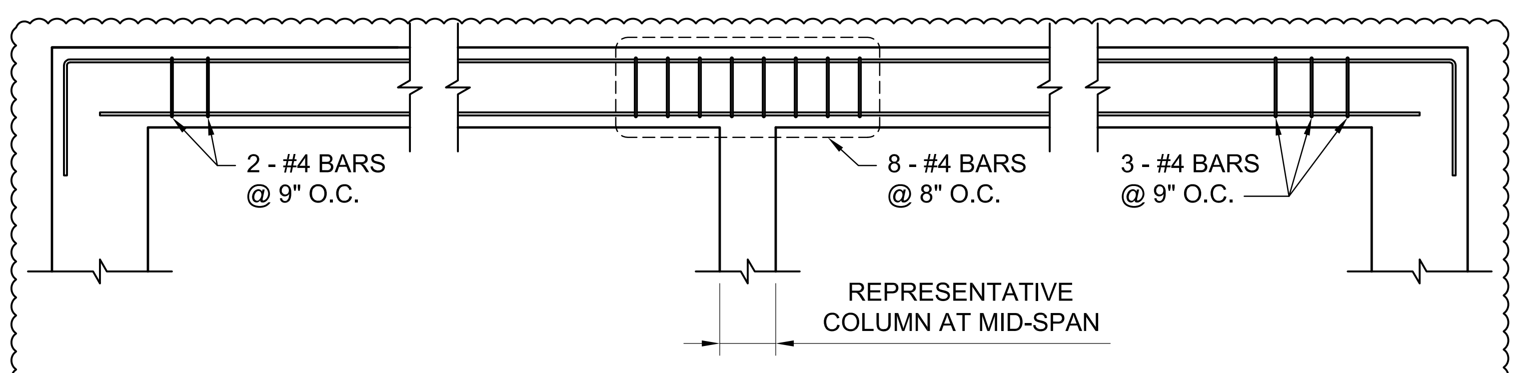


3 SECTION C-C
1" = 1'-0"



4 DETAIL D
1" = 1'-0"

NOTE:
1. NO SHEAR REINFORCEMENT DETECTED IN MIDDLE BEAMS.



5 SECTION E-E
1/4" = 1'-0"

CTLGROUP
CONSTRUCTION TECHNOLOGY LABORATORIES
ENGINEERS & CONSTRUCTION TECHNOLOGY CONSULTANTS

3737 Executive Center Dr., Suite 255
Austin, TX 78731-1633
Phone: 512-219-4057 Fax: 512-219-4077
www.CTLGroup.com

CTLGroup COA No.: F-3849

Austin Fire Department - Station #3

201 W. 30th Street
Austin, TX 78701

Structural Floor System Capacity Assessment

ISSUE

No.	Description	Date
1	Revision No. 1	25AUG17

CTLGroup No.: **231701**
Drawn: **SEH**
Checked: **BLE**
Date: **12MAY17**
Scale: **AS SHOWN**

Title:
Garage Details

Sheet No.:
2

Appendix B

Laboratory Test Results

Client:	**City of Austin**	CTLGroup Project No.:	**231701**
Project Name:	**Austin Fire Department Stations 3 & 22**	CTLGroup Project Mgr.:	**Bradley East**
	Structural Capacity Assessment	Analyst:	**WD, CA**
Contact:	**Karim Helmi**	Approved by:	**Bradley East**
Submitter:	**Bradley East**	Date Analyzed:	**June 27, 2017**
Date Received:	**June 21, 2017**	Date Reported:	**June 28, 2017**

ASTM C42 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
Section 7: Cores for Compressive Strength

Specimen Identification

CTLGroup Identification	4475701	4475702	4475703
Client Identification	No. 3-C6	No. 3-C7	No. 3-C8
Date Core Obtained from the Field	Not Stated	Not Stated	Not Stated
Date end preparation was completed and core was placed in sealed bag	6/22/17	6/22/17	6/22/17
Date Core was Tested	6/27/17	6/27/17	6/27/17

Concrete Description

Nominal Maximum Aggregate Size, in.	3/4	3/4	3/4
Concrete Age at Test	~65 years	~65 years	~65 years
Moisture Condition at Test	Per Standard	Per Standard	Per Standard
Length of Core, As Drilled, in.	6 1/2	7	6 3/4
Orientation of Core Axis in Structure	Vertical	Vertical	Vertical
Cylinder End Preparation	Capped	Capped	Capped

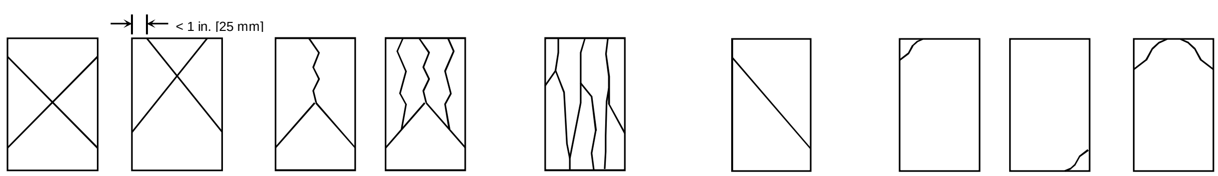
Concrete Dimensions

Diameter 1, in.	2.74	2.74	2.74
Diameter 2, in.	2.74	2.74	2.74
Average Diameter, in.	2.74	2.74	2.74
Cross-Sectional Area, in^2	5.90	5.90	5.90
Length Trimmed, in.	5.2	5.2	5.2
Length Capped, in.	5.3	5.3	5.4
Density, pcf	140	142	139

Compressive Strength and Fracture Pattern

Maximum Load, lb	17,620	18,006	15,659
Uncorrected compressive Strength, psi	2,990	3,050	2,650
Ratio of Capped Length to Diameter	1.95	1.95	1.97
Corrected Compressive Strength, psi	2,990	3,050	2,650
Fracture Pattern	Type 4	Type 1	Type 1

Schematic of Typical Fracture Patterns



Type 1
Reasonable well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps

Type 2
Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end

Type 3
Columnar vertical cracking through both ends, no well-formed cones

Type 4
Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type I

Type 5
Side fractures at top or bottom (occur commonly with unbonded caps)

Type 6
Similar to Type 5 but end of cylinder is pointed

Notes:

1. This report may not be reproduced except in its entirety.

Client:	**City of Austin**	CTLGroup Project No.:	**231701**
Project Name:	**Austin Fire Department Stations 3 & 22**	CTLGroup Project Mgr.:	**Bradley East**
	Structural Capacity Assessment	Analyst:	**WD, CA**
Contact:	**Karim Helmi**	Approved by:	**Bradley East**
Submitter:	**Bradley East**	Date Analyzed:	**June 27, 2017**
Date Received:	**June 21, 2017**	Date Reported:	**June 28, 2017**

ASTM C42 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
Section 7: Cores for Compressive Strength

Specimen Identification

CTLGroup Identification	4475704	4475705	4475706
Client Identification	No. 3-C9	No. 3-C10	No. 3-C11
Date Core Obtained from the Field	Not Stated	Not Stated	Not Stated
Date end preparation was completed and core was placed in sealed bag	6/22/17	6/22/17	6/22/17
Date Core was Tested	6/27/17	6/27/17	6/27/17

Concrete Description

Nominal Maximum Aggregate Size, in.	3/4	3/4	3/4
Concrete Age at Test	~65 years	~65 years	~65 years
Moisture Condition at Test	Per Standard	Per Standard	Per Standard
Length of Core, As Drilled, in.	6 1/2	6 3/4	5 1/4
Orientation of Core Axis in Structure	Vertical	Vertical	Vertical
Cylinder End Preparation	Capped	Capped	Capped

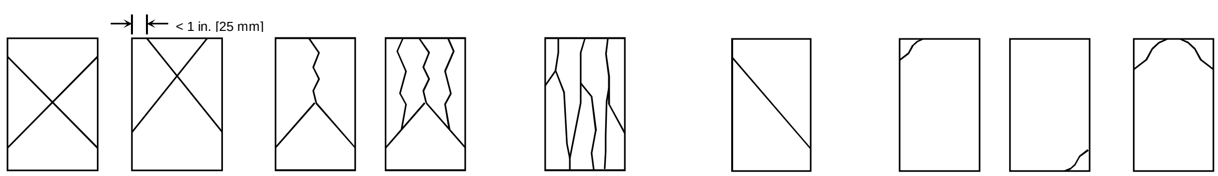
Concrete Dimensions

Diameter 1, in.	2.74	2.74	2.74
Diameter 2, in.	2.74	2.74	2.74
Average Diameter, in.	2.74	2.74	2.74
Cross-Sectional Area, in^2	5.90	5.90	5.90
Length Trimmed, in.	5.2	5.2	5.2
Length Capped, in.	5.4	5.3	5.4
Density, pcf	140	141	141

Compressive Strength and Fracture Pattern

Maximum Load, lb	15,534	15,388	18,126
Uncorrected compressive Strength, psi	2,630	2,610	3,070
Ratio of Capped Length to Diameter	1.96	1.95	1.96
Corrected Compressive Strength, psi	2,630	2,610	3,070
Fracture Pattern	Type 4	Type 1	Type 1

Schematic of Typical Fracture Patterns



Type 1
Reasonable well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps

Type 2
Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end

Type 3
Columnar vertical cracking through both ends, no well-formed cones

Type 4
Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type I

Type 5
Side fractures at top or bottom (occur commonly with unbonded caps)

Type 6
Similar to Type 5 but end of cylinder is pointed

Notes:

1. This report may not be reproduced except in its entirety.

Client:	**City of Austin**	CTLGroup Project No.:	**231701**
Project Name:	**Austin Fire Department Stations 3 & 22 Structural Capacity Assessment**	CTLGroup Project Mgr.:	**Bradley East**
		Analyst:	**WD, CA**
Contact:	**Karim Helmi**	Approved by:	**Bradley East**
Submitter:	**Bradley East**	Date Analyzed:	**June 27, 2017**
Date Received:	**June 21, 2017**	Date Reported:	**June 28, 2017**

ASTM C42 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
Section 7: Cores for Compressive Strength

Specimen Identification

CTLGroup Identification	4475707	4475708	4475709
Client Identification	No. 3-C12	No. 3-C13	No. 3-C14
Date Core Obtained from the Field	Not Stated	Not Stated	Not Stated
Date end preparation was completed and core was placed in sealed bag	6/22/17	6/22/17	6/22/17
Date Core was Tested	6/27/17	6/27/17	6/27/17

Concrete Description

Nominal Maximum Aggregate Size, in.	3/4	3/4	3/4
Concrete Age at Test	~65 years	~65 years	~65 years
Moisture Condition at Test	Per Standard	Per Standard	Per Standard
Length of Core, As Drilled, in.	6	5 1/4	5 3/4
Orientation of Core Axis in Structure	Vertical	Vertical	Vertical
Cylinder End Preparation	Capped	Capped	Capped

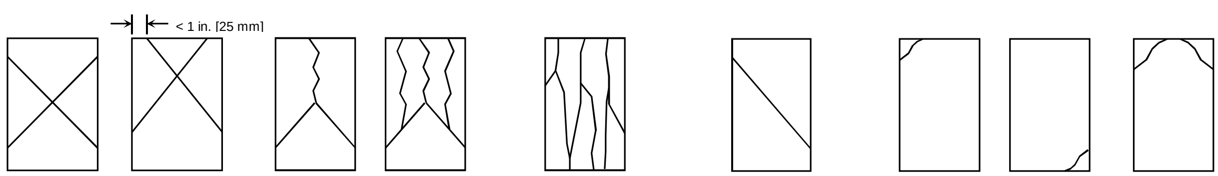
Concrete Dimensions

Diameter 1, in.	2.74	2.74	2.74
Diameter 2, in.	2.74	2.74	2.75
Average Diameter, in.	2.74	2.74	2.75
Cross-Sectional Area, in^2	5.90	5.90	5.94
Length Trimmed, in.	5.2	5.2	5.1
Length Capped, in.	5.4	5.4	5.3
Density, pcf	140	142	143

Compressive Strength and Fracture Pattern

Maximum Load, lb	18,485	20,585	17,159
Uncorrected compressive Strength, psi	3,130	3,490	2,890
Ratio of Capped Length to Diameter	1.96	1.95	1.94
Corrected Compressive Strength, psi	3,130	3,490	2,890
Fracture Pattern	Type 4	Type 1	Type 1

Schematic of Typical Fracture Patterns



Type 1
Reasonable well-formed cones on both ends, less than 1 in. [25 mm] of cracking through caps

Type 2
Well-formed cone on one end, vertical cracks running through caps, no well-defined cone on other end

Type 3
Columnar vertical cracking through both ends, no well-formed cones

Type 4
Diagonal fracture with no cracking through ends; tap with hammer to distinguish from Type I

Type 5
Side fractures at top or bottom (occur commonly with unbonded caps)

Type 6
Similar to Type 5 but end of cylinder is pointed

Notes:

1. This report may not be reproduced except in its entirety.

Copy No. 1

Report for
City of Austin
105 W. Riverside Drive, Austin, Texas 78704

CTLGroup Project No. 231701

Paste Carbonation Determination of Two Concrete Cores from the City of Austin Fire Department Station 3 Structural Capacity Assessment, Austin, Texas

August 9, 2017

Submitted by:
Meredith Strow
Jean L. Randolph

COA #F-3849

5400 Old Orchard Road
Skokie, Illinois 60077-1030
(847) 965-7500

Austin, TX • Bradenton, FL • Chicago, IL • Horsham, PA
Naperville, IL • Washington, DC • Doha, Qatar

www.CTLGroup.com

REPORT OF PASTE CARBONATION DETERMINATION

Date: August 9, 2017

CTLGroup Project No.: 231701

Paste Carbonation Determination of Two Concrete Cores from the City of Austin Fire Department Station 3 Structural Capacity Assessment, Austin, Texas

Two concrete cores, identified as FS #3 C15 and FS #3 C16 (Figs. 1 and 2), were received on August 1, 2017, by the CTLGroup Petrographic Laboratory from Mr. Bradley East, CTLGroup Engineer, on behalf of the City of Austin, Texas. Table 1 identifies and briefly describes the as-received cores.

TABLE 1 IDENTIFICATION AND BRIEF DESCRIPTION OF THE CORE SAMPLES

Core Identification	Brief Description	As-Received Photographs
FS #3 C15	Full-depth 1.7-in.-dia. core consisting of one concrete with a very thin layer of clear topping material on the top surface. A couple randomly-oriented hairline cracks are present on the top surface.	Fig. 1
FS #3 C16	Full-depth 1.7-in.-dia. core consisting of one concrete with a very thin layer of clear topping material on the top surface.	Fig. 2

Determination of the depth of paste carbonation of the two cores was requested, specifically from the core bottom surface up into the concrete. This report presents the details and results of the analysis.

FINDINGS AND CONCLUSIONS

Core FS #3 C15 does not contain rebar. Paste carbonation is present in both the top and bottom portions of the concrete core (Fig. 3a). From the top surface, the paste is carbonated to depths of 22 to 29 mm (0.9 to 1.1 in.). From the bottom surface, the paste is carbonated to depths of 10 to 27 mm (0.4 to 1.1 in.) into the concrete.

Core FS #3 C16 does not contain rebar. Paste carbonation is observed only in the bottom portion of the concrete core (Fig. 3b). From the bottom surface, the paste is carbonated to depths of 29 to 44 mm (1.1 to 1.7 in.) into the concrete.

All information obtained in the examination is presented in the laboratory data forms at the end of this report.

METHODS OF TEST

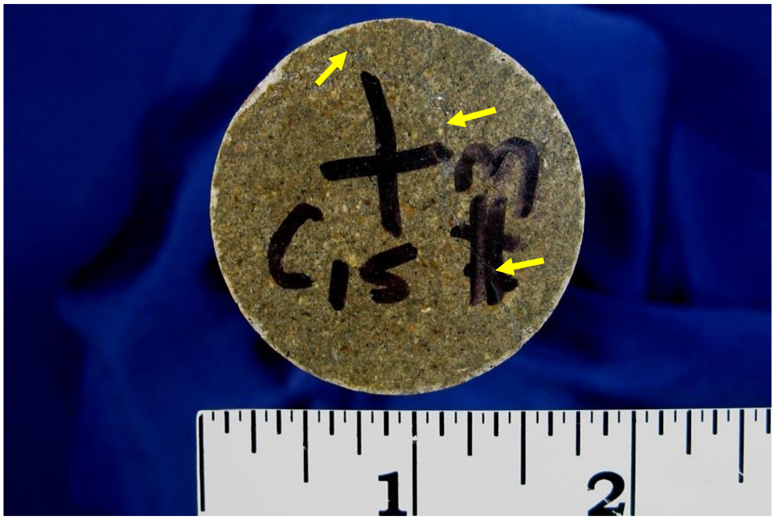
Depth and pattern of paste carbonation was determined by application of a pH indicator solution (phenolphthalein) to a freshly saw-cut, longitudinal concrete surface of each core. The solution imparts a deep magenta stain to high pH, non-carbonated paste. Carbonated paste does not change color.

Meredith Strow
Petrography Group

Jean L. Randolph
Senior Petrographer and Group Manager
Petrography Group

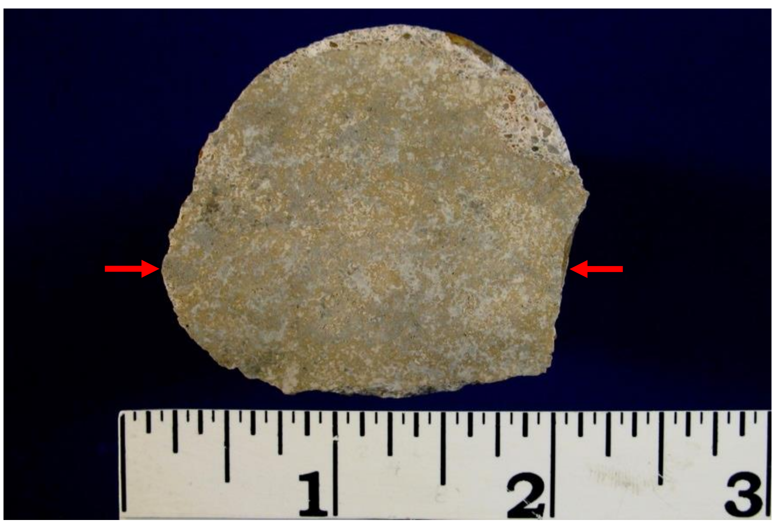
MLS/JLR/

Notes: 1. Results refer specifically to the samples submitted.
2. This report may not be reproduced except in its entirety.



1a. Core top surface. Surface is flat, even concrete surface with a very thin layer of clear topping material. Yellow arrows point to hairline cracks.

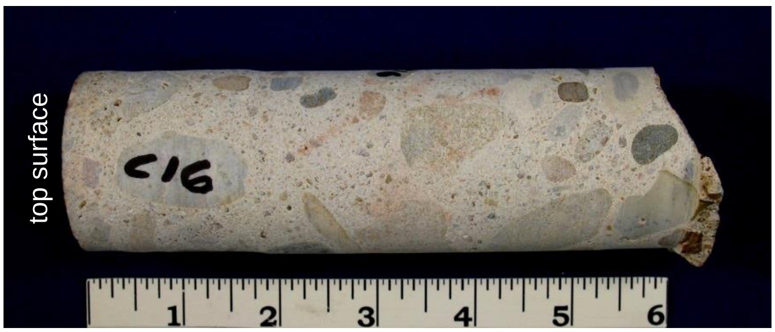
1b. Side view of core.



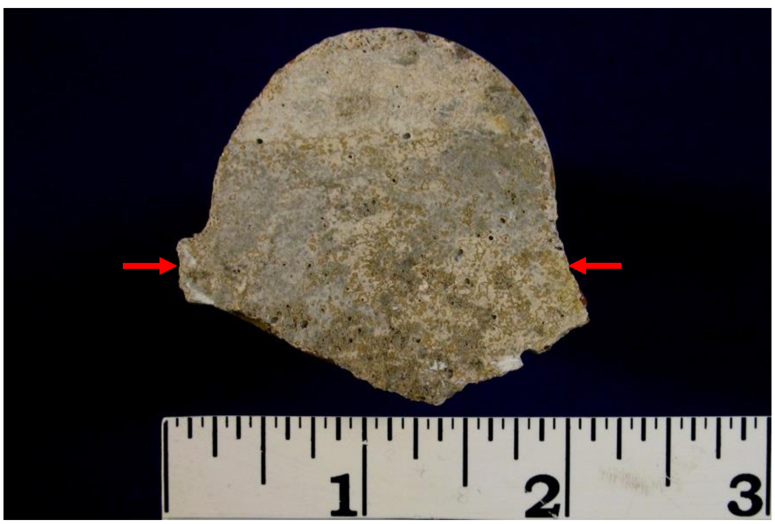
1c. Core bottom surface. The surface is a formed wavy shape. Red arrows point to corrugated ridge.

Fig. 1 Core FS #3 C15, as received in the Petrographic Laboratory for testing.

2a. Core top surface. Surface is flat, even concrete surface with a very thin layer of clear topping material.



2b. Side view of core.



2c. Core bottom surface. The surface is a formed wavy shape. Red arrows point to corrugated ridge.

Fig. 2 Core FS #3 C16, as received in the Petrographic Laboratory for testing.

3a. Core FS #3 C15

3b. Core FS #3 C16

Fig. 3 Saw-cut, cross-sectional concrete surfaces of Cores FS #3 C15 and FS #3 C16. Phenolphthalein (a pH indicator solution) was applied to the surface to determine paste carbonation levels. Non-carbonated paste is deep magenta; carbonated paste did not change color. Yellow bars and text designate depth into the concrete from the nearest surface. Scale is in inches.

LABORATORY DATA FORM

STRUCTURE: City of Austin Fire Station #3 *DATE RECEIVED*: August 1, 2017

LOCATION: Austin, Texas *EXAMINED BY*: Meredith Strow

SAMPLE

Client Identification: FS #3 C15.

CTLGroup Identification: 4506701.

Dimensions: Core diameter = 44 mm (1.7 in.), core length = 137 to 152 mm (5.4 to 6 in.); full structure thickness.

Top Surface: Flat, even, concrete surface with very thin layer of clear topping material. A couple long, randomly-oriented, hairline cracks extend across the full diameter of the core.

Bottom Surface: Wavy, fairly smooth, formed concrete surface with one corrugated ridge.

Cracks, Joints, Large Voids: No additional cracks present; no joints or large voids present.

Reinforcement: None present.

PASTE

Depth of Carbonation: 22 to 29 mm (0.9 to 1.1 in.) from top surface; 10 to 27 mm (0.4 to 1.1 in.) from bottom surface.

LABORATORY DATA FORM

STRUCTURE: City of Austin Fire Station #3 *DATE RECEIVED*: August 1, 2017

LOCATION: Austin, Texas *EXAMINED BY*: Meredith Strow

SAMPLE

Client Identification: FS #3 C16.

CTLGroup Identification: 4506702.

Dimensions: Core diameter = 44 mm (1.7 in.), core length = 136 to 150 mm (5.4 to 5.9 in.); full structure thickness.

Top Surface: Flat, even, concrete surface with very thin layer of clear topping material.

Bottom Surface: Wavy, fairly smooth, formed concrete surface with one corrugated ridge.

Cracks, Joints, Large Voids: None present.

Reinforcement: None present.

PASTE

Depth of Carbonation: Negligible from top surface; 29 to 44 mm (1.1 to 1.7 in.) from bottom surface.

Copy No. 1

Report for
City of Austin
105 W. Riverside Drive, Austin, Texas 78704

CTLGroup Project No. 231701

Paste Carbonation Determination on Core Samples from Austin Fire Department Stations 3 and 22 Structural Capacity Assessment, Austin, Texas

June 28, 2017

Submitted by:
Meredith Strow
Jean L. Randolph

COA #F-3849

5400 Old Orchard Road
Skokie, Illinois 60077-1030
(847) 965-7500

Austin, TX • Bradenton, FL • Chicago, IL • Horsham, PA
Naperville, IL • Washington, DC • Doha, Qatar

www.CTLGroup.com

REPORT OF PASTE CARBONATION DETERMINATION

Date: June 28, 2017

CTLGroup Project No.: 231701

Paste Carbonation Determination on Core Samples from Austin Fire Department Stations 3 and 22 Structural Capacity Assessment, Austin, Texas

Two concrete core samples were received June 23, 2017, in the CTLGroup Petrographic Laboratory from Mr. Bradley East, CTLGroup Engineer, on behalf of the City of Austin, Texas. Table 1 identifies and briefly describes the as-received specimens.

TABLE 1 IDENTIFICATION AND BRIEF DESCRIPTION OF THE CORE SAMPLES

Core Identification	Brief Description	As-Received Photographs
No. 3-Big Core	Full-depth 5.6-in.-dia. core, consisting of a terrazzo-type topping, then a thick mortar-like layer, then the substrate concrete.	Fig. 1
No. 3-C14	Specimen is the bottom 0.9-in. portion of a longer, 2.7-in.-dia. core. The bottom portion was saw-cut from the overlying core.	Fig. 2

Determination of the depth of paste carbonation of the two core specimens was requested, from the core bottom surface up into the concrete. This report presents the details and results of the analysis.

FINDINGS AND CONCLUSIONS

Core 3-Big Core contains three rebar segments, which are located in the bottom portion of the concrete. The rebar segments have concrete cover ranging from 0.5 to 1 in. from the bottom surface.

Carbonation in Core No. 3-Big Core is minimal and does not reach any of the four rebar segments present within the concrete (Fig. 3). The rebar segments have concrete cover ranging from 0.5 to 1 in. from the bottom surface. Four small, local regions of carbonation extend from the bottom surface to depths of 0.3 to 0.5 in. into the concrete. The carbonated region which extends 0.5 in. into the concrete is relatively far away from the rebar segments. The closest rebar segment to this carbonated region has 1 in. of concrete cover; the rebar is not comprised.

Core No. 3-C14 is a 0.9-in.-thick offcut from a longer core. No rebar is present in this core sample.

Carbonation in No. 3-C14 is substantial. The majority of the paste is carbonated throughout the full depth of the core sample, with small amounts of noncarbonated paste along the bottom surface (Fig. 4). The non-carbonated paste appears to extend upwardly into the concrete in a relatively random nature. Due to the amount of carbonation, it is likely that the carbonated paste is present beyond the 0.9 in. portion of the core evaluated in this examination.

All information obtained in the examination is presented in the laboratory data forms at the end of this report.

METHODS OF TEST

Depth and pattern of paste carbonation was determined by application of a pH indicator solution (phenolphthalein) to a freshly saw-cut, longitudinal concrete surface of each core. The solution imparts a deep magenta stain to high pH, non-carbonated paste. Carbonated paste does not change color.

Meredith Strow

Jean L. Randolph

Meredith Strow
Petrography Group

Jean L. Randolph
Senior Petrographer and Group Manager
Petrography Group

MLS/JLR/

Notes: 1. Results refer specifically to the samples submitted.
2. This report may not be reproduced except in its entirety.

1a. **Top surface. Surface is a terrazzo-like concrete material. Red arrows point to a thin reinforcement plate.**

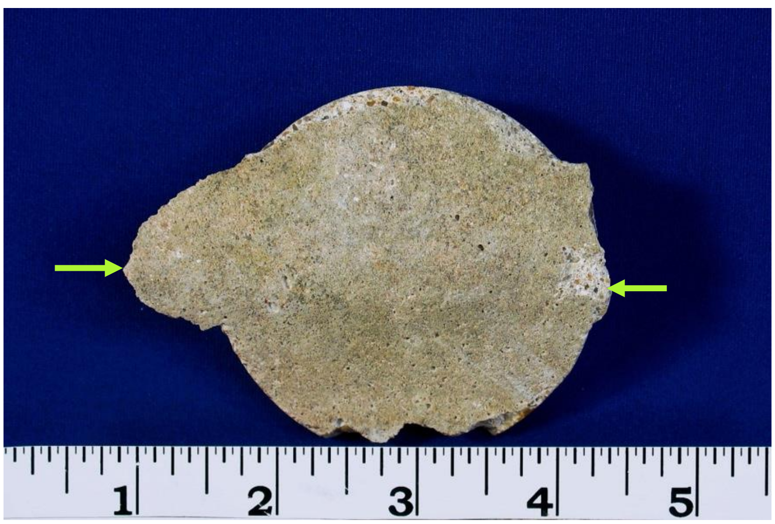
1b. **Side view of core. Core consists of a terrazzo-like concrete topping, with an underlying mortar-like layer, then the underlying substrate concrete. Three rebar segments (red arrows) are present in the bottom portion of the concrete. The concrete bottom surface is a formed, wavy corrugated shape.**

1c. **Core bottom surface. The surface is a formed wavy shape. Green arrows point to corrugated ridges.**

Fig. 1 Core No. 3-Big Core, as received in the Petrographic Laboratory for testing.

2a. Top of sample, which is a saw-cut surface.

2b. Side view of sample. The concrete bottom surface is a formed, wavy corrugated shape. Green arrow points to a corrugated ridge.



2c. Bottom of sample. Green arrows point to a corrugated ridge.

Fig. 2 Core No. 3-C14, as received in the Petrographic Laboratory for testing.

Fig. 3 Saw-cut, cross-sectional concrete surface of Core No. 3-Big Core. Phenolphthalein (a pH indicator solution) was applied to the surface to aid in carbonation assessment. Non-carbonated paste is deep magenta; carbonated paste did not change color. Four local regions of carbonated paste are present along the bottom surface; yellow arrows point to these regions and yellow bars and text designate depth into the concrete from the nearest bottom surface. Scale is in inches.

Fig. 4 Saw-cut, cross-sectional concrete surface of Core No. 3-C14. Phenolphthalein (a pH indicator solution) was applied to the surface to aid in carbonation assessment. Non-carbonated paste is deep magenta; carbonated paste did not change color. The majority of the paste is carbonated throughout the full depth of the concrete sample. A small amount of non-carbonated paste is present along the bottom surface and mottled upwardly into the concrete. Scale is in inches.

LABORATORY DATA FORM

STRUCTURE: Austin Fire Department
DATE RECEIVED: June 23, 2017

LOCATION: Austin, Texas
EXAMINED BY: Meredith Strow

SAMPLE

Client Identification: No. 3-Big Core.

CTLGroup Identification: 4402614.

Dimensions: Core diameter = 142 mm (5.6 in.), core length = 205 to 219 mm (8.1 to 8.6 in.); full structure thickness.

Top Surface: Flat, even, saw-cut terrazzo-type material surface.

Bottom Surface: Wavy, fairly smooth, formed concrete surface with corrugated ridges.

Cracks, Joints, Large Voids: None present.

Reinforcement:

- Three rebar segments are present in the bottom portion of the concrete; all three are oriented parallel to the top surface. Information regarding each segment is summarized below:
  - One 11-mm-dia. (0.4-in.-dia.) segment.
    - Located at depth of 168 mm (6.6 in.) from core top surface, or 112 mm (4.4 in.) from concrete top surface.
    - Concrete cover of 26 mm (1 in.) from the nearest bottom surface.
  - One segment has a diameter of 12 mm (0.5 in.) and has
    - Located at depth of 178 mm (7 in.) from core top surface, or 122 mm (4.8 in.) from concrete top surface.
    - Concrete cover of 17 mm (0.7 in.) from the nearest bottom surface.
    - This rebar segment was cut through at an angle and appears elongated on the lapped surface image.
  - One 6-mm-dia. (0.2-in.-dia.) segment.
    - Located at depth of 191 mm (7.5 in.) from core top surface, or 131 mm (5.2 in.) from concrete top surface.
    - Concrete cover of 12 mm (0.5 in.) from nearest bottom surface.

PASTE

Depth of Carbonation: Four local regions of carbonated paste are observed in the near-bottom region of the concrete. These regions extend from the bottom surface to depths of 7 mm (0.3 in.), 12.5 mm (0.5 in.), 10.5 mm (0.4 in.), and 10 mm (0.4 in.). No carbonated paste reaches rebar segments.

LABORATORY DATA FORM

STRUCTURE: Austin Fire Department
DATE RECEIVED: June 23, 2017

LOCATION: Austin, Texas
EXAMINED BY: Meredith Strow

SAMPLE

Client Identification: No. 3-C14.

CTLGroup Identification: 4475709-01.

Dimensions: Core diameter = 69 mm (2.7 in.). Core length = 22 mm (0.9 in.); partial structure thickness.

Top Surface: Flat, even, saw-cut concrete surface.

Bottom Surface: Wavy, fairly smooth, formed concrete surface with a corrugated ridge.

Cracks, Joints, Large Voids: None present.

Reinforcement: None present.

PASTE

Depth of Carbonation: The majority of the paste is carbonated throughout the full depth of the concrete sample. A small amount of non-carbonated paste is present along the bottom surface and mottled upwardly into the concrete.

Copy No. 1

Report for
City of Austin
105 W. Riverside Drive, Austin, Texas 78704

CTLGroup Project No. 231701

Paste Carbonation Determination on Core FS#22-C12 from Austin Fire Department Stations 3 and 22 Structural Capacity Assessment, Austin, Texas

August 8, 2017

Submitted by:
Jaclyn Ferraro
Jean L. Randolph

COA #F-3849

5400 Old Orchard Road
Skokie, Illinois 60077-1030
(847) 965-7500

Austin, TX • Bradenton, FL • Chicago, IL • Horsham, PA
Naperville, IL • Washington, DC • Doha, Qatar

www.CTLGroup.com

REPORT OF PASTE CARBONATION DETERMINATION

Date: August 8, 2017

CTLGroup Project No.: 231701

Paste Carbonation Determination on Core FS#22-C12 from Austin Fire Department Stations 3 and 22 Structural Capacity Assessment, Austin, Texas

One concrete core sample, identified as FS#22-C12, was received August 1, 2017, in the CTLGroup Petrographic Laboratory from Mr. Bradley East, CTLGroup Engineer, on behalf of the City of Austin, Texas. The core was received with saw-cut ends that are covered with a capping compound. Determination of paste carbonation in the concrete core was requested. This report presents the details and results of the analysis.

FINDINGS AND CONCLUSIONS

No paste carbonation is observed in the concrete of Core FS#22-C12 (Fig. 1). The sample contains one 4-mm-diameter (0.2-in.-dia.) wire mesh segment. All information obtained in the examination is presented in the laboratory data form at the end of this report.

METHODS OF TEST

Pattern of paste carbonation was determined by application of a pH indicator solution (phenolphthalein) to a freshly saw-cut, longitudinal concrete surface and fresh fractured surface of the core. The solution imparts a deep magenta stain to high pH, non-carbonated paste. Carbonated paste does not change color.

Jaclyn Ferraro
Petrography Group

Jean L. Randolph
Senior Petrographer and Group Manager
Petrography Group

JMF/JLR/

Notes: 1. Results refer specifically to the sample submitted.
2. This report may not be reproduced except in its entirety.

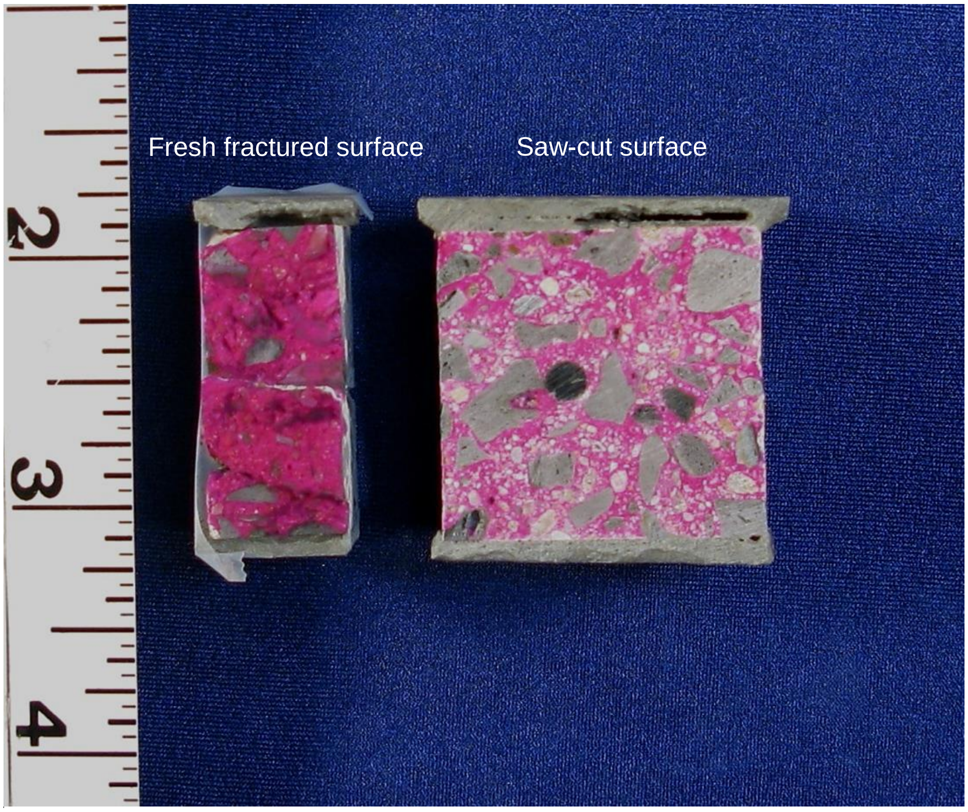


Fig. 1 Core FS#22-C12, after being saw-cut longitudinally in the Petrographic Laboratory. One resultant longitudinal saw-cut surface is shown on the right. The other longitudinal saw-cut surface was freshly fractured in the laboratory (left). Phenolphthalein (a pH indicator solution) was applied to these surfaces to determine localities of paste carbonation in the concrete. Non-carbonated paste is deep magenta; carbonated paste does not change color. In the core specimen, no carbonation is observed. Scale is in inches.

LABORATORY DATA FORM

STRUCTURE: Austin Fire Department

DATE RECEIVED: August 1, 2017

LOCATION: Austin, Texas

EXAMINED BY: Jaclyn Ferraro

SAMPLE

Client Identification: FS#22-C12.

CTLGroup Identification: 4475713.

Dimensions: Core diameter = 32 mm (1.3 in.). Core length without capping compound = 31 mm (1.2 in.); partial structure thickness.

Top and Bottom Surfaces: Saw-cut concrete surface covered by a capping compound.

Cracks, Joints, Large Voids: None present.

Reinforcement: One 4-mm-diameter (0.2-in.-dia.) wire mesh segment is present within the core.

PASTE

Depth of Carbonation: None observed.

Appendix C

Structural Analyses – Fire Station No. 3

STRUCTURAL ANALYSIS AND DESIGN REVIEW OF

CITY OF AUSTIN

FIRE STATION NO. 3

PHASE 2

This appendix describes the analysis and design review of Fire Station No. 3 floor system.

DESCRIPTION OF THE FLOOR SYSTEM

Fire Station No. 3 floor system is described in the main body of the report.

CODES AND STANDARDS

The design review of Fire Station No. 3 floor system is based on ACI 318-14.

MATERIAL PROPERTIES

An equivalent concrete compressive strength of 2639 psi is obtained from the statistical analysis of the concrete core test data. An elastic modulus of 2928 ksi is calculated per ACI 318-14 Equation 19.2.2.1.b. A weight density of 141 pcf is obtained from the concrete core test data and used in the structural analysis.

Mild deformed reinforcing steel is assumed to have a minimum yield strength equal to 40,000 psi based on the age of the structure. The structure reportedly was constructed in the 1950’s.

The material properties used in the analyses are summarized in Table 1.

MEMBER CAPACITIES

The flexural capacity of slab and beams are calculated using the spColumn computer program as shown in Figures 1 to 7.

The shear capacity of slab and beams are calculated and summarized in Table 2.

LOADS

Gravity dead load includes the self-weight of the floor system. The self-weight of the floor system is calculated using a weight density of 141 pcf.

Gravity live load includes a ladder truck in one bay and an engine truck in the other bay. The ladder truck weight and wheel footprint calculations are shown in Figure 8. The engine truck weight and wheel footprint calculations are shown in Figure 9.

In structural analysis, the length of the tire footprint (parallel to traffic direction) is assumed 10 inches and the width of the footprint (normal to traffic direction) is assumed 20 inches similar to those of a standard truck per AASHTO LRFD 2010.

No other live loads besides the truck loads are considered in the structural analyses.

FLOOR SLAB ANALYSIS

A three-span strip of the floor slab is analyzed under dead and live loads. The analysis model is shown in Figures 10 and 11. The effective width of one-way slab is calculated per AASHTO LRFD 2010 as shown in Table 3. Based on these results, an effective width of 99 in. is assumed for a single axle and an effective width of 151 in. is assumed for a tandem axle with 52 in. spacing between the parallel axles.

An analysis of the slab strip is conducted under a 27-kip axle load in the left bay and a 27-kip axle load in the right bay as shown in Figures 12 and 13. In this analysis, possible truck/axle positions are considered to be anywhere between a far left position and a far right position within the bay.

The shear force and bending moment envelopes are shown in Figures 14 and 15. The maximum shear force, positive bending moment, and negative bending moment from these envelope diagrams constitute the maximum demand (D) on the slab.

FLOOR SLAB DESIGN REVIEW

The slab strip capacity (C) is obtained by multiplying the unit-wide strip capacities and the strip width.

The slab shear force and bending moment demand capacity ratios (DCR) are summarized in Table 4.

The slab punching shear demand capacity ratio (DCR) under a wheel load is calculated in Table 5.

FLOOR SLAB REACTIONS

Figure 16 shows the slab reactions as the axle is positioned from one side of the left bay to the other side of the left bay. Figure 17 shows the slab reactions as the axle is positioned from one side of the right bay to the other side of the right bay. These reactions are used to calculate the percentage of the axle load that is carried by each support as shown in Table 6.

WIDE BEAM ANALYSIS

A four-span continuous beam model of the wide beam is analyzed under dead and live loads. The analysis model is shown in Figures 18 and 19.

Moving load analyses of the wide beam are conducted under a ladder truck and an engine truck as shown in Figures 20 to 23. The results of these analyses are scaled by the percentages shown in Table 5 and combined.

The shear force and bending moment envelopes are shown in Figures 24 and 25. The maximum shear force, positive bending moment, and negative moment from these envelope diagrams constitute the maximum demand (D) on the wide beam.

WIDE BEAM DESIGN REVIEW

The wide beam shear force and bending moment demand capacity ratios (DCR) are summarized in Table 7.

NARROW BEAM ANALYSIS

A four-span continuous beam model of the wide beam is analyzed under dead and live loads. The analysis model is shown in Figures 26 and 27.

Moving load analyses of the wide beam are conducted under a ladder truck and an engine truck similar to those shown in Figures 20 to 23. The results of these analyses are scaled by the percentages shown in Table 5 and combined.

The shear force and bending moment envelopes are shown in Figures 28 and 29. The maximum shear force, positive bending moment, and negative moment from these envelope diagrams constitute the maximum demand (D) on the wide beam.

NARROW BEAM DESIGN REVIEW

The wide beam shear force and bending moment demand capacity ratios (DCR) are summarized in Table 8.

RESULTS SUMMARY

The slab, wide beam, and narrow beam shear force and bending moment demand capacity ratios (DCR) are summarized in Table 9.

ANALYSIS NOTES

In the current analyses, the ends of the slab strip and the ends of beams are assumed fixed against rotation. An alternative pinned assumption will also be considered in the final retrofit design.

In the current analyses, the shear demand is evaluated at the face of the supports. A small reduction in the shear demand will be considered in the final retrofit design by evaluating shear at a distance equal to effective depth from the face of the support.

In the current analyses, two different types of truck in the left and right bays of the fire station are considered. Per information provided by client, the case of two heavy ladder trucks on adjacent bays need not be considered.

Table 1: Material properties

Concrete compressive strength	f'_c	psi	2639
Concrete modulus of elasticity	E_c	ksi	2928
Concrete Poisson's ratio	n	---	0.2
Concrete weight density	g	pcf	141
Concrete modulus of rupture	f_r	psi	385
Concrete direct tensile strength	f_t	psi	205
Reinforcementyield stress	f_y	ksi	40

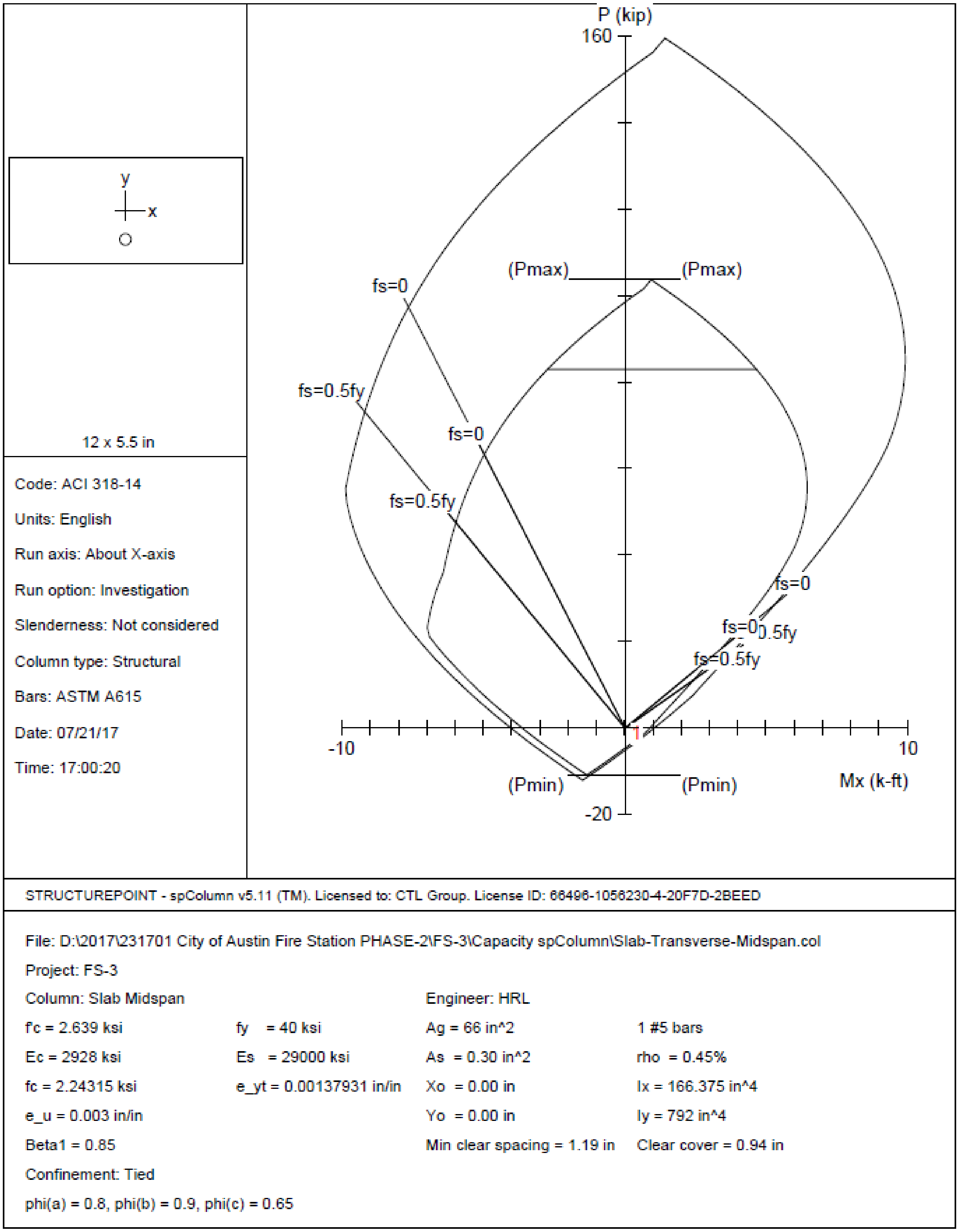
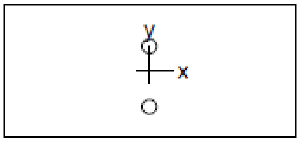


Figure 1: Flexural capacity of 1-ft wide slab strip in transverse direction at midspan



12 x 5.5 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

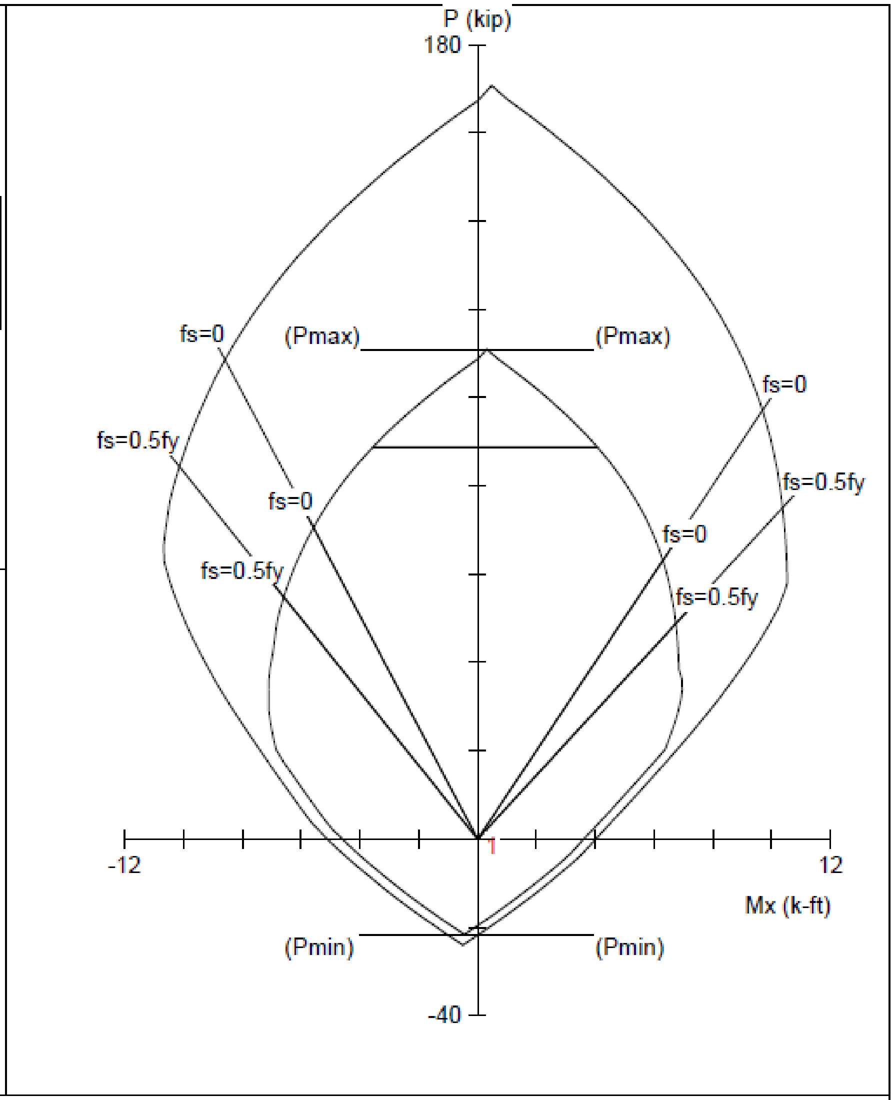
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 07/21/17

Time: 17:01:42



STRUCTUREPOINT - spColumn v5.11 (TM). Licensed to: CTL Group. License ID: 66496-1056230-4-20F7D-2BEED

File: D:\2017\231701 City of Austin Fire Station PHASE-2\FS-3\Capacity spColumn\Slab-Transverse-Support.col

Project: FS #3

Column: Slab Support		Engineer: HRL	
f'c = 2.639 ksi	fy = 40 ksi	Ag = 66 in^2	2 #5 bars
Ec = 2928 ksi	Es = 29000 ksi	As = 0.60 in^2	rho = 0.91%
fc = 2.24315 ksi	e_yt = 0.00137931 in/in	Xo = 0.00 in	Ix = 166.375 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 792 in^4
Beta1 = 0.85		Min clear spacing = 1.88 in	Clear cover = 0.94 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

Figure 2: Flexural capacity of 1-ft wide slab strip in transverse direction at support

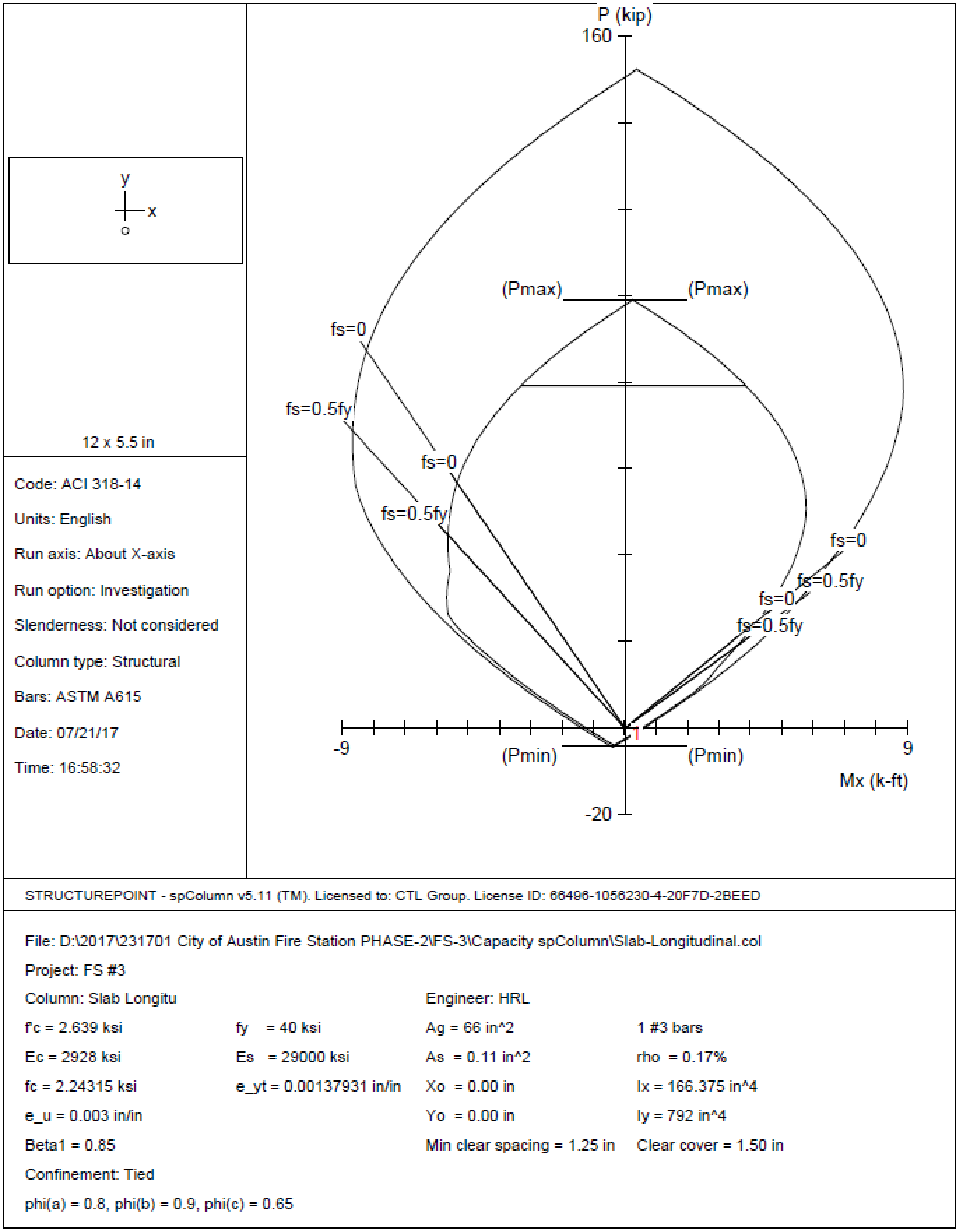
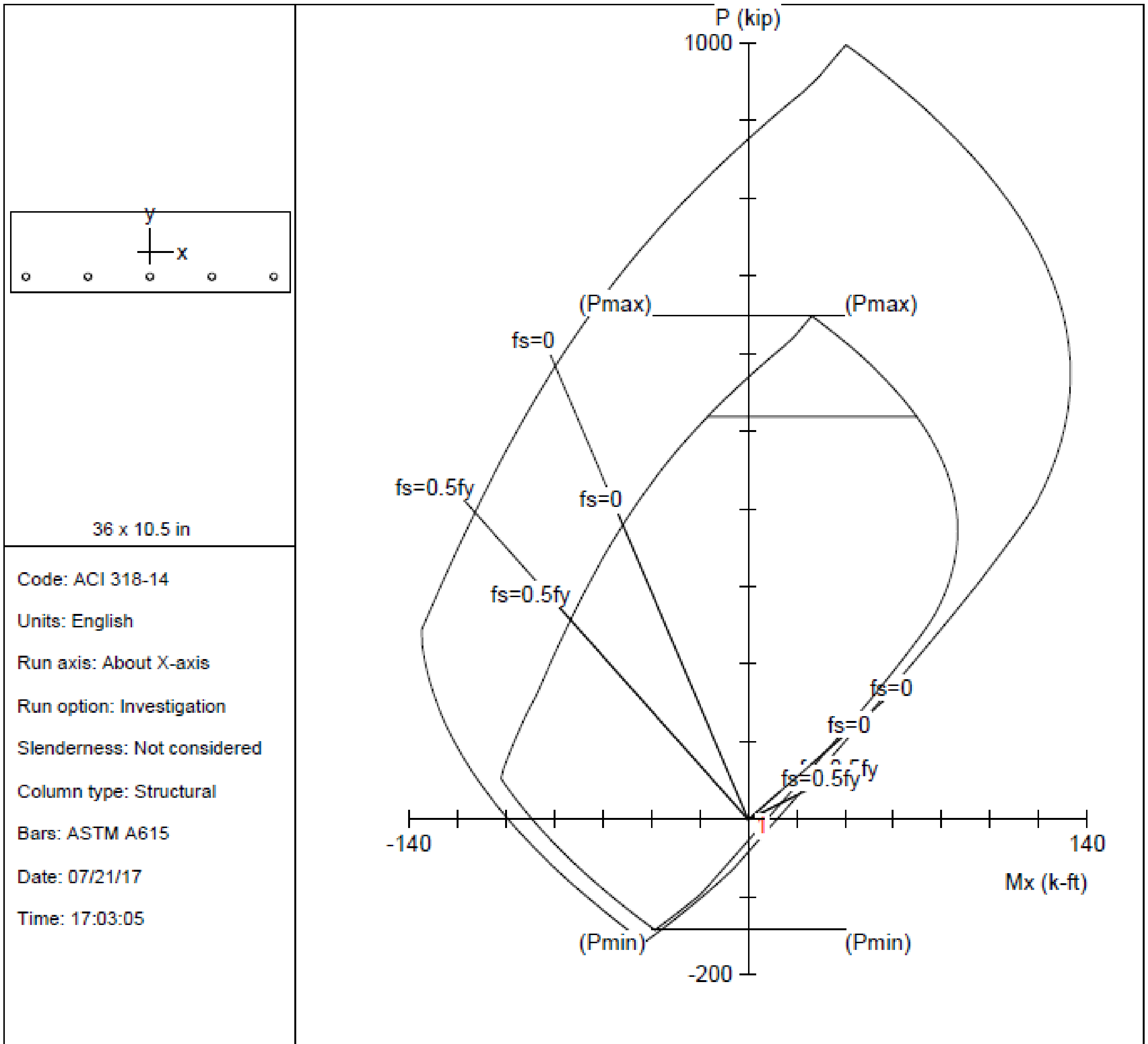


Figure 3: Flexural capacity of 1-ft wide slab strip in longitudinal direction



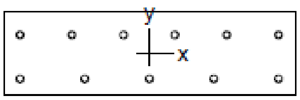
STRUCTUREPOINT - spColumn v5.11 (TM). Licensed to: CTL Group. License ID: 66496-1056230-4-20F7D-2BEED

File: D:\2017\231701 City of Austin Fire Station PHASE-2\FS-3\Capacity spColumn\Wide-Beam-Midspan.col

Project: FS #3

Column: 3-ft Midspan		Engineer: HRL	
f'c = 2.639 ksi	fy = 40 ksi	Ag = 378 in^2	5 #8 bars
Ec = 2928 ksi	Es = 29000 ksi	As = 3.95 in^2	rho = 1.04%
fc = 2.24315 ksi	e_yt = 0.00137931 in/in	Xo = 0.00 in	Ix = 3472.88 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 40824 in^4
Beta1 = 0.85		Min clear spacing = 7.00 in	Clear cover = 1.50 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

Figure 4: Flexural capacity of wide beam at midspan



36 x 10.5 in

Code: ACI 318-14

Units: English

Run axis: About X-axis

Run option: Investigation

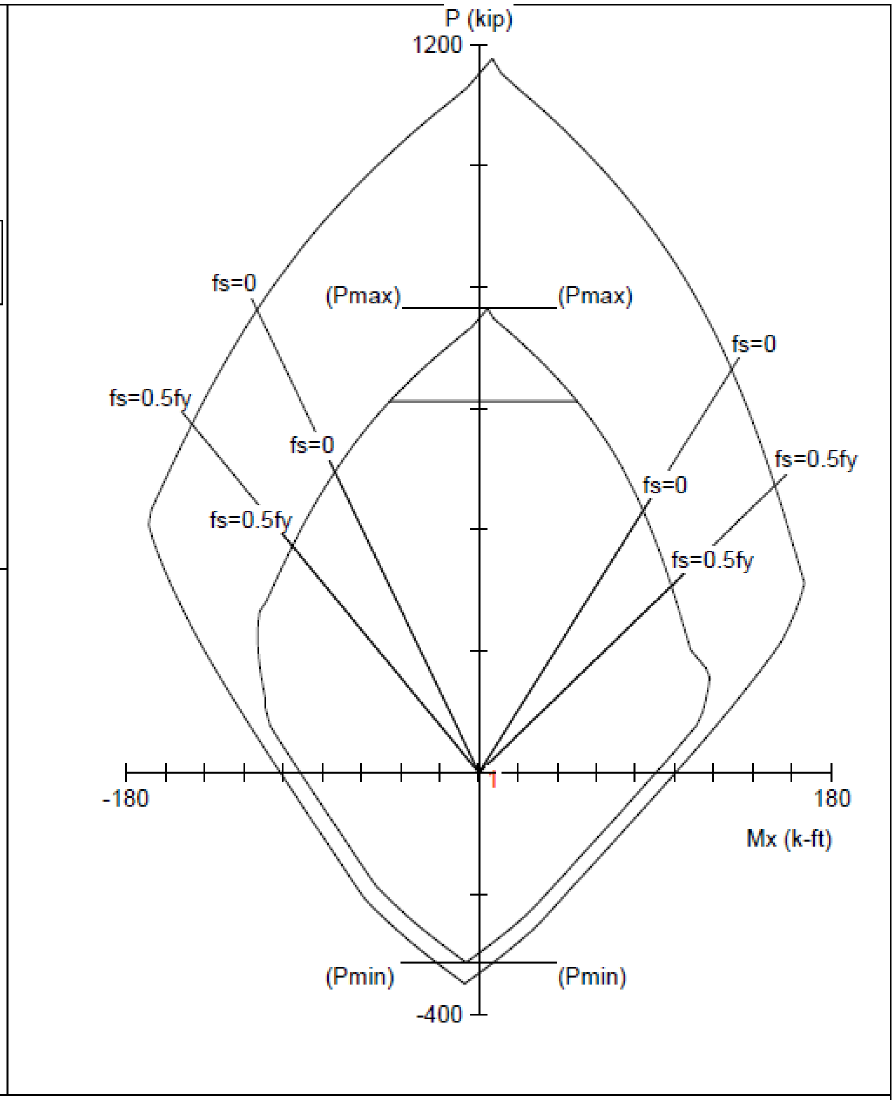
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 07/21/17

Time: 17:04:21



STRUCTUREPOINT - spColumn v5.11 (TM). Licensed to: CTL Group. License ID: 66496-1056230-4-20F7D-2BEED

File: D:\2017\231701 City of Austin Fire Station PHASE-2\FS-3\Capacity spColumn\Wide-Beam-Support.col

Project: FS #3

Column: 3-ft Support		Engineer: HRL	
f'c = 2.639 ksi	fy = 40 ksi	Ag = 378 in^2	11 #8 bars
Ec = 2928 ksi	Es = 29000 ksi	As = 8.69 in^2	rho = 2.30%
fc = 2.24315 ksi	e_yt = 0.00137931 in/in	Xo = 0.00 in	Ix = 3472.88 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 40824 in^4
Beta1 = 0.85		Min clear spacing = 4.50 in	Clear cover = 1.50 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

Figure 5: Flexural capacity of wide beam at support

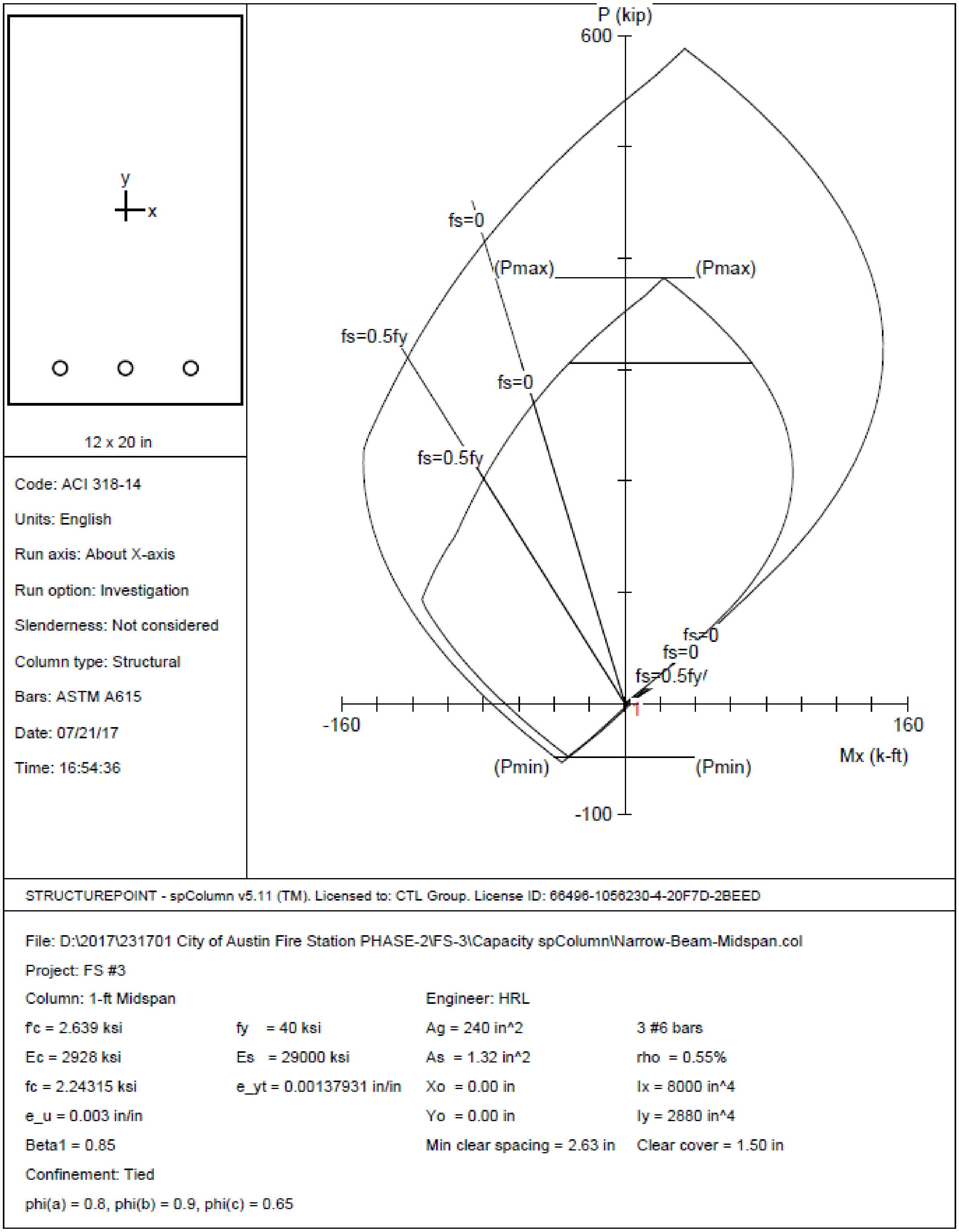


Figure 6: Flexural capacity of narrow beam at midspan

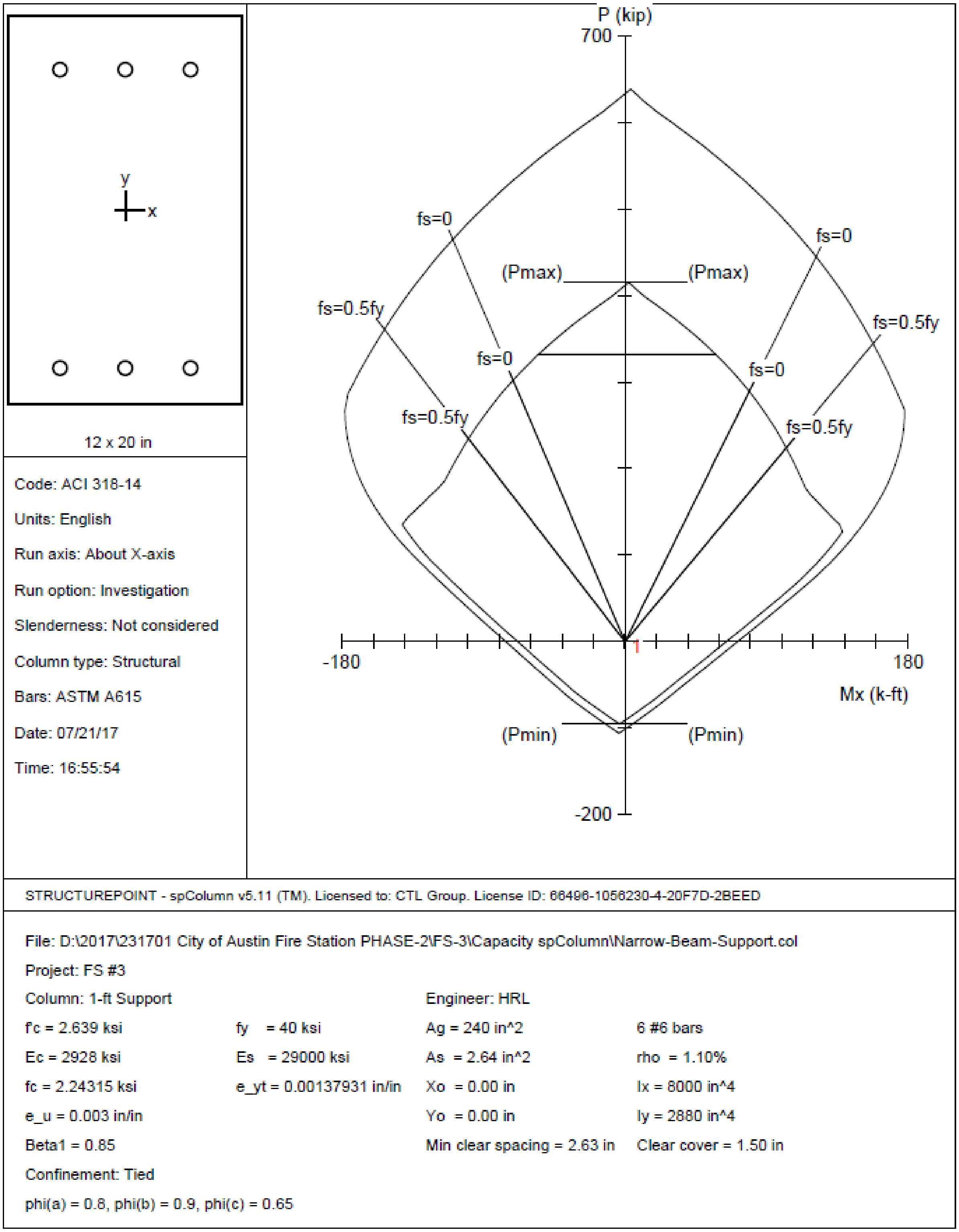
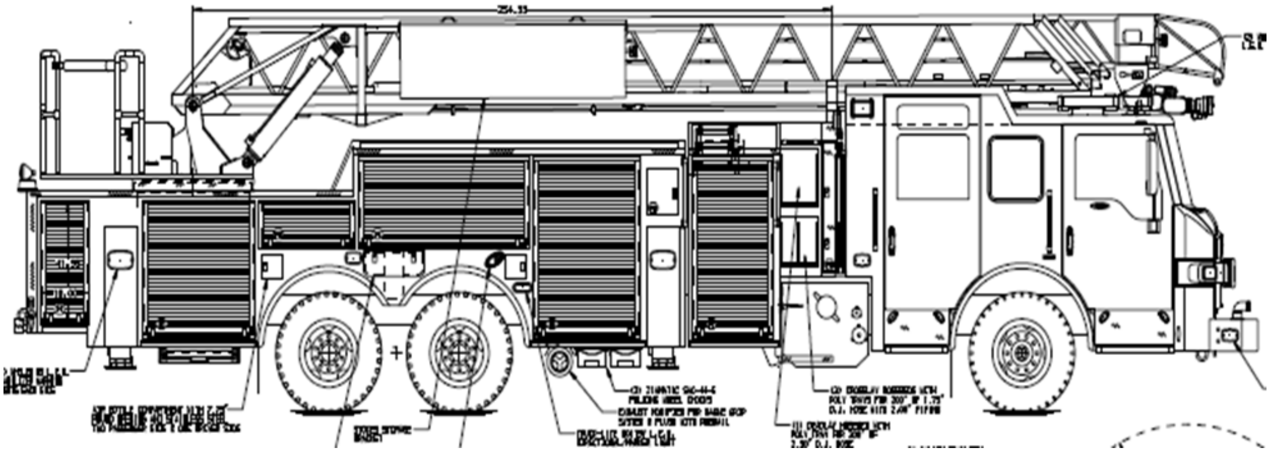


Figure 7: Flexural capacity of narrow beam at support

Table 2: Shear capacity

Member		Slab	Slab	Slab	Wide	Narrow
		1-ft Strip	99-in. Strip	151-in. Strip	Beam	Beam
f'c	psi	2,639	2,639	2,639	2,639	2,639
b	in	12	99	151	36	12
d	in	4.25	4.25	4.25	8.50	18.13
f	---	0.75	0.75	0.75	0.75	0.75
Vc	kips/ft	5.2	43.2	65.9	31.4	22.3
f Vc	kips/ft	3.9	32.4	49.5	23.6	16.8
Stirrups		---	---	---	---	#4@9
Av	in2					0.4
fy	psi					40
s	in					9
Vs	kip					32.2
f Vs	kip					24.2
f Vn	kip	3.93	32.4	49.5	23.6	40.9



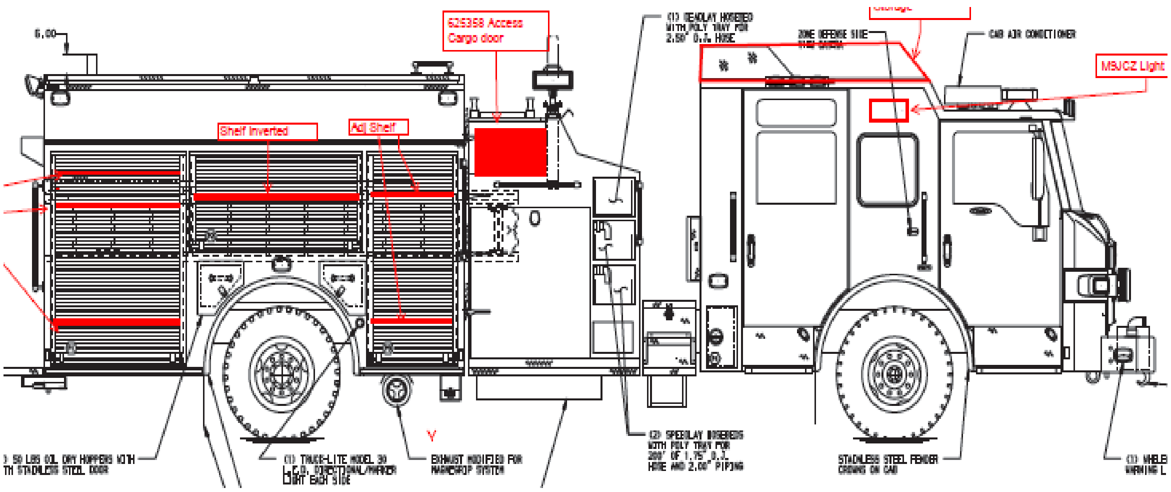
Front Axle	Rear Axle	Total
lb	lb	lb
22,800	54,000	76,800

	Axle	Axle Weight	Wheel Weight	Area	Length	Width	Pressure
Wheel footprint per CalTrans 2004 Section 3.3		(lb)	(lb)	(in^2)	(in)	(in)	(psi)
	front	22,800	11400	114	6.8	16.9	100
	rear	54,000	13500	135	7.3	18.4	100

	Axle	Axle Weight	Wheel Weight	Area	Length	Width	Pressure	g	IM
Wheel footprint per AASHTO 2010 Section 3.6.1.2.5		(lb)	(lb)	(in^2)	(in)	(in)	(psi)		
	front	22,800	11400	91.2	6.4	14.3	125	1	0
	rear	54,000	13500	108	6.4	16.9	125	1	0

Standard truck wheel footprint per AASHTO 2010 Section 3.6.1.2.5	Length	Width
	a	b
	(in)	(in)
	10.0	20.0

Figure 8: Ladder truck



Front Axle	Rear Axle	Total
lb	lb	lb
22,800	27,000	49,800

	Axle	Axle Weight	Wheel Weight	Area	Length	Width	Pressure
Wheel footprint per CalTrans 2004 Section 3.3		(lb)	(lb)	(in^2)	(in)	(in)	(psi)
	front	22,800	11400	114	6.8	16.9	100
	rear	27,000	13500	135	7.3	18.4	100

Wheel footprint per AASHTO 2010 Section 3.6.1.2.5	Axle	Axle Weight	Wheel Weight	Area	Length	Width	Pressure	g	IM
		(lb)	(lb)	(in^2)	(in)	(in)	(psi)		
	front	22,800	11400	91	6.4	14.3	125	1	0
	rear	27,000	13500	108	6.4	16.9	125	1	0

Standard truck wheel footprint per AASHTO 2010 Section	Length	Width
	a	b
	(in)	(in)
	10.0	20.0

Figure 9: Engine truck

Figure 10: Continuous 1-ft strip model of slab

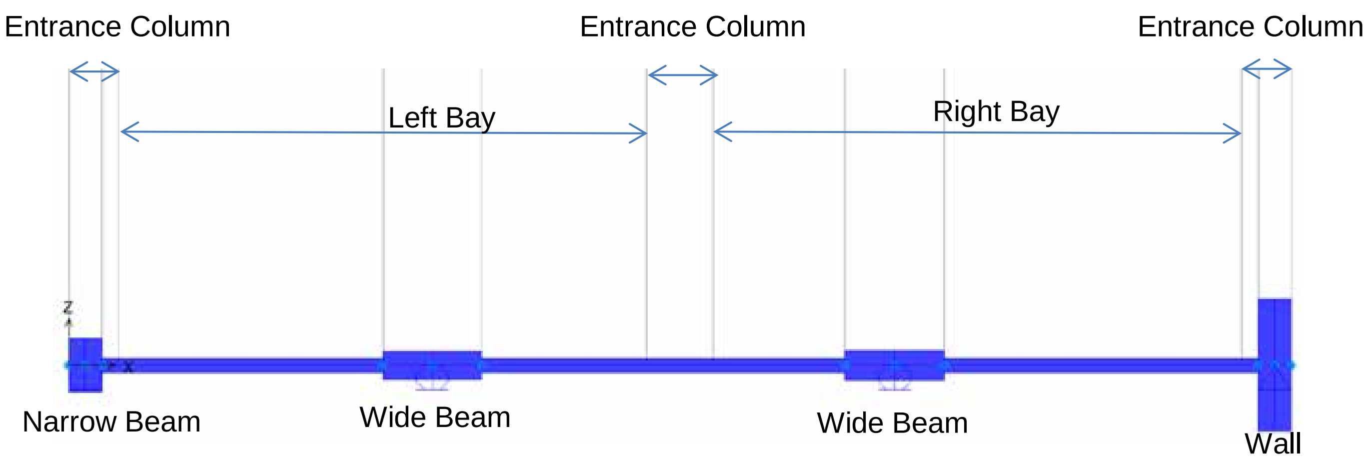


Figure 11: Continuous 1-ft strip model of slab showing member thicknesses

Table 3: Effective width of one-way slab per AASHTO LRFD 2010 Section 4.6.2.1.3

Span		Left	Middle	Right
Span length	ft	8.5	11	9.5
Width for M +ve	in.	82	99	89
Width for M -ve	in.	74	81	77

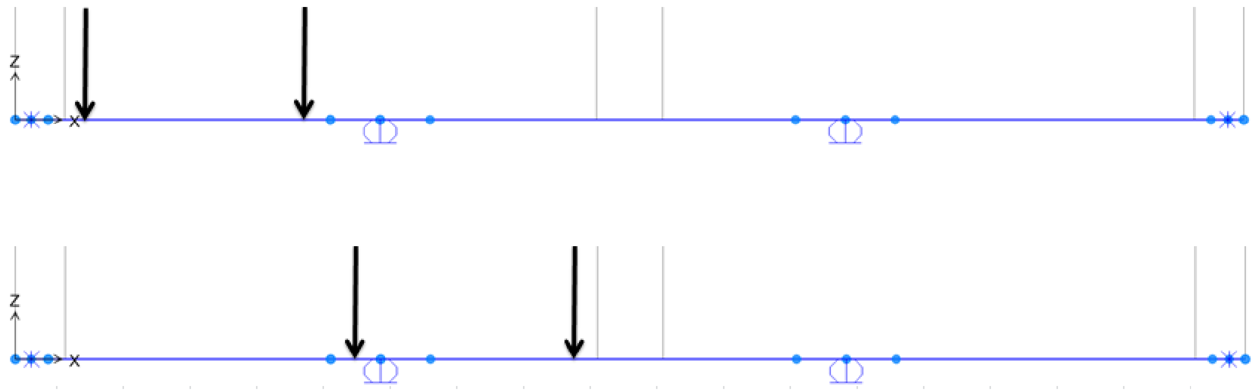


Figure 12: 27-kip axle extreme positions in the left bay

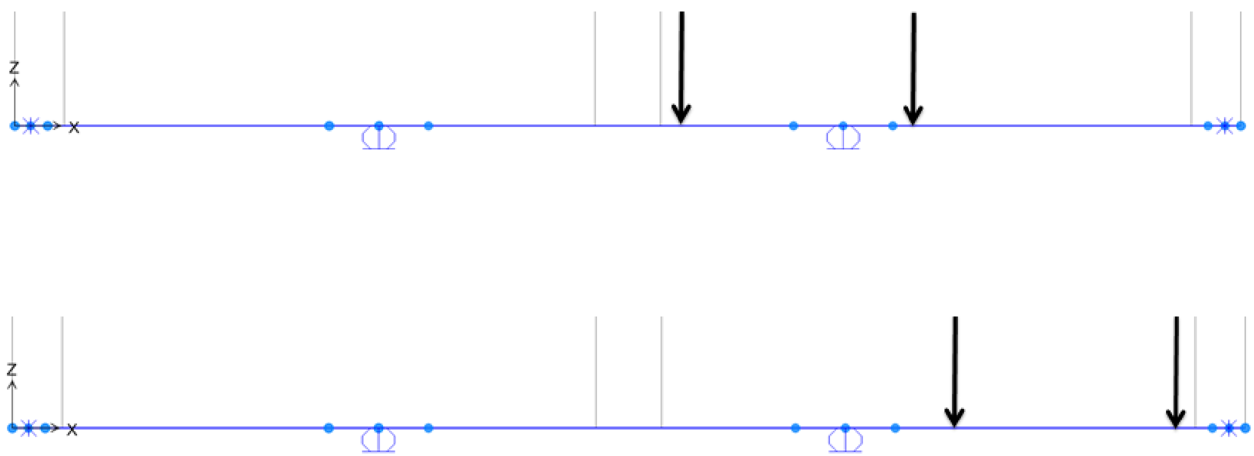


Figure 13: 27-kip axle extreme positions in the right bay

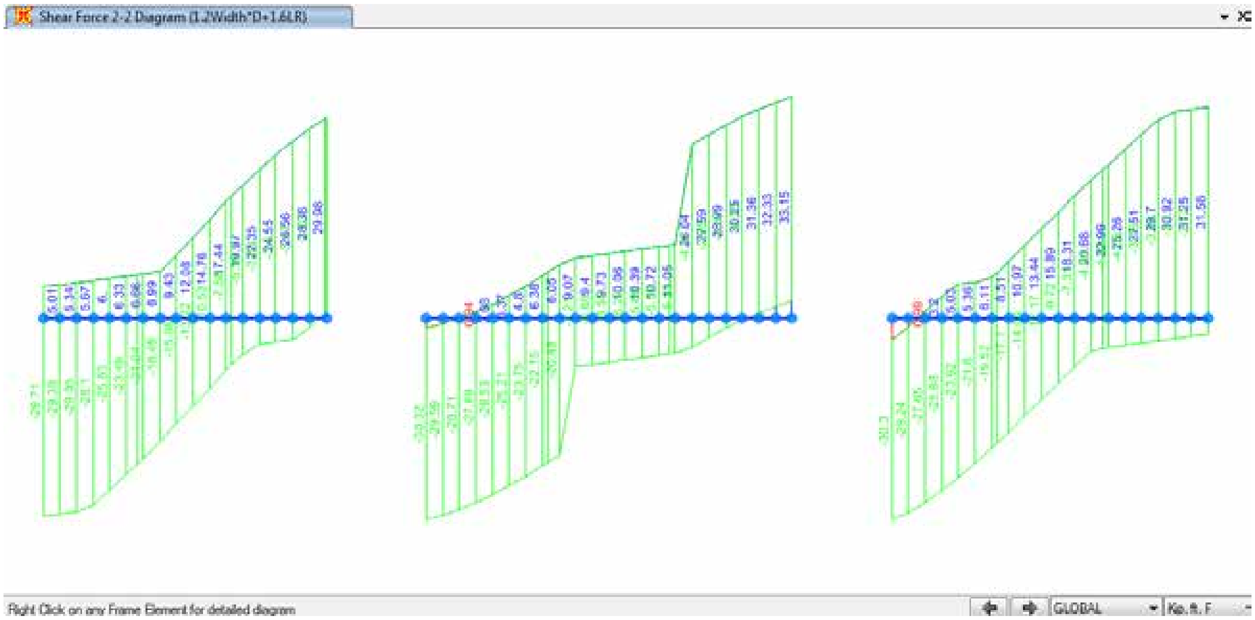


Figure 14: Shear force envelope due to factored self-weight of 99-in. strip plus factored truck loads in left and right bays

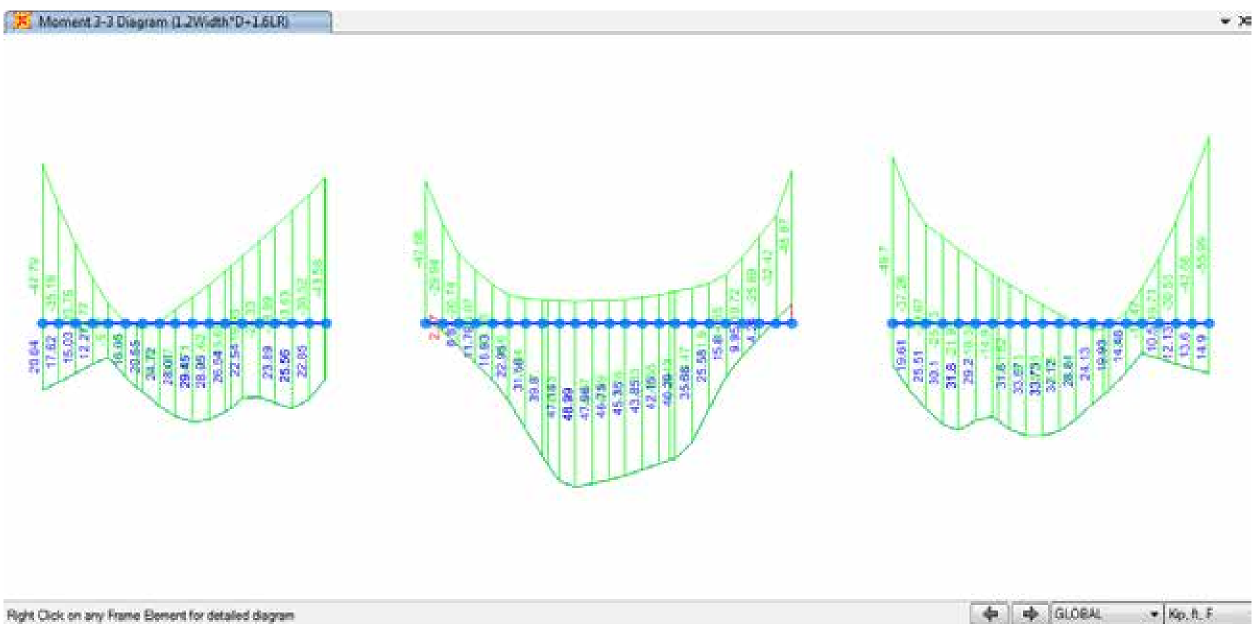


Figure 15: Bending moment envelope due to factored self-weight of 99-in. strip plus factored truck loads in left and right bays

Table 4: Slab shear force and bending moment demand capacity ratios (DCR)

99 in - Slab Strip under a Single Axle	Vu	f Vn	DCR-v	Mu+ve	f Mn+ve	DCR-M+ve	Mu-ve	f Mn-ve	DCR-M-ve
	kip	kip	---	ft-kip	kip	---	ft-kip	kip	---
	33.2	32.4	**1.02**	49.0	29.9	**1.64**	56.0	29.7	**1.89**

151 in - Slab Strip under a Tandem Axle	Vu	f Vn	DCR-v	Mu+ve	f Mn+ve	DCR-M+ve	Mu-ve	f Mn-ve	DCR-M-ve
	kip	kip	---	ft-kip	kip	---	ft-kip	kip	---
	66.3	49.5	**1.34**	98.0	45.6	**2.15**	112.0	45.3	**2.47**

Table 5: Slab punching shear demand capacity ratio (DCR)

Wheel Weight	kips	11.4	13.5
f'_c	psi	2639	2639
l	---	1	1
f	---	0.75	0.75
Type	---	interior	interior
c_1	in.	6.40	6.40
c_2	in.	14.30	16.90
d	in.	3.81	3.81
V_u	kips	18.2	21.6
M_x	kips.in	0	0
M_y	kips.in	0	0
b_0	in.	57	62
A_c	in^2	216	236
J_{cx}	in^4	4372	4889
J_{cy}	in^4	10330	14189
v_{u1}	psi	84	92
v_{ux}	psi	0	0
v_{uy}	psi	0	0
$\lvert v_{u1} \rvert$	psi	84	92
$\lvert v_{ux} \rvert$	psi	0	0
$\lvert v_{uy} \rvert$	psi	0	0
v_u	psi	84	92
b	---	2.23	2.64
a_s	---	40.00	40.00
4	---	4.00	4.00
2 + 4/b	---	3.79	3.51
$2 + a_s d / b_0$	---	4.69	4.47
f v_c	psi	146	135
DCR	---	0.58	0.68

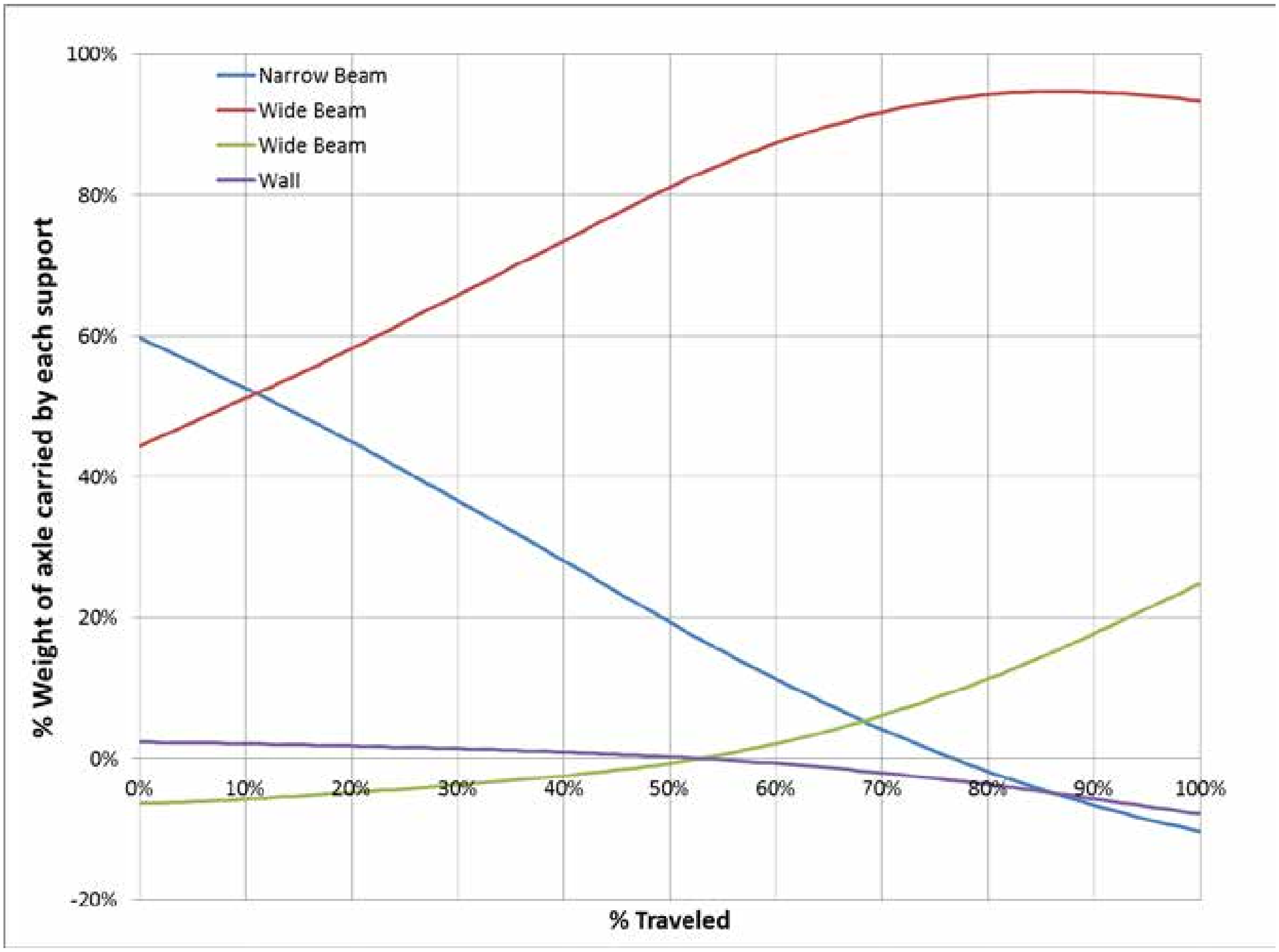


Figure 16: Reactions due to 27-kip axle positions in the transverse direction in left bay

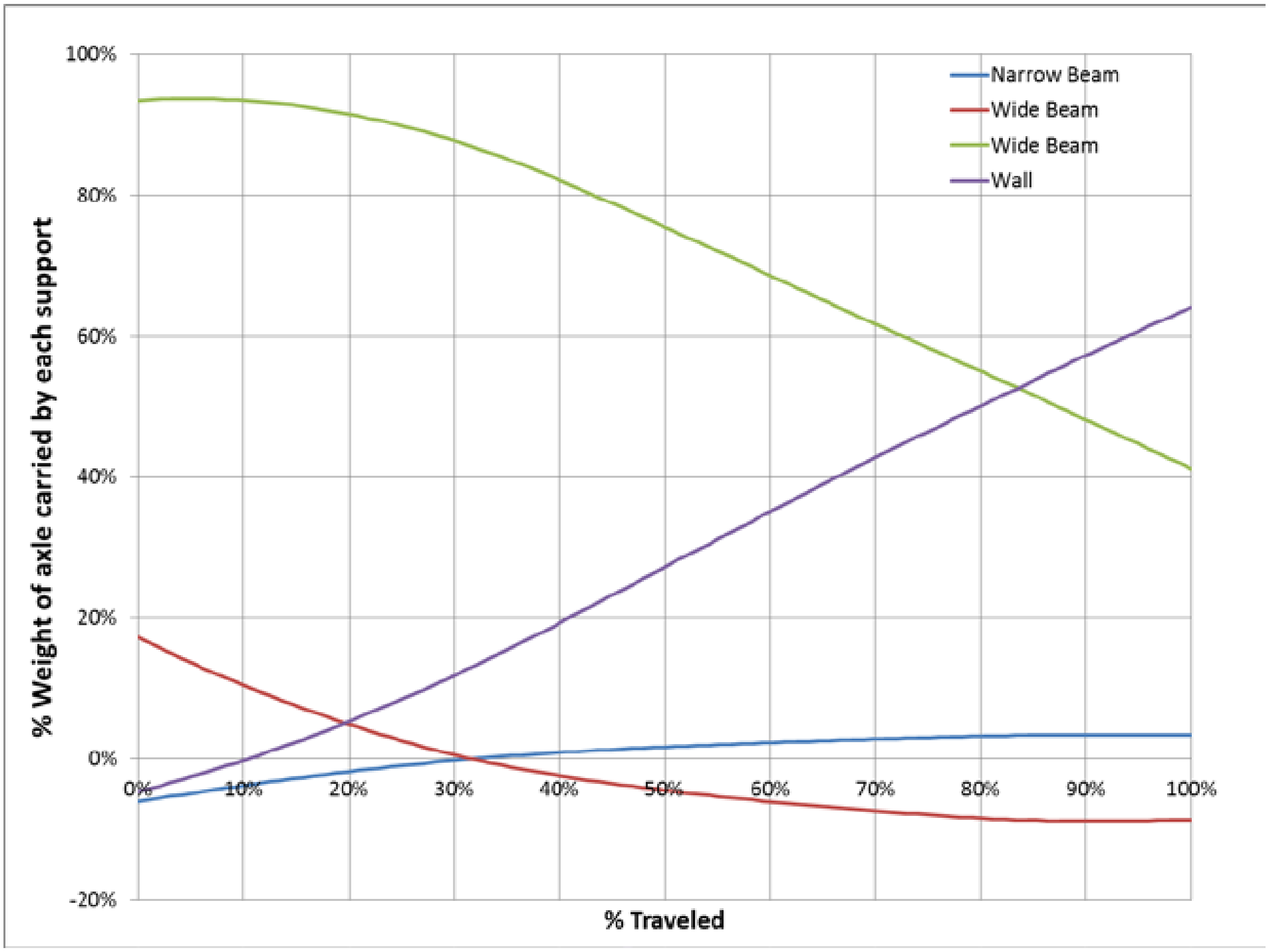


Figure 17: Reactions due to 27-kip axle positions in the transverse direction in right bay

Table 6: Percentage of the axle load that is carried by each support

	Narrow Beam	Wide Beam	Wide Beam	Wall
Truck on left bay	60%	95%	25%	2%
Truck on right bay	3%	17%	94%	64%

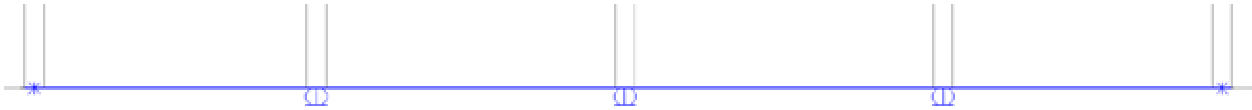


Figure 18: Continuous model of wide beam

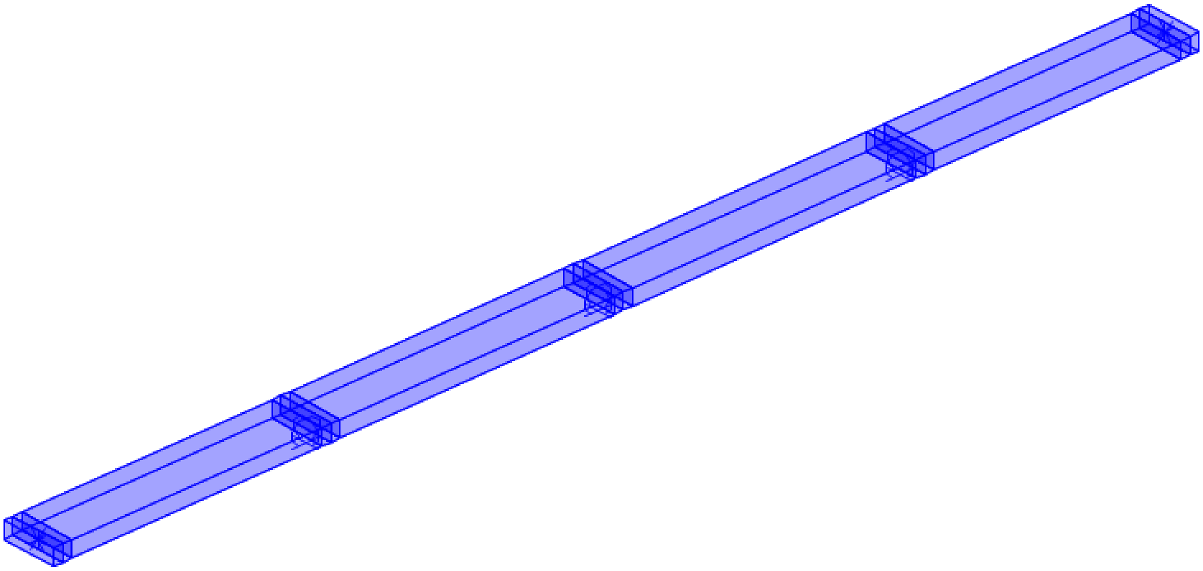


Figure 19: Model of wide beam showing member cross section

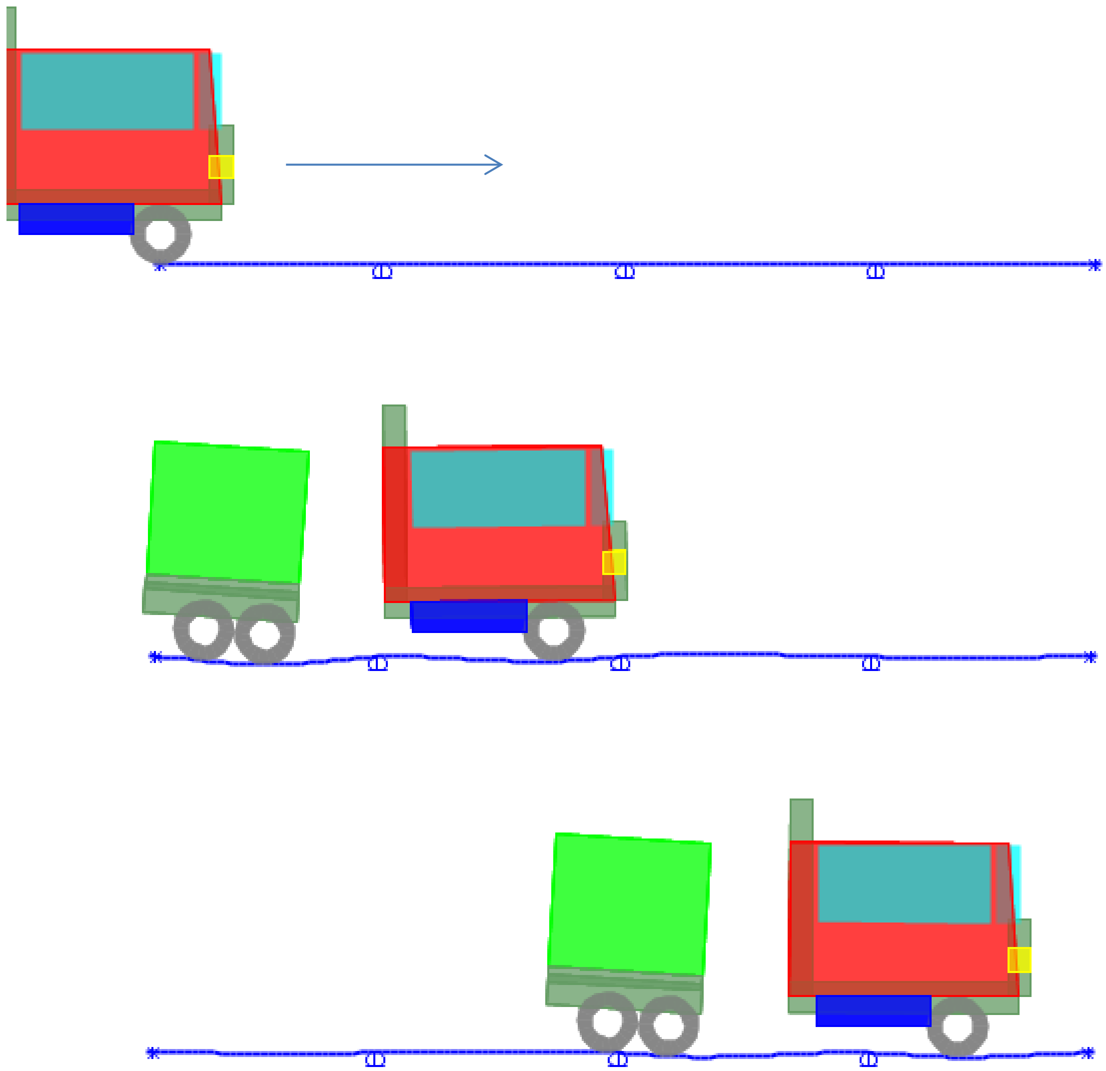


Figure 20: Ladder truck moving inside

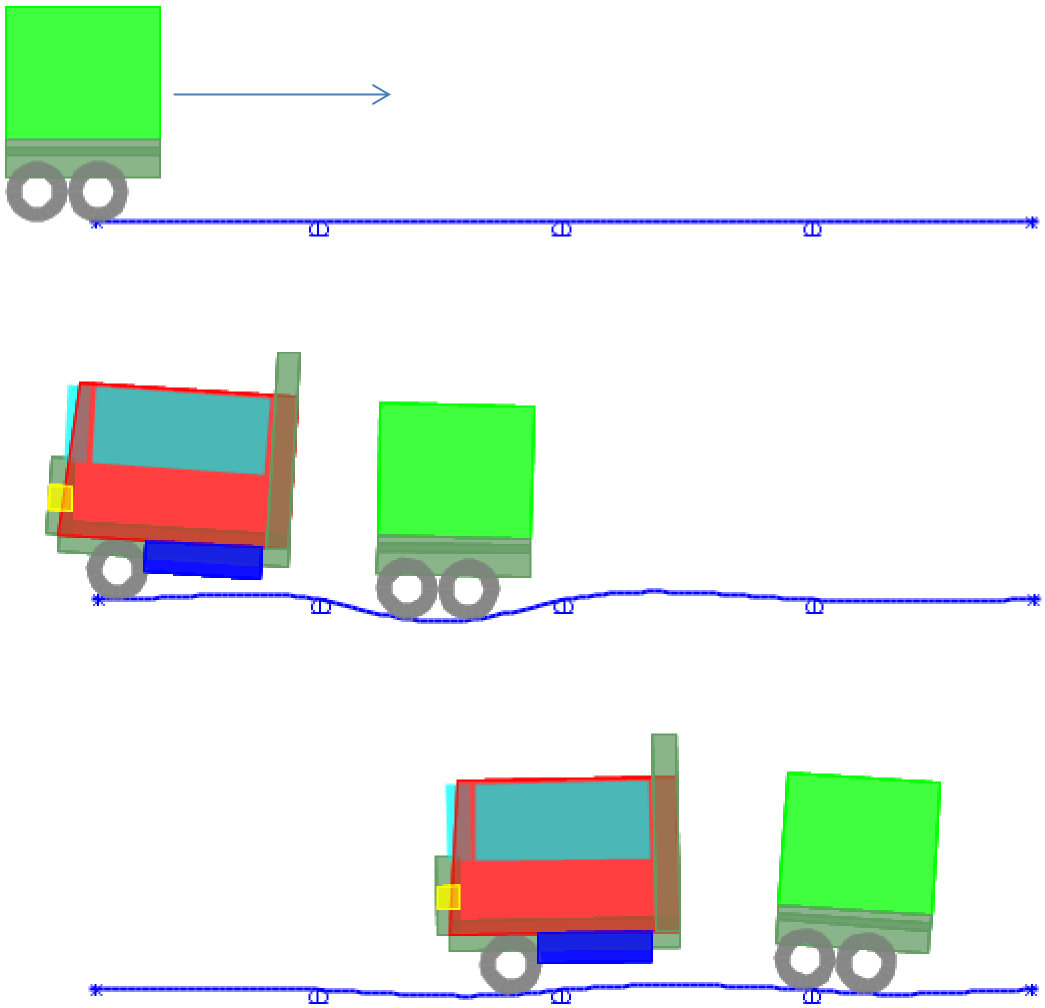


Figure 21: Ladder truck backing up

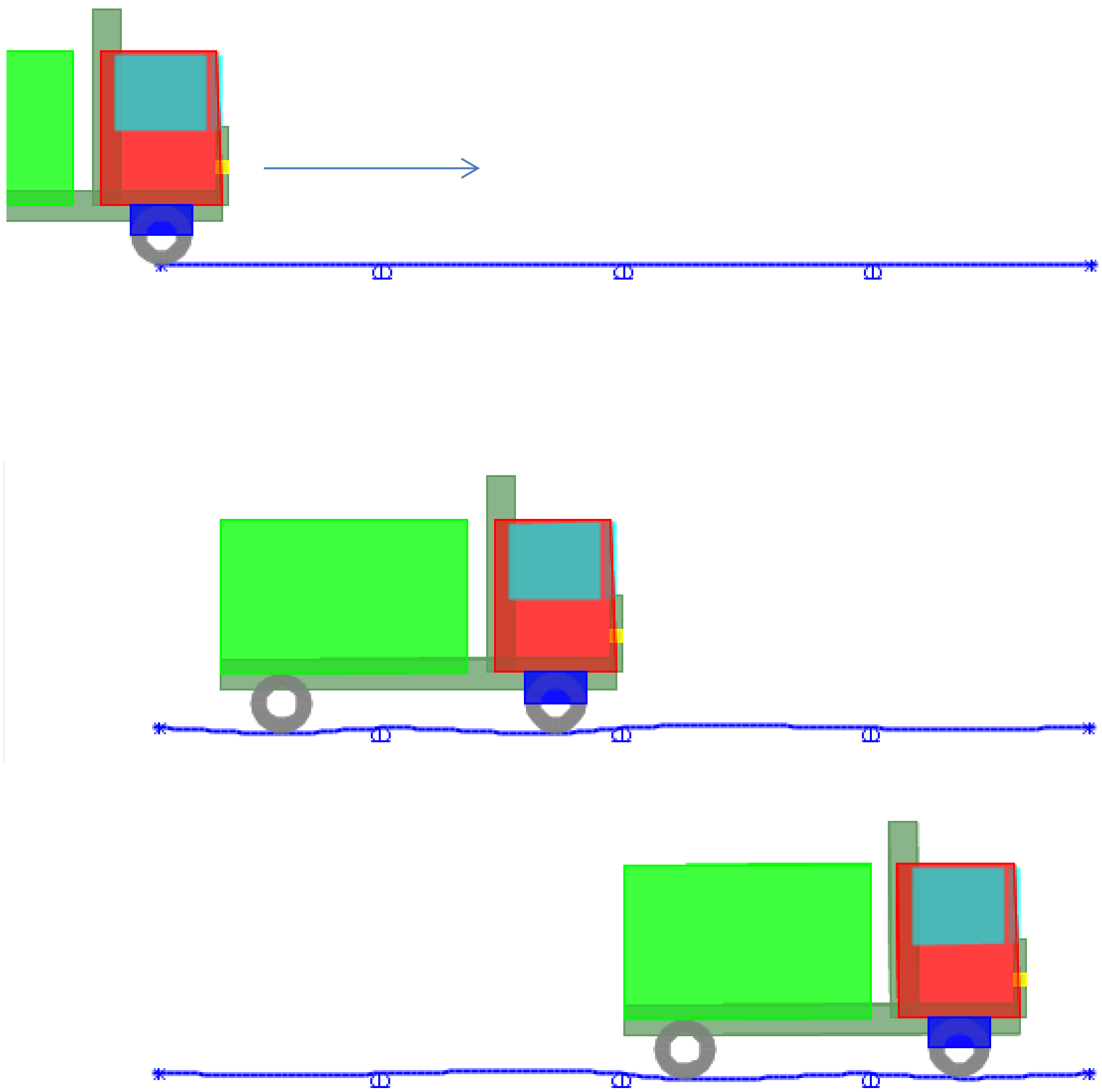


Figure 22: Engine truck moving inside

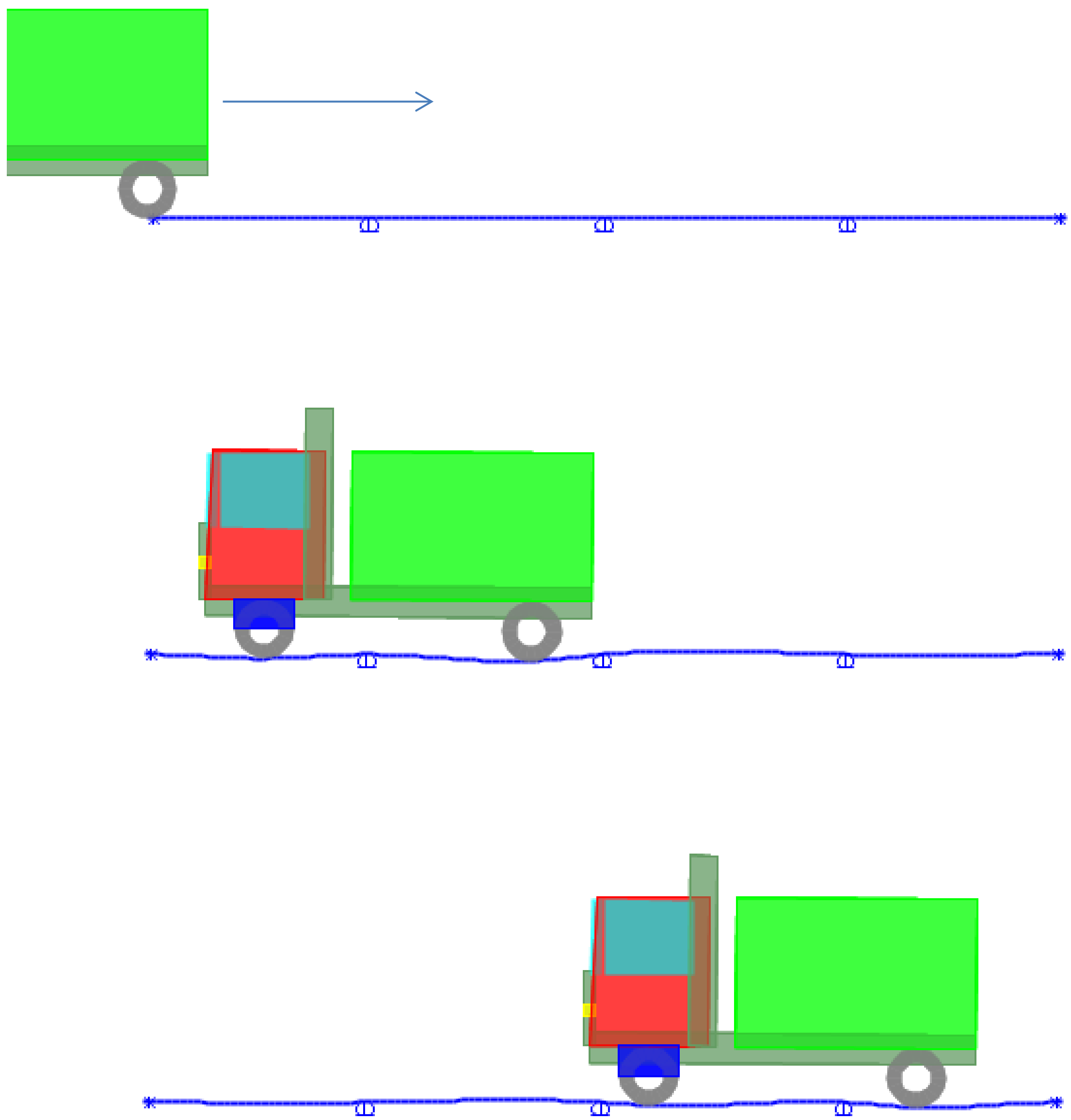


Figure 23: Engine truck backing up

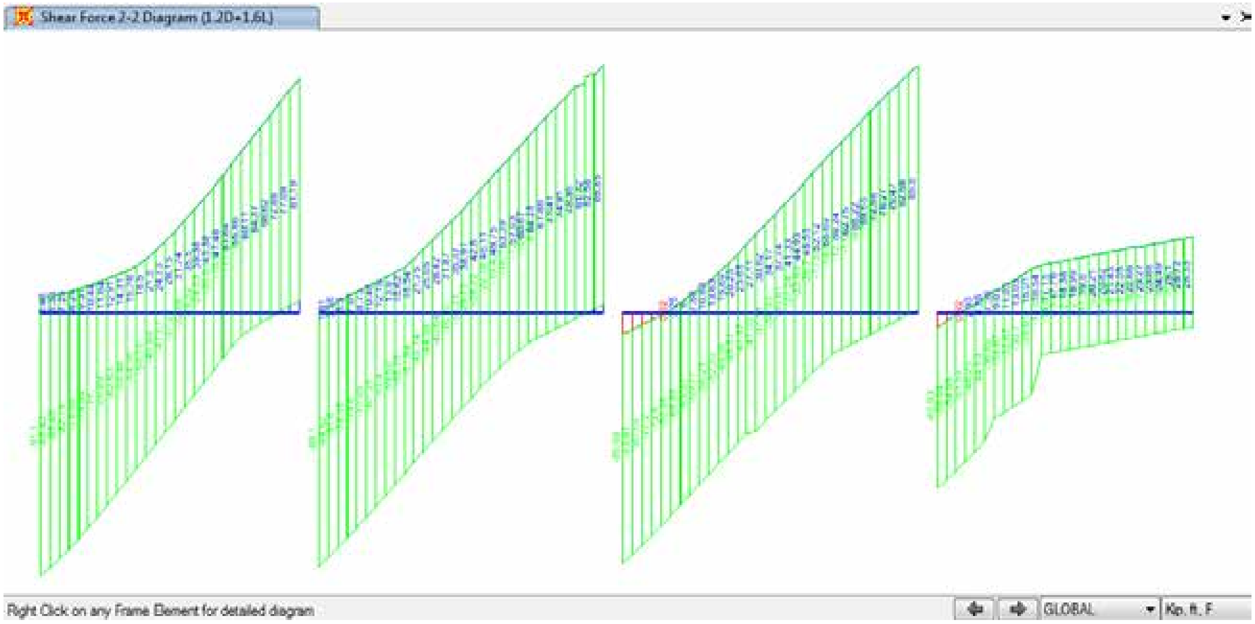


Figure 24: Wide beam shear envelope due to factored self-weight plus factored truck loads

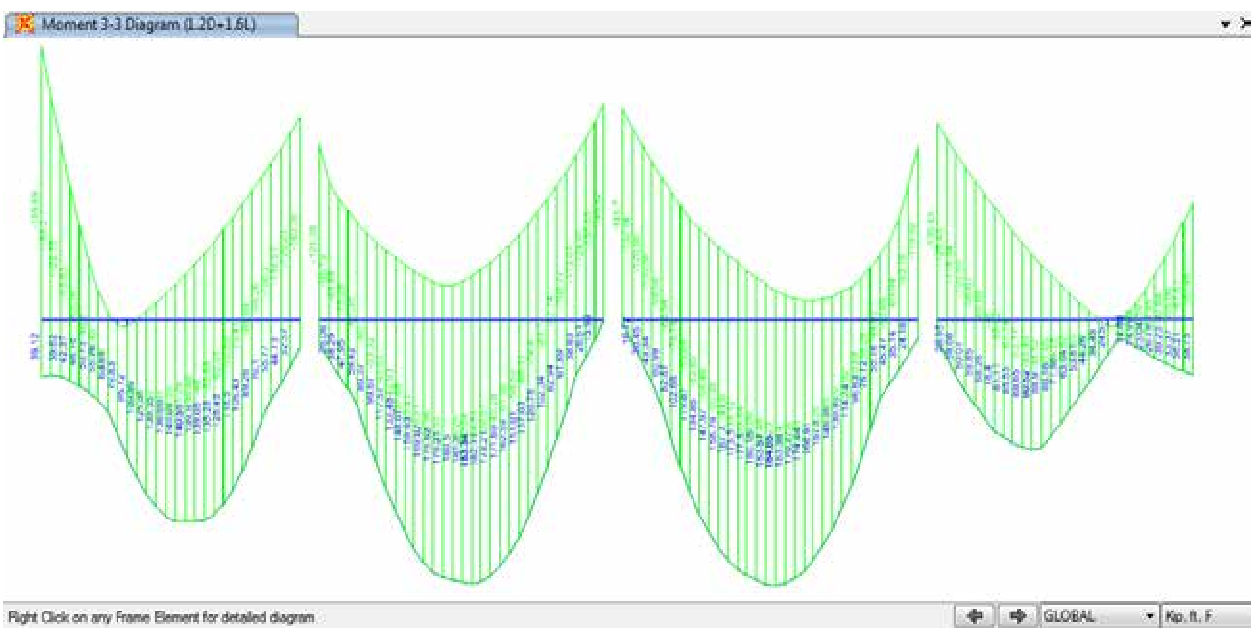


Figure 25: Wide beam moment envelope due to factored self-weight plus factored truck loads

Table 7: Wide beam shear force and bending moment demand capacity ratios (DCR)

	Vu	f Vn	DCR-v	Mu+ve	f Mn+ve	DCR-M+ve	Mu-ve	f Mn-ve	DCR-M-ve
	kip	kip	---	ft-kip	kip	---	ft-kip	kip	---
Wide Beam	91.1	23.6	**3.86**	184.7	89.1	**2.07**	189.7	90.3	**2.10**

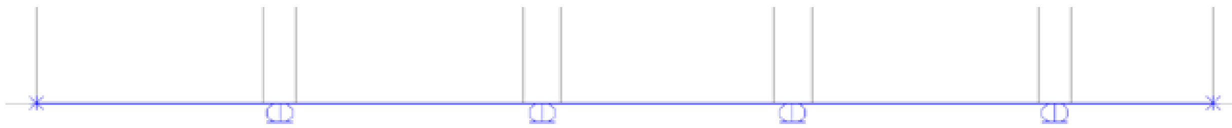


Figure 26: Continuous model of narrow beam

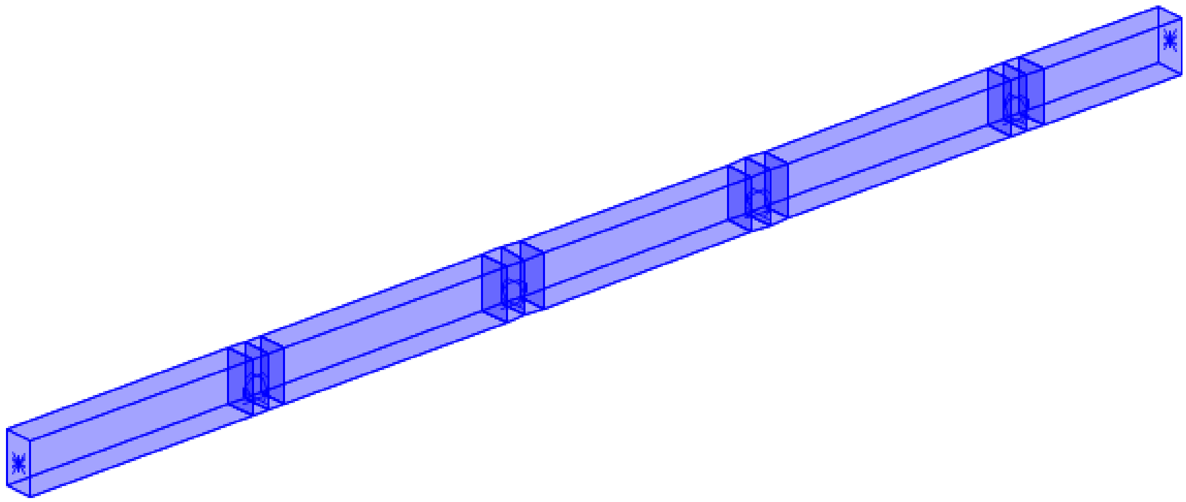


Figure 27: Model of narrow beam showing member cross section

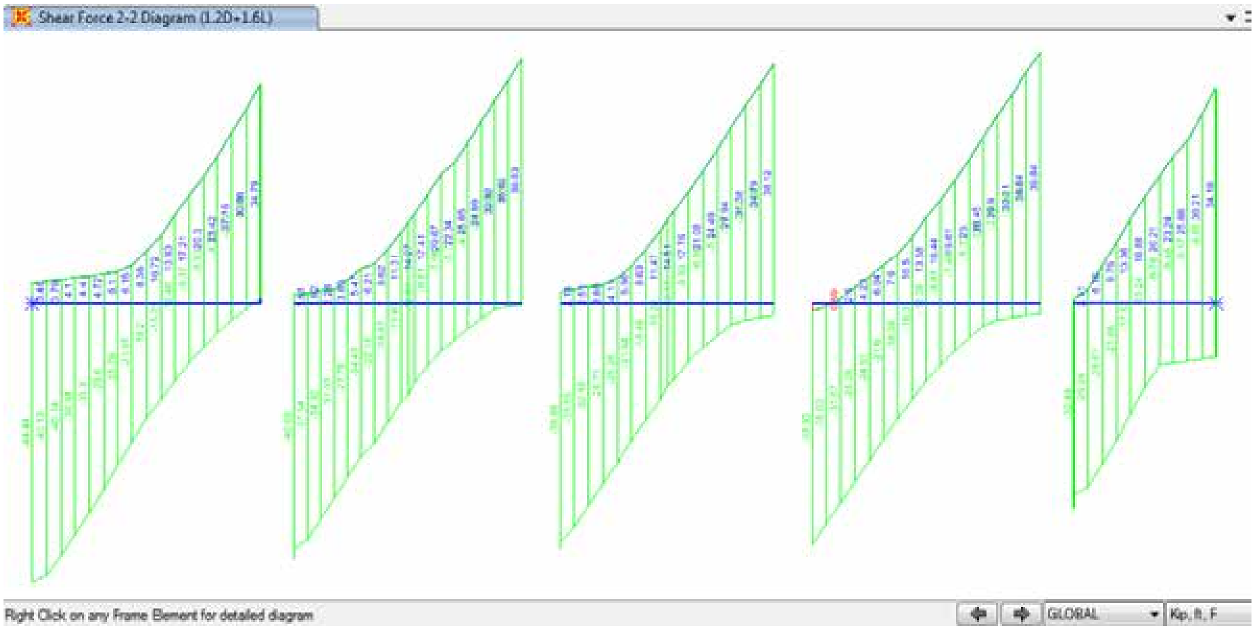


Figure 28: Narrow beam shear envelope due to factored self-weight plus factored truck loads

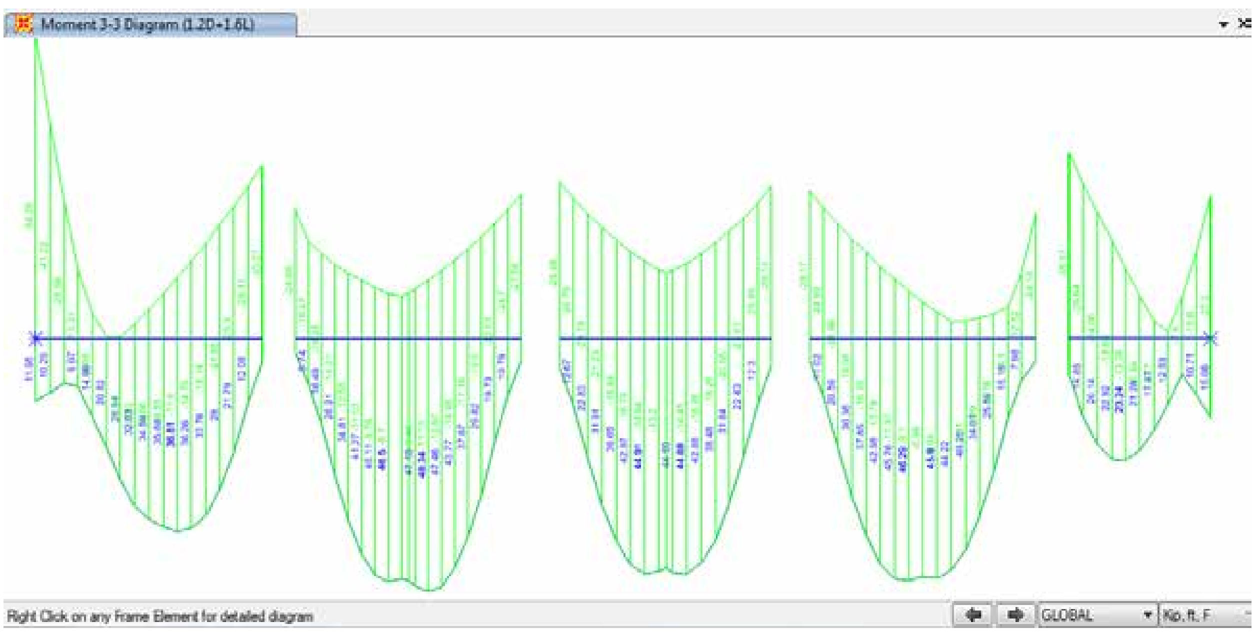


Figure 29: Narrow beam moment envelope due to factored self-weight plus factored truck loads

Table 8: Narrow beam shear force and bending moment demand capacity ratios (DCR)

	Vu	f Vn	DCR-v	Mu+ve	f Mn+ve	DCR-M+ve	Mu-ve	f Mn-ve	DCR-M-ve
	kip	kip	---	ft-kip	kip	---	ft-kip	kip	---
Narrow Beam	44.4	40.9	**1.09**	48.3	67.9	**0.71**	58.3	64.4	**0.91**

Table 9: Summary of shear force and bending moment demand capacity ratios (DCR)

	Vu	f Vn	DCR-v	Mu+ve	f Mn+ve	DCR-M+ve	Mu-ve	f Mn-ve	DCR-M-ve
	kip	kip	---	ft-kip	kip	---	ft-kip	kip	---
151 in - Slab Strip under a Tandem Axle	66.3	49.5	**1.34**	98.0	45.6	**2.15**	112.0	45.3	**2.47**
Wide Beam	91.1	23.6	**3.86**	184.7	89.1	**2.07**	189.7	90.3	**2.10**
Narrow Beam	44.4	40.9	**1.09**	48.3	67.9	**0.71**	58.3	64.4	**0.91**

City of Austin
Founded by Congress, Republic of Texas, 1839
Public Works Department, P.O. Box 1088, Austin, TX 78767-8839
Quality Management Division, 105 Riverside Drive, Suite 100, Austin, TX 78704

Alejandro Wolniewitz
Facilities Process Manager
Austin Fire Department
4201 Ed Bluestein Blvd
Austin, Texas 78721

DATE: September 1, 2017

RE: Forensic Investigation of the Existing Elevated Foundations for Fire Station 3 and Fire Station 22 to Determine If The Existing Elevated Foundations Could Safely Support New Vehicular Loads

Mr. Wolniewitz,

This letter is in regards to a forensic investigation that was performed on the existing elevated foundations at Fire Station 3 and Fire Station 22. The purpose of these forensic investigations were to determine if the existing foundations could safely support the higher loads of the new fire trucks. The forensic investigation was conducted by CTL Group, phase 2 report submitted August 31, 2017, to determine if the existing elevated foundations could safely support the new vehicular loads at Fire Station 3 and Fire Station 22.

The findings of both investigations indicate that the existing elevated foundations for Fire Station 3 and Fire Station 22 are not capable of safely supporting the higher loads of the new fire trucks. The following are the recommendations that were presented in the report from CTL Group with concerns/comments:

- Fire Station 3
    - Remove large portions of the existing foundation and replace with a new foundation that is designed to support the new loads.
        - Concerns/Comments: This is costly due to accessing and demolishing the existing foundation without damaging the remaining structure. There is a high risk associated with this recommendation.
    - Replacing the entire bay area.
        - Concerns/Comments: Initial cost could be higher by performing selective demolition to Fire Station 3 by removing the existing bay area and designing a foundation for the new required loads (and anticipated higher loads in the future) and rebuilding the bays. The new bay design will address the current new vehicular loads of the Fire Station and could be planned and implemented in a manner to fulfill the future needs of the Austin Fire Department.
    - Fill the crawlspace with flowable fill material.
        - Concerns/Comments: Fire Station is located in an area that has expansive soils, that is why the existing foundation system is suspended. Additional geotechnical investigation will be required to determine if adding the additional load of the flowable material will cause the soil to settle and create a void between the bottom of the existing foundation and the flowable fill. The soils can than rebound pushing upward against the bottom of the existing foundation causing it to move upward.

- Fire Station 22
    - The CTL Group's recommendations for Fire Station 22 are similar to the recommendations given for Fire Station 3. Please reference the above recommendations with concerns.

The forensic investigations that were performed by CTL Group of the existing elevated foundations of Fire Station 22 and Fire Station 3 revealed that the existing suspended foundations cannot safely support the vehicular loading from the new fire trucks. It is recommended that selective demolition be performed and replace the entire bays of Fire Station 3 and Fire Station 22. The new bays could be designed to accommodate the new vehicular loading requirements of the Fire Stations and anticipated future needs of the Austin Fire Department.

Please feel free to contact me if you have any questions.

Thanks

Karim Helmi, P.E.
City Structural Engineer - Quality Management Division
Public Works Department
City of Austin
Phone: (512) 974-6539

GEOTECHNICAL ENGINEERING STUDY
FIRE STATION #3 AND #22 BAY REPLACEMENT
201 W. 30TH STREET / 5309 EAST RIVERSIDE DRIVE
AUSTIN, TEXAS

KLEINFELDER PROJECT NO. 20190836.001A

October 24, 2018

1826 Kramer Lane, Suite M, Austin, TX. 78758 p | 512.926.6650 f | 512.833.5058

A Report Prepared for:

Mr. Alejandro Wolniewitz – Facilities Process Manager
Ms. Tica Chitrarachis – Rotation List Manager
City of Austin Fire Department
4201 Ed. Bluestein Boulevard
Austin, Texas 78721

Geotechnical Engineering Study
Fire Station #3 and #22 Bay Replacement
201 W. 30th Street / 5309 East Riverside Drive
Austin, Texas

Prepared by:

Ben Baugh
Benjamin Baugh, EIT
Staff Professional

Orlando Boscan
Orlando Boscan, PE
Project Manager

KLEINFELDER, INC
1826 Kramer Lane, Suite M
Austin, Texas 78758
Phone: 512.926.6650
Fax: 512.833.5058

October 24, 2018
Kleinfelder Project No.: 20190836.001A

October 24, 2018
Project No.: 20190836.001A

Mr. Alejandro Wolniewitz – Facilities Process Manager
Ms. Tica Chitrarachis – Rotation List Manager
City of Austin Fire Department
4201 Ed. Bluestein Boulevard
Austin, Texas 78721

Subject: **Geotechnical Engineering Study**
Fire Station #3 and #22 Bay Replacement
201 W. 30th Street / 5309 East Riverside Drive
Austin, Texas

Dear Mr. Wolniewitz:

Kleinfelder has completed the authorized subsurface exploration and geotechnical engineering study for the above-referenced project. The purpose of the geotechnical study was to explore and evaluate the subsurface conditions at Fire Station #3 and #22 and develop geotechnical design and construction considerations. The attached Kleinfelder report contains a description of the findings of our field explorations and laboratory testing program, our engineering interpretation of the results with respect to the design of building foundation and potential construction issues for the planned project.

As an additional service, we would be pleased to review the portions of the plans and specifications that were developed based on information from our Geotechnical Study. We can also provide construction phase services such as materials engineering, materials, testing, and foundation installation observation.

We appreciate the opportunity to be of service to you on this project. If we can be of additional assistance, please contact us at 512.926.6650.

Sincerely,

KLEINFELDER, INC.
Texas Registered Engineering Firm F-16438

Ben Baugh
Benjamin Baugh, EIT
Staff Professional

STATE OF TEXAS
ORLANDO L. BOSCAN
104906
LICENSED
PROFESSIONAL ENGINEER

Orlando Boscan
Orlando Boscan, PE
Project Manager

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING STUDY
FIRE STATION #3 AND #22 BAY REPLACEMENT
201 W. 30TH STREET / 5309 EAST RIVERSIDE DRIVE
AUSTIN, TEXAS

1 INTRODUCTION

1.1 PROJECT DESCRIPTION

We understand that the proposed project consists of the complete demolition and reconstruction of the fire engine bays for the City of Austin Fire Stations (FS) #3 and #22 in Austin, Texas. Reportedly, the results of a recent engineering forensic study indicated that the existing fire engine bay structures may be inadequate to support the loads from current, and likely future, fire-fighting vehicles. Reportedly, column loads for both existing bay structures are supported on drilled shaft foundations. Floor loads are supported by suspended structural slabs. We understand that the proposed reconstruction may include relatively minor expansion of the current bays footprints. The current planed dimensions for the existing bay structures are approximately 55 to 60 feet in length, and 35 to 40 feet in width. We also understand that the City of Austin is planning to support the new bays on drilled shaft and suspended floor slab foundation system.

Specific structural loading information was not available at the time of this report. Once available, loading information should be provided so that we can confirm the applicability of our recommendations.

1.2 PURPOSE AND SCOPE

Our study was generally performed based upon the Scope of Services presented in our proposal No. AUS18P77507R2 dated April 30, 2018. However, due to the encountered bedrock conditions in Fire Station 3, the borings were drilled deeper than originally planned to obtain the necessary subsurface information for foundation design.

The primary purpose of this geotechnical study is to provide recommendations for the design and construction of foundations for the proposed Fire Station #3 and #22 bays. To accomplish this purpose, our study included the following scope:

- Borings at FS #3 Site: Drilled and sampled 2 borings to a depth of approximately 45 feet below grade and 1 boring to a depth of 50 feet below grade. Hand-augered one boring south of the existing bay building to a depth of 5 feet below grade.

- Borings at FS #22 Site: Drilled and sampled 3 borings to a depth of approximately 60 feet below grade.
- Performed laboratory tests on select samples for classification and to estimate engineering properties of the subsurface materials.
- Performed engineering analyses using the field and laboratory data to develop geotechnical engineering recommendations for use during the design of the foundations of the proposed structures.

Design of the project including site civil and building structural design has not been performed, and the assumed locations and/or elevations of structures may change. Kleinfelder should be provided with the design information when it is available to evaluate whether recommendations presented herein are still applicable or require modifications, it is possible that modification of our recommendations may be required based upon the final design.

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

Subsurface conditions were explored by drilling and sampling 6 borings with a truck-mounted Mobile B-57 drill rig. An additional boring at FS #3 was advanced using hand-auger drilling. A schedule of the borings is presented in Table 2.1, and the approximate location of these borings is presented on Figures 1 and 2, Exploration Location Plan, and Vicinity Maps in Appendix A.

Table 2.1 - Schedule of Borings

Location	Boring No.	Depth	Date Drilled	Structure
FS #3	SB-1	50 feet	August 27, 2018	Engine Bay
FS #3	SB-2 and SB-3	45 feet	August 28 - 29, 2018	Engine Bay
FS #3	SB-4	5 feet	September 11, 2018	Engine Bay
FS #22	B-1 to B-3	60 feet	August 29 - 30, 2018	Engine Bay

Boring locations were established in the field by a representative of Kleinfelder. A hand-held Global Positioning System (GPS) with a horizontal accuracy of about 15 feet was used to record the boring locations. If required, a professional surveyor should be hired to obtain accurate boring location information.

Hand auguring, Shelby-tube sampling, split spoon sampling, rock coring, and solid-stem auger drilling techniques were used to complete the borings.

Relatively undisturbed samples of cohesive soils were collected by using the drilling rig to push a seamless, steel tube sampler into the soil (based upon ASTM D1587). The depths at which these samples were collected are indicated on the boring logs in Appendix A, Field Exploration Program. After a tube was recovered, the sample was extruded in the field, examined, and logged. The sample was then placed in a plastic bag to reduce moisture loss and protect the sample. During logging, an estimate of the sample consistency was obtained using a pocket penetrometer. This test provides relative strength data that is used as an approximate indicator of shear strength. The result of the penetrometer reading is recorded at a corresponding depth on the boring log.

At select locations, samples were also collected by driving a split-spoon sampler in conjunction with the Standard Penetration Test (SPT). This technique involves driving the spoon sampler a distance into the soil using a free-falling hammer (based upon ASTM D1586). During the test, the logger records the number of blows required to drive the spoon sampler over three successive 6-inch increments. The first 6 inches is the “seating drive,” while the number of blows required to drive the sampler the last two 6-inch increments is the “penetration” in blows per foot. Where resistance was high, the number of inches of penetration for 50 blows of the hammer is recorded. When less than 6 inches of penetration is obtained, the test is terminated regardless of the drive increment. The results of the penetration test are reported on the boring logs at the corresponding depth. Materials recovered from the split spoon sampler are then examined and placed in a plastic bag to reduce moisture loss and protect the sample.

Samples of rock and/or rock-like materials were collected with an NX size double-tube core barrel fitted with a carbide bit. Sample recovery and Rock Quality Designation (RQD) for each core run of rock and rock-like material were calculated and recorded on the field logs. The RQD is a modified core recovery percentage in which all the pieces of sound core over 4 inches long are summed and divided by the length of the core run. The RQD measurements and calculations were conducted in accordance with the procedures described in the Reference. Core breaks caused by the drilling process were fitted together and counted as one piece. Where it was difficult to discern natural breaks from drilling breaks, the break was considered a natural break, thus providing conservatism in the RQD calculation. The core run intervals for the project were typically 5 feet in length. RQD is categorized according to Table 2.2.

Table 2.2 – RQD Categorization

RQD (%)	Description of Rock Quality
0 – 25	Very Poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

At the completion of drilling, each boring was backfilled with 3/4-inch bentonite hole plug and auger cuttings up to and slightly above the existing ground surface except in borings that were drilled through concrete. The borings that were drilled through existing pavements were patched at the surface with concrete.

Boring logs are presented in Appendix A with soil and rock description keys. The logs indicate the material types, depths, and other details of materials encountered for each boring. Soil/rock descriptions presented upon the boring log resulted from a combination of field and laboratory test data. Stratigraphy lines in the boring logs correspond to the approximate boundary between strata. However, the in-situ subsurface transition can be, and is often gradual.

2.2 LABORATORY TESTING

Samples of subsurface materials from the borings were visually examined and the field classifications were verified by the engineer in the laboratory. Natural moisture content tests, Atterberg limits (liquid and plastic limits) determinations, unconfined compression tests, and sieve analysis tests were performed on select soil samples to establish index and strength properties and grain size characteristics, and to classify the soils according to the Unified Soil Classification System (USCS). The results of these tests are shown on the boring logs.

2.2.1 Chemical Tests

One combined soil sample for FS #3 and one combined soil sample for FS #22 were tested to determine the pH, soluble sulfate, chloride concentrations, and soil resistivity. A summary of these test results is listed in Section 4.6 of this report and the detailed test results are provide in Appendix B.

www.kleinfelder.com

KLEINFELDER 1826 Kramer Lane, Suite M, Austin, TX. 78758 p | 512.926.6650 f | 512.833.5058

3 GENERAL SITE CONDITIONS

3.1 GEOLOGY

The Austin Sheet of the Geologic Atlas of Texas locates the FS #22 project site within the Ozan Formation (Ko) of the Cretaceous-Late age. These materials primarily consist of highly-plastic clay, with various amounts of calcareous materials, silt, and sand. The site of FS #3 is situated within an outcropping of the Austin Chalk Formation. The Austin Chalk formation typically consists of clays overlying chalky limestone. The thickness of the clay above the limestone varies but is generally encountered at a shallow depth. The upper portions of the limestone are generally weathered, fractured, and very light brown to light yellow brown in color. Some zones of severely weathered limestone that are clay-like can be present above the weathered material. The underlying primary limestone is generally harder than the weathered limestone and is light to medium gray in color.

3.2 SUBSURFACE STRATIGRAPHY

The borings at FS #3 indicate the presence of moderate to high plasticity clay of depths varying from 26 to 28 feet. The clay overlays light gray limestone to the boring termination depth of approximately 50 feet below grade.

Based on the results of the borings at FS #22, the subsurface conditions at the site indicate the presence of alternating clay, sand, and gravel layers overlaying weathered gray shale. The gray shale was encountered at an approximate depth of 35 to 38 feet below grade.

The various types and depths of subsurface strata observed in the borings drilled for this study are shown on the Boring Logs presented in Appendix A of this Report. The strata thickness and general descriptions on the boring logs are based solely on the materials observed in the borings drilled for this study.

The descriptions are general and the range of depths approximate, because boundaries between different strata are seldom clear and abrupt in the field. In addition, the lines separating major strata types on the boring Logs do not necessarily represent distinct lines of demarcation for the various strata.

3.3 GROUNDWATER OBSERVATIONS

The borings were advanced using techniques that allow for direct and indirect observations of seepage and groundwater during drilling operations. Water was encountered in Boring B-3 at a depth of 35 feet below grade. 15 minutes after encountering water in boring B-3, the water depth was measured to be 34 feet below grade. Free water was not encountered in the remaining borings. Once rock coring is performed on a boring, water is introduced to the boring and water readings were not taken below the start of rock coring. These observations do not preclude the possibility of seepage or groundwater, and are only indicative of conditions at the time and place indicated.

The occurrence and variation of groundwater can vary due to many factors. These factors include seasonal changes, site topography, surface runoff, the layering and permeability of subsurface strata; water levels in waterways, utilities, and other factors not evident at the time of this study. Groundwater is likely perched above the limestone bedrock and within joints in the bedrock, especially during rainy seasons. The possibility of groundwater and its fluctuation should be considered when developing this project.

4 GEOTECHNICAL RECOMMENDATIONS

4.1 GENERAL

Based on the results of our evaluation, in our professional opinion, the project site can be developed for the proposed construction using conventional grading and excavation and foundation construction techniques, provided that the recommendations presented herein are incorporated into the design and construction of the project.

Recommendations submitted herein are based, in part, upon data obtained from our subsurface exploration. The nature and extent of subsurface variations that may exist at the proposed project site will not become evident until construction. Kleinfelder should be on site during foundation subgrade preparation to observe conditions. If significant variations are observed, the recommendations presented in this report may need to be revised. In addition, if changes in the nature, design, location or depth of the proposed structure are planned, Kleinfelder should be notified to review and modify the conclusions and recommendations contained in this report as appropriate. Changes in subgrade preparation and foundation design recommendations will not be considered valid unless provided in writing. General recommendations regarding geotechnical aspects of the project design and construction are presented below.

4.2 EXPANSIVE SOIL CHARACTERISTICS

An estimate of the potential vertical movement (PVM) was made using the Potential Vertical Rise (PVR) Method 124-E published by TxDOT, engineering judgment, and our experience. Based on this information, the estimated soil movement, or Potential Vertical Movement (PVM) for each site was estimated for a full seasonal moisture cycle based on the Potential Vertical Rise (PVR) Method 124-E published by TxDOT. The estimated PVM for each site is summarized in Table 4.1 below.

TABLE 4.1: Estimated PVM for FS #3 and FS #22

Location	Estimated PVM (inches)
FS #3	1 ½ to 3
FS #22	2 to 3

These soil movements can be caused by either shrink or swell movements, depending on seasonal moisture fluctuations. Recognize that this value range is not exact and is only an indication of the potential movements due to expansive soil for seasonal moisture fluctuations. Actual movements may be significantly larger than estimated due to inadequate site grading, poor drainage, ponding surface water, and/or leaks in utility lines. Significant changes to existing site grades can also alter actual movements by changing the thickness of the expansive soil and/or altering the active moisture zone depth. Recognize that this value is not an exact value but is only an indication of the potential movements due to expansive soil for seasonal moisture fluctuations.

4.3 DRILLED STRAIGHT-SIDED PIERS

4.3.1 Axial Capacity

In our opinion, the proposed FS #3 and FS #22 bays can be supported on straight-sided drilled shafts. Based on the encountered subsurface conditions at FS #3, the drilled shafts should terminate in the light gray limestone strata. If the drilled shafts terminate in the light gray limestone strata, then bearing capacity and side friction between the concrete and the limestone can be used to support the loads. The side friction and bearing capacity by depth is summarized in Table 4.2 below.

TABLE 4.2: Bearing Capacity and Side Friction by Depth (FS #3)

Stratum	Depth (ft)	Maximum Allowable Bearing Capacity (psf)	Maximum Allowable Side Friction (psf)
Light Gray Limestone	28-50	40,000	2,000

Side resistance values can be used for both compressive and tensile load resistance. The shafts should have a minimum penetration of 10 feet into the light gray limestone strata and have a minimum diameter of 24 inches to support the proposed structure. Final penetration should be determined by the structural engineer based on axial and lateral loadings.

We consider that the proposed FS #22 bay can also be supported on straight-sided drilled shafts. Based on the encountered subsurface conditions at FS #22, the drilled shafts should terminate in the dark gray weathered shale strata. If the drilled shafts terminate in the dark gray weathered shale strata, then bearing capacity and side friction between the concrete and the weathered shale can be used to support the loads. The side friction and bearing capacity is summarized in Table 4.3 below.

TABLE 4.3: Side Friction by Depth (FS #22)

Stratum	Depth (ft)	Maximum Allowable Bearing Capacity (psf)	Maximum Allowable Side Friction (psf)
Dark Gray Weathered Shale	36-38	5,000	1,200

Side resistance values can be used for both compressive and tensile load resistance. The shafts should have a minimum penetration of 15 feet into the dark gray weathered shale strata and have a minimum diameter of 24 inches to support the proposed structure. Final depths should be determined by the structural engineer based on axial and lateral loadings.

The expansive subgrade may subject the shafts to uplift pressures and create tensile forces within the shafts. Accordingly, each shaft should be steel reinforced to withstand these forces. The actual uplift forces will vary with depth and moisture condition, but steel reinforcement design for the soil uplift pressures may be modeled using 1,000 psf acting over the entire shaft perimeter that is within the upper 12 feet.

Settlements of properly designed and constructed shafts should be less than ¾ inch. It should be noted that the performance of the foundations will be more sensitive to the construction quality than the soil-structure interaction. Monitoring of the foundation installation by the geotechnical engineer or representative of the engineer is recommended.

Groundwater was not encountered during our field exploration at FS #3. At FS #22, free water was encountered in Borings B-1 and B-3 at a depth of approximately 34 feet below grade. Groundwater may be encountered during installation of the shafts, particularly if construction proceeds during a wet period of the year. In some cases, rapid placement of steel and concrete may permit shaft installation to proceed; however, the seepage rates could be sufficient to require the use of temporary casing for proper installation of the shafts. The casing should be seated in the bearing stratum with water and most loose material removed prior to beginning the design penetration. Care must be taken that a sufficient head of plastic concrete is maintained within the casing during extraction.

The concrete should have slump within 4 and 6 inches for uncased shafts and 5 and 7 inches for cased shafts. The concrete must be placed in a manner to avoid striking the reinforcing steel during placement. Compete installation of individual shafts should be accomplished within an 8-hour period in dry excavations and preferably as rapidly as possible in order to prevent deterioration of bearing surfaces.

Some intervals of the limestones are hard. These limestones can be difficult to penetrate, especially when drilling large diameter shafts. The drilled shaft excavations should be performed with hard rock drilling equipment suitable to perform this work by a contractor experienced in this area.

4.3.2 Group Effects

Some reduction for group effects should be considered where shafts will be installed in a group condition or where any shafts will be installed close together. To develop full load carrying capacity in side resistance, adjacent straight-sided drilled shafts should have a minimum center to center spacing of 2.5 times the diameter of the larger shaft. This spacing requirement includes proximity to existing shafts. Closer spacing will require some reductions in side resistance and/or changes in installation sequences. The design side shear for axial or uplift loads may be considered to vary linear from the full value at a spacing of 2.5 times the diameter of the larger shaft to 50 percent of the design value at a spacing of 1 times the diameter of the larger shaft.

4.3.3 LPILE Parameters (Version 7.0)

The LPILE parameters provided below are for the subsurface material described in the boring logs for the project. The depth of each layer can be generalized from the boring log. The top 5 feet of the subsurface profile in contact with the drilled shaft is neglected. p-y. Tables 4.4 and 4.5 provide the LPILE parameters for FS #3 and FS #22.

TABLE 4.4: Lpile Parameters for FS #3

Lpile p-y Curve Model	Depth (ft)	Cohesion (psf)	Friction Angle (deg.)	Effective Unit Wt. (pcf)(1)	Modulus k (pci)
Soft Clay	0-5	0	--	58	20
Stiff Clay w/o Free Water	5-15	4,000	--	58	270
Stiff Clay w/o Free Water	15-28	2,400	--	58	135
Stiff Clay w/o Free Water	28-50	7,000	--	83	540

TABLE 4.5: Lpile Parameters for FS #22

Lpile p-y Curve Model	Depth (ft)	Cohesion (psf)	Friction Angle (deg.)	Effective Unit Wt. (pcf)(1)	Modulus k (pci)
Soft Clay	0-5	0	--	58	20
Stiff Clay w/o Free Water	5-17	2,500	--	58	135
API Sand	17-34	--	30	53	25
Stiff Clay w/o Free Water	34-60	7,000	--	78	540

No reduction in individual lateral shaft capacity is required for drilled shafts spaced at a minimum center-to-center spacing of five diameters. Appropriate lateral reduction factors should be used, if the spacing between shafts is less than five diameters.

4.4 INTERIOR FLOOR SUPPORT

4.4.1 General

Near-surface soil conditions at this site are interpreted to be relatively uniform and consist of high plasticity clay soil. The high plasticity clay soils remain stable with constant moisture contents;

however, a change in the moisture content will cause the soil to swell or shrink thereby potentially causing movement and damage to the overlying structure

It is our understanding that the proposed bay reconstruction project includes structurally suspended floor slabs and crawl space. Based on this, potential shrink/swell movements associated with the near-surface highly-plastic clays should not affect the performance of the selected bay floor system. The crawl space will provide the necessary separation between the slab and soil movements associated with shrink/swell behavior. Similarly, structurally suspended grade beams will be isolated from soil movements by the crawl space.

4.5 SOLUBLE SULFATE

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) publication *Guide to Durable Concrete* (ACI 201.2R-08) provides guidelines for this assessment. The results of the sulfate testing indicate the potential for deterioration of concrete at FS #3 has a Class 0 exposure. For sites with Class 0 sulfate exposure, ACI does not have special requirements for sulfate resistance. The results of the sulfate testing indicate the potential for deterioration of concrete at FS #22 has a Class 1 exposure. For sites with Class 1 sulfate exposure, ACI recommends Type II cement or equivalent. The results from the sulfate content analysis can be seen below in Table 4.4.

TABLE 4.4: Sulfate Test Results

Location	Boring	Depth (feet)	Sulfate (ppm)
FS #3	SB-2	0.5 to 4	331
FS #22	B-3	2-4	20.9

4.6 SEISMIC HAZARDS SITE CLASS

This area of Texas is considered seismically inactive. Seismic designs in Texas are typically based upon the criteria established in the 2012 International Building Code (IBC). The seismic design is based upon the Site Class, as defined in Sections 1613.5.2 and 1613.5.5. Based upon the results of the site-specific borings and our experience with the local geologic conditions, the average subsurface conditions at both sites correspond to Site Class “C”. For this site class, the Mapped Spectral Response Acceleration at short periods (S_s) is about 0.064g, and the Mapped Spectral Response Acceleration at a 1 second period (S_1) is about 0.033g. For these accelerations, the Site Coefficients F_a and F_v are 1.2 and 1.7, respectively.

5 PAVEMENT DESIGN AND CONSTRUCTION CRITERIA

5.1 GENERAL

Based on information provided by City of Austin, we understand that the replacement of the existing driveways may be part of the proposed fire engine bays reconstruction at Fire Stations 3 and 22. The existing driveway pavements consists of Portland cement concrete, which is the material is commonly used for heavy-duty sections for projects similar to the proposed bay rehabilitation.

5.2 PAVEMENT THICKNESS FOR BAY DRIVEWAYS

The pavement section thickness recommendations presented in this section are based on the encountered subsurface conditions, our project understanding, and our previous experience with similar projects. It should be noted that a detailed pavement analysis was beyond our scope for this project. As such, the following table presents our recommended typical heavy-duty section for the proposed bays driveways. This section is not based on specific traffic loading information or pavement life expectancy.

TABLE 5.3: PAVEMENT THICKNESS RECOMMENDATIONS

Traffic	Pavement Section
Heavy Duty Pavement for Fire Engine Bay Driveways	8" Portland Cement Concrete Pavement over 8” Crushed Limestone Base

5.3 PAVEMENTS ON EXPANSIVE SOILS

At FS #3, we anticipate potential vertical movement of approximately 1 ½ to 3 inches. At FS #22, we anticipate potential vertical movement of approximately 2 to 3 inches. The sub base should extend a minimum of 12 inches outside the curb line. This will improve the support for the edge of the pavement and also lessen the "edge effect" associated with shrinkage during dry periods. The use of sand as a leveling course below pavement in expansive clay areas should be prevented as these porous soils can allow water inflow between the pavement and subgrade, facilitating heave and strength loss within the subgrade soil.

To reduce the potential vertical movement, we recommend excavating 1 foot of the in-situ fat clays and replacing with select fill. Prior to fill placement, the exposed subgrade should be scarified to a depth of 12 inches, moisture conditioned to +2 to 5% of optimum water content and compacted to 95% compaction.

It is important to reduce moisture changes in the pavement subgrade and sub base. The pavement and adjacent areas should be well drained. The pavement and surrounding grades must have positive drainage that quickly removes surface water and inhibits the absorption of surface water into the subgrade soils. Regular maintenance should be performed on cracks in the pavement surface to reduce water passing through to the base or sub base material. Even with these precautions, some distress may still occur, which will require periodic maintenance.

Consideration should be given to the location of existing and proposed trees, as they have been documented to desiccate surrounding subgrade soil and result in soil shrinkage and settlement. The zone of the desiccation varies by tree, but it is generally recommended that trees are set back so that the drip-line of the mature tree will not extend over or near the pavement structure. If existing mature trees are allowed to remain adjacent to the roadway, we recommend the installation of root barriers to keep these trees from causing differential movement of the new roadway.

6 CONSTRUCTION CONSIDERATIONS

6.1 DEMOLITION

Initial site preparation for the proposed project should commence with demolition of the existing pavements, fences, sidewalks, buildings, and other structures within the proposed construction areas. Demolition should also include removal of all utilities lines within the project site that will be abandoned as part of the construction. All broken asphaltic concrete and Portland cement concrete and other debris from demolition should be removed from the site. Areas disturbed during demolition should be approved by the geotechnical engineer prior to placement of structural fill. All disturbed soils should be undercut to expose competent, undisturbed, medium dense to dense or firm to stiff native soils prior to placement of structural fill.

We understand that the project consists of demolition and reconstruction of the existing fire engine bays. During demolition of the existing structures, any foundation element within 3 feet of slab level should be excavated and removed. Existing piers should have a minimum clearance of 3 feet from the slab level if it does not impede new construction. If the existing foundation impedes new construction then the foundation system should be removed, or new construction should be adjusted accordingly. Voids created due to the removal of existing foundation elements should be backfilled using on-site soil or structural fill material and compaction criteria provided in this report should be followed. Flowable backfill should be used to fill voids due to the removal of deep foundation elements.

6.2 EXISTING UTILITIES

Relocation/demolition of any existing utility lines within the zone of influence of proposed construction areas should also be completed as part of the site preparation. The lines should be relocated to areas outside of the proposed construction. Excavations created by removal/demolition of the existing lines should be cut wide enough to allow for use of heavy construction equipment to compact the backfill. In addition, the base of the excavations should be approved by the geotechnical engineer or approved representative prior to placement of backfill.

6.3 SITE PREPARATION

Before construction, care should be taken to see that any deleterious material present is removed from the site. Care should also be exercised during the grading operations at the site. The traffic of heavy equipment, including heavy compaction equipment, may create a general deterioration of the surficial clay soils. Therefore, it should be anticipated that some construction difficulties could be encountered during periods when these soils are saturated and that it may be necessary to improve, remove or avoid the saturated soils.

Proper drainage should be established so that ponding of surface runoff does not occur and cause construction delays. Where water seepage is encountered during construction, sloping excavation bottoms to a sump or a low point and use of conventional de-watering equipment may be necessary. Control of site surface drainage should be maintained at all times during construction so that drainage is directed away from open excavated areas.

6.3 EXCAVATION

6.3.1 General

Based on the subsurface conditions encountered in the borings, it appears that the overburden materials can be excavated using conventional soil excavation equipment. All excavations must comply with applicable local, state and federal safety regulations. The responsibility for excavation safety and stability of temporary construction slopes lies solely with the contractor. We are providing this information below solely as a service to our client. Under no circumstances should this information provided be interpreted to mean that Kleinfelder is assuming responsibility for construction site safety or the Contractor's activities, such responsibility is not being implied and should not be inferred.

6.4 MATERIAL REQUIREMENTS

Table 6.1 provides material, moisture, and density requirements for a variety of materials and applications. Compaction of each lift should be continuous over its entire area. Fill should be placed in loose horizontal lifts not exceeding 8 inches, with the intent of providing a compacted lift thickness of 6 inches.

When crushed limestone is used, the maximum allowable size is 1.5 inches and the maximum loose lift thickness should be reduced to 6 inches (or less if there is difficulty achieving compaction). Fill placed along slopes should be placed in horizontal lifts that are benched into the slope. The slopes should be overbuilt and cut back to final grades to ensure compaction along the face of the slopes.

TABLE 6.1: MATERIAL AND COMPACTION REQUIREMENTS

Material Use	Material Requirements	Proctor Test Method	(1)Density Requirement	(1)Moisture Requirement
Moisture Conditioned On-Site CH Soils	Organics < 2 %	ASTM D 698	95% minimum	+2 to +5 %
"Non-expansive" Select Fill	PI: 7 to 15, LL≤35 Passing #200 Sieve: ≤70% Organics < 2 %	ASTM D 698	98 % minimum	-1 to +3 %
Flexible Base: Pavement	TxDOT Item 247, Type A, Grade 1 or 2	ASTM D 698	98 % minimum	-3 to +3 %

The placement and compaction of fill material must be observed, monitored, and tested by Kleinfelder on a full-time basis. Prior to placing any fill material above existing materials, the exposed subgrade should be proofrolled. The exposed subgrade materials must be firm and able to support the construction equipment without displacement. Soft or yielding subgrade must be corrected and made stable before construction proceeds. Proof-rolling should be used to detect soft spots or pumping subgrade areas. Proof-rolling should be performed using a heavy pneumatic tired roller, loaded dump truck, or similar piece of equipment weighing at least 25 tons. Proof-rolling is intended to achieve additional compaction and to locate unstable areas and must be observed by Kleinfelder. Soft spots or areas of pumping subgrade must be undercut and reworked. Where fill placement is planned, the proof-rolling must occur once the exiting soils have been excavated and before the fill placement begins. Proof-rolling is intended not only for the foundation area, but also within all areas of pavements, sidewalks, walls, and other locations that will support surface loads.

Each lift of select fill material should be tested to confirm it has the specified moisture and compaction. One moisture/density test should be performed for every 5,000 square-feet of compacted area, or for every 150-linear foot of utility backfill. For smaller areas, a minimum of three tests should be provided for every lift. Subsequent lifts should not be placed until the exposed lift has the specified moisture and density. Lifts failing to meet the moisture and density requirements should be reworked to meet the required specifications.

The specified moisture content must be maintained until compaction of the overlying lift, or construction of overlying flatwork. Failure to maintain the moisture content could result in excessive soil movement, and can also have a detrimental effect on overlying plastic concrete. The contractor must provide some means of controlling the moisture content (such as water hoses, water trucks, etc.). Maintaining subgrade moisture is always critical, but will require the most effort during warm, windy, and/or sunny conditions. Density and moisture testing is recommended to provide some indication that adequate earthwork is being provided. However, the quality of the fill is the sole responsibility of the contractor. Satisfactory verification testing is not a guarantee of the quality of the contractor's earthwork operations.

7 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our preliminary conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The scope of services for this subsurface exploration and preliminary geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

KLEINFELDER
Bright People. Right Solutions.

Appendix A

N

LEGEND

SOIL BORING

NOTE:
BASE MAPPING AND VICINITY MAP CREATED FROM LAYERS COMPILED BY ESRI PRODUCTS AND 2018 MICROSOFT CORPORATION. COORDINATE SYSTEM: GCS WGS 1984

VICINITY MAP

NOT TO SCALE

THIS DRAWING IS NOT TO SCALE

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.	20190836
DRAWN BY:	MAP
CHECKED BY:	BB
DATE:	09-25-2018
REVISED:	-

EXPLORATION LOCATION PLAN AND VICINITY MAP

Fire Station #3 Bay Replacement
201 West 30th Street
Austin, Texas

FIGURE

2

PLOTTED: 09/25/2018 1:48:32 PM BY: MPalmer

GIS FILE PATH: \\azrgisstorp01\Working_clients_Automated_Exploration Plans

SAMPLE/SAMPLER TYPE GRAPHICS

- BULK / GRAB / BAG SAMPLE
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- PUSH TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING
- AIR ROTARY
- MUD ROTARY
- TEXAS CONE PENETRATION
- CORE SAMPLER

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major division	Subdivision	Fines	Gradation	Symbol	Description
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/ or 1>Cc >3	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu ≥4 and 1≤ Cc≤3	GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/ or 1>Cc >3	GP-GM	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/ or 1>Cc >3	SP	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu ≥6 and 1≤ Cc≤3	SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/ or 1>Cc >3	SP-SM	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

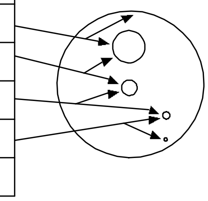
Major division	Subdivision	Symbol	Description
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	ML	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20190836	GRAPHICS KEY	FIGURE
DRAWN BY: MAP		
CHECKED BY: BB	Fire Station #3 and #22 Bay Replacement 201 W. 30th Street / 5309 East Riverside Drive Austin, Texas	A-1
DATE: 9/25/2018		
REVISED: -		

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

Term of Use	AMOUNT: Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20190836 DRAWN BY: MAP CHECKED BY: BB DATE: 9/25/2018 REVISED: -	SOIL DESCRIPTION KEY Fire Station #3 and #22 Bay Replacement 201 W. 30th Street / 5309 East Riverside Drive Austin, Texas

FIGURE
A-2

INFILLING TYPE

NAME	ABBR	NAME	ABBR
Albite	Al	Muscovite	Mus
Apatite	Ap	None	No
Biotite	Bi	Pyrite	Py
Clay	Cl	Quartz	Qz
Calcite	Ca	Sand	Sd
Chlorite	Ch	Sericite	Ser
Epidote	Ep	Silt	Si
Iron Oxide	Fe	Talc	Ta
Manganese	Mn	Unknown	Uk

DENSITY/SPACING OF DISCONTINUITIES

DESCRIPTION	SPACING CRITERIA
Unfractured	>6 ft. (>1.83 meters)
Slightly Fractured	2 - 6 ft. (0.061 - 1.83 meters)
Moderately Fractured	8 in - 2 ft. (203.20 - 609.60 mm)
Highly Fractured	2 - 8 in (50.80 - 203.30 mm)
Intensely Fractured	<2 in (<50.80 mm)

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	RECOGNITION
Pit (Pitted)	Pinhole to 0.03 ft. (3/8 in.) (>1 to 10 mm.) openings
Vug (Vuggy)	Small openings (usually lined with crystals) ranging in diameter from 0.03 ft. (3/8 in.) to 0.33 ft. (4 in.) (10 to 100 mm.)
Cavity	An opening larger than 0.33 ft. (4 in.) (100 mm.), size descriptions are required, and adjectives such as small, large, etc., may be used
Honeycombed	If numerous enough that only thin walls separate individual pits or vugs, this term further describes the preceding nomenclature to indicate cell-like form.
Vesicle (Vesicular)	Small openings in volcanic rocks of variable shape and size formed by entrapped gas bubbles during solidification.

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	CRITERIA
Unweathered	No evidence of chemical / mechanical alternation; rings with hammer blow.
Slightly Weathered	Slight discoloration on surface; slight alteration along discontinuities; <10% rock volume altered.
Moderately Weathered	Discoloring evident; surface pitted and alteration penetration well below surface; Weathering "halos" evident; 10-50% rock altered.
Highly Weathered	Entire mass discolored; Alteration pervading most rock, some slight weathering pockets; some minerals may be leached out.
Decomposed	Rock reduced to soil with relic rock texture/structure; Generally molded and crumbled by hand.

RELATIVE HARDNESS / STRENGTH DESCRIPTIONS

GRADE		UCS (Mpa)	FIELD TEST
R0	Extremely Weak	0.25 - 1.0	Indented by thumbnail
R1	Very Weak	1.0 - 5.0	Crumbles under firm blows of geological hammer, can be peeled by a pocket knife.
R2	Weak	5.0 - 25	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.
R3	Medium Strong	25 - 50	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of a geological hammer.
R4	Strong	50 - 100	Specimen requires more than one blow of geological hammer to fracture it.
R5	Very Strong	100 - 250	Specimen requires many blows of geological hammer to fracture it.
R6	Extremely Strong	> 250	Specimen can only be chipped with a geological hammer.

ROCK QUALITY DESIGNATION (RQD)

DESCRIPTION	RQD (%)
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

APERTURE

DESCRIPTION	CRITERIA [in (mm)]
Tight	<0.04 (<1)
Open	0.04 - 0.20 (1 - 5)
Wide	>0.20 (>5)

BEDDING CHARACTERISTICS

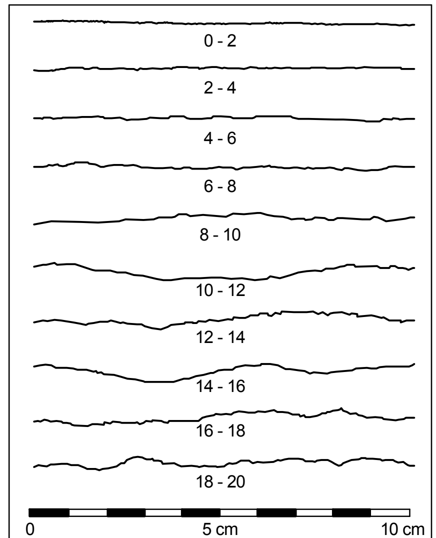
DESCRIPTION	Thickness [in (mm)]
Very Thick Bedded	>36 (>915)
Thick Bedded	12 - 36 (305 - 915)
Moderately Bedded	4 - 12 (102 - 305)
Thin Bedded	1 - 4 (25 - 102)
Very Thin Bedded	0.4 - 1 (10 - 25)
Laminated	0.1 - 0.4 (2.5 - 10)
Thinly Laminated	<0.1 (<2.5)

Bedding Planes — Planes dividing the individual layers, beds, or stratigraphy of rocks.

Joint — Fracture in rock, generally more or less vertical or traverse to bedding.

Seam — Applies to bedding plane with unspecified degree of weather.

JOINT ROUGHNESS COEFFICIENT (JRC)



From Barton and Choubey, 1977

RQD — Rock-quality designation (RQD) Rough measure of the degree of jointing or fracture in a rock mass, measured as a percentage of the drill core in lengths of 10 cm. or more.

CORE SAMPLER TYPE GRAPHICS

CORE SAMPLER

AQ CORE BARREL
(1.067 in. (27.1 mm.) core diameter)

AX CORE BARREL
(1.185 in. (30.1 mm.) core diameter)

BQ CORE BARREL
(1.433 in. (36.4 mm.) core diameter)

CONTINUOUS CORE SAMPLE
(2.000 in. (50.8 mm.) core diameter)

EX CORE BARREL
(0.846 in. (21.5 mm.) core diameter)

HQ CORE SAMPLE
(2.500 in. (63.5 mm.) core diameter)

NQ CORE SAMPLE
(1.874 in. (47.6 mm.) core diameter)

NO RECOVERY CORE SAMPLE

NX CORE SAMPLE
(2.154 in. (54.7 mm.) core diameter)

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	20190836	ROCK DESCRIPTION KEY	FIGURE
DRAWN BY:	MAP		
CHECKED BY:	BB	Fire Station #3 and #22 Bay Replacement 201 W. 30th Street / 5309 East Riverside Drive Austin, Texas	A-3
DATE:	9/25/2018		
REVISED:	-		

PLOTTED: 09/25/2018 01:52 PM BY: MPalmer
OFFICE FILTER: AUSTIN
PROJECT NUMBER: 20190836.001A
gINT FILE: Klf_gint_master_2017
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [LEGEND 3 (ROCK DESC KEY)]

Date Begin - End:	8/27/2018	**Drilling Company:**	Texas Geo Bore
Logged By:	J. Miller	**Drill Crew:**	Chris, Joey, Dauntez
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Mobile B-57
Plunge:	-90 degrees	**Drilling Method:**	Solid Stem Auger
Weather:	Sunny, 99° F	**Exploration Diameter:**	6 in. O.D.

BORING LOG SB-1

Hammer Type - Drop: 140 lb. Auto - 30 in.

Latitude: 30.29471°
Longitude: -97.73864°
Surface Condition: Concrete

Approximate Elevation (feet)	Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf RQD=%	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CONCRETE: 9"											
			Base: Crushed Limestone: light brown, loose (14")		BC=13 8 5									
			Fill: Lean CLAY with Sand and Gravel: brown and light brown, stiff, trace calcareous nodules,		BC=6 5 5			9.0			67	49	32	
			Fill: Fat CLAY: with sand pockets, dark brown, light brown, stiff to hard, with calcium calcareous nodules											
-5	5		**Fat CLAY (CH)**: dark brown, very stilff to hard, with calareous nodules, trace iron nodules		BC=4 6 6									
					PP=4.5									
			- with fine-grained gravel below 8 feet		PP=4.5+			8.8	130.3					**Unc. Comp. Str.=** q_u: 4.2 tsf Strain at Failure: 3.7%
			- light gray from 8 to 13.5 feet											
-10	10													
			- trace sand, calcareous nodules below 13.5 feet		BC=7 5 7									
-15	15		- light gray and light brown from 13.5 to 18 feet											
			- olive brown to dark brown, few calcareous nodules from 18 to 23 feet		PP=2.75									
-20	20		- laminated below 20 feet											
			- dark gray below 23 feet		PP=4.5+			21.4	107.7					**Unc. Comp. Str.=** q_u: 2 tsf Strain at Failure: 3.2%
-25	25													
			LIMESTONE: light gray, very weak to weak rock, highly fractured											
					BC=50/2"									
-30	30		- few shale seams below 30 feet		RQD=66	100%		5.6	140.9					**Unc. Comp. Str.=** q_u: 222.3 tsf Strain at Failure: 3.9%

Date Begin - End:	8/27/2018	Drilling Company:	Texas Geo Bore	**BORING LOG SB-1**
Logged By:	J. Miller	Drill Crew:	Chris, Joey, Dauntez	
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	Mobile B-57	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Solid Stem Auger	
Weather:	Sunny, 99° F	Exploration Diameter:	6 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 30.29471°
Longitude: -97.73864°
Surface Condition: Concrete

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			LIMESTONE: light gray, very weak to weak rock, highly fractured		RQD=17	90%								
-40	40				RQD=25	89%								
-45	45				RQD=55	81%		4.8	145.0					**Unc. Comp. Str.=** q_u: 144.9 tsf Strain at Failure: 2.0%
-50	50													
-55	55													
-60	60													
-65	65													

The boring was terminated at approximately 50 ft. below ground surface. The boring was backfilled with hydrated bentonite chips and patched with concrete at the surface on August 27, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	8/27/2018	Drilling Company:	Texas Geo Bore	BORING LOG SB-2
Logged By:	J. Miller	Drill Crew:	Chris, Joey, Jamel	
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	Mobile B-57	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Solid Stem Auger	
Weather:	Hot, Humid, 99° F	Exploration Diameter:	6 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 30.29469°
Longitude: -97.73857°
Surface Condition: Concrete

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Blow Counts(BC)= Uncorr. Blows/6 in. / Pocket Pen(PP)= tsf / RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		CONCRETE: 6"										
		Base: Crushed Limestone: light brown (14")	BC=3 4 4									
		Lean CLAY with Sand (CL): dark brown, stiff, trace calcium carbonate nodules										
		Fat CLAY (CH): trace fine to coarse-grained gravel, dark brown, stiff to hard, trace calcareous nodules and pockets	BC=3 8 8									
-5	5		BC=4 6 6									
		- with ferrous stains below 6 feet	PP=4.5+									
		- light brown and dark brown, 6 to 13 feet										
		- hard, 6 to 23 feet	PP=4.5+									
-10	10											
		light gray and brown mottled, 13 to 18 feet	PP=4.5+		CH	19.2			77	60	41	
-15	15											
		- dark brown, below 18 feet	PP=4.5+									
		- laminated, 19 to 20 feet										
-20	20											
			PP=4.5+			21.9	100.7					**Unc. Comp. Str.=** q_u: 2.7 tsf Strain at Failure: 6.3%
-25	25											
			RQD=0	42%								
		LIMESTONE: light gray, weak rock, few shale seams										
-30	30		RQD=50	100%								
		- vertical fracture/weathering from 31 to 32 feet										

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20190836	BORING LOG SB-2	FIGURE
	DRAWN BY: MAP		SB-2
	CHECKED BY: OB	City of Austin Fire Stations Fire Station #3 201 West 30th Street Austin, Texas	
	DATE: 9/25/2018		
	REVISED: -		PAGE: 1 of 2

Date Begin - End:	8/27/2018	Drilling Company:	Texas Geo Bore	**BORING LOG SB-2**
Logged By:	J. Miller	Drill Crew:	Chris, Joey, Jamel	
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	Mobile B-57	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Solid Stem Auger	
Weather:	Hot, Humid, 99° F	Exploration Diameter:	6 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 30.29469°
Longitude: -97.73857°
Surface Condition: Concrete

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			LIMESTONE: light gray, weak rock, few shale seams		RQD=0	40%								
-40	40				RQD=18	100%		4.7	147.6					**Unc. Comp. Str.=** q_u: 186.2 tsf Strain at Failure: 2.0%
-45	45													
-50	50													
-55	55													
-60	60													
-65	65													

The boring was terminated at approximately 45 ft. below ground surface. The boring was backfilled with hydrated bentonite chips and patched with concrete at the surface on August 27, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	8/28/2018	**Drilling Company:**	Texas Geo Bore		**BORING LOG SB-3**
Logged By:	J. Miller	**Drill Crew:**	Chris, Joey, Jamel		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Mobile B-57	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Solid Stem Auger		
Weather:	Sunny, Humid, 99° F	**Exploration Diameter:**	6 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 30.29462°
Longitude: -97.73857°
Surface Condition: Concrete

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CONCRETE: 7"											
			Base: Crushed Limestone: light brown (14")		BC=4 5 5									
			Fat CLAY (CH): trace sand, dark brown, hard, trace calcareous nodules		PP=4.5+									
					PP=4.5+									
-5	5													
			Clayey SAND (SC): trace fine-grained gravel, few calcareous nodules, brown		PP=4.5+		SC	6.6			47	47	31	
-10	10													
			Fat CLAY (CH): trace sand, trace gravel, light brown and gray, very stiff to hard		PP=3.5									
-15	15													
			- yellowish brown below 18 feet		PP=3.5		CH	24.4			97	64	45	
-20	20													
					PP=4.5+									
-25	25													
			LIMESTONE: light gray, few shale seams, very weak to weak rock		RQD=31	66%								
-30	30				RQD=78	100%								

Date Begin - End:	8/28/2018	**Drilling Company:**	Texas Geo Bore
Logged By:	J. Miller	**Drill Crew:**	Chris, Joey, Jamel
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Mobile B-57
Plunge:	-90 degrees	**Drilling Method:**	Solid Stem Auger
Weather:	Sunny, Humid, 99° F	**Exploration Diameter:**	6 in. O.D.

BORING LOG SB-3

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 30.29462°
Longitude: -97.73857°
Surface Condition: Concrete

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			LIMESTONE: light gray, few shale seams, very weak to weak rock		RQD=18	92%		6.6	141.0					**Unc. Comp. Str.=** q_u: 61.3 tsf Strain at Failure: 2.3%
-40	40				RQD=36	78%								
-45	45													
-50	50													
-55	55													
-60	60													
-65	65													

The boring was terminated at approximately 45 ft. below ground surface. The boring was backfilled with hydrated bentonite chips and patched with concrete at the surface on August 28, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	9/11/2018	Drilling Company:	Kleinfelder
Logged By:	B. Baugh	Drill Crew:	B. Baugh
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	Hand Auger
Plunge:	-90 degrees	Drilling Method:	Hand Auger
Weather:	Overcast, Light Rain	Exploration Diameter:	6 in. O.D.

BORING LOG SB-4

Latitude: 30.29456°
Longitude: -97.73882°
Surface Condition: Bare Earth

Approximate Elevation (feet)	Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf RQD=%	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Fat CLAY (CH): dark brown, moist, with roots					17.8						
			- with gravel below 2 feet											
			Sandy Lean CLAY (CL): trace gravel, light brown, moist				CL	6.4			68	42	29	
-5	5													
-10	10													
-15	15													
-20	20													
-25	25													
-30	30													

The boring was terminated at approximately 5 ft. below ground surface. The boring was backfilled with hydrated bentonite chips and patched with concrete at the surface on September 11, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20190836	BORING LOG SB-4	FIGURE
	DRAWN BY: MAP		
	CHECKED BY: OB	City of Austin Fire Stations Fire Station #3 201 West 30th Street Austin, Texas	SB-4
	DATE: 9/25/2018		
	REVISED: -		PAGE: 1 of 1

Bright People. Right Solutions.

Appendix B

Analytical Report 600445

for

Kleinfelder - Austin

Project Manager: Orlando Boscan

Fire Station 3 & 22 Reconstruction

20190836.001A

05-OCT-18

Collected By: Client

9701 Harry Hines Blvd
Dallas, TX 75220

Xenco-Houston (EPA Lab Code: TX00122):
Texas (T104704215-18-27), Arizona (AZ0765), Florida (E871002-24), Louisiana (03054)
Oklahoma (2017-142)

Xenco-Dallas (EPA Lab Code: TX01468):
Texas (T104704295-18-17), Arizona (AZ0809), Arkansas (17-063-0)

Xenco-El Paso (EPA Lab Code: TX00127): Texas (T104704221-18-13)
Xenco-Lubbock (EPA Lab Code: TX00139): Texas (T104704219-18-17)
Xenco-Midland (EPA Lab Code: TX00158): Texas (T104704400-18-18)
Xenco-San Antonio (EPA Lab Code: TNI02385): Texas (T104704534-18-4)
Xenco Phoenix (EPA Lab Code: AZ00901): Arizona (AZ0757)
Xenco-Phoenix Mobile (EPA Lab Code: AZ00901): Arizona (AZM757)
Xenco-Atlanta (LELAP Lab ID #04176)
Xenco-Tampa: Florida (E87429)
Xenco-Lakeland: Florida (E84098)

Table of Contents

05-OCT-18

Project Manager: **Orlando Boscan**
Kleinfelder - Austin
1826 Kramer Ln, Suite M

Austin, TX 78758

Reference: XENCO Report No(s): **600445**
Fire Station 3 & 22 Reconstruction
Project Address: ---

Orlando Boscan:

We are reporting to you the results of the analyses performed on the samples received under the project name referenced above and identified with the XENCO Report Number(s) 600445. All results being reported under this Report Number apply to the samples analyzed and properly identified with a Laboratory ID number. Subcontracted analyses are identified in this report with either the NELAC certification number of the subcontract lab in the analyst ID field, or the complete subcontracted report attached to this report.

Unless otherwise noted in a Case Narrative, all data reported in this Analytical Report are in compliance with NELAC standards. The uncertainty of measurement associated with the results of analysis reported is available upon request. Should insufficient sample be provided to the laboratory to meet the method and NELAC Matrix Duplicate and Matrix Spike requirements, then the data will be analyzed, evaluated and reported using all other available quality control measures.

The validity and integrity of this report will remain intact as long as it is accompanied by this letter and reproduced in full, unless written approval is granted by XENCO Laboratories. This report will be filed for at least 5 years in our archives after which time it will be destroyed without further notice, unless otherwise arranged with you. The samples received, and described as recorded in Report No. 600445 will be filed for 45 days, and after that time they will be properly disposed without further notice, unless otherwise arranged with you. We reserve the right to return to you any unused samples, extracts or solutions related to them if we consider so necessary (e.g., samples identified as hazardous waste, sample sizes exceeding analytical standard practices, controlled substances under regulated protocols, etc).

We thank you for selecting XENCO Laboratories to serve your analytical needs. If you have any questions concerning this report, please feel free to contact us at any time.

Respectfully,

Kalei Stout

Kalei Stout
Laboratory Manager

Recipient of the Prestigious Small Business Administration Award of Excellence in 1994.
Certified and approved by numerous States and Agencies.
A Small Business and Minority Status Company that delivers SERVICE and QUALITY

Houston - Dallas - Midland - San Antonio - Phoenix - Oklahoma - Latin America

Sample Cross Reference 600445

Kleinfelder - Austin, Austin, TX

Fire Station 3 & 22 Reconstruction

Sample Id	Matrix	Date Collected	Sample Depth	Lab Sample Id
SB-2	S	08-27-18 00:00	.5 - 4 ft	600445-001
B-3	S	08-27-18 00:00	2 - 4 ft	600445-002

CASE NARRATIVE

Client Name: Kleinfelder - Austin
Project Name: Fire Station 3 & 22 Reconstruction

Project ID:	*20190836.001A*	Report Date:	*05-OCT-18*
Work Order Number(s):	*600445*	Date Received:	*09/27/2018*

This laboratory is NELAC accredited under the Texas Laboratory Accreditation Program for all the methods, analytes, and matrices reported in this data package except as noted. The data have been reviewed and are technically compliant with the requirements of the methods used, except where noted by the laboratory.

Sample receipt non conformances and comments:

None

Sample receipt non conformances and comments per sample:

None

Certificate of Analytical Results
600445
Kleinfelder - Austin, Austin, TX
Fire Station 3 & 22 Reconstruction

Sample Id:	**SB-2**	Matrix:	Soil	Sample Depth:	.5 - 4 ft
Lab Sample Id:	600445-001	Date Collected:	08.27.18 00.00	Date Received:	09.27.18 08.20

Analytical Method: Soil pH by EPA 9045C | Prep Method:
Analyst: CHD | % Moist: | Tech: CHD
Seq Number: 3065144 | Date Prep:
Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
pH	12408-02-5	**10.8**			SU	10.03.18 09:55	K	
Temperature +	TEMP	**21.9**			Deg C	10.03.18 09:55	K	1

Analytical Method: Chloride, Mercuric Nitrate Method by SM4500 Cl-C | Prep Method:
Analyst: SDK | % Moist: | Tech: SDK
Seq Number: 3065358 | Date Prep:
Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
Chloride	16887-00-6	**4.94**	4.94	1.26	mg/kg	10.03.18 16:00	JK	1

Analytical Method: Sulfate by SW-846 9038 | Prep Method:
Analyst: SHT | % Moist: | Tech: SHT
Seq Number: 3065021 | Date Prep:
Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
Sulfate	14808-79-8	**331**	49.4	16.5	mg/kg	10.02.18 10:30	K	10

Analytical Method: Soil Resistivity (Saturated) by ASTM G57 | Prep Method:
Analyst: TRS | % Moist: | Tech: TRS
Seq Number: 3065227 | Date Prep:
Subcontractor: SUB: TX104704215-18-27 | Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
Resistivity (as saturated)	RESISTIVITY	**1445**			Ohm-cm	10.03.18 14:00	U	1

Certificate of Analytical Results
600445
Kleinfelder - Austin, Austin, TX
Fire Station 3 & 22 Reconstruction

Sample Id: **B-3** | Matrix: Soil | Sample Depth: 2 - 4 ft

Lab Sample Id: 600445-002 | Date Collected: 08.27.18 00.00 | Date Received: 09.27.18 08.20

Analytical Method: Soil pH by EPA 9045C | Prep Method:

Analyst: CHD | % Moist: | Tech: CHD

Seq Number: 3065144 | Date Prep:

Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
pH	12408-02-5	**8.47**			SU	10.03.18 09:55	K	
Temperature +	TEMP	**22.2**			Deg C	10.03.18 09:55	K	1

Analytical Method: Chloride, Mercuric Nitrate Method by SM4500 Cl-C | Prep Method:

Analyst: SDK | % Moist: | Tech: SDK

Seq Number: 3065358 | Date Prep:

Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
Chloride	16887-00-6	**2430**	497	127	mg/kg	10.03.18 16:00	K	99

Analytical Method: Sulfate by SW-846 9038 | Prep Method:

Analyst: SHT | % Moist: | Tech: SHT

Seq Number: 3065021 | Date Prep:

Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
Sulfate	14808-79-8	**20.9**	49.4	16.4	mg/kg	10.02.18 10:30	JK	10

Analytical Method: Soil Resistivity (Saturated) by ASTM G57 | Prep Method:

Analyst: TRS | % Moist: | Tech: TRS

Seq Number: 3065227 | Date Prep:

Subcontractor: SUB: TX104704215-18-27 | Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
Resistivity (as saturated)	RESISTIVITY	**1022**			Ohm-cm	10.03.18 14:00	U	1

Sample Id: **3065021-1-BLK** Matrix: Solid Sample Depth:

Lab Sample Id: 3065021-1-BLK Date Collected: Date Received:

Analytical Method: Sulfate by SW-846 9038 Prep Method:

Analyst: SHT % Moist: Tech: SHT

Seq Number: 3065021 Date Prep:

Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
Sulfate	14808-79-8	<16.4	49.2	16.4	mg/kg	10.02.18 10:30	U	10

Sample Id: **3065358-1-BLK** Matrix: Solid Sample Depth:

Lab Sample Id: 3065358-1-BLK Date Collected: Date Received:

Analytical Method: Chloride, Mercuric Nitrate Method by SM4500 Cl-C Prep Method:

Analyst: SDK % Moist: Tech: SDK

Seq Number: 3065358 Date Prep:

Prep seq:

Parameter	CAS Number	Result	MQL	SDL	Units	Analysis Date	Flag	Dil Factor
Chloride	16887-00-6	<1.26	4.95	1.26	mg/kg	10.03.18 16:00	U	1

Flagging Criteria

X In our quality control review of the data a QC deficiency was observed and flagged as noted. MS/MSD recoveries were found to be outside of the laboratory control limits due to possible matrix /chemical interference, or a concentration of target analyte high enough to affect the recovery of the spike concentration. This condition could also affect the relative percent difference in the MS/MSD.

B A target analyte or common laboratory contaminant was identified in the method blank. Its presence indicates possible field or laboratory contamination.

D The sample(s) were diluted due to targets detected over the highest point of the calibration curve, or due to matrix interference. Dilution factors are included in the final results. The result is from a diluted sample.

E The data exceeds the upper calibration limit; therefore, the concentration is reported as estimated.

F RPD exceeded lab control limits.

J The target analyte was positively identified below the quantitation limit and above the detection limit.

U Analyte was not detected.

L The LCS data for this analytical batch was reported below the laboratory control limits for this analyte. The department supervisor and QA Director reviewed data. The samples were either reanalyzed or flagged as estimated concentrations.

H The LCS data for this analytical batch was reported above the laboratory control limits. Supporting QC Data were reviewed by the Department Supervisor and QA Director. Data were determined to be valid for reporting.

K Sample analyzed outside of recommended hold time.

JN A combination of the "N" and the "J" qualifier. The analysis indicates that the analyte is "tentatively identified" and the associated numerical value may not be consistent with the amount actually present in the environmental sample.

** Surrogate recovered outside laboratory control limit.

BRL Below Reporting Limit.

RL Reporting Limit

MDL Method Detection Limit **SDL** Sample Detection Limit **LOD** Limit of Detection

PQL Practical Quantitation Limit **MQL** Method Quantitation Limit **LOQ** Limit of Quantitation

DL Method Detection Limit

NC Non-Calculable

SMP Client Sample **BLK** Method Blank

BKS/LCS Blank Spike/Laboratory Control Sample **BKSD/LCSD** Blank Spike Duplicate/Laboratory Control Sample Duplicate

MD/SD Method Duplicate/Sample Duplicate **MS** Matrix Spike **MSD:** Matrix Spike Duplicate

+ NELAC certification not offered for this compound.

***** (Next to analyte name or method description) = Outside XENCO's scope of NELAC accreditation

BS / BSD Recoveries

Project Name: Fire Station 3 & 22 Reconstruction

Work Order #: 600445 **Project ID:** 20190836.001A

Analyst: SDK **Date Prepared:** 10/03/2018 **Date Analyzed:** 10/03/2018

Lab Batch ID: 3065358 **Sample:** 3065358-1-BKS **Batch #:** 1 **Matrix:** Solid

Units: mg/kg

BLANK /BLANK SPIKE / BLANK SPIKE DUPLICATE RECOVERY STUDY

Chloride, Mercuric Nitrate Method by SM4500 Cl-C Analytes	Blank Sample Result [A]	Spike Added [B]	Blank Spike Result [C]	Blank Spike %R [D]	Spike Added [E]	Blank Spike Duplicate Result [F]	Blk. Spk Dup. %R [G]	RPD %	Control Limits %R	Control Limits %RPD	Flag
Chloride	<1.27	49.8	52.3	105	49.5	52.0	105	1	75-125	25	

Analyst: SHT **Date Prepared:** 10/02/2018 **Date Analyzed:** 10/02/2018

Lab Batch ID: 3065021 **Sample:** 3065021-1-BKS **Batch #:** 1 **Matrix:** Solid

Units: mg/kg

BLANK /BLANK SPIKE / BLANK SPIKE DUPLICATE RECOVERY STUDY

Sulfate by SW-846 9038 Analytes	Blank Sample Result [A]	Spike Added [B]	Blank Spike Result [C]	Blank Spike %R [D]	Spike Added [E]	Blank Spike Duplicate Result [F]	Blk. Spk Dup. %R [G]	RPD %	Control Limits %R	Control Limits %RPD	Flag
Sulfate	<16.3	195	192	98	194	192	99	0	80-120	20	

Relative Percent Difference RPD = 200*|(C-F)/(C+F)|
Blank Spike Recovery [D] = 100*(C)/[B]
Blank Spike Duplicate Recovery [G] = 100*(F)/[E]
All results are based on MDL and Validated for QC Purposes

Form 3 - MS / MSD Recoveries

Project Name: Fire Station 3 & 22 Reconstruction

Work Order # : 600445 **Project ID:** 20190836.001A

Lab Batch ID: 3065358 **QC- Sample ID:** 600445-001 S **Batch #:** 1 **Matrix:** Soil

Date Analyzed: 10/03/2018 **Date Prepared:** 10/03/2018 **Analyst:** SDK

Reporting Units: mg/kg

MATRIX SPIKE / MATRIX SPIKE DUPLICATE RECOVERY STUDY

Chloride, Mercuric Nitrate Method by SM4500 Cl-C	Parent Sample Result [A]	Spike Added [B]	Spiked Sample Result [C]	Spiked Sample %R [D]	Spike Added [E]	Duplicate Spiked Sample Result [F]	Spiked Dup. %R [G]	RPD %	Control Limits %R	Control Limits %RPD	Flag
Chloride	4.94	49.4	54.4	100	49.6	47.1	85	14	75-125	25	

Lab Batch ID: 3065021 **QC- Sample ID:** 600337-001 S **Batch #:** 1 **Matrix:** Soil

Date Analyzed: 10/02/2018 **Date Prepared:** 10/02/2018 **Analyst:** SHT

Reporting Units: mg/kg

MATRIX SPIKE / MATRIX SPIKE DUPLICATE RECOVERY STUDY

Sulfate by SW-846 9038 Analytes	Parent Sample Result [A]	Spike Added [B]	Spiked Sample Result [C]	Spiked Sample %R [D]	Spike Added [E]	Duplicate Spiked Sample Result [F]	Spiked Dup. %R [G]	RPD %	Control Limits %R	Control Limits %RPD	Flag
Sulfate	176	198	364	95	195	358	93	2	75-125	20	

Matrix Spike Percent Recovery [D] = 100*(C-A)/B
Relative Percent Difference RPD = 200*|(C-F)/(C+F)|

Matrix Spike Duplicate Percent Recovery [G] = 100*(F-A)/E

ND = Not Detected, J = Present Below Reporting Limit, B = Present in Blank, NR = Not Requested, I = Interference, NA = Not Applicable
N = See Narrative, EQL = Estimated Quantitation Limit, NC = Non Calculable - Sample amount is > 4 times the amount spiked.

Sample Duplicate Recovery

Project Name: Fire Station 3 & 22 Reconstruction

Work Order #: 600445

Lab Batch #: 3065227 **Project ID:** 20190836.001A
Date Analyzed: 10/03/2018 14:00 **Date Prepared:** 10/03/2018 **Analyst:** TRS
QC- Sample ID: 600445-001 D **Batch #:** 1 **Matrix:** Soil
Reporting Units: Ohm-cm

SAMPLE / SAMPLE DUPLICATE RECOVERY

Soil Resistivity (Saturated) by ASTM G57 Analyte	Parent Sample Result [A]	Sample Duplicate Result [B]	%RPD	RPD Limit	Flag
Resistivity (as saturated)	1446	1446	0	20	U

Lab Batch #: 3065144
Date Analyzed: 10/03/2018 09:55 **Date Prepared:** 10/03/2018 **Analyst:** CHD
QC- Sample ID: 600337-001 D **Batch #:** 1 **Matrix:** Soil
Reporting Units: Deg C

SAMPLE / SAMPLE DUPLICATE RECOVERY

Soil pH by EPA 9045C Analyte	Parent Sample Result [A]	Sample Duplicate Result [B]	%RPD	RPD Limit	Flag
Temperature	21.4	22.2	4	25	

Lab Batch #: 3065144
Date Analyzed: 10/03/2018 09:55 **Date Prepared:** 10/03/2018 **Analyst:** CHD
QC- Sample ID: 600337-001 D **Batch #:** 1 **Matrix:** Soil
Reporting Units: SU

SAMPLE / SAMPLE DUPLICATE RECOVERY

Soil pH by EPA 9045C Analyte	Parent Sample Result [A]	Sample Duplicate Result [B]	%RPD	RPD Limit	Flag
pH	7.53	7.93	5	20	

Log Difference Log Diff. = Log(Sample Duplicate) - Log(Original Sample)
Spike Relative Difference RPD 200 * | (B-A)/(B+A) |
All Results are based on MDL and validated for QC purposes.

BRL - Below Reporting Limit

Chain of Custody

XENCO LABORATORIES

Work Order No: 600445-D

Houston,TX (281) 240-4200 Dallas,TX (214) 902-0300 San Antonio,TX (210) 509-3334
Midland,TX (432-704-5440) EL Paso,TX (915)585-3443 Lubbock,TX (806)794-1296
Hobbs,NM (575-392-7550) Phoenix,AZ (480-355-0900) Atlanta,GA (770-449-8800) Tampa,FL (813-620-2000)

www.xenco.com Page ______ of ______

Project Manager:	Orlando Boscan	Bill to: (if different)	
Company Name:	Kleinfelder	Company Name:	
Address:	1826 Kramer Lane Suite M	Address:	
City, State ZIP:	Austin, TX 78758	City, State ZIP:	
Phone:	512-926-6650	Email:	Oboscan@Kleinfelder.com

Work Order Comments

Program: UST/PST ☐ PRP ☐ Brownfields ☐ RC ☐ Superfund ☐

State of Project:

Reporting: Level II ☐ Level III ☐ PST/UST ☐ RRP ☐ Level IV ☐

Deliverables: EDD ☐ ADaPT ☐ Other:

Project Name:	Fire Station 3&22 Reconstruction	Turn Around	
Project Number:	20190836.001A	Routine ☐	
P.O. Number:	20190836.001A	Rush:	
Sampler's Name:	Ben Baugh	Due Date:	

SAMPLE RECEIPT	Temp Blank: Yes No	Wet Ice: Yes No
Temperature (°C):	20.4	Thermometer ID: XDA
Received Intact:	Yes No	
Cooler Custody Seals:	Yes No N/A	Correction Factor: +0.3°
Sample Custody Seals:	Yes No N/A	Total Containers:

ANALYSIS REQUEST

Sample Identification	Matrix	Date Sampled	Time Sampled	Depth	Number of Containers	Soil Ph		Soi Resisitivity	Chloride Content	Sulfate Content	Sample Comments
SB-2	soil	8/27/2018	N/A	0.5-4ft	2	X		X	X	X	
B-3	soil	27-Aug	N/A	2-4ft	2	X		X	X	X	

Work Order Notes

TAT starts the day recevied by the lab, if received by 4:30pm

Total 200.7 / 6010 200.8 / 6020: 8RCRA 13PPM Texas 11 Al Sb As Ba Be B Cd Ca Cr Co Cu Fe Pb Mg Mn Mo Ni K Se Ag SiO2 Na Sr Tl Sn U V Zn

Circle Method(s) and Metal(s) to be analyzed **TCLP / SPLP 6010**: 8RCRA Sb As Ba Be Cd Cr Co Cu Pb Mn Mo Ni Se Ag Tl U **1631 / 245.1 / 7470 / 7471** : Hg

Notice: Signature of this document and relinquishment of samples constitutes a valid purchase order from client company to Xenco, its affiliates and subcontractors. It assigns standard terms and conditions of service. Xenco will be liable only for the cost of samples and shall not assume any responsibility for any losses or expenses incurred by the client if such losses are due to circumstances beyond the control of Xenco. A minimum charge of $75.00 will be applied to each project and a charge of $5 for each sample submitted to Xenco, but not analyzed. These terms will be enforced unless previously negotiated.

Relinquished by: (Signature)	Received by: (Signature)	Date/Time	Relinquished by: (Signature)	Received by: (Signature)	Date/Time
1 Ben Baugh	[signature]	9/25/18 12:00	2		
3 [signature]	FedEx		4 FedEx	Whitney Capps	9/27/18 0820
5			6		

Revised Date 051418 Rev. 2018.1

XENCO
LABORATORIES

Custody Seal / Shipping Record

Client Name: Kleinfelder-Austin

Lab Work Order #: 600445

THU - 27 SEP AA
HOT 2DAY

TRK# 0201 7733 2049 1640

40 KIPA

75220
TX-US
DFW

952943 26Sep 12:02 AFWH 547C1/F78C/A17C

XENCO LABORATORIES

Inter-Office Shipment

IOS Number **114730**

Date/Time: 09/27/18 21:57
Created by: Angelica Martinez
Please send report to: Kalei Stout

Lab# From: **Dallas**
Delivery Priority: Fedex
Address: 9701 Harry Hines Blvd

Lab# To: **Houston**
Air Bill No.: 773345816488
E-Mail: kalei.stout@xenco.com

Sample Id	Matrix	Client Sample Id	Sample Collection	Method	Method Name	Lab Due	HT Due	PM	Analytes	Sign
600445-001	S	SB-2	08/27/18 00:00	ASTM G57SAT	Soil Resistivity (Saturated) by ASTM G5	10/03/18	02/23/19	KLS		
600445-002	S	B-3	08/27/18 00:00	ASTM G57SAT	Soil Resistivity (Saturated) by ASTM G5	10/03/18	02/23/19	KLS		

Inter Office Shipment or Sample Comments:

Relinquished By: Angelica Martinez
Angelica Martinez

Date Relinquished: 09/27/2018

Received By: Shakhshir
Monica Shakhshir

Date Received: 09/28/2018 09:45

Cooler Temperature: 4.5

XENCO Laboratories

Inter Office Report- Sample Receipt Checklist

Sent To: Houston
IOS #: 114730

Acceptable Temperature Range: 0 - 6 degC
Air and Metal samples Acceptable Range: Ambient
Temperature Measuring device used : HOU-068

Sent By: Angelica Martinez   **Date Sent:** 09/27/2018 09:57 PM
Received By: Monica Shakhshir   **Date Received:** 09/28/2018 09:45 AM

Sample Receipt Checklist		Comments
#1 *Temperature of cooler(s)?	4.5	
#2 *Shipping container in good condition?	**Yes**	
#3 *Samples received with appropriate temperature?	**Yes**	
#4 *Custody Seals intact on shipping container/ cooler?	**Yes**	
#5 *Custody Seals Signed and dated for Containers/coolers	**Yes**	
#6 *IOS present?	**Yes**	
#7 Any missing/extra samples?	**No**	
#8 IOS agrees with sample label(s)/matrix?	**Yes**	
#9 Sample matrix/ properties agree with IOS?	**Yes**	
#10 Samples in proper container/ bottle?	**Yes**	
#11 Samples properly preserved?	**Yes**	
#12 Sample container(s) intact?	**Yes**	
#13 Sufficient sample amount for indicated test(s)?	**Yes**	
#14 All samples received within hold time?	**Yes**	

*** Must be completed for after-hours delivery of samples prior to placing in the refrigerator**

NonConformance:

Corrective Action Taken:

Nonconformance Documentation

Contact: ______________ **Contacted by :** ______________ **Date:** ______________

Checklist reviewed by: Monica Shakhshir   Date: 09/28/2018

XENCO Laboratories
Prelogin/Nonconformance Report- Sample Log-In

Client: Kleinfelder - Austin

Date/ Time Received: 09/27/2018 08:20:00 AM

Work Order #: 600445

Acceptable Temperature Range: 0 - 6 degC
Air and Metal samples Acceptable Range: Ambient

Temperature Measuring device used : XDA

Sample Receipt Checklist		Comments
#1 *Temperature of cooler(s)?	20.4	
#2 *Shipping container in good condition?	**Yes**	
#3 *Samples received on ice?	**No**	
#4 *Custody Seals intact on shipping container/ cooler?	**No**	
#5 Custody Seals intact on sample bottles?	**N/A**	
#6*Custody Seals Signed and dated?	**N/A**	
#7 *Chain of Custody present?	**Yes**	
#8 Any missing/extra samples?	**No**	
#9 Chain of Custody signed when relinquished/ received?	**Yes**	
#10 Chain of Custody agrees with sample labels/matrix?	**Yes**	
#11 Container label(s) legible and intact?	**Yes**	
#12 Samples in proper container/ bottle?	**Yes**	
#13 Samples properly preserved?	**Yes**	
#14 Sample container(s) intact?	**Yes**	
#15 Sufficient sample amount for indicated test(s)?	**Yes**	
#16 All samples received within hold time?	**Yes**	
#17 Subcontract of sample(s)?	**Yes**	Xenco Stafford Resistivity
#18 Water VOC samples have zero headspace?	**N/A**	

*** Must be completed for after-hours delivery of samples prior to placing in the refrigerator**

Analyst: PH Device/Lot#:

Checklist completed by: Angelica Martinez Date: 09/27/2018
Angelica Martinez

Checklist reviewed by: Kalei Stout Date: 09/28/2018
Kalei Stout