Preliminary Engineering
Geotechnical Design Memorandum for Preliminary 20% Design

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Subject:	**GDM 11 for Concept Design** Geotechnical Design Recommendations for Critical Structures and Summary of Criteria
Revision:	Revision A

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SUMMARY

Subject/Objective

This Geotechnical Design Memorandum (GDM) presents a geotechnical evaluation of and recommendations for the proposed foundation, retaining wall and underground structural systems for planned portal, cut-and-cover station, ventilation and station entrance structures. Geotechnical design parameters for soil and rock, and a summary of the assumptions is provided as input for subsequent geotechnical analyses including continuum and soil-structure interaction. Feasible support-of-excavation (SOE) and foundation systems are presented.

The geotechnical design parameters, evaluations, recommendations and design pressure diagrams for this memorandum have been revised from Technical Memorandum (TM #11) based on the following information:

- Final Geotechnical Data Report
- Geotechnical Design Memorandum GDM #3
- Structural Tunnel Drawings dated December 20, 2019

This GDM includes updates from the above-mentioned information and incorporates the most recent project alignment when approved by DART. The project structures described herein are based on December 20, 2019 project alignment.

Conclusions

The primary conclusions from this evaluation are that non-driven drilled-in pile foundation systems are viable SOE for the unique project constraints including geotechnical and surface site conditions. Namely, the following SOE systems are feasible based on planned underground structures: pre-drilled rock-socketed soldier piles-and-lagging system for portal approaches, braced rigid systems such as slurry wall or secant pile wall keyed into bedrock for cut-and-cover station boxes, and internally-braced flexible systems for ventilation and station entrance shafts. Pre-support measures such as ground improvement and/or canopy arch spiling will need to be installed for ground control at critical locations such as the shallow cover condition at the transition between cut-and-cover and SEM tunnel (interface of Reach 6 and Reach 7) at Harwood Street. Design verification during construction should include a compatible instrumentation and monitoring plan for adjacent sensitive structures.

1 INTRODUCTION

1.1 Purpose

The purpose of this geotechnical design memorandum is to summarize the geotechnical evaluation including recommended geotechnical design parameters for performing preliminary 20% design of portal, cut-and-cover station, ventilation and station entrance structures, and the SOE of these structures for Conceptual Design. The geotechnical evaluation described herein is based on Geotechnical Design Memorandum #3 – Preliminary Ground Characterization Revision A, February 19, 2020, DART D2 Project, and the underground alignment current as of December 20, 2019.

1.2 Scope

The scope of this memorandum is a final draft preliminary geotechnical evaluation for foundation, ground retention and underground structures associated with excavation and construction of planned portal, station entrance and ventilation shaft structures for the alignment and configuration current as of December 20, 2019.

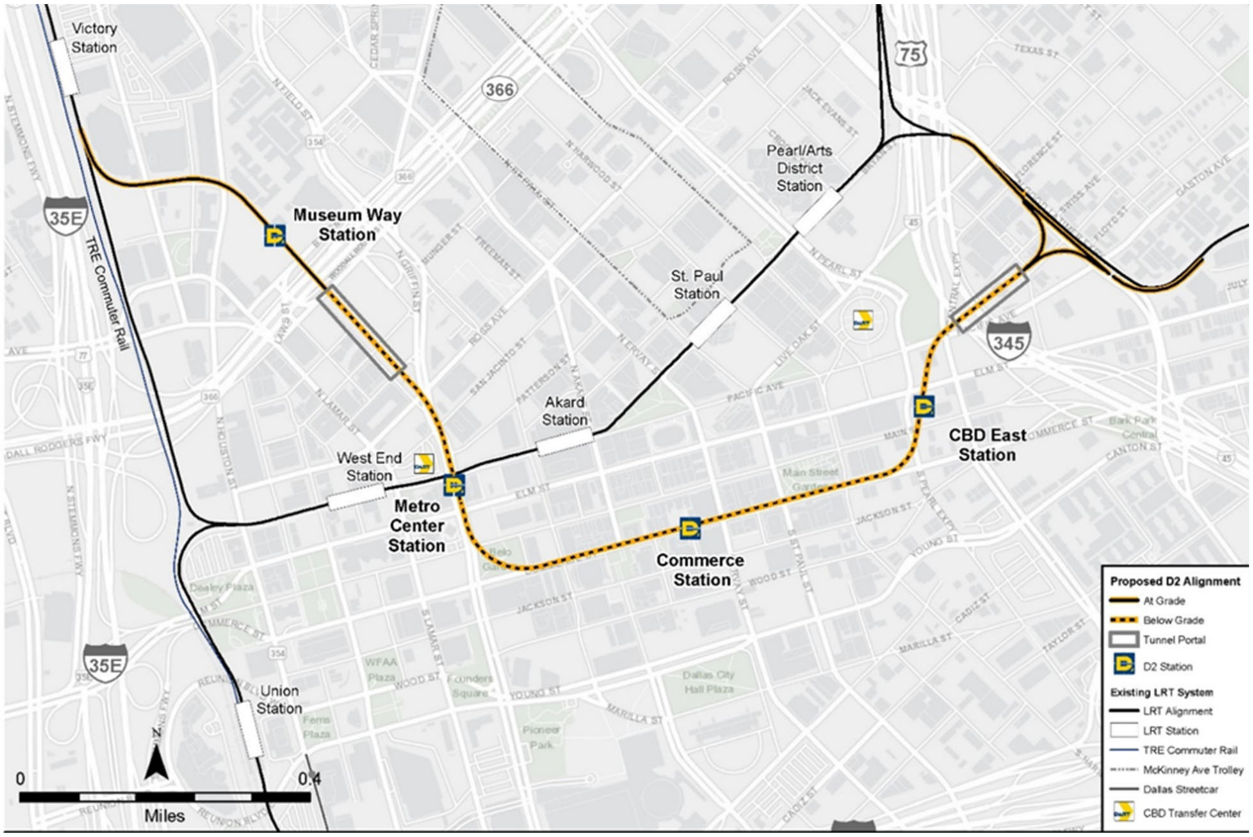
This design memorandum will be included in the Concept Geotechnical Inventory and Conceptual Design Report.

2 BACKGROUND

2.1 Project Description (Standardized)

The Dallas Central Business District (CBD) DART Second Light Rail (D2 Subway), Locally Preferred Alignment (LPA) current as of latest received on December 20, 2019, includes east and west portals, twin bored or mined tunnels, three underground transit stations from Station 41+50 to 101+55.23 totaling approximately 6005-ft (Figure 2-1). These planned underground structures will feature foundation and retaining wall systems associated with the retained portal approaches, cut-and-cover stations, and cut-and-cover tunnels as well as shafts for station entrances and ventilation structures along the LPA. Future at-grade double crossovers are planned near the portals.

FIGURE 2-1. LPA ALIGNMENT UPDATED VERSION DECEMBER 20, 2019



2.2 Inputs

Relevant available subsurface data were compiled and reviewed to prepare this GDM including the following:

Revision A Geotechnical Design Memorandum #3 –Ground Characterization Revision A, February 19, 2020, DART D2 Project.

This GDM is based on DART D2 20% design alignment and configuration dated December 20, 2019.

2.3 Assumptions and Limitations

2.3.1 ASSUMPTIONS

The DART D2 project alignment and configuration is current as of December 20, 2019. Any changes in this alignment or configuration could affect assumptions regarding construction considerations associated with revised locations and/or geometry of portal and cut-and-cover station structures.

It has been assumed that interpretations of ground conditions provided in GDM #3 – Preliminary Ground Characterization dated February 19, 2020 are representative of the anticipated conditions during project construction.

2.3.2 LIMITATIONS

The ground characterization presented in GDM 3 was inferred from limited currently available site-specific geotechnical information and supplemented by published information and data from other relevant Dallas area projects. Actual site-specific ground and groundwater conditions encountered during construction of retaining structures described herein may differ from those presented in GDM #3.

This GDM does not specifically address hazardous substances or contaminated soil, rock, or groundwater since project geotechnical investigation performed to-date have excluded environmental sampling and testing scope.

This GDM presents geotechnical recommendations for design of foundation, ground retention and underground structures associated with excavation and construction of planned portal, station entrance and ventilation shaft structures based on project alignment and configuration current as of December 20, 2019. Hence, geotechnical recommendations provided herein may be subject to change should the project alignment and configuration be revised.

3 PORTAL STRUCTURES

3.1 West Portal and Approaches

The 20% project alignment's planned west portal extends southeast parallel to North Griffin Street from south of Spur 366 (Woodall-Rodgers) to approx. 85-ft east of Corbin Street. The portal will consist of a 620-ft long retained cut and cut-&-cover approach structures from McKinney Avenue to Hord Street, with depths up to 27-ft and approximate 75-ft width at the headwall. The tunnel portal will be constructed such that future development could span over the U-wall section. Pedestrian access is planned along both sides of the portal. An at-grade double crossover is planned under Woodhall-Rodgers viaduct based on March 7, 2019 alignment.

The north approach and portal limits are located across N. Griffin Street from the Dallas World Aquarium building and within associated parking lots east of North Griffin Street. Use of drilled foundation and retaining wall systems will help minimize construction impacts such as settlement and noise and vibration on adjacent buildings during construction.

3.1.1 EXISTING CONDITIONS

The planned north portal limits currently consist of at-grade property adjacent to and east of North Griffin Street and south of Woodall-Rodgers to Hord Street. The portal headwall is located between Corbin Street and Hord Street and lies within the existing parking lot area located north of Griffin Street.

SUBSURFACE CONDITIONS

Based on General Geologic Profile presented in Figure 4_C of GDM #3 the generalized subsurface stratigraphy consists of successive strata of 1-ft surficial asphalt and concrete pavement, 2 to 4-ft thick fine Sand fill, 17-ft to 23-ft of Alluvium primarily consisting of cohesionless material ranging from silty sands / sands and gravel to clayey sands. The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. A thin 2-ft thick layer of weathered Limestone underlies the Alluvium at approximate 20-ft to 25-ft depth below existing ground surface. Limestone bedrock may be encountered in the bottom 3-ft to 4-ft of invert excavation south of Corbin Street for the alignment extending approximately 50-ft northwest from headwall of the west portal. Proposed invert depth of U-walls varies from 13-ft at start of U-wall increasing to 31-ft at portal headwall below existing ground surface. Rock core samples recovered from nearby test borings within the anticipated rock socket depth of soldier piles and lagging and/or secant pile wall support of excavation system extending from top of rock to 10-ft to 18-ft depth below rock had recoveries ranging from 90% to 100%, and RQD values of 90% to 100%. Groundwater elevation El +411 measured/ observed in test borings corresponding to approximate 18-ft depth below ground surface.

The planned portal headwall east of Corbin Street extends to approximately 37-ft invert depth and includes about 10-ft of anticipated excavation in limestone bedrock. Joints dipping from 45° to 60° have been reported in the Limestone within the project vicinity. Based on available nearby test borings, presence of Shale is not anticipated within the excavation limits of the north portal structures.

GEOTECHNICAL DESIGN PARAMETERS

Based on review of currently available test boring data consisting of GDM #3 in-progress Draft Ground Characterization developed from Final Geotechnical Data Report dated August 29, 2019, preliminary design parameters have been developed and are summarized in Tables 3-1 and 3-2 for soil strata and rock formations, respectively.

TABLE 3-1. PRELIMINARY GEOTECHNICAL DESIGN PARAMETERS FOR WEST PORTAL RETAINING STRUCTURES

Stratum	Approx. Thickness (ft)	Moist Unit Weight (pcf)	Friction Angle (°)	Cohesion (psf)		Earth Pressure Coefficient	
				C_u	C'	K_a	K_o
Fill	2 – 5	107	30	0		0.333	0.5
Alluvium – A1 medium stiff Clay	0 - 8	112	24		470	*	
Alluvium – A2 Loose to medium dense Sand	2 - 8	109	28	0		0.307	0.47
IGM – Weathered Rock	0 - 2	117	33			0.3	0.455
Limestone bedrock	0 - 3	129		-		-	

Preliminary design groundwater elevation El +411 based on observation during drilling of boring T-1 and T-6. *The Alluvium A1 soils are capable of developing swell pressures greater than at-rest earth pressures.

TABLE 3-2. PRELIMINARY ROCK MASS DESIGN PROPERTIES FOR WEST PORTAL RETAINING STRUCTURES

Formation	RQD (Core Run numbers)	Estimated UCS (psi) (field description)	Discontinuity properties			
			Type	Dip Angle (°)	Joint roughness J_r	Joint alteration J_a
Limestone – hard to very hard, unweathered	98% to 100% (RC-1 through RC-3)	1000 – 1500 (hard, strong) 1500 – 2500 (very hard, very strong)	*	*	*	*

Note *Project-specific discontinuity data will be provided when available.

3.1.2 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Under these site conditions, a feasible temporary braced support of excavation (SOE) system consisting of rock-socketed drilled soldier piles and lagging wall or where groundwater level encountered above bottom of excavation a temporary braced sheet-pile wall may be considered and should be designed based on the geotechnical design parameters presented in Tables 3-1 and 3-2. However, a rigid SOE system such as secant pile wall may be necessary based on site-specific settlement assessment. Design of these SOE systems should consider potential impacts on existing structures and include a compatible instrumentation and monitoring program.

During construction, surface protection measures should be implemented to avoid introducing water into clays possessing high swell potential and erosion-susceptible sand. Specifically, consider a diversion structure at the top of excavated invert ramp to protect of any exposed cohesive and granular subgrade and/or surfaces. After completion of SOE walls for “U”-shaped portal approach section, a mud slab should be installed over the compacted backfill bearing on the subgrade. The waterproofing membrane should be placed over the mud slab and against shotcrete-lined internal face of SOE walls prior to installing final cast-in-place (CIP) lining and roadway pavement. Diligent field supervision and quality control is key to construction of the undrained portal approach structure.

3.2 East Portal

The planned location of the east portal structure extends from North Central Expressway, east of North Cesar Chavez Boulevard and north of Pacific Avenue crossing underneath the existing Interstate 345 (I-345) running northeast north of Swiss Avenue to North Hawkins Street. The portal will commence along Swiss Avenue at North Hawkins Street and consists of a 600- ft long retained cut and cut-&-cover structure of 30-ft depth and approx. 80-ft width at the headwall near North Cesar Chavez Boulevard and Pacific Avenue. Pedestrian access is planned along both sides of the portal.

3.2.1 EXISTING CONDITIONS

Adjacent building developments along Swiss Avenue in Deep Ellum include Buell & Co. Public Storage, Sherwin-Williams, Bottled Blonde, Lizard Lounge and various unnamed, undeveloped and/or vacated buildings.

SUBSURFACE CONDITIONS

Based on General Geologic Profile (GDM #3), the generalized subsurface profile within the east portal excavation limits consists of 1-ft to 7-ft thick surficial Fill overlying successively strata of Alluvium Type A1 (approx. 5-ft to 16-ft), up to 2-ft thick weathered Limestone and maximum 2-ft Limestone bedrock at the Cesar Chavez Boulevard corresponding to western limit of Reach 10. The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. Limestone bedrock was encountered at depths exceeding 20-ft below existing grade and only reaching bottom of invert elevation within limits of Reach 10 at Cesar Chavez Boulevard. Groundwater level was encountered within the Alluvium at 14-ft depth corresponding to El +456 during drilling of Boring P-102. Rock cores recovered from 21-ft to 51-ft depth corresponding to approximately 20-ft below invert excavation represent anticipated rock socket depths. Rock core recoveries ranged from 93% to 100% and RQD values from 95% to 99% at these anticipated rock socket depths. Subsurface conditions at the planned headwall (Station 95+00) near Pacific Avenue will be provided based on results of proposed test boring T-112 located some 200-ft northeast of the headwall. The planned headwall excavation will extend approximately 60-ft below existing grade.

GEOTECHNICAL DESIGN PARAMETERS

Based on review of currently available test boring data, preliminary design parameters have been developed and are summarized in Table 3-3 for soil strata below:

TABLE 3-3. PRELIMINARY GEOTECHNICAL DESIGN PARAMETERS FOR EAST PORTAL RETAINING STRUCTURES

Stratum	Approx. Thickness (ft)	Moist Unit Weight (pcf)	Friction angle (°)	Cohesion (psf)		Earth Pressure Coefficient	
				C_u	C′	K_a	K_o
Fill - miscellaneous	1 – 7	107	30	0		0.333	0.5
Alluvium – A1 medium stiff Clay	5 – 16	112	24		470	*	
IGM - Weathered Limestone	0 - 2	117	33	0		0.3	0.455

Preliminary design groundwater elevation El +456 based on observations during drilling of boring P-102.
***The Alluvium A1 soils are capable of developing swell pressures greater than at-rest earth pressures.**

3.2.2 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Under these site conditions, a braced support of excavation system consisting of rock-socketed drilled soldier piles and lagging wall with pattern rock dowels and shotcrete support applied for the underlying rock excavation is considered feasible for the portal structures and should be designed based on the geotechnical design parameters presented in Table 3-2. For situations where groundwater level encountered above the bottom of excavation, or where the presence of nearby buildings susceptible to damage from surface settlement induced by groundwater drawdown exists, a rigid watertight SOE system such as secant pile may be appropriate. Selection of suitable SOE system should provide necessary ground control measures to mitigate potential for ground loss due to loss of fines, or surface subsidence due to consolidation or lateral movements. Ground improvement is also considered as a viable option for groundwater cut-off. For excavations exceeding 25-ft depth, rigid SOE systems may be required. A concrete collar or frame as applicable, may be installed at the top of rock to provide additional lateral support to the soldier piles. Upon receipt of updated geotechnical data, including soil and rock mass laboratory test results, an assessment of construction-impact should be performed to identify any necessary structural reinforcement and/or mitigation strategies.

During construction, surface protection measures should be implemented to avoid introducing water into clays possessing high swell potential and erosion-susceptible sand. Specifically, diversion devices at the top of the excavated ramp should be considered to protect any exposed cohesive and granular subgrade and/or surface. After completion of SOE walls for “U”-shaped portal approach section, a mud slab should be installed over the compacted backfill bearing on the subgrade. The waterproofing membrane should be placed over the mud slab and against shotcrete-lined internal face of SOE walls prior to installing final cast-in-place (CIP) lining and roadway pavement. Diligent field supervision and quality control is key to construction of the undrained portal approach structure.

4 STATION CAVERNS

4.1 Metro Center Station – Cut-and-Cover Structures

For 20% Project Alignment current as of December 20, 2019, the planned Metro Center Station is located within the North Griffin Street ROW from Patterson Street to Elm Street, and oriented in the north-south direction. This station is currently planned to be constructed as a cut-&-cover structure due to lack of available rock cover necessary for a stable mined cavern construction including a mezzanine level. The station box and associated shaft structures are located within Reach 3 as described in Geotechnical Design Memorandum #3. Potential shaft locations under consideration include: north end station entrances at both the west and east sides of the station on North Griffin Street, north of Pacific Avenue and adjacent to existing DART West Transfer Center to the west and at existing at-grade parking to the east, and south end station entrances at the northeast corner of Elm Street and North Griffin Street within the street ROW. DART-owned Rosa Parks Plaza may also be considered for access to businesses, residences and destinations west of the station. Final shaft locations will be determined based on availability of sites for easement and property acquisition to accommodate operational (passenger circulation and ventilation) requirements.

4.1.1 CUT-AND-COVER STATION BOX

The planned 410-ft long station box excavation located within the street ROW will extend to approximate invert El +356 corresponding to about 72 feet below grade. Shale is anticipated to be encountered in the lower 5 to 6 feet of the excavation. Based on available test borings B-1 (northwest end) and TS-104 (eastern portion) of the station, the quality of the shale is expected to be relatively poorer at the northwest end of the station and improving to the east.

SUBSURFACE CONDITIONS

The General Geologic Profile provided in GDM #3 dated February 19, 2020 includes test boring B-1, located at the intersection of Griffin Street and Pacific Avenue towards the northern end of the station limits, and boring TS-104, located on Griffin Street between Pacific Avenue and Elm Street. The generalized subsurface profile may be described as 1.5-ft asphalt/ concrete pavement with granular base course overlying successive strata of loose Sandy Clay to very loose Sand (possibly Fill) to 5-ft depth, 9-ft of very hard Sandy Clay (pp > 4.5 tsf), approx. 11-ft of very loose Sand extending to 25-ft, and 4-ft of weathered moderately hard to hard Limestone with clay layers. The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. Limestone bedrock was encountered at 29-ft depth. The Limestone bedrock containing locally interlayered shaley seams (thicknesses ranging from 6 inches to 20 inches) with typical core recovery and RQD values exceeding 98%. Shale was encountered below the Limestone at 66-ft depth. Water seepage was observed during drilling at 20-ft depth, providing preliminary estimate of groundwater level. The approximately 72-ft depth of station box excavation is anticipated to include 25-ft of soil, 4-ft of Intermediate Geomaterial (IGM) representing weathered Limestone, approximately 36-ft of Limestone bedrock and 6-ft of Shale. "Fish Bed Conglomerate" or "Transition Zone" has been reported at the base of the Limestone per GDM #3. Hence, considering the upper 1 to 4-feet of the Shale may be susceptible to sloughing upon excavation. GDM #3 reports slickensides indicative of faulting on several fracture sets at various orientations in Boring T-104.

4.1.2 SHAFTS FOR STATION ENTRANCE/ EGRESS AND VENTILATION

The potential station entrance locations described in Section 4.1 should lead to the consideration of extending the support of excavation limits to encompass the shafts beyond the station cavern footprint. Shaft excavation outside the North Griffin Street ROW would likely require measures to address presence of potential utility conflicts. Anticipated depth of entrance shafts would be on the order of 35-ft corresponding to approximate mezzanine level.

The need for ventilation shafts at this station must consider safety and operational requirements for the entire underground alignment and will be determined per NFPA 130 and DART Design Criteria. If needed, ventilation shaft depths on the order of 50-ft would extend to the future platform level. Feasibility of potential locations at either end of the station would be subject to review.

SUBSURFACE CONDITIONS

Based on completed nearby test boring B-1 located at intersection of North Griffin Street and Pacific Avenue towards the northern end of the station limits, the generalized subsurface profile may be described as 1.5-ft asphalt/ concrete pavement with granular base course overlying successive strata of loose Sandy Clay to very loose Sand (possibly Fill) to 5-ft depth, 9-ft of very hard Sandy Clay (pocket penetrometer (pp) > 4.5 tsf), approx. 11-ft of very loose Sand extending to 25-ft, and 4-ft of weathered, moderately hard to hard Limestone with clay layers overlying Limestone bedrock at 29-ft depth. The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. The Limestone bedrock containing locally interlayered shaley seams (thicknesses ranging from 6 inches to 20 inches has typical core recovery and RQD values exceeding 98%. Shale stratum was encountered below the Limestone at 66-ft depth. Invert excavation in the Shale ranges from 1-ft to 7-ft. Laboratory testing results performed on Shale core samples measured slake durability index (SDI) values ranging from 40% to 44%. These SDI values correspond to very low durability per Gamble's Slake Durability Classification. Water seepage was observed during drilling at 20-ft depth, providing preliminary estimate of groundwater level corresponding to approximate El +409. The approximately 35-ft depth of entrance shaft excavation is anticipated to include 25-ft of soil, 4-ft of weathered Limestone and approximately 6-ft of Limestone bedrock. Consider the following soil profile for approximate 50-ft depth of ventilation shaft excavation: 25-ft soil, 4-ft weathered Limestone and 21-ft of Limestone bedrock.

GEOTECHNICAL DESIGN PARAMETERS

Based on review of GDM #3, preliminary design parameters have been developed and are summarized in Tables 4-1 and 4-2 for soil strata and rock formations, respectively.

TABLE 4-1. PRELIMINARY GEOTECHNICAL DESIGN PARAMETERS FOR METRO CENTER STATION VICINITY RETAINING STRUCTURES

Stratum	Approx. Thickness (ft)	Moist Unit Weight (pcf)	Friction angle (°)	Cohesion (psf)		Earth Pressure Coefficient	
				C_u	C′	K_a	K_o
Fill - miscellaneous	2 – 3	107	30	0		0.333	0.5
Alluvium – A1 medium stiff Clay	4 – 10	112	24		470	*	
Alluvium – A2	8 - 15	109	28	0		0.307	0.47
IGM - Weathered Limestone	2 - 3	117	33	0		0.3	0.455
Limestone bedrock	35 - 40	129					
Shale bedrock	1 - 7	136					

Preliminary design high groundwater elevation El +410 based on well measurements of boring TS-104.
***The Alluvium A1 soils are capable of developing swell pressures greater than at-rest earth pressures.**

TABLE 4-2. PRELIMINARY ROCK MASS DESIGN PROPERTIES FOR METRO CENTER STATION VICINITY

Formation	RQD (Core Run numbers)	Estimated UCS (psi) (field description)	Discontinuity properties			
			Type	Dip Angle (°)	Joint roughness J_r	Joint alteration J_a
Limestone – hard to very hard, unweathered w/ occasional shale seams	98% to 100% (RC-1 through RC-4)	1000 – 1500 (hard, strong) 1500 – 2500 (very hard, very strong)	*	*	*	*
Shale	NA	NA	*	*	*	*

Note *Project-specific discontinuity data will be provided when available.

4.1.3 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Under these site conditions, a braced rigid support of excavation system consisting of slurry wall or secant pile wall system toed into top of rock is considered feasible for the relatively large footprint and depth of the cut-and-cover station box. For the shallower and smaller footprint entrance and ventilation shaft excavations, an internally braced flexible SOE system, such as a rock-socketed drilled soldier piles and lagging wall is considered appropriate. For both these SOE systems, pattern rock dowels and shotcrete support within the underlying rock excavation is considered feasible. Design of these SOE wall systems would be based on the geotechnical design parameters presented in Tables 4-1 and 4-2. The design of station invert slab and wall support should consider estimated swell pressures based on the very low durability associated with the Slake Durability Index values reported in GDM #3 and future swell test data. Upon receipt of updated geotechnical data, including soil and rock mass laboratory testing results, an assessment of construction impact will be performed to identify any necessary structural reinforcement and/or mitigation strategies.

Preliminary screening using available subsurface data and as-built building records should be performed to assess construction impact on adjacent structures. This screening will help identify buildings susceptible to damage from shaft construction, which would be candidates for further study, including pre-construction condition survey, settlement analysis, recommended instrumentation and monitoring program, and proposed mitigation measures. A summary of the impact assessment performed for these structures should be summarized in construction impact assessment reports. During construction, the approved instrumentation and monitoring program should be coordinated with construction, and include both pre- and post-construction condition surveys to document building performance. During excavation preparatory measures including grading and diversion of groundwater inflow and rainwater runoff should be implemented to protect the shale exposed in sidewalls and subgrade. Upon completion of excavation, a mud slab should be installed over the exposed shale subgrade to protect against potential swelling and air-slaking due to moisture. The waterproofing membrane should be placed over the mud slab and over shotcrete-lined internal face of SOE walls prior to construction of final CIP walls of the shaft and subsequent backfill restoration to finished pavement. Diligent field supervision and quality control is key to construction of the undrained shaft structure.

4.2 Commerce Station – Shaft Structures

This station is currently planned to be constructed as a mined cavern which limits surface impacts to localized shaft penetrations to accommodate future station, entrance/egress and ventilation facilities. Based on the December 20, 2019 project alignment, the planned shaft locations include: west end main station entrances north of Commerce Street within limits of Pegasus Plaza featuring an underground passageway connection to the mined station as well as an east end station emergency egress located south of Commerce Street between Lane Street and Ervin Street. This parcel is currently occupied by Jack Boles elevated parking structure.

4.2.1 SHAFTS FOR STATION ENTRANCE/ EGRESS AND VENTILATION

The main station entrances will be located at Pegasus Plaza to provide convenient station access for occupants of the Federal buildings, AT&T Headquarters Campus, The Adolphus and Magnolia Hotels, and multiple other businesses and residences as well as destinations, respectively. These off-line shafts would be constructed beyond the station cavern footprint and will require underground adit connections to the station. Anticipated depth of entrance shafts would be on the order of 55-ft, corresponding to mezzanine level, or may extend somewhat deeper, should an under-platform mezzanine configuration be adopted.

Two ventilation shafts for this station will be located to consider safety and operational requirements for the entire underground alignment and mined station caverns and will be determined per NFPA 130 and DART Design Criteria. Specifically, off-line ventilation shaft structures will be located at the north side of Commerce Street along east side of existing Magnolia Hotel and south of Commerce Street on east side of Browder Street.

Assuming ventilation shaft location at center of planned station, shafts would extend to approximately 50-ft depth to reach the future platform level.

SUBSURFACE CONDITIONS

Based on General Geologic Profile Figure 4-E and 4-F provided in GDM #3, including completed nearby test borings TS-202 and B-3, the generalized subsurface profile may be described as surficial fill (1-ft to 4-ft thick) underlain by Alluvium A1– Clay up to 5-ft thick, Sandy Alluvium A2 up to 4-ft thick and weathered rock (approx. 2-ft to 4-ft thick) underlain by Limestone bedrock. The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. Bedrock was encountered at depths ranging from approx. 10-ft to 15-ft below existing grade. Slickensided fractures dipping at 45 to 50 degrees and at 60 degrees were observed in the limestone bedrock within the lower 25-ft of the station cavern excavation.

GEOTECHNICAL DESIGN PARAMETERS

Based on review of currently available test boring data which is to be updated upon receipt of soil laboratory test data results, preliminary design parameters have been developed and are summarized in Tables 4-3 and 4-4 for soil strata and rock formations, respectively.

TABLE 4-3. PRELIMINARY GEOTECHNICAL DESIGN PARAMETERS FOR COMMERCE STATION VICINITY RETAINING STRUCTURES

Stratum	Approx. Thickness (ft)	Moist Unit Weight (pcf)	Friction angle (°)	Cohesion (psf)		Earth Pressure Coefficient	
				C_u	C′	K_a	K_o
Fill - miscellaneous	1 - 4	107	30	0		0.333	0.5
Alluvium – A1 medium stiff Clay	4 – 5	112	24		470	*	
Alluvium – A2	–2 - 4	109	28	0		0.307	0.47
IGM - Weathered Limestone	2 - 4	117	33	0		0.3	0.455
Limestone bedrock	70 - 83	129					
Shale bedrock	0 - 4	136					

Preliminary design groundwater elevations El +422 in overburden and El +414 in bedrock based on well measurements observations during drilling of test boring TS-202. *The Alluvium A1 soils are capable of developing swell pressures greater than at-rest earth pressures.

TABLE 4-4. PRELIMINARY ROCK MASS DESIGN PROPERTIES FOR COMMERCE STATION VICINITY

Formation	RQD (Core Run numbers)	Estimated UCS (psi) (field description)	Discontinuity properties			
			Type	Dip Angle (°)	Joint roughness J_r	Joint alteration J_a
Limestone – hard to very hard, unweathered w/ occasional shale seams	94% to 100% (RC-1 through RC-9)	1000 – 1500 (hard, strong) 1500 – 2500 (very hard, very strong)	*	*	*	*
Shale	NA	NA	*	*	*	*

Note *Project-specific discontinuity data will be provided when available.

4.2.2 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Under these site conditions, a braced support of excavation system consisting of rock-socketed drilled-in soldier pile and lagging wall; or where groundwater level is encountered above the bottom of excavation or where existing adjacent buildings would be exposed to potential damage from groundwater induced surface settlement, a secant pile wall with pattern rock dowels and shotcrete support applied within the underlying rock excavation is considered feasible for shaft structures and should be designed based on the geotechnical design parameters presented in Tables 4-3 and 4-4. For excavations greater than 25-ft depth, rigid SOE systems may be required. A concrete collar or frame, as applicable, may be installed at the top of rock to provide additional lateral support to the soldier piles. Selection of suitable SOE system should provide necessary ground control measures to mitigate any potential ground loss due to loss of fines, or surface subsidence due to consolidation and lateral movement. Ground improvement techniques are also considered a viable option to achieve groundwater cut-off at the top of rock. Upon receipt of updated geotechnical data, including soil and rock mass laboratory testing results, an assessment of construction-impact will be performed to identify any necessary structural reinforcement and/or mitigation strategies.

Preliminary screening using available subsurface data and as-built building records should be performed to assess potential construction impact on adjacent structures. This screening will help identify buildings susceptible to damage from shaft construction, which would be candidates for further study including pre-construction condition survey, settlement analysis, recommended instrumentation and monitoring program, and proposed mitigation measures. A summary of the impact assessment performed for these structures will be summarized in a construction impact assessment report(s). During construction, the approved instrumentation and monitoring program should be coordinated with construction and followed by a post-construction condition survey to document building performance. Upon completion of excavation, a mud slab should be installed over the compacted backfill bearing on the subgrade. The waterproofing membrane should be placed over the mud slab and shotcrete lined internal face of the SOE walls prior to construction of final CIP walls of the shaft and subsequent backfill restoration to finished pavement. Diligent field supervision and quality control is key to construction of the undrained shaft structure.

4.3 Central Business District (CBD) East Station – Cut-&-Cover structures

For the 20% Project Alignment, the planned CBD East Station is located east of Pearl Street stretching northwest from Main Street to Elm Street. The station is oriented in the northeast-southwest direction. This station is currently planned to be constructed as cut-&-cover station box. Currently shaft locations are still under study. Final shaft locations will be determined based on availability of sites for easement and property acquisition to accommodate operational (passenger circulation and ventilation) requirements.

4.3.1 SHAFTS FOR STATION ENTRANCE/ EGRESS AND VENTILATION

It is assumed that potential station entrances will be selected to provide convenient station access for occupants of the UNT System College of Law and surrounding future college campus buildings, Statler Hotel and Residences, Hotel Indigo, the Majestic Theatre and multiple other businesses, restaurants and residences as well as destinations. These off-line shafts would be constructed beyond the station cavern footprint and would require underground adit connections to the station mezzanine level. Anticipated depth of entrance shafts would be on the order of 35-ft, corresponding to the mezzanine level.

The number of ventilation shafts for this station must consider safety and operational requirements for the entire underground alignment and mined station caverns and will be determined per NFPA 130 and DART Design Criteria.

Assuming ventilation shaft location at the center of planned station, the shafts would extend to approximately 50-ft depth to reach the future platform level.

SUBSURFACE CONDITIONS

Based Figure 4-G and 4-H General Geologic Profile provided in GDM #3, the generalized subsurface profile in the vicinity of planned CBD East Station may be described as surficial Fill underlain by Alluvium – Clay, and IGM or weathered rock. The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. Limestone bedrock was encountered at 33-ft to 40-ft depth and extended to approx. 120-ft below grade. Shale was not encountered in any of these test borings within 120-ft depth below the ground surface. The 14-inch thick Bentonite Marker Bed was encountered at El +426 corresponding to approximate depths ranging from 35-ft to 40-ft below existing ground surface.

GEOTECHNICAL DESIGN PARAMETERS

Based on review of currently available test boring data which is to be updated upon receipt of soil laboratory test results, preliminary design parameters have been developed and are summarized in Tables 4-5 and 4-6 for soil strata and rock formations, respectively.

TABLE 4-5. PRELIMINARY GEOTECHNICAL DESIGN PARAMETERS FOR CBD EAST STATION VICINITY RETAINING STRUCTURES

Stratum	Approx. Thickness (ft)	Moist Unit Weight (pcf)	Friction angle (°)	Cohesion (psf)		Earth Pressure Coefficient	
				C_u	C'	K_a	K_o
Fill - miscellaneous	2 – 9	107	30	0		0.333	*
Alluvium – A1 medium stiff Clay	11 - 25	112	24		470	*	
Alluvium – A2	4	109	28	0		0.307	0.47
IGM - Weathered Limestone	1 - 4	117	33	0		0.3	0.455
Limestone bedrock	2	129					

Preliminary design groundwater elevations El +452 in overburden based on well measurements observations during drilling of test borings TS-206 and TS-209. *The Alluvium A1 soils are capable of developing swell pressures greater than at-rest earth pressures.

TABLE 4-6. PRELIMINARY ROCK MASS DESIGN PROPERTIES FOR CBD - EAST STATION VICINITY

Formation	RQD (Core Run numbers)	Estimated UCS (psi) (field description)	Discontinuity properties			
			Type	Dip Angle (°)	Joint roughness J_r	Joint alteration J_a
Limestone – hard to very hard, unweathered w/ occasional shale seams	97% to 100% (RC-1 through RC-4)	1000 – 1500 (hard, strong) 1500 – 2500 (very hard, very strong)	*	*	*	*
Shale	NA	NA	*	*	*	*

Note *Project-specific discontinuity data will be provided when available.

4.3.2 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Under these site conditions, a braced rigid support of excavation system consisting of slurry wall or secant pile wall system toed into top of rock is considered feasible for the relatively large footprint and depth of the cut-and-cover station box. For the shallower and smaller footprint entrance and ventilation shaft excavations, an internally braced flexible SOE system such as a rock-socketed drilled strutted soldier piles and lagging wall is considered appropriate. For shaft locations where groundwater level is encountered above the bottom of excavation or where existing adjacent buildings are susceptible to damage from potential surface settlement due to groundwater drawdown, a secant pile wall may be required. Groundwater cut-off methods including ground improvement may be required at the soil-weathered rock to bedrock interface. For both these SOE systems, pattern rock dowels and shotcrete support within the underlying rock excavation is considered feasible. Support design should be based on the geotechnical design parameters presented in Tables 4-5 and 4-6. Upon receipt of updated geotechnical data, including soil and rock mass laboratory testing results, an assessment of construction-impact should be performed to identify any necessary structural reinforcement and/or mitigation strategies.

Preliminary screening using available subsurface data and as-built building records should be performed to assess potential construction impact on adjacent structures. This screening will help identify buildings susceptible to damage from shaft construction, which would be candidates for further study including pre-construction condition survey, settlement analysis, recommended instrumentation and monitoring program, and proposed mitigation measures. A summary of the impact assessment performed for these structures will be summarized in a construction impact assessment report(s). During construction, the approved instrumentation and monitoring program should be coordinated with construction and followed by a post-construction condition survey to document building performance. Upon completion of excavation, a mud slab should be installed over the compacted backfill bearing on the subgrade. The waterproofing membrane should be placed over the mud slab and shotcrete lined internal face of SOE walls prior to construction of final CIP walls of the shaft and subsequent backfill restoration to finished pavement. Diligent field supervision and quality control is key to construction of the undrained shaft structure.

5 CUT-AND-COVER RUNNING TUNNELS

Cut-and-cover construction is currently anticipated and appears feasible for three distinct relatively shallow depth tunnel sections (designated Reaches 2, 7, and 9 in GDM #3) located within curved locations of the project alignment.

5.1 Cut-and-Cover Tunnel between West Portal and Metro Center Station

From the west portal headwall, the 20% project alignment proceeds underground extending southeast under Hord Street then curves south under Ross Avenue and San Jacinto Avenue before connecting to the west limit of Metro Center Station. This section of running tunnel designated as Reach 2 in GDM #3 consists of a 777-ft long relatively shallow depth section with a 1000-ft radius horizontal curve and a maximum rock cover of 10-ft in Limestone at the interface with Metro Center Station. The width of cut-and-cover box increases from approximately 38-ft at sta. 41+50 (west portal headwall) to approx. 66-ft at Sta. 49+27 (west limit of Metro Center Station). Due to steeply descending grade from portal to the station the tunnel excavation transitions rapidly from predominantly overburden with about 4-ft invert in rock to full-face rock under Ross Avenue. Cut-and-cover tunnel construction is favored due to the shallow (27-ft to 61-ft invert depth) profile with relatively limited rock cover and curved alignment conditions. Construction of the cut-and-cover tunnel will be staged such that traffic disruption on existing cross streets are minimized and completed in a manner that would accommodate any future deck-over development plans. Sequential Excavation Method construction may be considered for the deeper portion of Reach 2 however this option would require comprehensive pre-excavation measures and ground monitoring, this would be subject to further evaluation and analysis.

5.1.1 EXISTING CONDITIONS

The limits of Reach 2 cut-and-cover tunnel include existing paved parking lot east of North Griffin Street between Corbin Street and Hord Street at north end, existing two-story brick building at 1708 North Griffin Street) with surrounding paved parking lot, narrow triangular parcel bounded on the east by elevated North Griffin Street roadway, and continue under the elevated North Griffin Street roadway. The close proximity of adjacent multi-story building at 1001 Ross Street bordering the west side of South Griffin will impose noise and vibration limits and associated monitoring on construction.

SUBSURFACE CONDITIONS

Based on General Geologic Profile (GDM #3), the generalized subsurface profile within the excavation horizon of Reach 2 consists of successive strata of surficial Fill (less than 5-ft), Alluvium (15-ft to 21-ft thick), weathered Limestone (3-ft thick), Limestone bedrock, and the underlying Shale. The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. Depth to top of Limestone bedrock varies approx. 20-ft to 26-ft descending east towards Metro Center Station. Based on limited available data, excavations for proposed cut-and-cover tunnel will encounter rock ranging from about 4-ft to full face within the limits of Reach 2. Groundwater was encountered within the Alluvium at approximate 18-ft depth below existing ground surface corresponding to approximate El +410 based on water level measurements taken during drilling of nearby test borings. This groundwater level is approximately 4-ft to 7-ft above the top of rock within the limits of Reach 2. The excavation will extend approximately 10 to 40-ft below ground water level El +410.

5.1.2 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Based on review of the geotechnical data and ground characterization provided in GDM #3, preliminary design parameters were developed (Table 5-1)

TABLE 5-1. PRELIMINARY GEOTECHNICAL DESIGN PARAMETERS FOR REACH 2 CUT-&-COVER TUNNEL

Stratum	Approx. Thickness (ft)	Moist Unit Weight (pcf)	Friction angle (°)	Cohesion (psf)		Earth Pressure Coefficient	
				C_u	C'	K_a	K_o
Fill	2 – 4	107	30	0		0.333	0.5
Alluvium – A1 medium stiff Clay	7 - 11	112	24		470	*	
Alluvium – A2 Loose to medium dense Sand	6 - 11	109	28	0		0.307	0.47
IGM – Weathered Rock	2 – 3	117	33	0		0.3	0.455
Limestone bedrock	4 - 32	129		-		-	

Preliminary design groundwater elevation El +410 based on observations during drilling of borings T-102 and T-103. *The Alluvium A1 soils are capable of developing swell pressures greater than at-rest earth pressures.

Under these site conditions and considering the planned slightly curved plan alignment, key considerations for selection of SOE system for the overburden include: (i) excavation depth and width, (i) protection of existing adjacent and overhead structures, (ii) groundwater control, (iii) right-of-way restrictions, and (iv) limited surface space for construction.

The site groundwater condition would eliminate soldier-pile and lagging system recommended for U-wall section of the portal structures. Considering selection factors listed above, top-down construction sequence would address urban setting and need to expedite surface restoration. Rigid SOE systems such as slurry walls, secant pile or tangent pile walls keyed into top of competent rock should be considered. Hence, current recommendation is for a rigid watertight “bathtub”, internally braced SOE system keyed into rock for groundwater cut-off. Depending on available surface space for associated support equipment, i.e. de-sander and silo for slurry wall, less intrusive tangent or secant pile wall system. Low headroom construction equipment combined with pre-excavation trenching to lower existing grade will be required to install SOE walls under existing flyover structure.

Subsequent rock excavation support comprised of pattern rock dowels and shotcrete support applied for the underlying rock excavation is considered feasible. A concrete collar or frame as applicable, may be installed at the top of rock to provide additional lateral support to the soldier piles. Confirmatory field packer permeability testing of the rock mass is recommended to assess the required embedment cut-off into top of rock. The actual footprint location of the support of excavation scheme will require coordination with local City DOT including permit application for temporary sheet closures. The excavation support system should be designed using the geotechnical design parameters presented in Table 5-1. Site-specific settlement assessment may also identify adjacent buildings requiring additional pre-support measures such as pre-loaded struts to restrict ground movements and protect against construction damage. If actual depth to top of rock is encountered at greater than 25-ft depth for a significant portion of this section, rigid SOE systems may be required.

5.2 Cut-and-Cover Tunnel between Commerce Street and CBD East Station

Reach 7 of the project alignment features an approximate 320-ft horizontal radius curve extending eastward under Commerce Street from Harwood turning northeast under Pearl Street to west end of CBD East Station. This 683-ft long curved tunnel having excavation depths ranging from 27-ft at the western limit to approximate 61 ft at the eastern end is currently anticipated as a cut-&-cover structure due to insufficient available rock cover i.e. less than 10-ft. At the west limit near Harwood Street, the cut-and-cover tunnel transitions to mined SEM tunnel of Reach 8 which will require ground stabilization of arch. . Advanced utility relocation will be required within Reach 7, as described in Section 5.2.1 below

5.2.1 EXISTING CONDITIONS

The limits of Reach 7 are currently lined by a handful of existing buildings on both sides of Commerce Street as well as on the west side of South Pearl Expressway including four-story UNT Dallas College of Law at 106 South Harwood Street and five-story brick Dallas Municipal Court at 2014 Main Street on the north side of Commerce Street; two-story brick commercial building at 2008 & 2014 Commerce Street, two-story building at 2024 & 2026 Commerce Street, and abutting three-story (2036 Commerce Street) and two-story (2038) brick buildings at southwest corner of Commerce and South Pearl Expressway on the south side of Commerce Street. These buildings are separated by paved at-grade parking lots. At the northern end of Reach 7 a seven-story commercial building is situated at southwest corner of Main Street and South Pearl Expressway. This building is also surrounded by paved at-grade parking lots. The presence of an existing 8’-6” horseshoe-shaped storm sewer extending west-east beneath the north side of the Commerce Street right-of-way will require advance utility relocation prior to commencing cut-and-cover construction.

SUBSURFACE CONDITIONS

Within the excavation horizon of Reach 7, the generalized subsurface profile consists of successive strata of surficial Fill (approx. 3-ft to 10-ft thick), Alluvium A1 (4-ft to 23-ft thick), Weathered Limestone (2-ft to 4-ft thick), and Limestone bedrock (approx. 19-ft to 31-ft thick). The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. Depth to top of Limestone bedrock ranges from 15-ft under Pearl Expressway to 30-ft east of Harwood corresponding to western limit of Reach 7 at western limit near Harwood and about 23-ft at south side of Main Street. Based on limited available data, rock excavations for proposed cut-and-cover tunnel will range from 14-ft to 31-ft thickness. GDM #3 (Table 12) reports cut-&-cover tunnel excavation of 41% Limestone Class I and 8% Limestone Class II by volume within Reach 7. Per Table 3 of GDM #3, Class I Limestone is massive to moderately jointed predominantly Limestone with some shale having RQD > 90% and fracture spacing greater than 6-ft; and Class II Limestone is moderately blocky predominantly Limestone with some shale possessing RQD values between 50% and 90% and fracture spacing between 2 to 6 feet. Unconfined Compressive Strength values range from 1540 psi to 5800 psi based on laboratory testing (GDM #3, Table 16). Groundwater level measured in monitoring wells installed in TS-207 and TS-208 ranged from El +434 to El + 452 for well screened in overburden and El +428 and El +435 for well screened in rock.

5.2.2 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Based on review of the geotechnical data and ground characterization provided in GDM #3, preliminary design parameters were developed (Table 5-2)

TABLE 5-2. PRELIMINARY GEOTECHNICAL DESIGN PARAMETERS FOR REACH 7 CUT-&-COVER TUNNEL

Stratum	Approx. Thickness (ft)	Moist Unit Weight (pcf)	Friction angle (°)	Cohesion (psf)		Earth Pressure Coefficient	
				C_u	C′	K_a	K_o
Fill	3 – 10	107	30	0		0.333	0.5
Alluvium – A1 medium stiff Clay	4 – 22	112	24		470	*	
IGM – Weathered Rock	2 – 4	117	33	0		0.3	0.455
Limestone bedrock	12 - 31	129		-		-	

Preliminary design groundwater elevation El +452 in overburden and El+430 in rock based on observation well readings in borings TS-207 and TS-208. *The Alluvium A1 soils are capable of developing swell pressures greater than at-rest earth pressures.

Pre-construction activities will include relocation of existing storm water sewer line under Commerce Street to enable cut-and-cover tunnel construction to commence. Considering the presence of adjacent multi-story buildings, less than 20-ft depth to top of rock and ground water level measured within the Alluvium, a rigid watertight SOE system such as a secant pile wall system is feasible. The secant piles will need to be keyed into competent bedrock to establish effective groundwater cut-off and permit subsequent rock excavation to the tunnel invert extending approx. 23-ft to 31-ft into Limestone). Rock excavation using non-blasting methods i.e. appropriately sized roadheader, or hoe-ram combined with chemical rock-splitting agents as-needed, appear feasible based the UCS values provided above. Rock support system comprised of pattern rock dowels and shotcrete support is feasible. A tieback anchored waler installed at the base of the secant pile wall would provide additional lateral resistance and help protect the top of rock ledge perimeter. Alternatively, a watertight flexible SOE system such as braced sheet-pile wall system may also be considered at locations where overburden cuts are less than 25-ft depth. Actual limits of SOE installation will require coordination with local City DOT including permit application for temporary street closures. The excavation support system should be designed using the geotechnical design parameters provided in Table 5-2. Site-specific settlement assessment may also identify adjacent buildings requiring additional pre-support measures such as pre-loaded structs to restrict ground movements and protect against construction damage. The interface between SEM Tunnel (Reach 6) and Cut-and-Cover Tunnel (Reach 7) corresponds to about max 9-ft rock cover for approx. 62-ft to 71-ft excavation width. Arch stabilization consisting of canopy spiles will be required as pre-support for subsequent SEM mined tunnel in Reach 6 at this location under Harwood Street.

5.3 Cut-and-Cover Tunnel between CBD East Station and East Portal

Reach 9 of the project alignment features an approximate 400-ft horizontal radius curve extending from CBD East Station northeast crossing under Pacific Avenue to connect to East Portal headwall at Cesar Chavez Boulevard. The 360-ft long cut-and-cover tunnel structure has excavation depths ranging from approx. 24-ft to 34-ft. Box widths taper from 70-ft at the station to 45-ft at the portal headwall to accommodate existing IH 35 piers. Invert excavation will encounter up to 2-ft weathered Limestone underlain by up to 3-ft Limestone bedrock.

5.3.1 EXISTING CONDITIONS

The limits of Reach 9 currently extend northeast through existing at-grade paved parking lot (Platinum Parking) under Pacific Avenue and Cesar Chavez Boulevard. The elevated Southbound Interstate Highway 345 structure abuts the northeastern limit of Reach 9.

SUBSURFACE CONDITIONS

Based on Figure 4-H General Geologic Profile presented in GDM #3, the generalized subsurface profile within limits of Region 9 consist of surficial Fill (4-ft to 7-ft thick) overlying successive strata of Alluvium A1 (approx. 15-ft to 20-ft thick) and Alluvium A2 (approx. 2-ft to 5-ft), up to 2-ft Weathered Limestone and Limestone bedrock. The A1 – Cohesive Alluvium is highly expansive, moisture-sensitive and swell pressures will develop upon changes in moisture content. Invert excavation will include up to 3-ft Limestone bedrock consisting of Limestone Ground Class L-II. Per GDM #3, L-II is characterized as moderately blocky predominantly Limestone with some shale generally having RQD values between 50% to 90% and fracture spacing 2 to 6-feet. Typical discontinuity data includes one set of slickensided, polished fracture surfaces.

Ground water observed at 16-ft depth within the overburden corresponding to El +449 during drilling of Boring T-112.

5.3.2 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

Based on review of the geotechnical data and ground characterization presented in GDM #3, preliminary geotechnical design parameters were developed (Table 5-3).

TABLE 5-3. PRELIMINARY GEOTECHNICAL DESIGN PARAMETERS FOR REACH 9 CUT-&-COVER TUNNEL

Stratum	Approx. Thickness (ft)	Moist Unit Weight (pcf)	Friction angle (°)	Cohesion (psf)		Earth Pressure Coefficient	
				C_u	C’	K_a	K_o
Fill	3 – 7	107	30	0		0.333	0.5
Alluvium – A1 medium stiff Clay	5 – 24	112	24		470	*	
Alluvium – A2	3 - 5	109	28				
IGM – Weathered Rock	1 – 2	117	33	0		0.3	0.455
Limestone bedrock	12 - 31	129		-		-	

Preliminary design groundwater elevation El +449 in overburden based on observations during drilling of boring T-112. *The Alluvium A1 soils are capable of developing swell pressures greater than at-rest earth pressures.

Considering the relative absence of nearby buildings but with existing bridge piers, presence of groundwater in the Alluvium and planned excavation depths exceeding 22-ft in overburden, a watertight SOE system such as braced sheet-pile wall (flexible) or braced rigid secant pile wall is feasible. As-built location of adjacent bridge viaduct foundations should be field verified and if necessary, installation measures such as preclusion of driven methods should be implemented to avoid excess vibration. The selected SOE system should be keyed into competent bedrock to establish groundwater cut-off at the interface between overburden, weathered Limestone and Limestone bedrock. Invert excavation is expected to include up to 5-ft of Ground Class L-II Limestone bedrock. Rock reinforcement including rock dowels and shotcrete should be installed to stabilize moderately blocky Limestone characterized by slickensided and/or clay-coated or disintegrated rock altered surfaces for 5-ft excavation height. Design of SOE elements should use geotechnical design parameters provided in Table 5-3. Installation of SOE will require coordination with local City of Dallas DOT including permit application for temporary street closures.

6 SUMMARY OF CRITERIA FOR GEOTECHNICAL ANALYSES

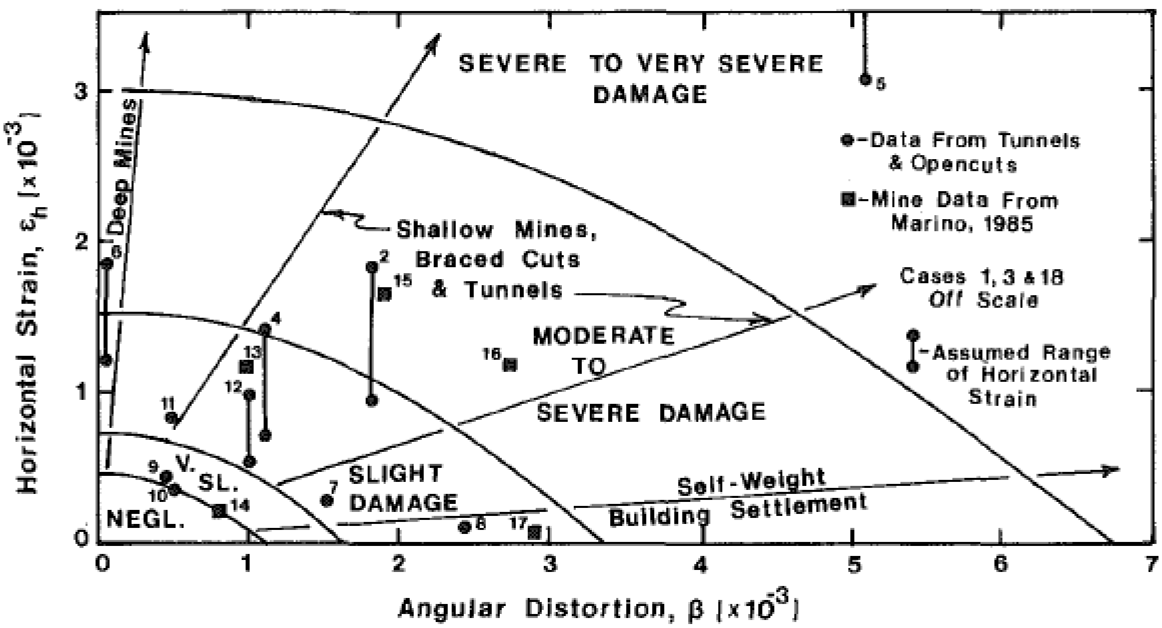
6.1 General Approach

Typically, building impact assessments of structures that are near excavations involve the following two phases: During preliminary design, an initial phase (Phase I) screening of potential susceptible structures is performed based on empirical and semi-analytical approaches. However, it should be noted that numerical analysis may be necessary to consider the anticipated complex subsurface site conditions. Then, the screening results are used to develop a list of vulnerable structures. These structures will be subject to a more rigorous/ comprehensive impact assessment, Phase II, involving soil-structure interaction analysis.

6.1.1 BUILDING IMPACT DAMAGE CRITERIA

The building impact evaluation should be performed considering the actual building type, such as brick and frame structures, including historic buildings, and then compared with acceptable range of angular distortion and lateral (tensile) strain values as shown in Figure 5-1. Key factors affecting building impact assessment include depth and size of excavation, geologic conditions (soil and rock types), excavation and support systems, distance between excavation limit and existing building structure as well as volume loss. Based on this building impact assessment, a project-specific building instrumentation and monitoring program will be developed which establishes suitable limits as well as threshold values for actions to be taken during construction.

FIGURE 5-1. RELATIONSHIP OF DAMAGE TO ANGULAR DISTORTION AND HORIZONTAL EXTENSION STRAIN (BOSCARDIN AND CORDING, 1989)



6.1.2 FINITE ELEMENT METHOD USING SOIL-STRUCTURE INTERACTION

Soil-structure interaction analysis using project-specific geotechnical data will be performed to confirm the initial building impact assessment. Specifically, the need for mitigation measures such as structural underpinning, compensation grouting, or ground treatment will be identified.

7 CONCLUSIONS

The current LPA includes portal approach structures, one mined and two cut-and-cover stations as well as associated shafts for station entrances and ventilation structures. This technical memorandum provides design considerations and construction recommendations for support of excavation systems which must be in compliance with applicable local ordinances, mitigate construction impacts on existing structures and accommodate transit patrons as well as occupants of nearby business within the dense commercial corridor of downtown Dallas surrounding the planned DART D2 alignment. Final design of critical cut-and-cover structures under both temporary construction and permanent conditions will be based on design pressure diagrams developed from available geotechnical data as well as the criteria for subsequent geotechnical analyses.

8 RECOMMENDATIONS FOR PE 20% DESIGN

8.1 Design Recommendation 1 – Use pre-drilled rock-socketed soldier piles-and-lagging or secant pile wall SOE system for portal approaches

8.1.1 DESIGN RECOMMENDATION

Use non-driven support of excavation systems for portal approaches and headwall such as pre-drilled rock socketed and internally braced soldier piles-and-lagging or a strutted secant pile wall where groundwater level is encountered above the bottom of excavation or where potential of surface settlement induced by groundwater drawdown may cause damage to nearby existing structures. Ground improvement techniques may be required to achieve secure groundwater cut-off at the interface between overburden-weathered Limestone and Limestone bedrock.

8.1.2 BASIS FOR RECOMMENDATION

Local ordinances placing noise and vibration restrictions near existing buildings surrounding the portal locations would restrict use of viable construction methods. Pre-drilling would provide practical method of addressing potential obstructions while minimizing settlement as well as noise and vibration. The use of pre-drilled elements for SOE system would help optimize embedment depths based on ground loads and variable excavation depth of the portal approaches. The relative shallow depth to bedrock anticipated to be on the order of less than 25-feet at the portal locations would favor use of rock-socketed embedment as-needed based on planned excavation depth. This SOE system has previously been used in the Dallas area.

8.1.3 SOURCES OF UNCERTAINTY

The sources of uncertainty include actual bedrock depths, groundwater levels, presence of obstructions, utilities, building foundations, and additional surcharge loads due to future development along the portal limits.

8.2 Design Recommendation 2 – Use braced rigid SOE systems such as slurry wall or secant pile wall for cut-and-cover station box structures

8.2.1 DESIGN RECOMMENDATION

Use a braced rigid support of excavation system keyed into rock such as slurry wall or secant pile wall for cut-and-cover station box structures.

8.2.2 BASIS FOR DESIGN RECOMMENDATION

The linear geometry and uniform excavation depth, anticipated to be on the order of 70-ft, would favor a continuous high-production technique such as slurry wall or secant pile wall system. Use of a rigid internally braced SOE system designed to support the existing ground and surcharge loads allowing optimization of final lining thickness designed for hydro-static loads would accommodate the limited North Griffin Street right-of-way at Metro Center Station.

8.2.3 SOURCES OF UNCERTAINTY

The sources of uncertainty include actual depth to bedrock, groundwater levels, presence of obstructions, and surcharge due to future development within the planned Metro Center Station limits.

8.3 Design Recommendation 3 – Use appropriate swell pressure for developing lateral pressure in A1 Alluvium and/or Shale

8.3.1 DESIGN RECOMMENDATION

Use swell pressures provide in Table 14 of GDM #3 and swell pressures based on Slake Durability Index values provided in Table 16 of GDM #3 for lateral pressure computed for design of walls in A1 – Alluvium and Shale, respectively.

8.3.2 BASIS FOR DESIGN RECOMMENDATION

Similar projects in the Dallas area have reported swell pressures of up to 10,000 to 12,000 psf (Richards, 1977). These values greatly exceed computed at-rest earth pressures, and therefore must be considered.

8.3.3 SOURCES OF UNCERTAINTY

The sources of uncertainty include the actual presence of cohesive Alluvium and /or Shale within the excavation horizon during construction of the planned structures described herein.

9 CONSTRUCTION CONSIDERATIONS

9.1 Use non-expansive materials for construction of walls in cut-and-cover and U-wall portal structures

Use non-swelling backfill soil or reduce swell loads by placing a compressible material between back of wall and backfill to absorb some of the lateral swelling and associated pressures.

9.2 Protect expansive clay soils and shale from exposure to water during excavation for walls and invert slab prior to placement of final lining

Protect newly excavated surfaces in clay soils and/or shale from exposure to moisture by applying shotcrete and/or placing mud slab.

9.3 Special Monitoring Requirements

Project-specific instrumentation and monitoring plan should include existing structures in proximity to above-mentioned excavation locations which are susceptible to vibration and settlement damage.

10 REFERENCES

10.1 Sources of Data

1. Dallas Area Rapid Transit DART Light Rail Project Design Criteria Manual dated January 31, 2003
2. DART D2 GPC6 FIRST DRAFT Technical Memorandum TM #3 – Preliminary Ground Characterization Rev A dated January 26, 2019
3. New York City Transit Structural Design Guidelines Subway and Underground Structures DG452A Issue No. 2 December 2006
4. Boscardin, M. and Cording, J., Building response to excavation-induced settlement, Journal of Geotechnical Engineering, 115, 1989, p1-21.
5. Gamble, J.C., Durability-plasticity classification of shales and other argillaceous rocks, Ph.D. thesis, 1971. University of Illinois.
6. Richards, B.G. "Pressures on a Retaining Wall by an Expansive Clay," Proceedings 9th International Conference on Soil Mechanics and Foundation Engineering, (1977), Tokyo, Japan.