Cut and Cover Tunnel in LBJ Express Highway in Dallas Texas USA

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Abstract

The LBJ Expressway Highway improved one of the busiest and most congested highways in Dallas. The fact that the majority of the Project must be built within the existing facility and keeping the existing traffic, poses significant challenges from the view point of maintenance of traffic, construction phasing and schedule. All this has required an extraordinary effort of planning and design, incorporating innovative ideas to the design of the cut and cover tunnel, especially for the retaining walls and the covering structure in order to meet the schedule, adapt the design to the temporary traffic plan, reduce traffic detours and improve safety during construction.

The final design of the cut and cover tunnel proved to be successful in complying with the existing constrains of space, maintenance of traffic and programme. This was possible due to the selection for each specific section of the project of the right type of retaining wall and designing a covering structure formed by precast elements that proved to meet programme requirements with a minimal disruption to existing traffic and as well as providing a cost efficient design.

Key words

Cut and cover tunnel, project constrains, traffic management, retaining wall strategy, precast concrete structure

1 Introduction

The IH-635 Managed Lanes Project (LBJ Freeway) rebuilt one of the busiest and most congested highways in North Texas by 2016. This project is one of the largest private-public partnerships undertaken in the United States in terms of complexity and investment value.

The most significant part of the project consists in a full reconstruction of IH-635 general purpose lanes from IH-35E to US-75 and adding six managed lanes in a depressed section for a total length of nine miles. The depressed managed lanes are excavated between retaining walls of approximately half the width of the general purpose lanes on a bridge overhang above the depressed managed lanes supported by straddle bents across the whole width of the six managed lanes.

The fact that the new managed lanes must be built within the existing facility while keeping the existing traffic, with more than 250,000 vehicles driving through the corridor every day, poses significant challenges from the view point of maintenance of traffic, construction phasing and schedule. All this has required an extraordinary effort of planning and design, incorporating innovative ideas to the design of the retaining structures in order to improve schedule, adapt the construction needs to the traffic management plan, reduce traffic detours and improve safety during construction.

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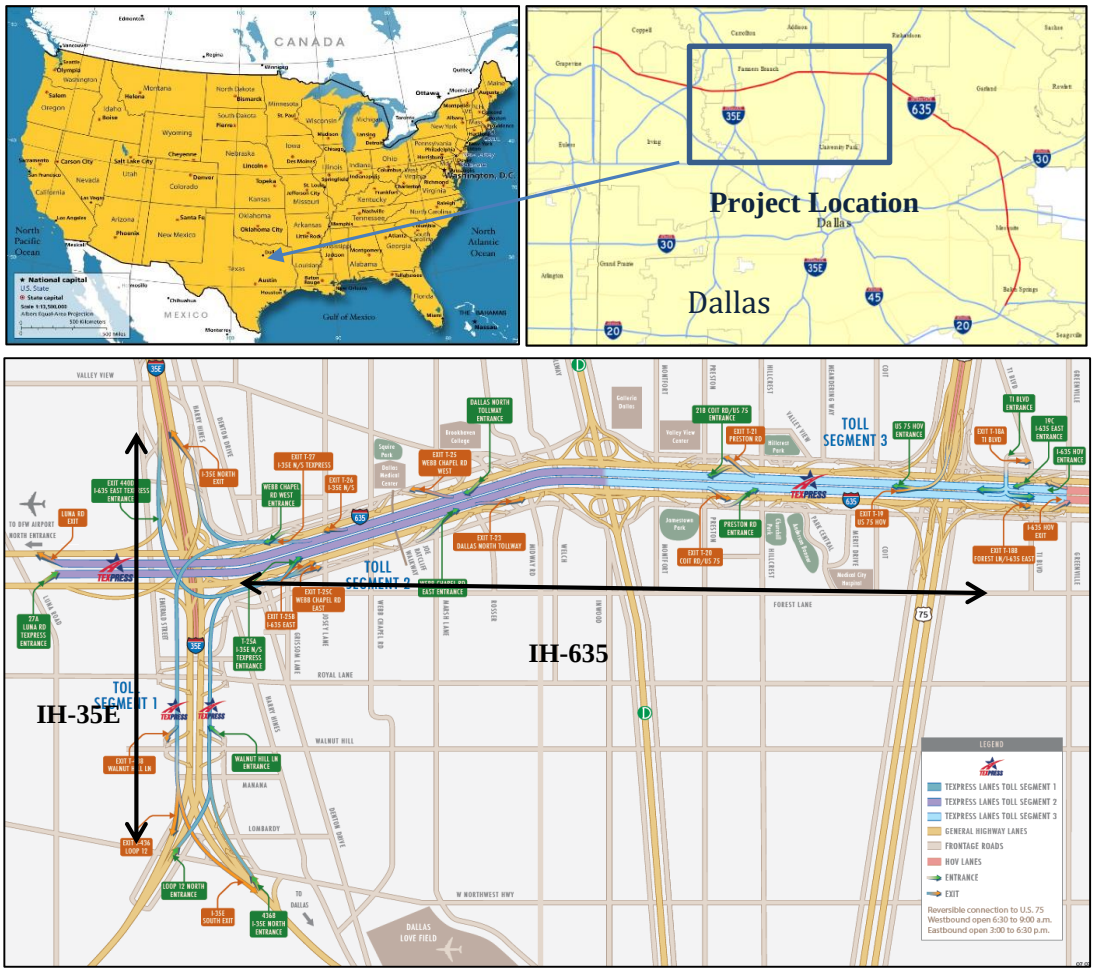


Fig. 1 LBJ Express project location and Plan View

2 Description of the Project

TxDOT awarded this Comprehensive Development Agreement "CDA" project to a Cintra-led Developer consortium, following a competitive process. This innovative 22 -km project addresses increased traffic demands through a multi-level highway system. Construction cost is approximately $2 billion, with $2.2 billion total project investment with a concession period of 52 years.

The project has two distinct sections:

1. Express toll direct connectors along IH 35 E, connecting with IH 635 easterly. This section, on a North to South alignment along IH 35 E consists of an elevated direct connector carrying four new toll lanes (with ultimate expansion to six) on long viaducts to avoid congestion on the existing IH 35E general purpose (non-tolled) lanes (Figure 2).

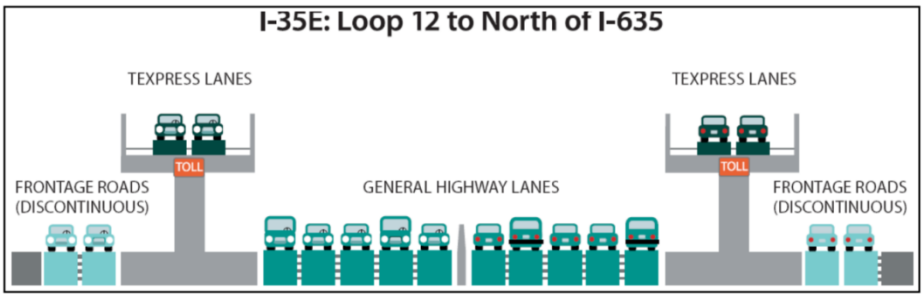


Fig. 2 LBJ. Express toll connectors to/from IH 635

2. Managed lanes along IH 635. Considering the limited availability of Right of Way (ROW) along the corridor, a depressed U-section to accommodate the managed lanes had to be excavated and partially covered by the general purpose lanes, which were previously shifted outwards from their present position to create enough room in the median for excavation.

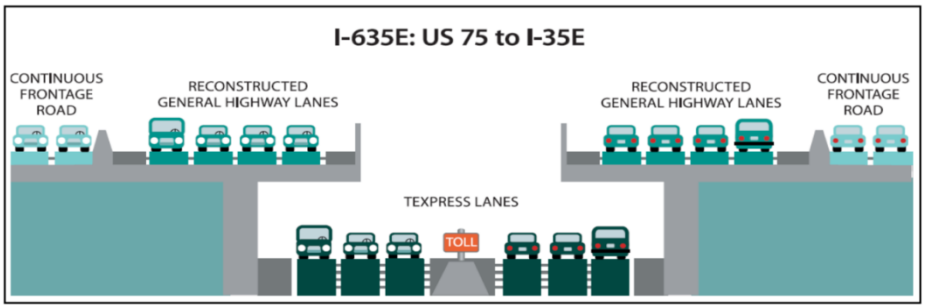


Fig. 3 LBJ Express IH 635 Typical Section

For this paper we are going to discuss the latter section since it's the one that presents the cut a cover part of the project with the majority of the features that require underground retaining structures.

Managed Lanes along IH 635

The basic structural configuration of the depressed Managed Lanes consists of a six-lane facility with narrow median, 1.2 meter inside shoulders and 3 meter outside shoulders. The depressed Managed Lanes are excavated between retaining walls of different types according to the geologic formations encountered along the length of the depressed section. In order to allow for the excavation of the Managed Lanes, the existing General Purpose lanes are displaced outwards and built in the new configuration as a four-lane facility. Approximately half the width of the General Purpose Lanes is on a bridge overhang over the depressed Managed Lanes, supported by straddle bents across the whole width of the Managed Lanes.

The overhang section leaves an opening over the median of the depressed Managed Lanes in both directions with a typical width of 13 meters. As can be seen from this description, structural issues have required applying innovation and the best engineering solutions.

The fact that the new Managed Lanes must be built within the existing facility poses challenges from the view point of maintenance of traffic; this has required an extraordinary effort of planning and design. The structures of the IH635 Section consist primarily of concrete beams with a length of 30-37 meters supported by bents caps spanning the managed lanes at widths of 20 to 30 meters in each direction of traffic.

Several preliminary designs of the straddle bents for the U-section bridges were considered during the first stages of the design. Two primary alternatives were evaluated, the first consisted of precast prestressed bent cap segments, and the second alternative utilized cast-in-place post-tensioned bent caps. Due to the magnitude of loads and the span lengths, a prestressed or post-tensioning solution was recommended.

3 Geology of the Project

Three major geologic formations are present along the IH-635 Managed Lane project. These formations include alluvial and terrace deposits at the west and east ends of the project, the Eagle Ford Formation at the western end of the project, and the Austin Chalk Formation outcropping along the majority of the length of the project. The outcrop limits of these formations and descriptions are listed in Table 1.

Tab. 1 Geologic Formations

Formation	Approximate Limits of Outcrop	Description
Alluvial and Terrace Deposits	West End of Project Sta. 65+00 to 95+00 and East End of Project Sta. 355+00 to 400+00	• Deposits typically consist of layers of fat clays, lean clays, clayey sands and sands. Fill soils are present. • Deposits at the west end of the project are from Elm Fork of the Trinity River and are underlain by the Eagle Formation at depths greater than 17 to 20 meters. • Deposits at the east end of the project are from White Rock Creek and are underlain by the Austin Chalk formation at depths typically less than 8 meters.
Eagle Ford Shale	Sta. 95+00 to 150+50	• Arcadia Park upper member of the Eagle Ford Formation outcrops in the project foot print. • Residual soils consist of fat clays and shaley clays. The thickness of the residual soils is typically less than 8 meters. • Hard concretions and thin limestone flagstone layers are present. Occasional bentonite seams can be present. • Massive clay shale formation with nearly horizontal beds. Upper 1.5 to 5 meters of shale exhibits lower strengths than those below these depths. The shale is a very weak rock. • Faults with slickensided shear planes and near vertical joints can be present. • Overlain by Austin Chalk east of about Sta. 150+50.
Austin Chalk Limestone	Sta.150+00 to 355+00	• Lower and Middle Members of Austin Chalk Formation outcrops in this project. • Residual soils consist of primarily of fat clays with lean clays present at the top of the limestone. • Austin Chalk is not a crystalline limestone. Beds ranging from 18 to 36 inches are nearly horizontal • except at faults. • The Austin Chalk is massive with widely spaced faults and joints. Bentonite layers less than 2 inches thick to about 14 inches thick are present. Thin shale layers are also present. • The weathered section of the Austin Chalk is tan to yellowish brown in colour. The unweathered section is light to dark grey. Numerous horizontal and vertical joints typically form in the upper 1 to 1.5 meters in the more highly weathered section. The jointing becomes less frequent with depth in the less weathered tan limestone section.

Figures 4 and 5 depict a geotechnical profiles of the project along IH 635 from West to East.

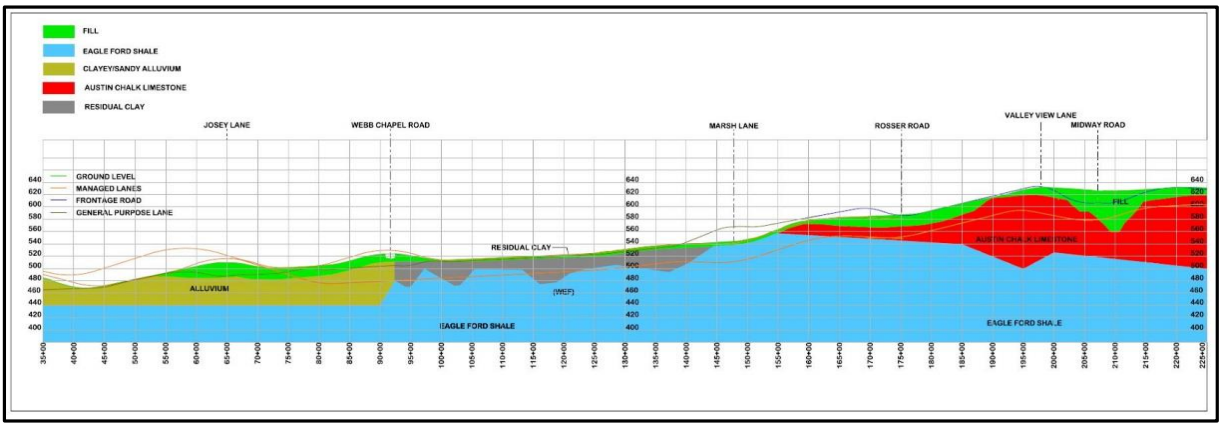


Fig. 4 Geotecnical profile Along IH–635 [1 of 2]

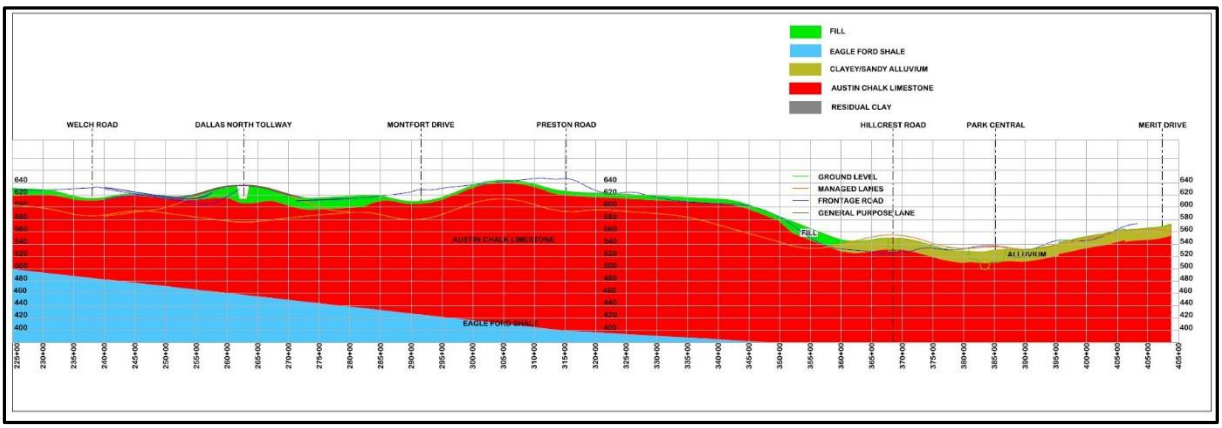


Fig. 5 Geotecnical profile Along IH–635 [2 of 2]

3.1 Alluvial Soils

Two broad classes of alluvial soils exist in the Project area, described here as sandy and clayey alluvial soils. The sandy alluvial soils are attributable either to riverbed deposits or lower terraces of the ancestral Trinity River (Eubank 1965), while the clayey alluvial soils are attributable either to upper terrace deposits of the Trinity River (Eubank 1965), or are soils that are the product of erosion and nearby deposition of residual soils or intermittent streams.

Sandy alluvial deposits exist in the Trinity River floodplain and extend down to the top of bedrock, which may be as much as 20 meters below existing grade in the Project area, and potentially deeper in the actual floodplain. These soils tend to be composed of sands and gravels. Where these granular deposits exist along the Project, generally west of Webb Chapel Road, they tend to be found below a depth of 5 meters, and are typically of limited thickness (e.g., about 1.5 meters), suggesting their origin could be a buried channel or stream meander.

The clayey alluvial soils that are believed to be remnants of terraces from the ancestral Trinity River (Eubank 1965) are generally found from the ground surface down to a depth of approximately 4.5 meters, and are mainly high plasticity clays. Such clayey alluvial soils are encountered in borings at the west end of the Project along the gentle slope rising out of the Trinity River bottom up to the ridge formed in the more resistant Austin Group, and on the east end of the Project on the bluff dropping down into the White Rock Creek drainage.

The other class of clayey alluvial soils, described above as colluvial soils, which are higher up on the hillsides along the alignment, originated primarily as residual clays and are likely to exist on the eastern and western slopes.

3.2 Eagle Ford Shale

Portions of the cut-and-cover tunnel and U-wall sections will be excavated within the Eagle Ford Shale. The Eagle Ford Shale is classified as a very weak rock (ISRM 1981) and upon wetting is known to swell, slake and lose strength rapidly. In terms of discontinuities, the Eagle Ford Shale is considered massive, moderately jointed rock (Proctor and White 1977), and except in isolated cases, the joints are generally widely spaced and tight. Of the joints and fractures in the Eagle Ford Shale that were identified in the test borings approximately half were classified as slickensided.

Core recovery in the Eagle Ford Shale during the Phase 1 and Phase 2 geotechnical investigations was generally excellent, ranging from 0 percent to 100 percent. The average core recovery was equal to 93 percent. Approximately 82 percent of the individual core runs are reported to have core recovery of 90 percent or more. Rock Quality Designation (RQD) values ranged from 0 to 100 percent and averaged 90 percent. Approximately 71 percent of the individual core runs are reported to have an RQD of 90 or more. According to rock classification methods using these parameters, the Eagle Ford Shale would be classified as "good to excellent".

The results of laboratory tests performed on samples collected during the geotechnical investigations indicate that the weathered Eagle Ford Shale that was tested has an average dry unit weight of 1,680 kg/m^3, and average moisture content of 23 percent. The liquid limits of the weathered shale ranged from 45 to 96, and the plasticity indices ranged from 26 to 63.

For the fresh Eagle Ford Shale samples tested for this Project, an average dry unit weight of 1,870 kg/m^3, and an average moisture content of 17 percent were determined. Four tests were run to determine liquid limit and plasticity index in the fresh Eagle Ford Shale.

In these tests, the liquid limit of the shale ranged from 58 to 59, and the plasticity index ranged from 34 to 36. These values appear to be on the low end of reported literature values which suggest that the liquid limit for the Eagle Ford Shale can typically range from 60 to 80 and the plasticity index can typically range from 34 to 48 (Font 1979).

Unconfined compressive strength tests performed on the fresh Eagle Ford Shale indicated strengths ranging from 200 to 4,800 kPa, with an average of 1,300 kPa. Young's modulus and Poisson's ratio was determined in three of the unconfined compressive strength tests and average values of 275 MPa and 0.25 were determined, respectively.

Based on the roadway geometry presented in the Reference Schematic, essentially all excavation for the cut-and-cover tunnel and U-wall sections located in the fresh Eagle Ford Shale will extend at least 3 meters into the shale below the Austin Chalk/Eagle Ford Shale contact. As presented earlier, the Eagle Ford Shale underlies the Austin Chalk along an unconformable contact, with erosional periods occurring in the Eagle Ford Shale prior to deposition of the Austin Chalk.

3.3 Austin Chalk

The Austin Chalk is a massive rock formation with limited structural features such as fault surfaces and bentonite layers. Shears in the limestone randomly occur. The bentonite layers tend to be more frequent east of Midway Road. The cut section stability between about Sta. 218+00 and 295+00 and between about Sta. 340+00 and 353+00 will be impacted by bentonite layers.

Photographs of fault shears in the Austin Chalk and weathering results in the less resistance chalk layers receding back from more resistance layers are shown on Figure 6. The chalk weathering results in closely spaced horizontal and vertical joints at the top of the rock and more widely spaced vertical joints with depth.

Fig. 6 Examples of Austin Chalk Limestone Weathering

The shear planes affecting cuts in the north facing vertical cut and affecting cuts in the south facing vertical slopes are presented on Figures 7 an 8. A friction circle for a 40 degree friction angle is included on the exhibits. A friction circle for = arcTan(Tan 40/1.5) = 29 degrees is also shown.

Wedge planes intersecting outside of the friction circle for 29 degrees are expected to have a factor of safety of 1.5 or greater with respect to a wedge failure for dry conditions. Wedge planes that intersect within the 40 degree friction circle are expected to fail without the addition of rock nails. The available data indicate that wedge failures are possible and that strengthening sections of the rock cuts where wedges are present will be needed.

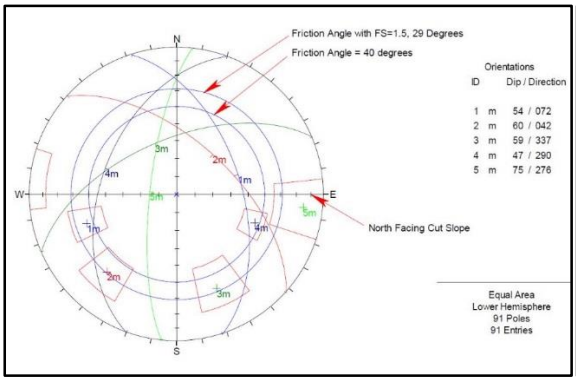


Fig. 7 Great Circle Plots North facing

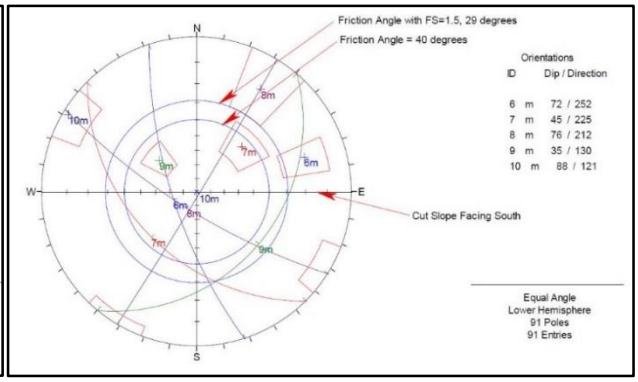


Fig. 8 Great Circle Plots South facing

4 Retaining wall types used in the Cut and Cover Tunnel

4.1 Retaining wall strategy

One of the first tasks of the design of this project was to establish the different types of retaining walls along the cut and cover section. Two major challenges were needed to take into account for this task:

- Space constrains due to the ROW available.
- The necessity of maintaining four lanes of traffic along IH635 in each direction.

These two project constrains had to be checked against the different types of soils as described previously in the paper.

The different types of retaining walls used for the cut and cover section of the project were:

- MSE Walls: where there was enough space and no traffic constrains this was the type of wall to be used. In some occasions where traffic needed to be maintained and the back sloping of the structural fill was not possible due to space restrictions, a temporary soil nail wall was placed at the back end of the reinforced block.
- Drilled Shaft Wall: where the height of the wall started to be over 3-4 metres, it was impractical to build MSE walls. The solution finally chosen was drilled shaft walls evenly spaced in cantilever or with several rows of tie backs (up to 6 rows depending on the final height). This was the solution implemented in the areas with Eagle Ford Shale.
- Hybrid Walls, MSE Wall over Drilled Shaft Wall/Rock Nail Wall: in areas of Eagle Ford shale, where space was available, a solution of MSE wall on top of drilled shaft wall was chosen as the most efficient. In areas where Austin Chalk was present the solution was the use of Rock Nail walls instead of the Drilled Shaft Wall.
- Rock Nail Wall: in areas of Austin Chalk where a wall was needed this was the solution estimated as the most efficient one.

4.2 MSE Walls

Mechanically Stabilized Earth (MSE) walls will be used where there is sufficient space to allow construction. Where there is insufficient space to construct full-height MSE walls, hybrid walls consisting of drilled shaft lower wall section supporting an MSE-wall are planned.

MSE Wall design parameters

Tab. 2 MSE Wall Design Parameters

Material Type		Unit Weight [kg/m^3]	Effective Stress Strength	
			Cohesion [kPa]	Friction Angle ϕ [o]
Reinforced Zone Select Fill: TxDOT Item 423, Type B		1,680 – 2,080	0	34
Retained Active Wedge Zone: TxDOT Item 423, Type B		1,680 – 2,080	0	34
Retained Active Wedge On-Site Clay		2,000	0	22
Wall Bearing Resistance	**On site Clay**	2,000	0	22
Wall Sliding Resistance	**On site Clay**	2,000	0	34
	Select fill	2,000	0	30

The nominal factored bearing resistance in clay was determined to be 240 kPa. MSE walls present in some cases temporary shoring for the placement of structural fill as can be seen in Fig. 9.

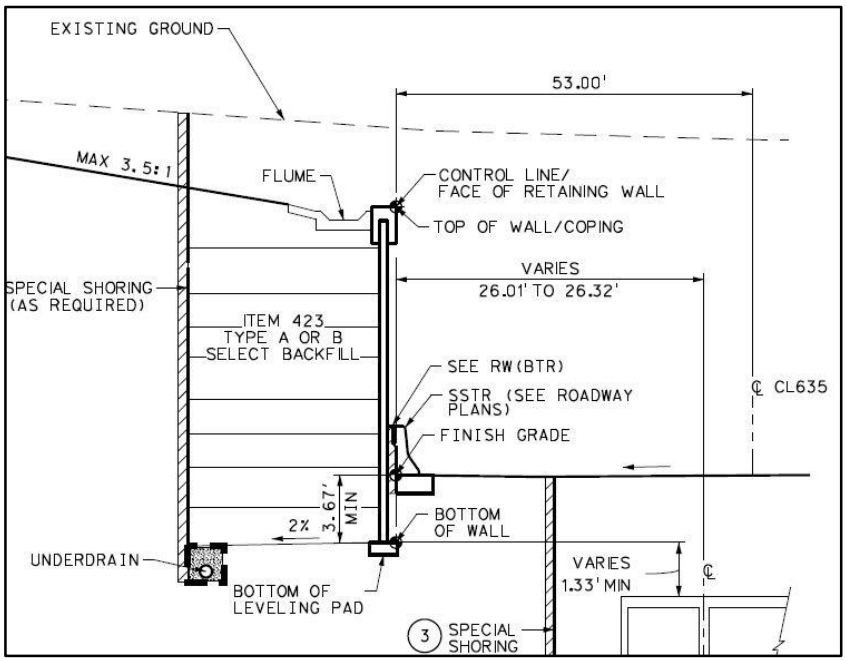


Fig. 9 MSE Wall with temporary shoring (soil nail)

4.2 Hybrid Walls: MSE Walls over Drilled Shaft Wall

A hybrid wall consisting of a lower drilled shaft section with an upper MSE wall section is planned in areas with space constraints.

The vertical surcharge load and lateral loads from the MSE wall must be included in the drilledshaft wall design. The horizontal forces from earth pressures and surcharge loads acting on the MSE wall are to be applied to the top of the drilled shaft cap beam as a horizontal line force. There will be a shear wall at the top of the drilled shaft to resist the lateral forces from the MSE wall. The wall panels must bear directly against the shear wall. If needed concrete must be placed between the panel and the shear wall to ensure a tight contact.

Minimum MSE wall anchor length was estimated to be 0.7 H, where H is the height of the MSE wall where lateral loads resisted by drilled shaft wall.

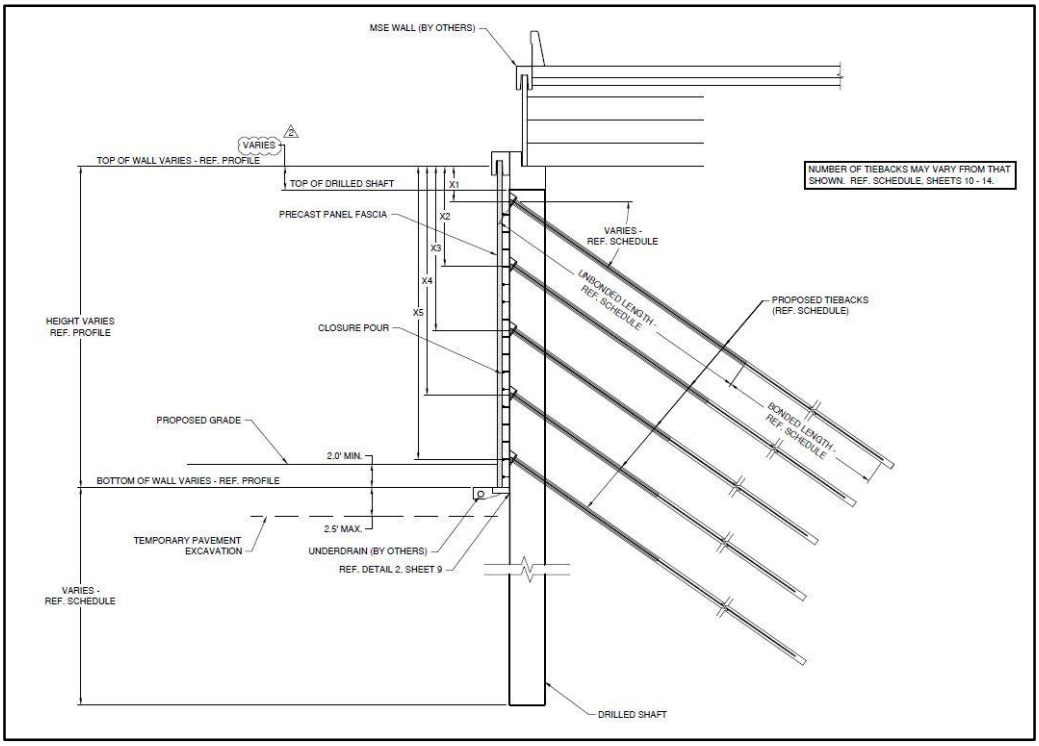


Fig. 10 Hybrid Wall: Drilled Shaft with Tie Backs + MSE on the top part

Design parameters for Axial capacity of Drilled Shaft Walls were as per Table 3. Other design parameters considered for the shale are:

- Unit Weight: 2,000 kg/m^3
- At-Rest Earth Pressure Coefficient, ko 1 – 1.5
- Cohesion, c 14 kPa
- Effective Stress Friction Angle, degrees 30
- Soil Modulus, k 34,000 kN/m^3
- Nominal Unfactored Bond Stress 190 kPa

Tab. 3 Drilled Shaft Walls Design Parameters. Axial capacity

Design Notes	Parameter]
Use of End Bearing and Skin Friction Values	Drilled shafts can be designed for a combination of tip resistance and side resistance for penetration depths into design stratum of 2 shaft diameters or greater.
Dry Construction	The drilled shaft design end bearing and skin friction values are based on dry construction methods in the shale.
Estimated Settlement	Less than 0.5 % of shaft diameter for shaft tip bearing in gray limestone
Minimum Center to Center Drilled Shaft Spacing	3 times the diameter of the larger shaft. Closer spacing will result in reduction in the skin friction values and possibly requires special installation sequences. As a general guide, the design skin friction will vary linearly from the full value at a spacing of 3 diameters to 50 percent of the design value at 1 times the diameter. This minimum drilled shaft spacing requirement for axial resistance should be considered at the same time as the minimum spacing

4.3 Drilled Shaft Walls

Drilled-shaft walls are expected to be constructed where space limitations prevent the use of full-height MSE walls. Drilled shafts were spaced normally to 30 cm between faces of the shafts.

Wall drainage was required between the drilled shafts. Drainage panels placed between the drilled shafts can be used to intercept groundwater. The drainage panels were connected to a drain at the base of walls.

Fig. 11 Drilled Shaft with Tie Backs

4.4 Rock Nail Walls

All sections of the cut and cover that required a retaining wall in the Austin Chalk formation were design with rock nails. In the areas where there was a weaker upper part and there were no space constrains, MSE wall replaces soil overlying rock, creating a hybrid MSE/Rock Nail type of wall where.

Fig. 12 Rock nail wall in the Austin chalk

The following table summarizes the minimum required block width to height ratio to satisfy block sliding and overturning criteria. The analyses included the factors of safety for sliding and overturning of 1.5. The conditions considered included sliding for assumed vertical crack filled with water and sliding on bentonite seam, overturning for water pressures acting at base of block and vertical crack, and for sliding for water pressures acting on the base and vertical crack.

Tab. 4 Rock Nail Wall Block width

Failure Criteria	**Block Width over Height Ratio, L/H**				
	Bentonite Layer	**Bentonite Layer**	**Class 1 Tan Limestone**	**Class 2 Tan Limestone**	**Gray Tan Limestone**
Sliding due to vertical crack 100 % water filled	0.66				
Sliding due to vertical and horizontal crack 100 % water filled		0.84	0.75	0.61	0.51
Overturning moment due to vertical and horizontal crack 100 % water filled	0.61	0.61	0.61	0.61	0.61

5 Structural design for the cut and cover tunnel

The cut a cover section presented an area with repetitive structural elements, which are most suitable for using precast elements as a viable option.

The structures of the IH635 Section consist primarily of concrete beams with a length of 30 - 37 meters supported by bents caps spanning the managed lanes at widths of 20 to 30 meters in each direction of traffic.

The design and dimensions of the bent caps was a challenge since they had to be adequate for different construction phases, different span lengths, and at the same time their details and dimensions had to be similar for all of them in order to prefabricate them quickly, meeting the tight deadlines.

Several preliminary designs of the straddle bents for the U-section bridges were considered during the first stages of the design. Two primary alternatives were evaluated, the first consisted of precast prestressed bent cap segments, and the second alternative utilized cast-in-place post-tensioned bent caps. Due to the magnitude of loads and the span lengths, a prestressed or post-tensioning solution was recommended.

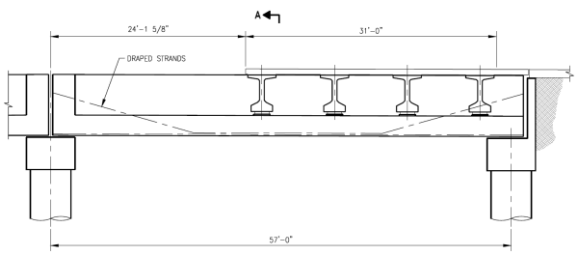


Fig. 13 Alternative 1 – Precast/prestressed

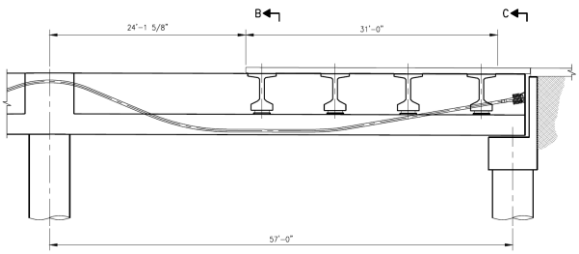


Fig. 14 Alternative 2 – Cast-in-place post-tensioned

Alternative 1 was chosen for simply supported bent caps that could be precast in a plant, and installed in one night. Caps rest on two elastomeric bearings in each side, and column reinforcement does not have to be extended into the cap as it was typically done in the state of Texas.

An inverted-T section was chosen in order to reduce the vertical clearance, the shape of this section allowed a raise in the profile of the managed lanes, which reduces the height of the retaining walls, the excavation, and results in a more cost efficient design of the overall U-section.

There were several different type of bent caps depending on the span lengths and the construction phasing:

a) <u>Precast design</u>

1. Reinforced concrete
2. Pre-stressed concrete
3. Precast concrete pieces with post-tension

b) <u>Cast in place design</u>

5.1 Precast design

The standard type of cap used in Texas is a fixed cast-in-place cap. The time needed for the construction of a cast-in-place takes approximately two to three weeks per set of form, while in one night, five or six precast caps can be placed when the precast option is used.

There were three main designs depending on the geometrical and phasing constraints of the Project:

1. <u>Precast with only reinforced steel:</u>

These bent caps were mostly used when the span lengths were between 6 and 9 meters, although some caps had spans around 14 meters where the reinforced steel was designed to handle the load, especially when the caps were supporting only one or two beams.

2. <u>Precast cap with pre-stressed tendons:</u>

The typical U-Section spans are around 18 meters, with variations from one cap to other. The precast plant was able to cast all the bent caps with only adjustments between one cap and other. With the precast caps, the construction team was able to place five or six caps per night.

Figure 15 shows a typical case which includes one cap with only reinforced concrete, and another cap with longer span where additional prestressed strands were required.

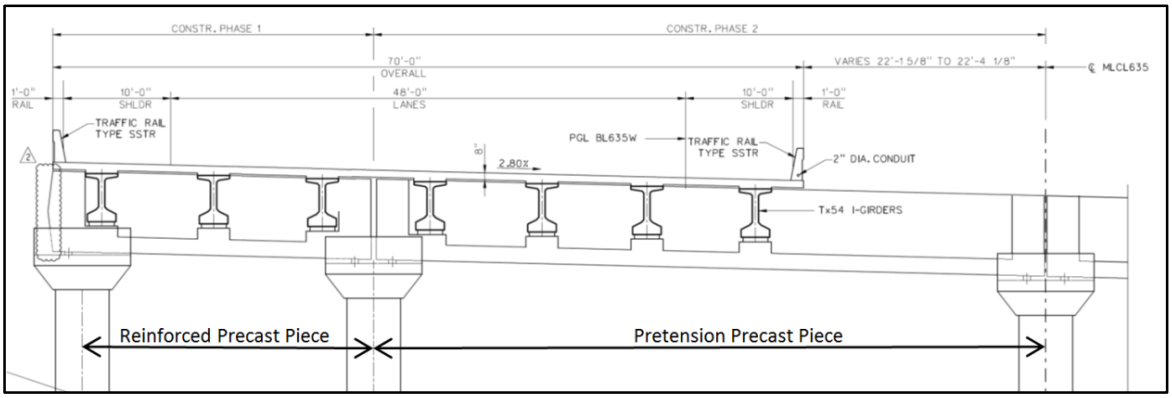
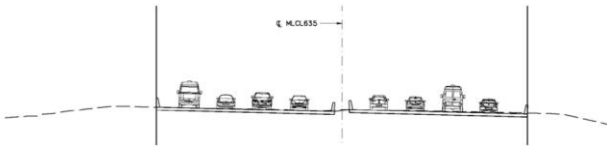


Fig. 15 Precast caps with only reinforced vs precast and pre-stressed

3. <u>Precast concrete bent caps with post-tension</u>

In order to maintain the amount of lanes to accommodate the heavy traffic, the bent caps were required to be constructed in several phases. See below for a description of the phases to explain the need of posttensioning between the different caps.



The original configuration had 4 lanes per direction in the IH-635.

Figure a: Original situation, 4 lanes per direction on IH-635

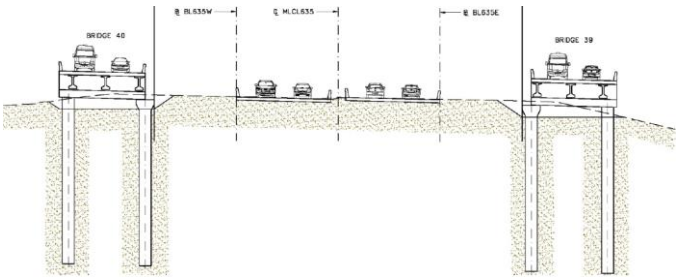


Figure b: Construction of exterior bridges

Phase 1:

Temporary walls were constructed to maintain the traffic in the inner section while exterior bridges were built. For this phase precast prestressed bent caps were sufficient to resist the traffic loads.

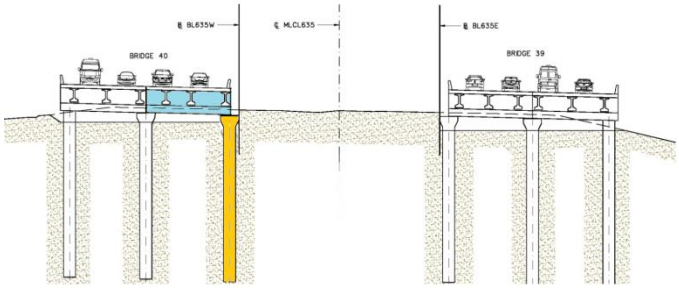


Figure c: Second stage construction of exterior bridges

Phase 2:

At this stage, provisional drilled shafts that were acting as a temporary columns had to be built since a permanent column would be in conflict with the future traffic of the managed lanes as shown in Figure c. Prestressed bent caps with post-tensioning ducts were installed and ready to be connected to the adjacent cap in a later phase.

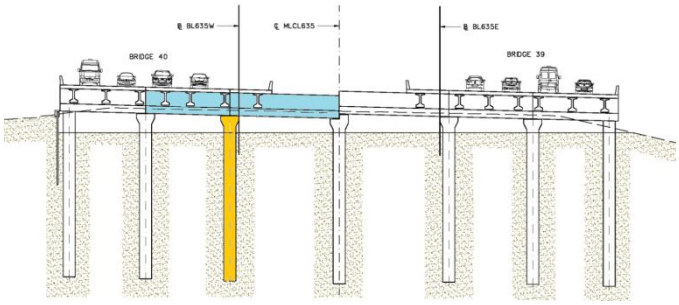


Figure d: Bridge Completion

Phase 3

Another cap spanning in most of the cases more than 20 meters was placed between the existing bent caps and the center pier of the managed lanes. Post-tensioning of bars and tendons was required to connect the bent cap with to the previous cap. Details are explained later in the article.

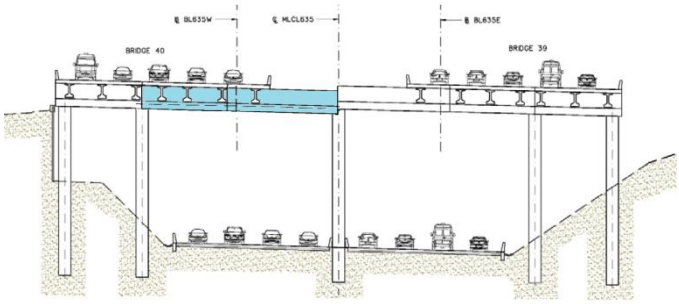


Figure e: Excavation and Managed lane construction

Phase 4

Once both bents were connected and post-tensioned, the provisional drilled shaft was removed, the beams could be placed, and the rest of the deck poured.

The final step consisted in the excavation and the construction of the managed lanes. Face of drilled shafts were cleaned and aesthetics were included in the columns.

Post-tensioning caps were carefully designed due to the high amount of reinforcement needed in order to allow the concrete to be poured easily around the reinforcement. Three-

dimensional drawings with real dimensions were needed to make sure that the upper tendons, lower tendons and reinforcement were not interfering.

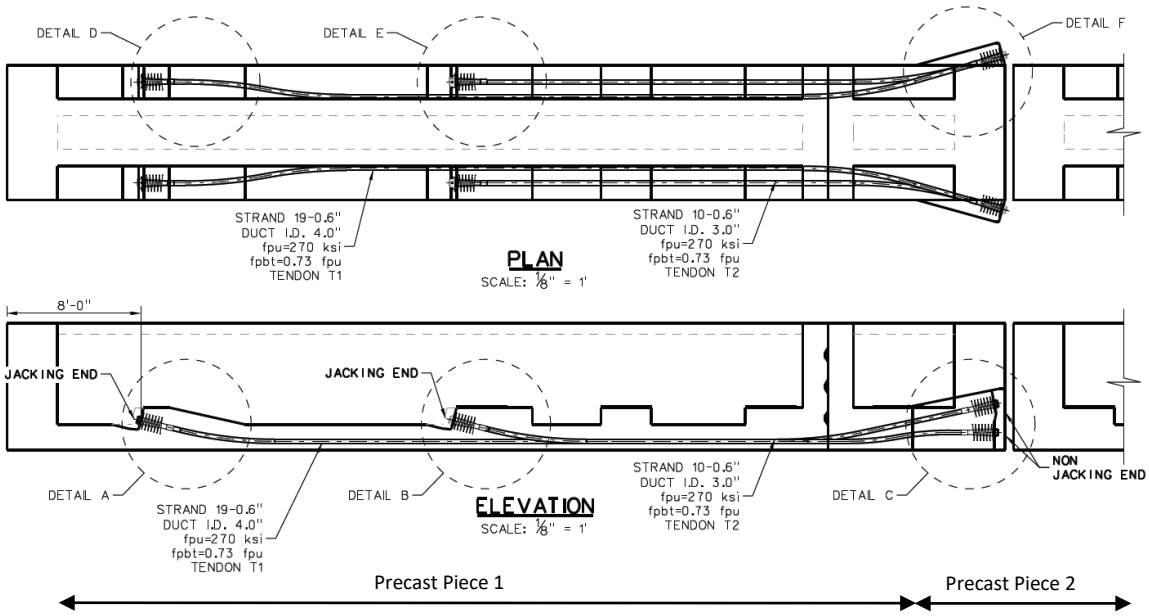


Fig. 16 Bent general details with post-tensioning tendons.

Fig. 17 (a) Post-tensioning bent cap; (b) Precast Bent caps.

5.2 Cast in place design

Cast in place design was considered in 27 bents out of a total of 217, with a total of 86 cast-in-place portions of bent caps versus 491 of precast bent caps. The span length of the cast-in place caps vary from 15 to 20 meters. No prestressing was needed since the cap was designed continuous along the section. In order to achieve this continuity, mechanical couplers were installed at the construction joints between caps so the reinforcement was continuous. For very few bent caps (less than 20) with longer spans, up to 30 meters, posttensioning was needed due to this increase of length.

To describe the importance of this innovation, Figure zz shows one of the few original fixed cast-in-place caps constructed. The use of cast-in-place caps as shown in Figure zz would have not only increased the time needed to finish the project, but also would have increased the potential risks for accidents due to employees working at great heights.

Fig. 18 (a) Reinforcement placement in cast-in-place cap; (b) Reinforcing cage; (c) Concrete pouring in cast-in-place cap; (d) Cast-in-place cap finished.

6 Conclusions

When facing a tight schedule with complex traffic phasing and space restrictions innovative ideas are needed to succeed under these challenges.

These innovative ideas have to be adjusted to each specific problem encountered in the project. The two main areas affected by the project constrains of space, programme and traffic maintenance were retaining walls and structures in the cut and cover tunnel of the depressed sections.

The final design of the cut and cover tunnel proved to be successful in part due to the selection for each specific section of the project of the right type of retaining wall, and designing a covering structure formed by precast elements that proved to meet programme requirements with a minimal disruption to existing traffic.

For the retaining wall, a clear strategy for their design was early adopted in the design process which enabled the selection of the most efficient design that could meet the space and traffic maintenance challenges. This strategy was adapted to the different types of soils that were encountered along the cut a cover section.

It has been shown how well precast elements have worked when similar designs allowed to use a standard model. These precast elements had to be adjusted to the construction phasing and post-tension became the solution to solve some of the most difficult phases of the construction process. In addition, in order to solve a different type of problem, a composite section helped to minimize traffic disruption to a very heavily trafficked highway.

Precasting has been one of the keys to the success of this projects and ought to have a special consideration for this section. Some of the main advantages of using this method are listed below:

- Schedule improvement: in addition to the time savings when precasting in a yard on site versus cast in place, construction of the caps could begin prior to excavation which allows several other activities to be overlapped.
- Ability to adapt the structures to the traffic control plan: some bridges were built in different phases. In order to connect the bridge, as a temporary solution, the caps were placed on temporary columns with temporary seats and then it was assembled with the rest of the caps with post-tensioning.
- Construction safety is improved: precast caps are built in a controlled facility. Conditions on site such as heights, heavy machinery, traffic, etc., impose more hazards and risks to the workers.
- Improved Quality: At the facility, the workers have a routine, they have specific tasks and it is also easier for quality inspectors to perform their jobs. Additionally the forms in the plant were provided with external vibration which meant a better finish product.
- Easier to transport and place: precast caps were less heavy than the cast in place option and could be easily transported and place in their final position.

7 Acknowledgements

The authors wish to express their respect and admiration for TxDOT and their leadership in the project described above. They also want to acknowledge the General Engineering Consultant working for TxDOT Jacobs and the Independent Engineer URS for their participation in this project.

This process of incorporating innovative design ideas for their inclusion in the final product was possible with the collaboration between all parties which lead to a successful delivery of the project ahead of schedule. The technical descriptions that appear in this paper are from the authors' viewpoint as a committed but private proposer and are not intended to represent TxDOT's position.