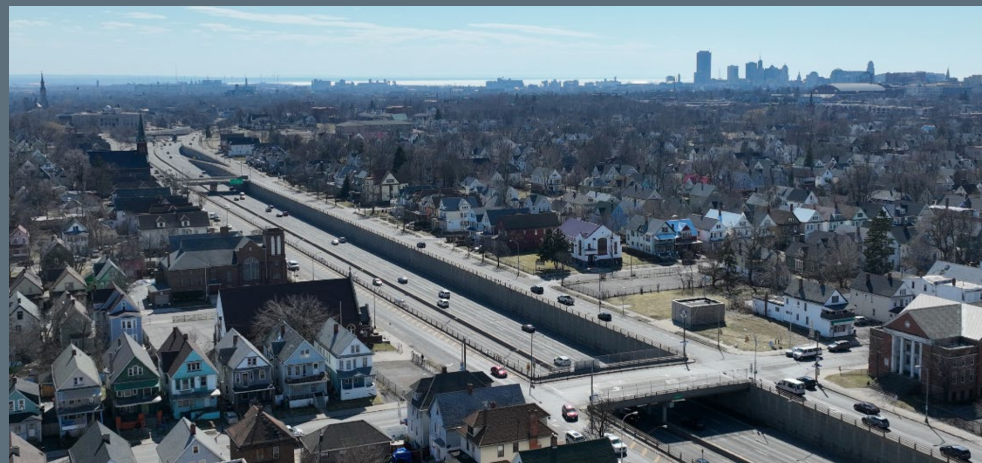
Appendix A12

Technical Memorandum:

Tunnel Structure Type



TECHNICAL MEMORANDUM:

TUNNEL STRUCTURE TYPE

KENSINGTON EXPRESSWAY PROJECT, PIN 5512.52

FINAL

September 10, 2023

PREPARED FOR

NEW YORK STATE OF OPPORTUNITY. Department of Transportation

HNTB

LaBella

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KENSINGTON EXPRESSWAY PROJECT, PIN 5512.52
TECHNICAL MEMORANDUM: TUNNEL STRUCTURE TYPE

1. Introduction

1.1. Project Location and Description

The Kensington Expressway Project is seeking to reconnect communities surrounding a stretch of the currently depressed NYS Rte. 33, Kensington Expressway corridor, Figure 1. The project includes the reconstruction of the Kensington Expressway with a tunnel extending approximately 4,150 feet, with southern portal at Dodge Street and northern portal at Sidney Street, Figure 2.

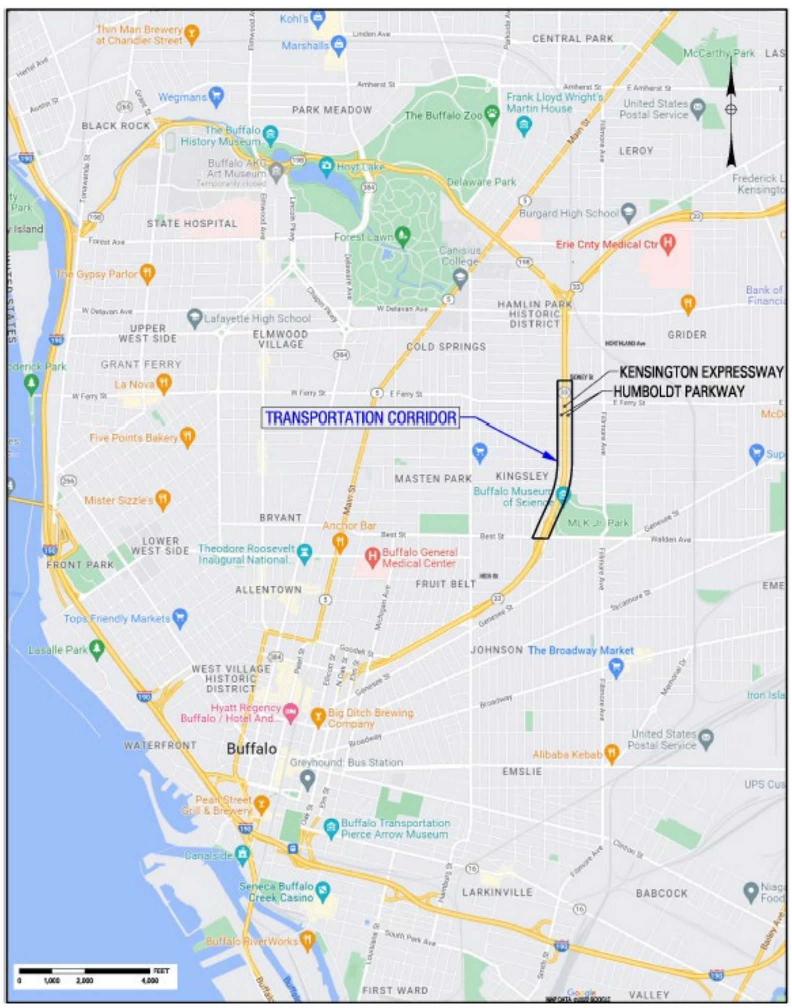


Figure 1 Project Location Map

Figure 2 Proposed Plan of Tunnel – Project Limits

1.2. Objectives

This Technical Memorandum serves to compare potential structural solutions for the tunnel construction and provide recommendations for a structural solution that is feasible with respect to constructability, costs, and impacts, including to the traveling public (both on local city streets and the Kensington Expressway), to adjacent City of Buffalo residents, and right-of-way (ROW).

1.3. Background

LaBella and HNTB collaboratively worked towards a tunnel structure and staging scheme that:

- Minimize impacts to the daily traffic on the Expressway.
  - Maximize the duration of Work Zone Traffic Control (WZTC) measures (stages) where 3 lanes would be provided on both the eastbound (EB) and westbound (WB) lanes.
  - Maintain at least 2 lanes of traffic in each direction through all primary stages.
- Minimize impacts along the Humboldt Parkway (traffic, parking, pedestrians and bicycles, as well as private property access).
  - Attempt to limit the operations that hinder these functions to short term (daily) closures.
- Minimize the quantity, depths, costs, and impacts of the usage of temporary support of excavation (SOE) walls.

- Create staging and work areas, where possible, that we think would be beneficial for efficient construction.
- Minimize the need for ROW to mitigate the above. Knowing that the private properties along the corridor have minimal front lawn and parking areas, we knew it would be important to try to establish a design that precluded the need for temporary street widening to accommodate the primary functions along the Humboldt Parkway.

1.4. Considerations

In order to accomplish the above, we explored optional (and optimal) locations of the permanent walls, taking into account the location of utilities, existing retaining walls, including footings and battered piles (where present), as well as any required SOE walls to install the permanent infrastructure.

Additionally, the conditions along the project length varied along the corridor with some of the major mitigated features being:

- Existing 84" BSA (Buffalo Sewer Authority) sewer along Humboldt northbound (NB) from East Ferry to the north.
- The location of the existing retaining walls widen near the on and off ramps to East Utica Street in the central portion of the corridor.
- Retaining wall footing types vary through the corridor as the rock depth varies.
- The Best Street on ramps within the tunnel at the south end require that the outside tunnel walls be pushed further out.

With these changing conditions, it was important to consider transitioning the design schemes through the different areas of the corridor. Some of the proposed highway geometric design changes to accommodate the optimal location of the permanent walls (and SOE walls) were:

- Shifting the Kensington Expressway mainline (tunnel) to the east at the northern portion of the project provided the best solution for the north end of the project.
- Removal of the horizontal compound curve in the middle of the project, replacing it with a single curve that met (tangent to) the offset alignment at the north end of the project, and with the existing alignment at the south end.

These changes allow us to minimize the impacts of the tunnel construction during all major phases of the project. Additionally, we are able to meet our traffic goals on the Expressway as defined above.

2. Existing Conditions

2.1. Geotechnical Subsurface Conditions

Existing subsurface information is available from Record Plan information from (1) NYSDPW FACC 59-19: Section 1, Contract 2 (Hickory Street to East Utica) from 1955 to 1959; (2) NYSDOT C 68-2: Kensington Expressway Arterial Highway – Section II (Northampton Street to Northland Avenue) from 1965 and 1966; (3) several supplemental soil borings that were obtained in 1999, 2007 and 2017. A supplemental boring program is being performed as part of this Contract, and current preliminary engineering assumptions will be revised based on additional findings and development of the Foundation Design Report, as necessary.

The record soil boring locations (with estimated rock elevations and year of the boring), as well as estimated rock elevations beneath the expressway retaining walls and bridge substructures have been plotted on plan sheets of the Kensington Expressway, extending from the vicinity of the Best Street Bridge (BIN 1022609) north beyond the East Ferry Street Bridge (BIN 1022640) to establish an anticipated top of rock profile and anticipated rock removal limits. See Appendix A for plans and profiles of assumed soil excavation limits, limits of anticipated rock removal by mechanical methods, and anticipated limits of rock removal requiring blasting.

Most of the currently available record soil borings are located within the "retaining wall" section, with few soil borings located behind the existing retaining walls within the Humboldt Parkway. Of the 59 soil boring records, it was found that 45 soil borings had rock core information included. Most of the rock cores were typically limited to a depth of 5 ft., and typically had rock sample recoveries of approximately 90%. Findings noted that the rock was hard, seamed and broken. Existing subsurface information appears to show existing bedrock elevation that are relatively consistent and shallow.

Existing borings show soils consisting primarily of silt or sandy silt, some clay, and occasional occurrences of sand, gravel or fractured stone, including limestone fragments.

Existing borings show variable ground water depth, with the invert of the proposed tunnel anticipated to be below the water table for the full length of the tunnel. The design ground water table elevation ranges from elevation 617 to 641 along the length of the tunnel, while the tunnel invert elevation ranges from elevation 606 to 626.

The following design parameters were assumed for high rock conditions (Sta. 115+00):

Adopted Design Parameters for STA 115+00

Depth From (ft)	Depth To (ft)	Thickness (ft)	Strength	SPT-N	Su (ksf)	Young's Modulus (ksf)
0	5	5	Very Stiff	26	2.2	340
5	14	9	stiff	11	1.5	485
14	100	86	Rock			

Soil Properties from SPT-N Value and Unified Classification System

Boring Log	DH-W9	-
Surf EL	635.44	ft
Station	115+00	-

Depth (ft)	EL (ft)	SPT-N	USCS	Unit Weight (pcf)	S_u (tsf)	Consistency	Friction Angle (deg)	Relative Density	Young's Modulus (ksf)
2	633.44	26	ML	121	2.2	Very Stiff			342
6	629.44	26	ML	121	2.2	Very Stiff			342
11	624.44	11	ML	127	1.5	Stiff			487
14.3	621.14								

Depth (ft)	EL (ft)	Rock Name	Discontinuity	REC (%)	RQD (%)	Weather	Strength	Qu (ksf)	Young's Modulus (ksf)
14.33	621.11	ROCR	Seamed and Broken	95.8					
15.33	620.11	ROCR	Seamed and Broken	66.7					
41.66	593.78	ROCR	Seamed and Broken	94.4					

The following design parameters were assumed for low rock conditions (Sta. 86+00):

Adopted Design Parameters for STA 86+00

Depth From (ft)	Depth To (ft)	Thickness (ft)	Strength	SPT-N	Su (ksf)	Young's Modulus (ksf)
0	12	12	Very Stiff	22	5	350
12	39	27	stiff	13	1.3	440
39	100	61	Rock			

Soil Properties from SPT-N Value and Unified Classification System

Boring Log	DH-E3	-
Surf EL	654.36	ft
Station	86+00	-

Soil Properties

Depth (ft)	EL (ft)	SPT-N	USCS	Unit Weight (pcf)	S_u (tsf)	Consistency	Friction Angle (deg)	Relative Density	Young's Modulus (ksf)
2	652.36	20	ML	122	2.7	Very Stiff			357
6	648.36	19	ML	122	3	Very Stiff			361
11	643.36	26	ML	121	2.2	Very Stiff			342
16	638.36	12	ML	125	1.3	Stiff			444
21	633.36	11	ML	127	1.5	Stiff			487
26	628.36	12	ML	125	1.3	Stiff			444
31	623.36	12	ML	125	1.3	Stiff			444
36	618.36	15	ML	123	1.2	Stiff			388

Depth (ft)	EL (ft)	Rock Name	Discontinuity	REC (%)	RQD (%)	Weather	Strength	Qu (ksf)	Young's Modulus (ksf)
38.6	615.76	ROCR	Seamed and Broken	95.8					
40.16	614.2	ROCR	Seamed and Broken	66.7					
41.66	612.7	ROCR	Seamed and Broken	94.4					

2.2. Existing Retaining Walls

With the variability of top of rock elevations along the Kensington Expressway within the project limits, the existing retaining walls along the depressed highway are founded either on rock or on piles. Below is a general description of the existing retaining walls based on foundation type. All existing retaining walls have horizontal rustications and either a steel railing or a concrete ornamental railing.

2.2.1. Existing Retaining Walls on Rock

There are two types of details for existing retaining walls founded on rock: cantilevered retaining walls on spread footing on rock and buttressed retaining wall on spread footing keyed into rock, with keys aligned with buttresses.

Cantilevered retaining walls on spread footing on rock exist at the following locations within the proposed tunnel limits:

- South of and adjacent to the Dodge Street Bridge west abutment and along the west side of the Best Street ramp for three of the seven retaining wall panels (Wall No. 8 from Contract 59-19)
- Between Riley and Sidney Streets, along the west ramp, all panels of Wall No. 2 from Contract 68-02.
- Between Girard Place and Sydney Street, on the east side, all panels of Wall No. 1 from Contract 68-02.

Buttressed retaining walls on spread footing keyed into rock, with keys aligned with buttresses, exist at the following locations:

- South of the Dodge Street Bridge and along the west side of the Best Street ramp for one of the seven retaining wall panels (Wall No. 8 from Contract 59-19)
- Between Dodge and Northampton Streets, on the west side, for six of the eight retaining wall panels (Wall No. 9 from Contract 59-19).

2.2.2. Existing Retaining Walls on Piles

Existing retaining walls have variability of pile details based on wall heights.

Cantilever retaining walls on steel H-pile foundations exist at the following locations:

- South of the Dodge Street Bridge and along the west side of the Best Street ramp for three of the seven wall panels (Wall No. 8 from Contract 59-19)
- Between Dodge and Northampton Streets, on the west side, for two of the eight retaining wall panels (Wall No. 9 from Contract 59-19).
- Between Dodge and Northampton Streets, on the east side, for all seven retaining wall panels (Wall No. 11 from Contract 59-19).
- Between Northampton and East Utica Streets, along the east ramp, 12 panels of Wall No. 3 from Contract 68-02.
- Between Northampton and East Utica Streets, on the west side, all three panels of Wall No. 10 from Contract 59-19 and 15 of the 18 panels of Wall No. 4 from Contract 62-86.

2.3. Existing Bridges

There are four existing bridges over the Kensington Expressway within the tunnel limits, located at Dodge, Northampton, East Utica, and East Ferry Streets. The Dodge and Northampton Street Bridges were constructed in 1963 under Contract 59-19. The East Utica and East Ferry Street Bridges were constructed in 1967 under Contract 68-02.

2.3.1. Superstructures

Each of the four bridges consists of two simple spans, with Span 1 over Kensington Expressway eastbound and Span 2 over Kensington Expressway westbound. All four bridges have steel multi-girder superstructures with composite reinforced concrete decks. The Dodge Street Bridge has an asphalt overlay.

The existing Dodge Street Bridge is at the southern limit of the proposed Kensington Expressway Tunnel. The bridge carries a 30′-0″ roadway for a single travel lane and shoulder in each direction and a 6′-0″ sidewalk with metal railing and pedestrian fence at each fascia. The bridge has a minimum vertical clearance of 14′-3″ over the Kensington Expressway.

The existing Northampton Street Bridge carries a 48′-0″ roadway for a single travel lane and shoulder in each direction and an 8′-0″ sidewalk with metal railing and pedestrian fence at each fascia. The bridge has a minimum vertical clearance of 14′-6″ over the Kensington Expressway.

The existing East Utica Street Bridge carries a 52′-0″ roadway for a travel lane, turn lane, and shoulder in each direction and a 6′-0″ sidewalk with concrete railing and pedestrian fence at each fascia. The bridge has a minimum vertical clearance of 15′-4″ over the Kensington Expressway.

The existing East Ferry Street Bridge carries a 52′-0″ roadway for a travel lane, turn lane, and a 6′-0″ sidewalk with concrete rail and pedestrian fence at each fascia. The bridge has a minimum vertical clearance of 15′-2″ over the Kensington Expressway.

2.3.2. Substructures

The begin and end abutments at Dodge Street are reinforced concrete buttressed abutments on spread footings with keys into rock aligned with buttresses. The begin and end abutments at East Utica and East Ferry Streets are reinforced concrete cantilever abutments on spread footings on rock. The begin and end abutments of the Northampton Street Bridge are founded on steel piles, with one row of vertical piles under the heel and two rows of battered piles under the toe. All existing abutments have horizontal rustications, similar to the existing retaining walls.

Pier 1 for each structure is a multicolumn pier with reinforced concrete columns and cap beam. At Dodge, East Utica, and East Ferry Streets, the pier is founded on spread footing keyed into rock. At Northampton Street, the pier is founded on three lines of steel piles, with the center line vertical and the other two lines battered away from the centerline of the pier.

2.3.3. Utilities and Other Appurtenances

The existing Dodge Street Bridge currently has street lighting and roadway sign panels mounted to Span 1 girder G1 and Span 2 Girder G6.

The existing Northampton Street Bridge currently carries two 8″ gas lines and one 10″ water main, roadway lighting, and a roadway sign panel mounted to Span 1 girder G1 for the East Utica Street Exit.

The existing East Utica Street Bridge currently carries several utilities, including nine 4″ electrical conduits, nine 3½″ communication (telephone) conduits, and one 12″ water main. There is a roadway sign panel mounted to Span 2 Girder G9 for the Best Street Exit.

The existing East Ferry Street Bridge currently carries several utilities, including twelve 4″ electrical conduits and two 12″ water mains. There are roadway sign panels mounted to Span 1 girder G1.

3. Proposed Cross Section Assumptions

3.1. Horizontal Clearance

The proposed Kensington Expressway Tunnel would consist of two adjacent traffic corridors, one for eastbound traffic and the other for westbound traffic. Each corridor would have three 12′-0″ lanes, an 8′-0″ right shoulder and a 6′-0″ left shoulder for an overall width of 50′-0″ between walls (see Figure 3).

There would be a single wall separating the two directions of traffic and retaining walls to the outside, generally in similar locations to the existing retaining walls along the Kensington Expressway.

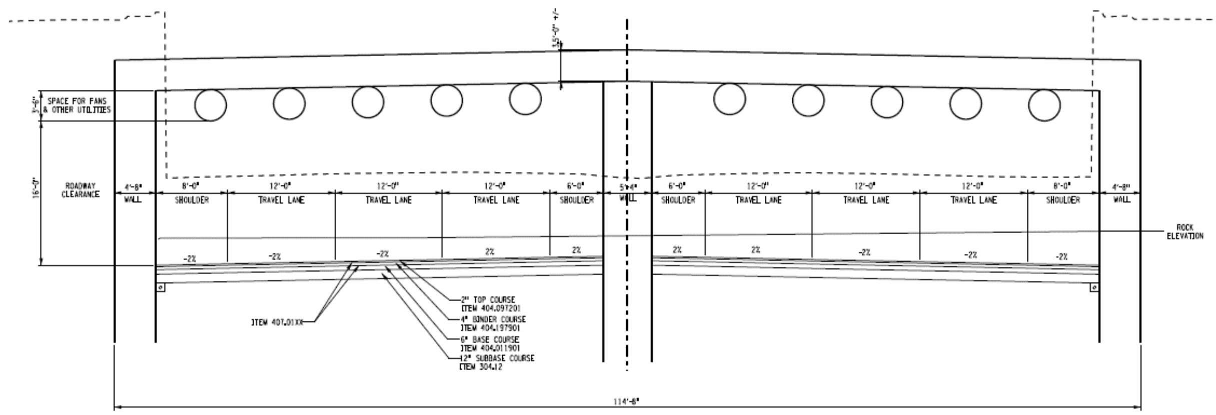


Figure 3 Tunnel Roadway Cross Section

3.2. Vertical Clearance

Many bridges on either end of the proposed tunnel limits have between 14′ and 15′ vertical clearance, therefore each tunnel tube is proposed to maintain a minimum 16′-0″ vertical clearance, consistent with Highway Design Manual Chapter 2 tunnel design criteria (Section 2.7.5.9) from top of roadway to the lowest element over the roadway, whether structure or appurtenance. These appurtenances include tunnel ventilation (jet fans), sprinkler system, lighting, roadway signs, ITS equipment and utility ducts.

3.3. Overburden

The proposed Kensington Expressway Project seeks to reconnect communities along the Humboldt Parkway, with each direction of traffic separated by a park. The tunnel roof would accommodate a minimum of 3′-6″ depth of special organic soil to support growth of trees of a similar species as the original parkway. In addition, there would be an additional approximate 0′-6″ of depth provided for a drainage layer, waterproofing, and insulation over the tunnel cap. In the case of an arch structure, a reduced soil depth would be permitted at the crown of the arch to minimize the impact to the overall tunnel depth and associated soil and rock removal limits. The minimum permissible overburden at the crown of the arch would be 3'-0" total, to include 2'-6" of soil and 6″ for drainage and insulation.

The cap would continue to carry existing cross streets at Dodge, Northampton, East Utica, and East Ferry Streets. Additional cross streets would be established at Riley Street, Winslow Avenue, and Sidney Street.

Additionally, the proposed alignment of the northbound and southbound Humboldt Parkway would be shifted inward from their current locations by 11′-6″. A portion of these roadways would be over the proposed tunnel. Roadway geometry will account for cross slopes, curb heights, etc. and would be adjusted as needed to maintain appropriate depths over the structure for subbase, pavement, etc.

4. Design Criteria

Structural design of the proposed Tunnel would be performed in accordance with the NYSDOT *LRFD Bridge Design Specifications* (AASHTO *LRFD Bridge Design Specifications*, 9th Edition and NYSDOT *LRFD Blue Pages*) and as supplemented by design parameters related to the unique nature of the project and the environmental nature of its geographic location, as further proposed and detailed in this section.

The proposed Tunnel would primarily support a park-like setting as well as local city streets, both transverse to and longitudinal to the tunnel alignment. The loading conditions described below would be considered on the roof structure for the park-like and the roadway areas. The load factors indicated below would be modified with an operational importance modifier (η_i = 1.05).

It is anticipated that future maintenance of the park-like areas over the tunnel cap would be the City of Buffalo's responsibility, while maintenance of the tunnel structure would remain NYSDOT responsibility. It is envisioned that a maintenance agreement may be required with the City of Buffalo, which could include limitations on storage of materials so that loading as defined in below criteria is not exceeded during maintenance activities. Additionally, with the tunnel being NYSDOT property, highway work permits would be required for events or work in the park, which will provide a means to have permit stipulations that would limit loading such that established design criteria are not exceeded (may require load ratings, as appropriate).

In the event of future changes in operational use over the tunnel, the tunnel structure will be designed for vehicular live loading (HL-93) in park-like areas and roadway areas alike. As such, all park-like areas will be designed for both loading defined for "park-like areas" and "roadway areas" as defined in the following sections. These live loads will be considered traveling both parallel to and perpendicular to the tunnel alignment.

4.1. Strength and Service Limit States

4.1.1. Dead Loads

Dead load would consist of the self-weight of the tunnel cap structure (preliminary design is based on 155 PCF unit weight for reinforced concrete), and superimposed dead loads consisting of:

1. Park-like areas

   Waterproofing and protective concrete: 80 Pounds per Square Foot (PSF), with DC (Dead load effect due to structural components and nonstructural attachments) Load Factor

   Mechanical-Electrical systems: 10 PSF, with DW (Dead load effect due to wearing surface and utilities) Load Factor

   Trees, shrubs and temporary facilities: 60 PSF, with DW Load Factor

   Total .... 150 PSF

2. Roadway areas

   Waterproofing and protective concrete: 80 PSF, with DC Load Factor

   Mechanical-Electrical systems: 10 PSF, with DW Load Factor

   Total .... 90 PSF

Substantial concentrated loads such as weight of fans suspended from the roof would be considered as applicable. All equipment and utilities would be treated as DW loading.

4.1.2. Superimposed Dead Load/Earth Fill

1. Park-like areas:

   Unit weight of 3′-6″ depth of compacted fill is assumed to be 130 pcf, with EV (vertical pressure from dead load of earth fill) Load Factor.

2. Roadway areas:

   Weight of 2′-9″ depth of compacted fill, with EV Load Factor

   Weight of 9″ depth of asphalt roadway, with DW Load Factor

3. Construction Loading (Strength only): 250 PSF, with 1.25 Load Factor

   Modified as required by design-builder.

4.1.3. Live Load

1. Park-like areas:

   a. Pedestrian Loading (no snow): 85 PSF, with LL (vehicular live load) Load Factor

      OR

   b. Snow load within park: 150 PSF, with LL Load Factor

      Equivalent to weight of 6-foot-high unplowed snow with a unit weight of 25 PCF. See Section 4.2.2 for further discussion on snow loads.

      And

   c. Snow load within 15′ of roadway: 180 PSF, with LL Load Factor

      Equivalent to weight of 4 feet of piled up wet snow/rain mix with a unit weight of 45 PCF. Considered along each side of Humboldt Parkway and cross streets. See Section 4.2.2 for further discussion on snow loads.

2. Roadway areas:

   a. Vehicular live load on the tunnel roof would be AASHTO HL-93 live load with dynamic load allowance and with distribution of load through earth fill in accordance with Article 3.6.1.2.6. The cap shall have an LRFR Inventory Rating Factor of 1.2 or greater in the as-designed/as-built condition, in accordance with Section 2.5.1 of the NYSDOT Bridge Manual.

      Load factor = 1.75

      OR

   b. Sidewalk live load will be in accordance with pedestrian loads established in NYSDOT LRFD Section 3.6.1.6 and will use NYSDOT LRFD LL Load Factor.

3. Construction Loading (Strength Limit State only):

   Live load directly on the tunnel roof will be as required in construction and AASHTO HL-93 truck (no lane) without dynamic load allowance, single lane multiple presence factor. Construction live load will be placed on the roof slab to create the maximum load effect for the tunnel element being designed. Additionally, the roof slab will not be considered in the distribution of the truck load.

   Load Factor = 1.4.

4.2. Extreme Event Limit States

4.2.1. Dead Load and Earth Fill

As per Sections 4.1.1 and 4.1.2.

4.2.2. Snow

Snow loading is not defined by NYSDOT LRFD. However, since much of this structure will be covered by park-like areas, which will not be plowed during the winter, snow loading must be considered, especially given the heavy snow demands in Buffalo due to lake effect snows.

The 2018 International Building Code does not provide ground snow loads for Buffalo, indicating extreme local variations in ground snow loads in the area, which did not allow for defining loading at the scale of mapping provided within the code. The code recommends site-specific considerations.

The ASCE 7 Hazard Tool recommends a ground snow load of 76 PSF within the project limits for Risk Category IV.

https://roofonline.com/weight-of-snow/ provides a table of weights of snow for different conditions (from light, dry snow to slush) based on references including The International Classification for Seasonal Snow on the Ground, International Association of Cryospheric Sciences, 2009. The variation in loading can be substantial and is affected by many factors, including (but not limited to) how wet the original snow fall is, the overall depth of accumulated snow (which becomes more compact with greater depth), and the amount of wind exposure. Given the large variation in potential snow loading (see Table 1), the snow loads used are based on a conservative 25 PCF for typical snows to cover dry, new snow to wet snow. Additionally, slush is considered at 45 PCF, in accordance with this documentation. See Table 1 for excerpts from this documentation.

1. Park-like areas:

   Snow load: 250 PSF,with 0.5 Load Factor

   Equivalent to weight of 10-foot-high unplowed snow with a unit weight of 25 PCF.

   No other live load.

2. Roadway areas:

   Snow load: 250 PSF, with 0.5 Load Factor

   Equivalent to weight of 10-foot-high unplowed snow with a unit weight of 25 PCF.

   No other live load.

Table 1 Weight of Snow (https://roofonline.com/weight-of-snow/)

Weight of Snow						
Type of Snow	Lbs per Inch of Depth per Square Foot (Average)	Lbs per Cubic Foot (lb/ft^3) (Average)	Lbs per Cubic Foot (lb/ft^3) (Range)	Grams per Cubic Centimeter (g/cm^3) (Range)	Kg per Cubic Meter (kg/m^3) (Average)	Kg per Centimeter of Depth per Square Meter (Average)
Ordinary New Snow (Snow immediately after falling, in below-freezing temperatures, with no wind; fresh, uncompacted snow that has a high volume of trapped air.)	0.3 lbs	3.59 lb/ft^3	3.12 – 4.06 lb/ft^3	0.05 – 0.065 g/cm^3	57.5 kg/m^3	0.58 kg
Settling Snow (Snow less than a day old that is starting to experience some wind and temperature variation.)	0.68 lbs	8.12 lb/ft^3	4.37 – 11.86 lb/ft^3	0.07 – 0.19 g/cm^3	130 kg/m^3	1.30 kg
Damp New Snow (Snow immediately after falling, in slightly above-freezing temperatures, with little wind exposure.)	0.78 lbs	9.37 lb/ft^3	6.24 – 12.49 lb/ft^3	0.1 – 0.2 g/cm^3	150 kg/m^3	1.50 kg
Settled Snow (Typical after more than one day in place. Snow that has experienced some temperature and wind variation.)	1.3 lbs	15.61 lb/ft^3	12.49 – 18.73 lb/ft^3	0.2 – 0.3 g/cm^3	250 kg/m^3	2.5 kg
Average Wind-Toughened Snow (Compacted snow after wind exposure in below-freezing temperatures.)	1.46 lbs	17.48 lb/ft^3	17.48 lb/ft^3	0.28 g/cm^3	280 kg/m^3	2.8 kg
Wet Snow (Dense, sticky snow in relatively warm temperatures with little wind. Good snow for making snowballs.)	1.75 lbs	21 lb/ft^3	17 – 25 lb/ft^3	0.27 – 0.40 g/cm^3	335 kg/m^3	3.35 kg
Wind-Packed Snow (Hard Wind Slab. Compacted snow after prolonged and heavy wind exposure.)	1.98 lbs	23.73 lb/ft^3	21.85 – 25.6 lb/ft^3	0.35 – 0.41 g/cm^3	380 kg/m^3	3.8 kg
Slush (Advanced melting snow; snow/water mix.)	3.75 lbs	45 lb/ft^3	35 – 55 lb/ft^3	0.56 – 0.88 g/cm^3	720 kg/m^3	7.2 kg

5. Cap / Superstructure Alternatives

Two alternative configurations for the cap/superstructure are considered: a flat slab roof and a double arched roof. For cross sections of these alternatives, see Appendix B.

5.1. Structural Basis of Design

Given the age, condition, and design/detailing of the existing retaining walls, incorporation of these walls into the structural system of the proposed structure, which would subject them to loading patterns that are different from the current/as-designed conditions, has not been considered. As such, the present evaluation focuses on types of roof structure to be supported by new retaining walls and a central dividing wall (walls are discussed in Section 6).

In this respect, two basic configurations, a flat slab roof and a double arched roof, were evaluated as described below.

In all cases, a deepening of the Expressway's roadway profile would be necessary to provide the headroom to accommodate vehicle envelope and tunnel ventilation equipment and other utilities and appurtenances, as discussed in Section 3.

Structural solutions assume the use of the following material properties:

- 3,000 PSI for cast-in-place concrete 28-day compressive strength. Permitting higher strength mixes (say 5,000 PSI 28-day compressive strength) is recommended for this project to achieve more efficient structural designs
- 8,000 PSI for precast concrete 28-day compressive strength (prestressed solutions not considered due to variable loading conditions leading to time dependent parameters that would be difficult to predict)
- 60 KSI for epoxy-coated reinforcement steel yield strength
- 100 KSI (75 KSI with use of couplers) for chromium steel reinforcement steel yield strength

Concrete roof thicknesses include additional allowance to Code-specified clear cover to steel elements at the underside of the roof to accommodate for 3 inches of sacrificial concrete or fireproofing material.

To achieve a 75+ year design life for the tunnel structure, chromium steel reinforcement and high performance internally curing concrete (HPIC) will be used for the roof slab. Additionally, a three (3) inch minimum clear cover will be provided for reinforcement within the walls and slabs. Epoxy coated reinforcement will be used in the center and exterior retaining walls as well as the bottom slab.

5.1.1. Flat Roof Slab

A flat roof solution might include one of the following options:

- Cast-in-place (CIP) reinforced concrete slab,
- A series of precast reinforced concrete panels,
- A CIP concrete slab with a series of structural steel filler beams, or
- Precast concrete units with structural steel filler beams.

The limited soil depth above the tunnel roof slab will not permit the use of standard traffic signal and light pole foundations for such appurtenances along the Humboldt Parkway. Project-specific details will be established for connection of these elements directly to the roof slab while allowing for future maintenance repairs or replacements of the appurtenances. These project-specific details will require City of Buffalo review and approval, which will occur in development of details in final design.

The reinforced concrete options offer the flexibility, if needed, of rigid connections to the walls, which would allow for more economical exterior walls by taking advantage of the lateral support that would be offered by the slab acting as a strut between exterior walls. This advantage, however, would be lost if it is desired to design the walls as cantilevered structures, with the anticipation of a future removal of the cap in full for rehabilitation or replacement. For this reason and also for increased durability, the design of the roof slab will not consider any increase in capacity due to the presence of compressive forces exerted between the exterior walls by the adjacent soil mass.

These slab alternatives are anticipated to have a superstructure depth of 3'-6".

5.1.2. Precast Concrete Arch

In order to minimize the depth of the tunnel invert in an arched roof solution, to minimize required soil excavation and rock removal, the arch would be relatively flat, with a span (50 ft) to rise (7 ft) ratio of just over 7. As such, the arch must be designed for combined axial and flexural loading (pinned/roller assumption). Each arch is anticipated to be 3'-6" thick.

While the arch itself is based on pinned/roller assumptions, the detailing of the connection of the arch to the supporting retaining walls would cause the arched geometry to introduce outward horizontal thrust reactions at the supports from dead loads of the arch itself as well as any superimposed dead or live loads placed upon the arch. For the exterior walls, this horizontal thrust acts in opposition to lateral earth pressure, and as such is not problematic. However, the center wall needs to be designed to take the horizontal thrust acting at the top of the wall and generating considerable bending moment at the base of the center wall from the unbalanced dead loads of a single vault during construction (see Figure 4).

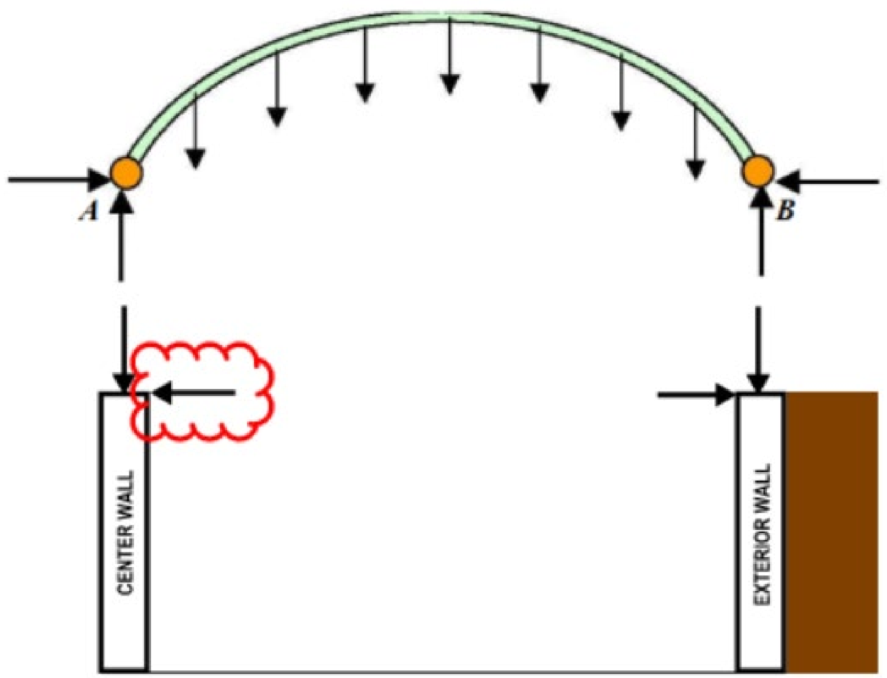


Figure 4 Unbalanced Horizontal Thrust on Center Wall during Construction

Due to the difficulties and ensuing cost of forming a double arched roof, this configuration lends itself to a precast reinforced concrete solution. There would likely need to be a joint between the double arches.

5.2. Evaluation of Proposed Roof Configurations

The feasibility and the advantages and disadvantages of the proposed two cap alternatives are evaluated from the aspects of constructability, performance/durability, impacts, and cost (see Section 8).

5.2.1. Constructability Issues

Important factors affecting constructability can be identified as:

- Structural considerations
- Fabrication and installation considerations
- Work zone traffic control
- Adequate contractor staging areas

Specific constructability issues of different alternatives are discussed below.

Structural Considerations:

The flat roof slab options allow for achieving continuity of the cap over the two spans. This allows for the elimination of a joint at the center pier, which improves the durability of the structure. The arch solution would require a joint between the two arch vaults at the center wall, posing a durability concern due to increased potential for water infiltration, especially with the arches creating a trough that would be prone to collecting water in this area.

The proposed slab-type roof is preferred over the arched type of roof structure on the basis of lateral loading, particularly as the tunnel structure would be constructed in stages. With the slab-type structure, the loads induced into the exterior and center walls from temporary and permanent loadings are primarily vertical. On the other hand, an arch solution introduces lateral loads at the supports in addition to vertical loads. The issue is that when only one traffic corridor of the tunnel is completed, the center wall must resist an unbalanced lateral load from the arch. This imbalance is not reconciled until the second traffic corridor is constructed. Thus, the arch solution requires a more robust design for the center wall than a flat slab solution. This is especially true in areas of low rock, where the moment arm for the horizontal thrust would be substantial.

Fabrication and Installation Considerations:

Precast solutions may be preferred from the perspectives of quality control and the possibility of using higher strength concrete. Fabrication of precast elements in shop allows for working in controlled environments (temperature and humidity). Additionally, all units are inspected for defects prior to shipping.

On the other hand, it may be easier to accommodate changes in alignment/geometry with cast-in-place construction than with precast elements, for which repeated use of similar forms is convenient.

The flat roof alternatives allow for construction of the cap by cast-in-place or precast concrete methods, at the contractor's discretion. These solutions also allow for pre-casting of units onsite or offsite, allowing a contractor flexibility in operations and potential to reduce shipping. For the arch roof, precast reinforced concrete is preferred, and given the more complex geometric forming requirements, shop fabrication would likely be required.

Cast-in-place construction is more permissive to field inaccuracies, such as misalignments of the supporting walls. In addition, installing precast slab or arched cap segments over cast-in-place walls requires a higher degree of precision. Misalignments between adjacent precast units create steps that may prove difficult to adequately waterproof, with potential for increased long term durability concerns.

Work Zone Traffic Control:

It is important to note that the flat slab structure of the tunnel, as depicted in the WZTC sections (see Appendix E), is critical in quickly returning the Humboldt Parkway back to existing user levels. Being able to backfill the area over the tunnel roof after it is constructed allows for the re-establishment of turn lanes and bicycle lanes on Humboldt Parkway. Additionally, it accomplishes this without any temporary widening along Humboldt Parkway, thereby avoiding temporary ROW impacts, utility and sign relocations, tree removals, as well as temporary sidewalk and driveway accommodations. It is not feasible to achieve similar accommodations with an arch structure, as the imbalanced horizontal loading and resulting moment at the base of the center wall become too great.

Contractor Staging Areas:

The flat slab type design alternatives provide a natural location for the contractor to stage equipment and materials, as well as establishing an invaluable work area for construction staging. A contractor could be working directly on the flat slab itself with temporary or permanent fill placed on the slab. Additionally, the staging area becomes progressively larger as construction progresses.

The arch alternative does not allow for this benefit, as it is not feasible to design for the additional horizontal thrust from the superimposed dead and live loads that the arch solution would impose on the center wall.

5.2.2. Performance of Completed Structure

The long-term durability of a structure is critical to its success. Some factors that contribute to long term durability include:

- Drainage
- Joints
- Bearings

Specific discussions on anticipated performance of structural alternatives are discussed below.

Drainage:

The arched-type roof structure presents a challenge regarding appropriate drainage of the overburden soils on the roof, as a trough is created between the two vaults of the overall tunnel structure. Drainage pipes would need to be placed in the center wall and tied to a drainage system. Since the tunnel drainage is setup to be collected and pumped due to being potentially hazardous, the drainage system for the drainage in the center wall would need to be its own separate system, creating an additional cost to the project.

Joints:

Precast concrete solutions require the use of units of a size and weight that are feasible to lift and transport. As such, the precast solutions, whether for flat slab or arch, would have many joints between adjacent precast units. These joints would need special attention in their detailing to achieve watertight solutions. As compared to cast in place solutions, the number of joints susceptible to potential leakage is much larger with precast alternatives.

Bearings:

The alternatives under consideration (flat slab solutions and precast arch alternative) would not make use of traditional bearings. Instead, a sliding interface would likely be provided between the superstructure and retaining walls. While these sliding surfaces might not have the same maintenance needs as traditional bearings, their durability and effectiveness does change with time. It might be expected that the friction coefficient at the sliding interface would change with time, which might mean additional horizontal loading being transferred from the superstructure to the retaining walls for loading considerations such as thermal effects. Cast-in-place alternatives and precast alternatives with closure pours may allow for integral construction of retaining walls and caps, which would eliminate the need for these sliding interfaces.

5.2.3. Other Considerations and Impacts

Park Considerations:

The project seeks to reconnect communities along the Humboldt Parkway via the addition of greenspace on the proposed cap and cover tunnel structure. A 3′-6″ soil depth allows for growth of up to 60-foot-tall trees. The flat slab design solutions allow for uniform 3′-6″ soil depth and freedom to plant trees at any location within the park. To maintain the same tunnel invert as the flat slab solution, the overburden at the crown of the arch would need to be reduced such that plantings would be limited to areas away from the crown. An offset alignment further restricts locations where trees can be planted while maintaining appropriate depth for root structure. Additionally, in such a case, there would be concerns that roots might infiltrate drainage and insulation layers (possibly damaging integrity of plantings) and ultimately into the structure itself. To mitigate this, the same minimum cover could be applied to the arch structure, which then would require a deeper tunnel invert.

Utilities:

The existing cross street bridges carry various utilities which would need to be accommodated by the proposed structure. Slab solutions could accommodate utilities within the slab itself. In the arch solution, these utilities would need to be accommodated within the rise of the arch, meaning they would need to be coordinated with tunnel ventilation equipment, fire suppression system, lighting, and associated utility runs.

Additionally, in a utility coordination meeting, the Buffalo Water Authority conveyed that waterlines under the proposed tunnel would not be acceptable. Instead, the project would need to accommodate replacement waterlines either over the tunnel (buried), embedded within the cap, or against the underside of the tunnel roof slab, and insulated as necessary. The slab design can accommodate these options more effectively than the arch solution.

Soil Excavation and Rock Removal Limits:

As noted above, in order to maintain a minimum 3′-6″ soil depth over the arch for planting of trees and mitigating potential infiltration of roots into drainage and insulation layers or tunnel roof, the invert of the arch alternative would need to be deeper than that for a flat slab alternative. This thereby increases soil excavation limits and rock removal limits, whether by mechanical means or blasting. The topic of rock removal has been a contentious one with the public and stakeholders. As such, solutions that minimize rock removal for the project are preferred.

6. Retaining Wall / Substructure Alternatives

6.1. Retaining Wall Concept

The construction of the proposed tunnel requires that the vertical roadway profile be lowered by 0 feet at the tie-ins to a maximum of approximately 20 feet. The tunnel walls must support not only earth and water pressures but also considerable vertical load from the roof slab and overlying park and roadway development. In addition, the tunnel alignment would be below the groundwater table, limiting flow beneath the tunnel is essential for preventing the need for constant water pumping. These desired attributes lead to a recommendation that interlocking secant piles be used for the lower portions of the tunnel walls. Unlike slurry walls, secant piles can be drilled to considerable depth into rock and can be readily extended from invert level with conventional CIP wall construction.

All 4,150 feet of the new tunnel invert would be below ground water level. The recommended secant pile design would provide an effective barrier to limit the amount of water that can seep into the tunnel area and eventually into the tunnel's subdrain system. Limiting the seepage would also substantially reduce sump pumping and long-term maintenance costs.

Secant piles would be drilled into bedrock from existing grade. Cast in place walls would extend the walls from the top of the secant piles at existing grade to the support level of the tunnel roof slab. The section of secant piles that would be exposed when excavation is completed to the final tunnel invert elevation would be lined with a concrete fascia tied into the secant piles and finished to match the cast-in-place (CIP) wall above for a uniform appearance.

Two alternative wall designs are considered. For cross sections of these alternatives, see Appendix B. For discussion of advantages, disadvantages, and design limitations of these alternatives, see Section 6.3.

6.1.1. Braced Wall Alternative

In this alternative, the walls are assumed to be braced by the tunnel roof acting as a strut between the exterior tunnel walls. Connections between the roof and the walls would be similar to integral abutment construction, creating a rigid frame structure.

The proposed scheme for the tunnel exterior walls consists of 3′-0″ diameter secant piles 5′-6″ on center (see Figure 5) with 4′-0″ cast-in-place walls extending to the underside of the tunnel roof.

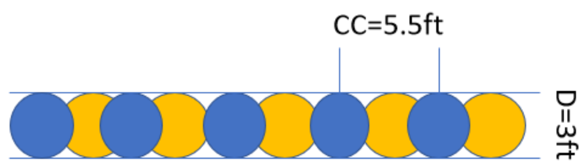


Figure 5 Proposed Secant Pile Configuration (Note: Orange Circles Represent unreinforced Secant Piles and Blue Circles Represent Reinforced Secant Piles)

While the center wall will be braced in the final condition, temporary conditions during construction require the center wall to be designed as a cantilever wall. An unbalanced load condition occurs when one side is excavated and the other remains at existing elevations and supporting live load surcharge. The maximum differential in soil depth on either side of the center wall is anticipated to be approximately 22 feet to accommodate placement of subbase and potential drainage or utilities. The center wall consists of 4′-0″ diameter secant piles 6′-0″ on center (see Figure 6) with 5′-4″ cast in place walls above.

6.1.2. Cantilever Wall Alternative

In this alternative, the walls are assumed to act as cantilevered structures, without bracing at the top of the wall.

The proposed scheme for both tunnel exterior walls and center wall consists of 4′-0″ diameter secant piles 6′-0″ on center (see Figure 6) with 4′-8″ exterior and 5′-4″ center cast-in-place walls extending to the underside of the tunnel roof.

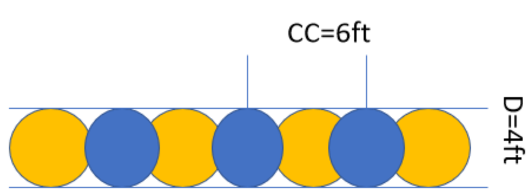


Figure 6 Proposed Secant Pile Configuration

6.2. Retaining Wall Construction Concept

6.2.1. New Retaining Walls Founded on Rock

In areas where the new tunnel walls would be founded on rock, the anticipated structural scheme entails the removal of the existing cantilever retaining walls along the east side of the highway, and the partial preservation of the existing retaining walls along the west side of the highway to serve as temporary support of excavation during construction. In the final condition, the existing walls would be fully removed or abandoned in place so that the final proposed structure does not rely on existing retaining walls in any capacity.

6.2.2. New Retaining Walls in Low Rock Areas

Similar to proposed walls founded on rock, the recommended structural scheme for proposed walls in low rock areas entails the removal of the existing cantilever retaining walls along the east side of the Kensington Expressway.

For the west walls, where the existing walls cannot practically be reused as temporary support of excavation due to their pile foundations, the existing walls would also be removed. Where the secant pile alignment intersects the existing piles, these piles would likely need to be pulled out for deep conflicts or burned for shallow conflicts to allow construction of the secant piles. The depth of the conflict varies along the alignment and typically occurs within a depth of 5 feet. Given the vertical profile change between the existing and proposed walls the existing piles will be exposed during the construction of the new walls.

6.3. Evaluation of Proposed Retaining Wall Configurations

The feasibility and the advantages and disadvantages of the proposed retaining wall alternatives are evaluated from the aspects of constructability, future replacement needs, impacts, and cost.

6.3.1. Constructability Issues

The final design of the 3′-0″ secant pile walls relies on the tunnel roof acting as a strut between the exterior walls, effectively bracing the wall. The temporary SOE used for the construction of the east wall would be required to remain effective until the completion of the adjoining EB and WB tunnel cap structures between exterior walls. As this is anticipated to require more than 18 months, the temporary SOE will need to be designed as permanent and will need to have corrosion protection. To allow access to the cap of the first tunnel corridor prior to completion of the adjacent corridor, flowable fill could be placed between the temporary SOE and the proposed wall. To minimize hydrostatic pressure effects on the proposed wall, the flowable fill would be placed in small lifts. As an alternate, the use of 4′-0″ secant piles would allow for the temporary SOE to be removed or abandoned in place after the construction of the proposed wall.

6.3.2. Future Roof Slab Removal

Replacement of the tunnel roof slab without additional measures would require the exterior walls to be designed to take lateral earth pressures and surcharge in cantilever action. The high and low rock were evaluated to determine load effects in the wall both in braced (via roof) condition and in cantilevered condition to allow for potential future removal and replacement of the roof slab (see Table 1). In the case of high rock, maximum moment condition was markedly higher for the cantilever condition (535 k-ft/ft) versus the braced condition (176 k-ft/ft), a 200% increase. In addition, the required embedment of the secant piles increased from approximately 15 ft to 25 ft. To have sufficient capacity in the cantilevered condition, the exterior retaining walls require the use of 4′-0″ diameter secant piles. For the braced condition, with the roof acting as a strut, 3′-0″ diameter secant piles are sufficient to withstand the loading.

Table 2: Wall Foundation Reactions for Braced (Roof) and Cantilever Wall Conditions

Loading Conditions	Low Rock Condition			High Rock Condition		
	Roof Strut	Cantilever	Increase (%)	Roof Strut	Cantilever	Increase (%)
Maximum Moment (kip-ft/ft)	144	438	200	176	535	200
Maximum Shear (kip/ft)	25	40	60	28	45	60

If detailed correctly, durability would be such that replacement would not be required, similar to subway tunnel construction. Considering the substantial difference in required secant pile size based on the support conditions at the top of the wall and associated costs, we considered another approach to any future roof slab replacement. Should the tunnel roof need to be removed and replaced sometime in the future, its removal could be staged such that the impacted wall length could be propped to the adjoining supported wall sections. The lateral loads from the unsupported wall section could be distributed using a temporary 'waler'. The loads would be transferred to the adjacent roof slab. With temporary closure of the shoulders within the tunnel, there would be room for the waler and jacks. This approach would allow the use of the smaller 3′-0″ diameter secant piles.

To balance the initial capital costs associated with designing the proposed exterior retaining walls for future replacement of the full cap with flexibility of the final design to accommodate the potential future replacement of the cap, a blend of the above two options is recommended. The exterior walls will be designed to span an unbraced length of 20 feet. This will allow for future roof slab replacement in segments along the length of the tunnel without necessitating additional bracing of the retaining walls.

6.3.3. Other Considerations and Impacts

The use of 3′-0″ secant pile walls allows for the following benefits:

- The overall width of the tunnel section would be 1′-4″ narrower, slightly reducing project excavation and rock removal needs.
- The smaller exterior walls accommodate an additional 8″ clearance from the 84″ sewer lines along northbound Humboldt.

6.3.4. Quantities and Cost

The cross-sectional area of the 3′-0″ diameter secant piles is 7.1 SF, while for the 4′-0″ diameter secant piles it is 12.6 SF. The premium for the cantilevered wall equates to a 78% increase in material quantities and costs for the exterior wall secant piles. Additionally, the CIP walls above the secant pile walls would see a 17% increase in material quantities and costs.

The smaller footprint of the overall tunnel section with the use of the 3′-0″ diameter secant piles would slightly reduce soil excavation and rock removal quantities and costs.

6.4. Temporary Support of Excavation

In areas of high rock, the existing east wall would be fully removed and the proposed east wall would be constructed via the use of soldier pile and lagging wall or interlocking sheeting with tiebacks placed behind the existing retaining wall foundation. Interlocking sheeting would provide the additional advantage of providing groundwater cut-off and minimize the need for pumping of ground water. The deflections of the temporary walls can be predicted and designed to limit horizontal deflections to less than 2 inches. The existing west wall would generally be maintained to serve as temporary support of excavation for the construction of the proposed west wall. Construction of the proposed wall would require removal of the wall toe. To ensure that the existing retaining wall maintains sufficient sliding and overturning capacity, under pinning would be used to tieback the existing foundation via rock bolts through the proposed secant pile wall and anchoring of the top of the secant pile wall to the existing footing. This work would be staged such that the toe of the existing footing is only partially removed and underpinning made effective prior to proceeding with the next adjacent area. See Appendix C for details.

In areas of low rock, both the existing east and west walls would be removed and proposed walls would be constructed with the use of temporary support of excavation. Since the low rock areas are well below the water table, interlocked steel sheet piling is recommended for the SOE to provide for groundwater cut-off. See Appendix D for details.

7. Constructability and Staging/Phasing

7.1. Work Zone Traffic Control (WZTC) Designs and Drawings

LaBella developed several drawings to depict the WZTC schemes in both plan and sections as follows:

(1) WZTC sections depict the primary stages of construction. (See Appendix E.)

(2) WZTC overview plan depicts the location of proposed SOE walls, permanent walls as well as the limits of where each of the WZTC sections would be applicable. (See Appendix F.)

(3) WZTC staging crossover plans – Plans that depict the primary traffic crossovers necessary for the EB and WB tunnel construction as noted on the WZTC staging sections. (See Appendix G.)

We feel that the WZTC design schemes represent an acceptable level of disturbance to the traveling public (local and commuters) as well as for the residents and users of the Humboldt and other City streets. We feel that this design, although not exclusive to others providing similar results, may help the NYSDOT establish reasonable parameters to build into construction contract requirements.

7.2. Constructability Challenges and Other Project Impacts

7.2.1. Change in Profile

The proposed final Kensington Expressway roadway profile within the tunnel limits varies from about 7′ to 20′ lower than the existing roadway profile. Due to this large grade differential, each tunnel tube would need to be constructed the full length prior to construction of the adjacent tube. Thus, one tube would need to be lowered in its entirety prior to lowering the adjacent tunnel tube.

7.2.2. Existing Retaining Walls

Given the change in roadway profile noted above, the proposed retaining wall foundations would be lower than those of the existing foundations. Any construction schemes that maintain existing retaining walls through construction of proposed retaining walls would need to be closely evaluated to ensure existing foundations are not undermined during construction. This becomes especially challenging in areas where existing foundations are on piles, with battered piles towards the Kensington Expressway. Existing piles are quite small, which does not make them feasible as soldier piles. Therefore, proposed excavation limits would sever these piles. Groundwater would pose an additional challenge in trying to use the existing piles in a soldier pile and lagging configuration. Thus, these existing walls would need to removed and replaced as construction progresses.

Additionally, the age and condition of the existing retaining walls coupled with the desired 75+ year design life of the proposed tunnel would preclude the reuse of the existing retaining walls in any capacity as part of the final design. All existing retaining walls will either be removed or abandoned in place in the final configuration.

7.2.3. Superstructure Solutions

Use of arches would provide substantial limitations on construction of the project. With only one arch in place, there would be an unbalanced horizontal thrust at the top of the center wall during construction. This horizontal thrust results in a large moment at the base of the center wall. To minimize the moment in the center wall and its required size, the loads on the center wall would need to be balanced with the arch from the adjacent span prior to placing any fill, and the fill would be placed in lifts to maintain near balanced loading conditions over the two spans. This loading restriction would result in a long construction zone (over 4,150′) along the full length of one direction of the Humboldt Parkway and a portion of the opposing direction in which traffic would be shifted.

Concrete slab or filler beam caps solutions allow for more flexibility in construction. These structural solutions could be loaded as construction progresses, generating construction staging areas as construction advances. Additionally, it could allow for potential of establishing partial park-like as construction progresses. More importantly, it would allow for re-establishing the parking lane along the Humboldt Parkway as construction progresses, thereby reducing impacts to local residents that utilize on street parking.

7.2.4. Existing Bridges

During construction, overpasses within the tunnel limits need to be maintained as follows based on coordination with the Buffalo Fire Department and for construction traffic/emergency access considerations:

- Vehicle and pedestrian traffic will be maintained at Northampton and East Ferry Streets via temporary bridges.
- Dodge Street and East Utica Street may be closed at times during the construction sequence. Pedestrian-only temporary bridges would be used as appropriate to maintain east-west connectivity during the construction period. Pedestrian crossings over the Kensington Expressway would be located at a maximum spacing of approximately 1,300 feet.
- The Best Street bridge will be replaced in stages to maintain vehicle and pedestrian access.

To achieve the above requirements, existing bridge structures would need to be replaced with temporary bridge superstructures (as needed) on proposed retaining walls as early work items. (See Appendix G.) These temporary bridges would then be replaced with the final roof decking one half at a time, successively diverting traffic to the reconstructed half. The proposed realignment causes an offset between the existing bridge piers and the center wall, which does not allow for partial removal/replacement of one span at a time while also maintaining sufficient horizontal clearances to maintain traffic in all construction stages.

Placement of new permanent bridges at the existing or proposed cross street locations in lieu of temporary bridges was investigated. The final surface elevations at the proposed cross street locations provide approximately 13′-3″ vertical clearance to the existing Kensington roadway surface. As such, the Kensington would need to be lowered by approximately 9" to achieve 14'-0" or by approximately 15" to achieve 14'-6" minimum vertical clearance. Note that the existing bridges within the project limits generally have 14'-6" to 15'-0" vertical clearance, with the controlling structure currently having 14'-5" vertical clearance. As such, it would not be recommended to permit less than 14′-6″ vertical clearance for temporary conditions. Ideally, with these structures intended to have a 75+ year design life, it would be preferrable to provide greater vertical clearance to minimize the potential for vehicular impact of the new structure during construction.

The lowering of Kensington poses several challenges and additional construction efforts. Temporary drainage would be required to avoid ponding in otherwise new low points in the temporary condition. The top of existing footings of adjacent retaining walls may be exposed, which would require potential removal or would impact available space for travel lanes/shoulders while maintaining traffic through the various construction stages. Temporary pavement would be required for a substantial stretch of the Kensington to accommodate a profile change for the temporary condition, which will be replaced with full/final tunnel construction.

Further, the horizontal offset between existing and proposed alignments is such that the proposed tunnel center wall (or bridge pier in this case) would not align with piers of existing adjacent bridges, limiting available space for travel lanes to accommodate traffic during various construction stages. The new west abutment and existing east retaining walls would need to be protected from oncoming traffic (blunt ends) due to the horizontal offset between existing and proposed alignments.

Operationally, a faux surface would need to be applied at the bottom of tunnel top slab to match adjacent tunnel segments (such as panels) to achieve uniform air flow within tunnel in final condition. Panels would be connected in sustained tension and may introduce risk of panels falling onto traffic. Additionally, panels would need to be removed to perform bridge superstructure inspection and maintenance.

7.3. Suggested Construction Sequence

The construction sequence is anticipated to be similar for all proposed structural solutions. See Appendix E for construction staging drawings.

- Where needed, replace existing bridges with temporary bridges (either staged construction or one bridge at a time, as deemed acceptable by the Department). (See Appendix G.)
- Place Temporary SOE for Construction of East Tunnel Wall (Stage 1)
    - Humboldt Parkway SB and Kensington Expressway operate normally. Shift traffic on Humboldt Parkway NB (temporary traffic impacts).

  - Establish work zone and install temporary SOE.
  - Shift temporary concrete barrier block by block as temporary SOE is installed to accommodate parking and one lane of traffic.
- Construct East Tunnel Wall (Stage 2)
  - Humboldt Parkway SB operates normally. Maintain reduced width operations on Humboldt Parkway NB. Shift Kensington Expressway EB and WB traffic towards the west retaining walls, maintaining 3 lanes in each direction but with reduced shoulders.
  - Drill secant piles for east wall from work zone along Humboldt Parkway NB.
  - Commence removal of existing retaining wall and fill to face of temporary SOE. For soldier pile and lagging wall, install lagging as removals progress.
  - Once approximately 5 feet to 8 feet of embankment is removed, install tiebacks.
  - Confine removal of existing retaining wall and embankment to temporary SOE/secant pile wall.
  - Locally (adjacent to the east wall) excavate to final tunnel invert elevation and complete construction of east wall.
  - Place and compact fill to existing subbase and place temporary pavement.
- Construct Center Wall (Stage 3)
  - Humboldt Parkway SB operates normally. Maintain reduced width operations on Humboldt Parkway NB. Reduce Kensington Expressway to 2 lanes in each direction with EB traffic shifted towards east wall and WB traffic shifted towards west retaining wall.
  - Drill secant piles for center wall.
  - Excavate on both sides of center wall to final tunnel invert elevation and complete construction of the center wall.
  - Place and compact fill to existing subbase and place temporary pavement.
- Lower Kensington Expressway EB to Final Grade (Stage 4)

  - Humboldt Parkway SB operates normally. Maintain reduced width operations on Humboldt Parkway NB. Maintain Kensington Expressway with 2 lanes in each direction, crossing over EB traffic to the WB side.
  - Excavate Kensington Expressway EB to below tunnel invert slab elevation.
  - Construct tunnel drainage and place aggregate and underdrains.
  - Construct eastbound tunnel roadway invert slab.
- Construct West Tunnel Wall (Stage 5)
  - Humboldt Parkway SB operates normally. Maintain reduced width operations on Humboldt Parkway NB. Crossover Kensington Expressway to the EB side and maintain 2 lanes in each direction.
  - Tieback existing west retaining wall (as needed) and partially remove existing west retaining wall toe.
  - Drill secant piles for west wall.
  - Locally (adjacent to the west wall) excavate to final tunnel invert elevation and complete construction of the west wall.
- Lower Kensington Expressway WB to Final Grade and Place WB Cap (Stage 6)
  - Humboldt Parkway SB operates normally. Maintain reduced width operations on Humboldt Parkway NB. Maintain 2 lanes in each direction on EB side for Kensington Expressway.
  - Continue excavation of Kensington Expressway WB to below tunnel invert slab elevation
  - Excavate Kensington Expressway EB to below tunnel invert slab elevation.
  - Construct tunnel drainage and place aggregate and underdrains.
  - Construct westbound tunnel roadway invert slab.
  - Construct westbound tunnel roof.
  - Construct realigned Humboldt Parkway SB (shift traffic as needed).
  - Install and perform testing of WB tunnel electrical, mechanical, and

communication systems.

- Place EB Tunnel Cap (Stage 7)
    - Humboldt Parkway SB operates normally. Maintain reduced width operations on Humboldt Parkway NB. Cross over Kensington Expressway to WB side and maintain 2 lanes in each direction.
    - Construct eastbound tunnel roof.
    - Construct realigned Humboldt Parkway NB (shift traffic as needed).
    - Install and perform testing of EB tunnel electrical, mechanical, and communication systems.
- Complete Tunnel Construction (Stage 8)
    - Humboldt Parkway NB and SB operate normally. Shift Kensington Expressway EB traffic to EB side and commence normal operations (3 lanes in each direction) on the Kensington Expressway.
    - Install landscaping and trees.
- Remove temporary bridges and construct new roadway at existing cross street locations.

8. Cost Comparison

Table 3 presents a comparison of materials quantities and costs between the flat slab and double arched roof alternatives, estimated based on preliminary designs with loading conditions as defined in Section 4.

Table 3 Comparison of Arched and Flat Roof Alternatives

ITEM	ROOF TYPE	
	ARCH	FLAT SLAB
INTERIOR SPAN EACH SIDE	50'-0''	50'-0''
THICKNESS	3’-6”	3'-6''
RISE	7'-0''	N/A
CONCRETE QUANTITY PER LINEAL FOOT [CY]	14.90	14.30
CONCRETE UNIT PRICE	1,800.00	1,800.00
CONCRETE COST PER LINEAL FOOT	**26,820.00**	**25,740.00**
REINFORCEMENT QUANTITY PER LINEAL FOOT [LBS]	2,360	3,484
REINFORCEMENT UNIT PRICE	2.75	2.75
REINFORCEMENT COST PER LINEAL FOOT	**6,490.00**	**9,581.00**
TOTAL COST OF CONCRETE AND REINFORCEMENT	**33,310**	**35,321**

Quantity and cost estimates from preliminary analyses show that the arch roof results in material costs that are 6% lower than material costs for flat slab alternates.

9. Recommendations and Conclusions

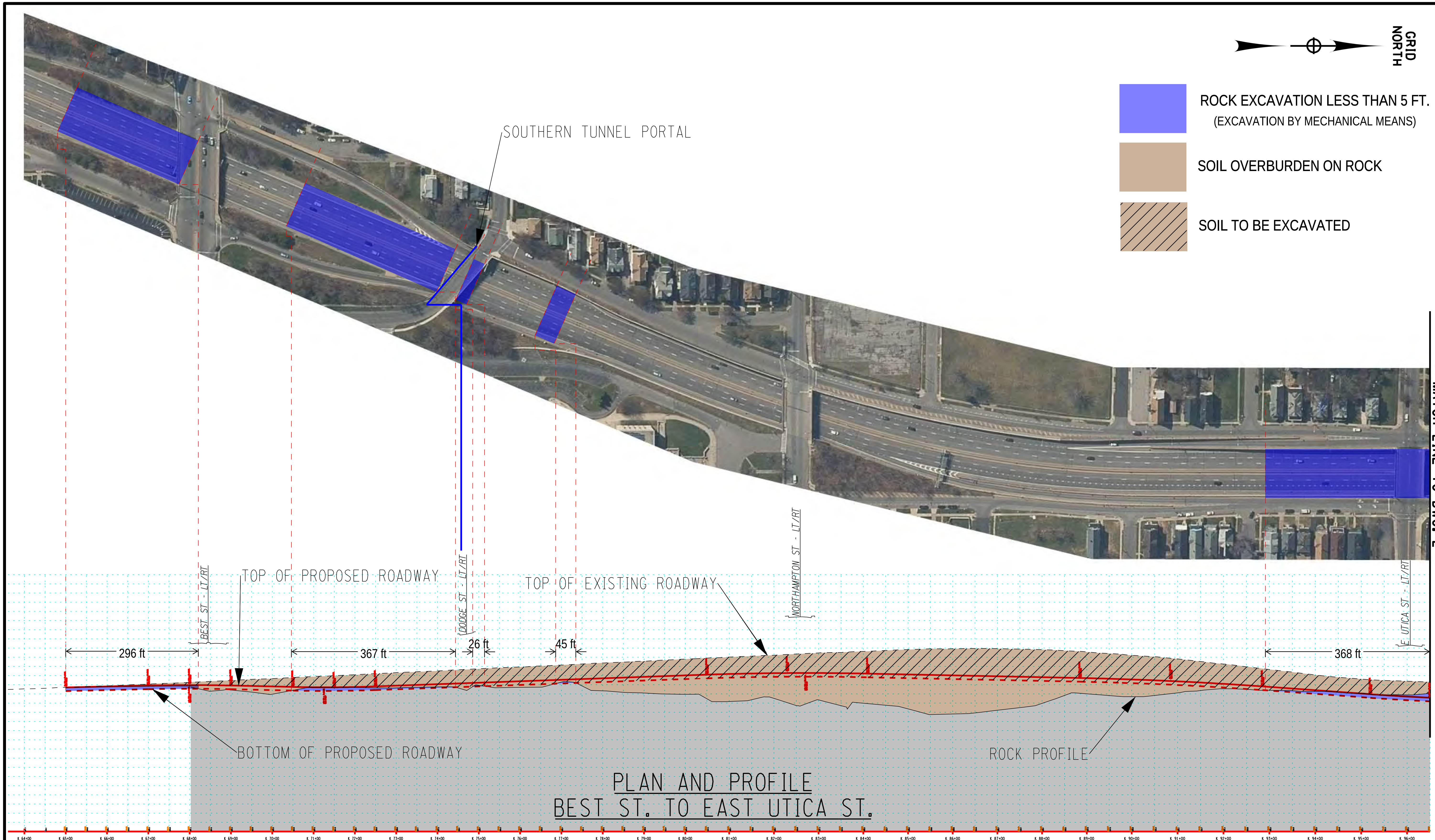
There is a relatively small material cost difference between the flat slab and arch roof/superstructure solutions. These material costs do not account for the following additional retaining walls costs for the arch solution:

- Independent drainage system for the full length of the tunnel that would be required to properly drain the trough between the two arches.
- Additional material costs for center wall to provide sufficient capacity to withstand horizontal thrust from arch dead load imbalanced during construction.
- Additional soil excavation and rock removal costs to maintain minimum 3′-6″ soil overburden over arch and avoid root penetration into drainage and insulation layers and structure.
- Additional material costs for exterior and center walls for additional overburden and deeper tunnel invert elevation to maintain minimum 3′-6″ soil overburden over arch and avoid root penetration into drainage and insulation layers and structure.

Additionally, the flat roof slab option offers substantial construction phasing and staging benefits as compared to the arch solution, reducing impacts to the traveling public and improving constructability and efficiency for the contractor.

Based on the above, we recommend a flat slab construction for the tunnel cap/superstructure. For the retaining walls, we recommend reinforced concrete walls with a braced (roof slab) design on foundations consisting of secant piles embedded into rock to effectively and efficiently cutoff groundwater seepage into the tunnel. Finally, we recommend that these braced walls be designed to accommodate an unbraced length of 20 feet to allow for future roof slab replacement in segments along the length of the tunnel without necessitating additional bracing of the retaining walls.

Appendix A:
Soil and Rock Excavation
Plans and Profiles



PLAN AND PROFILE
BEST ST. TO EAST UTICA ST.

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AS-BUILT REVISIONS DESCRIPTION OF ALTERATIONS:	COUNTY: REGION:	PIN	BRIDGES	CULVERTS	ALL DIMENSIONS IN ft UNLESS OTHERWISE NOTED	CONTRACT NUMBER DRAWING NO. SHEET NO.

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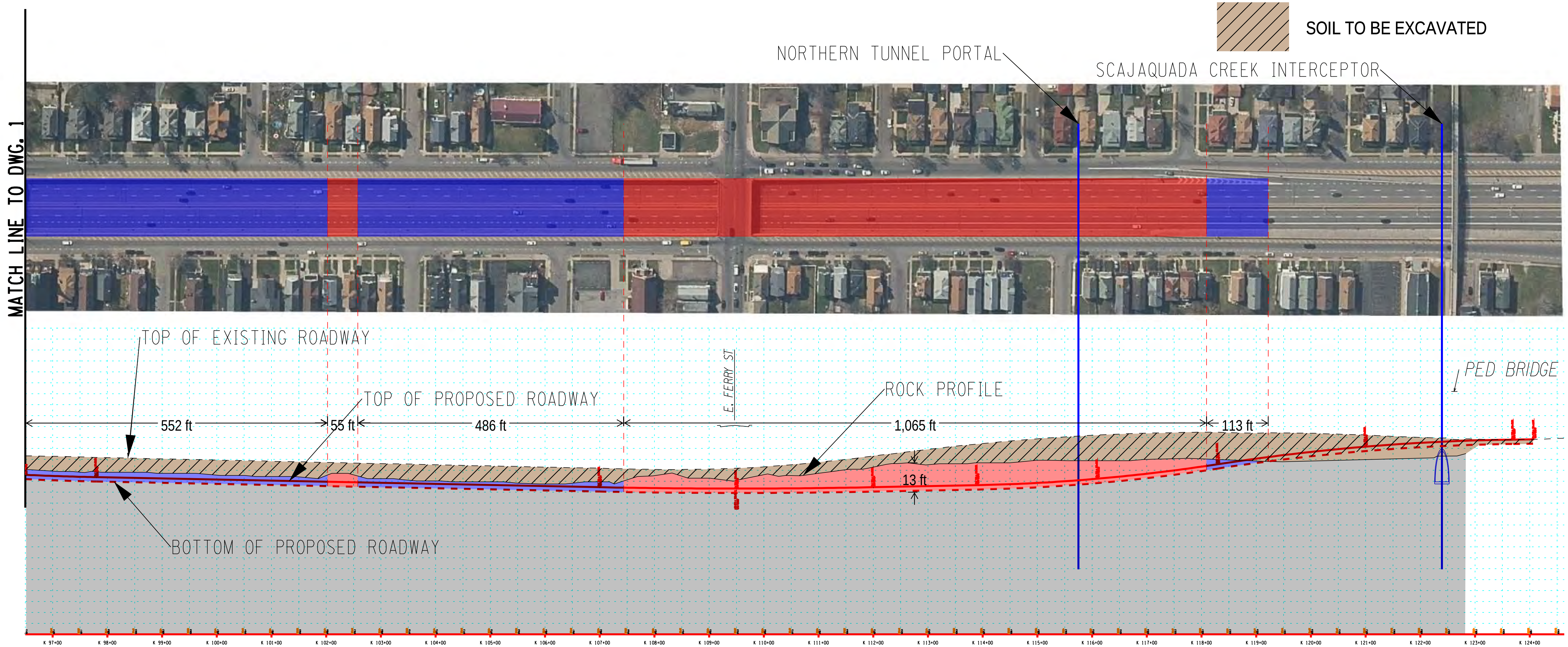
GRID NORTH

ROCK EXCAVATION LESS THAN 5 FT.
(EXCAVATION BY MECHANICAL MEANS)

ROCK EXCAVATION IN EXCESS OF 5 FT.
(BLASTING OF ROCK LIKELY; SEE NOTE 1)
MAXIMUM DEPTH = 13 FT.

SOIL OVERBURDEN ON ROCK

SOIL TO BE EXCAVATED



PLAN AND PROFILE
EAST UTICA ST. TO PED BRIDGE

NOTE:

1. LOCATION AND LIMITS OF BLASTING WILL BE DETERMINED DURING CONSTRUCTION WITH THE DEVELOPMENT OF A BLASTING PLAN. THE BLASTING PLAN WILL TAKE INTO ACCOUNT PROXIMITY TO EXISTING AND PROPOSED STRUCTURES AND ESTABLISHED VIBRATION LIMITS PRIOR TO ANY BLASTING OPERATIONS.

AS-BUILT REVISIONS DESCRIPTION OF ALTERATIONS:	COUNTY: REGION:	PIN	BRIDGES	CULVERTS	ALL DIMENSIONS IN ft UNLESS OTHERWISE NOTED	CONTRACT NUMBER / DRAWING NO. / SHEET NO.

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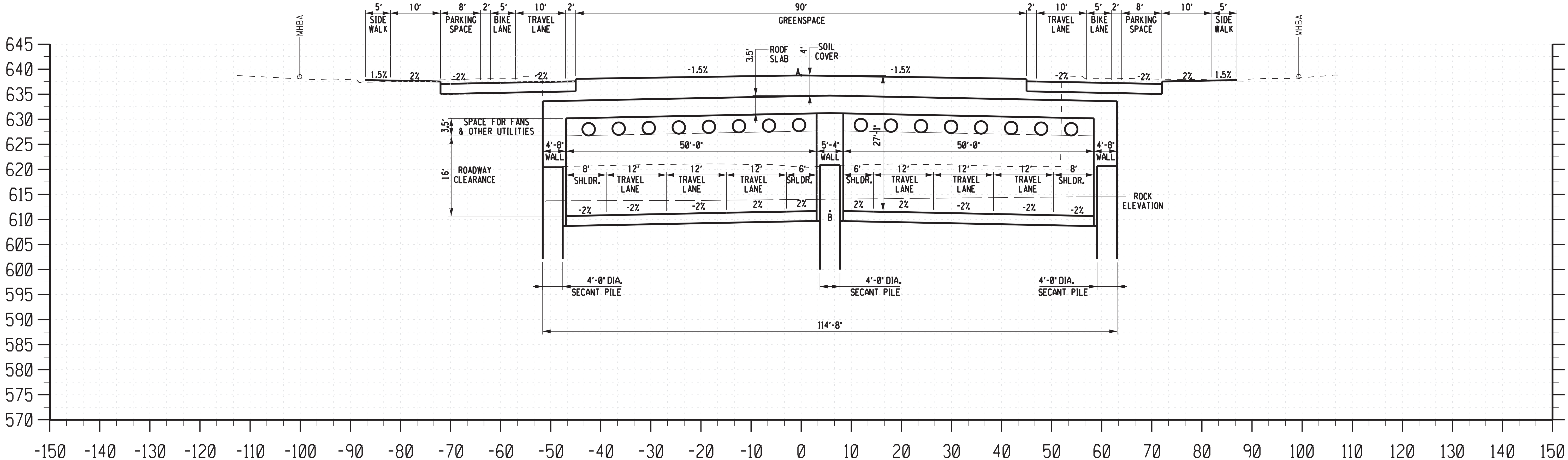
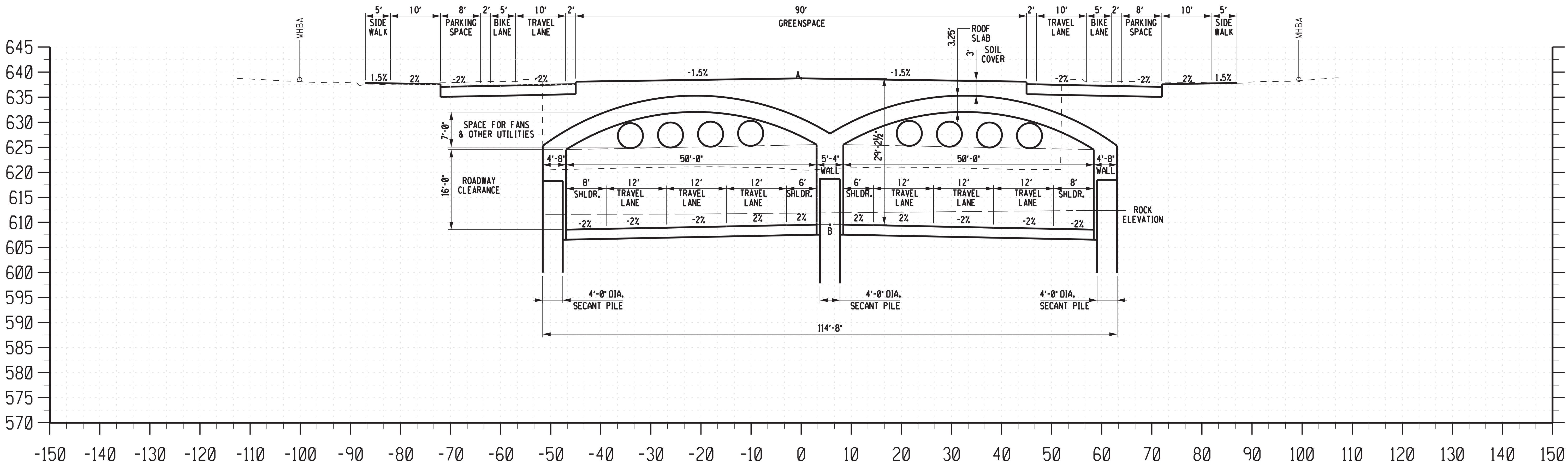
Appendix B:
Proposed Cross Sections
Flat Slab and Arch Alternatives

P.I.N.
CONTRACT NO.
ALIGNMENT:



TYPICAL NORMAL SECTION (STA. 98+00 SHOWN) - FLAT ROOF SLAB WITH 4' SECANT PILES



TYPICAL NORMAL SECTION (STA. 98+00 SHOWN) - FLAT ROOF SLAB WITH 4' SECANT PILES

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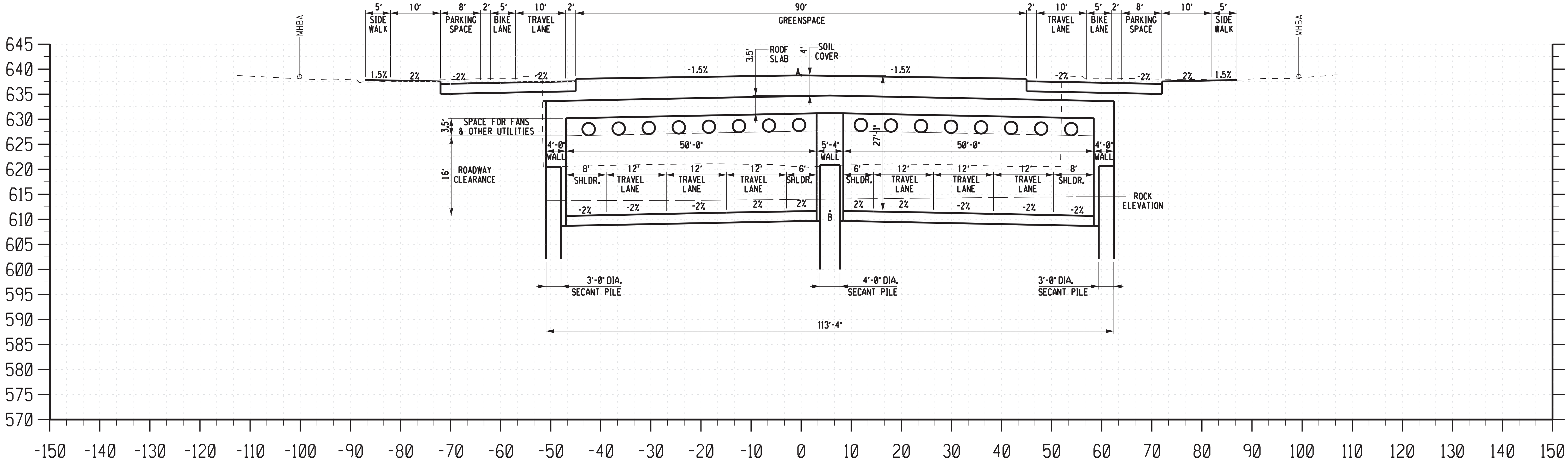
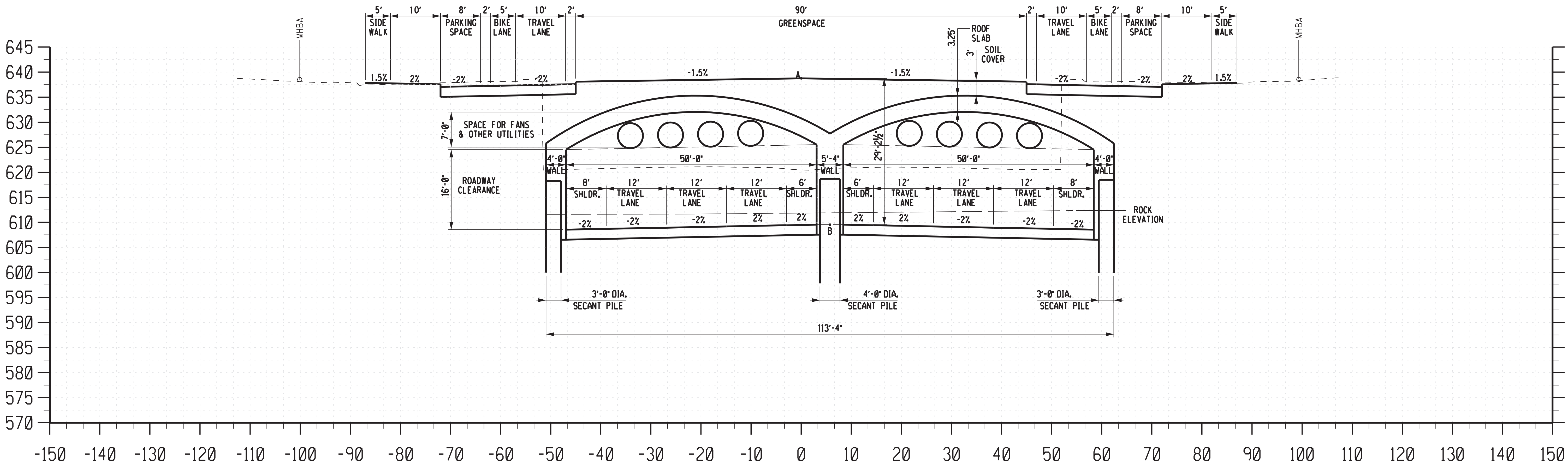


TYPICAL NORMAL SECTION (STA. 98+00 SHOWN) - FLAT ROOF SLAB WITH 3' SECANT PILES



TYPICAL NORMAL SECTION (STA. 98+00 SHOWN) - FLAT ROOF SLAB WITH 3' SECANT PILES

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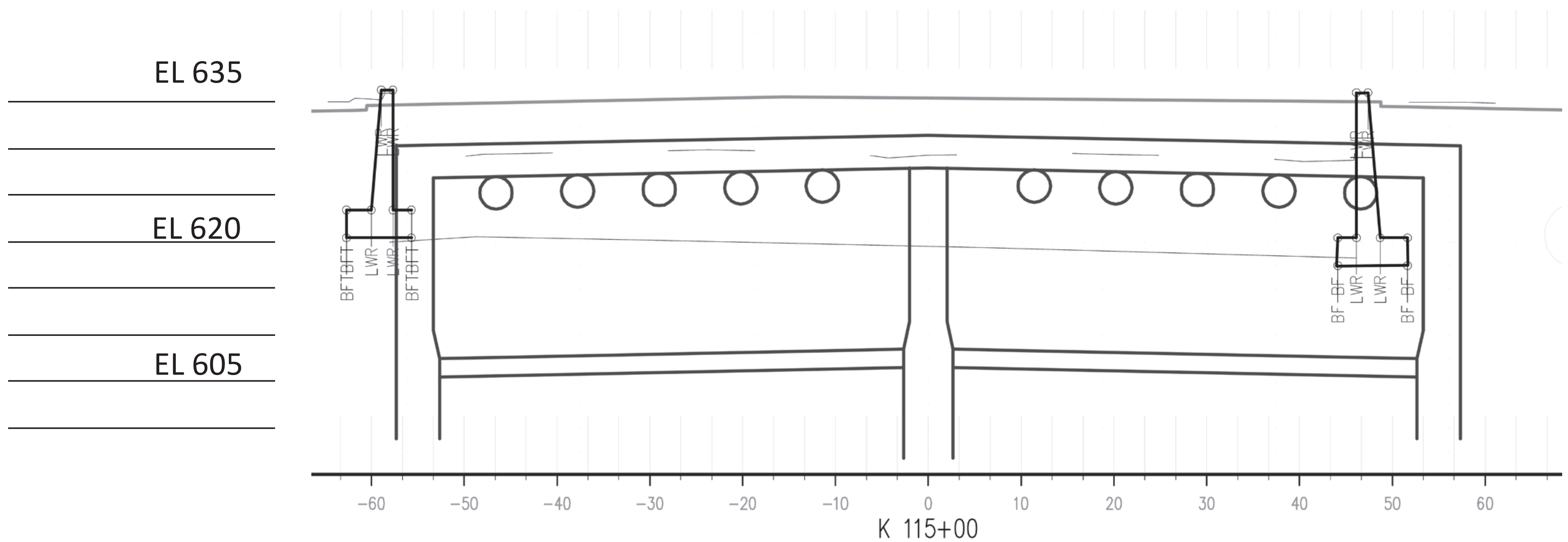
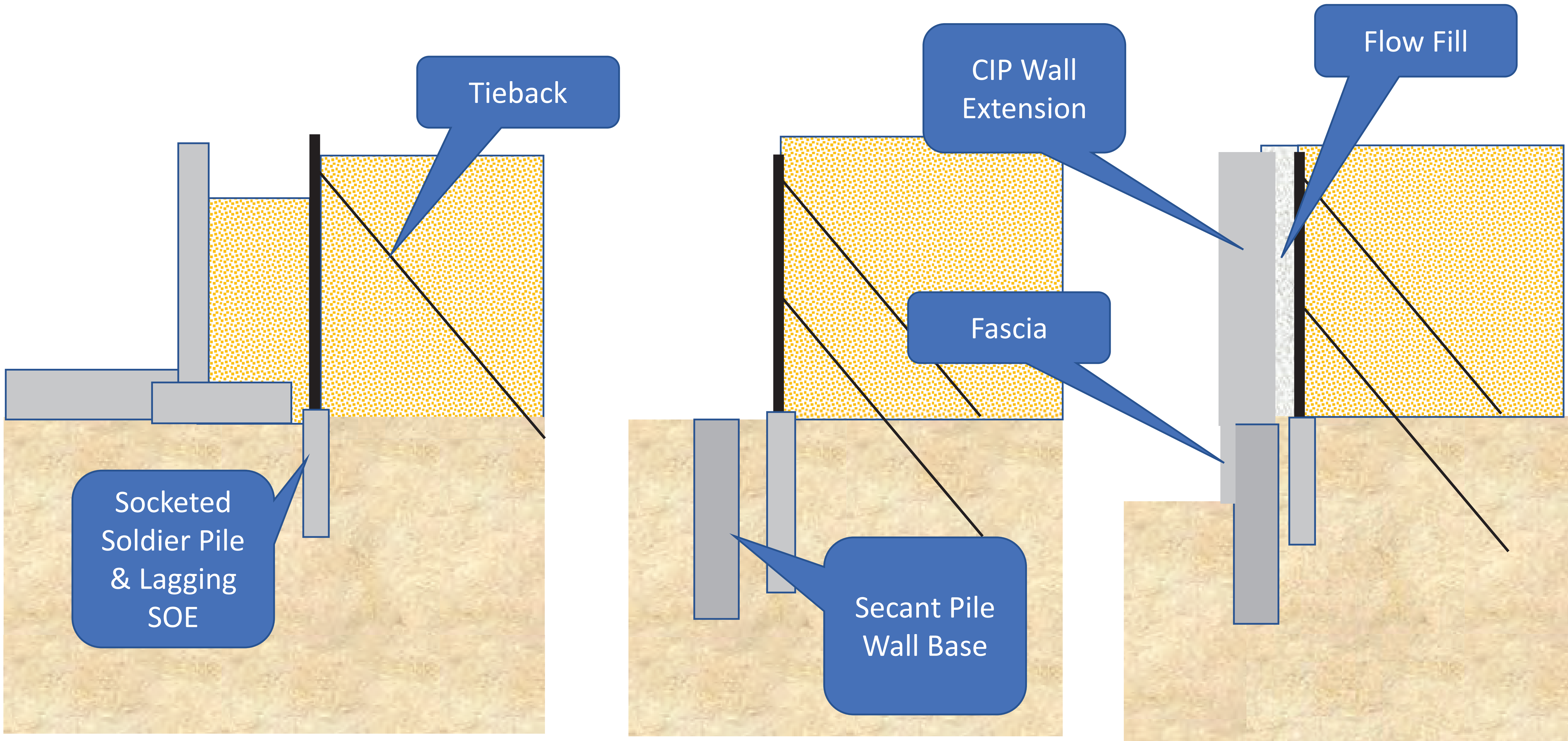
Appendix C:
Retaining Wall Concepts in High Rock Areas

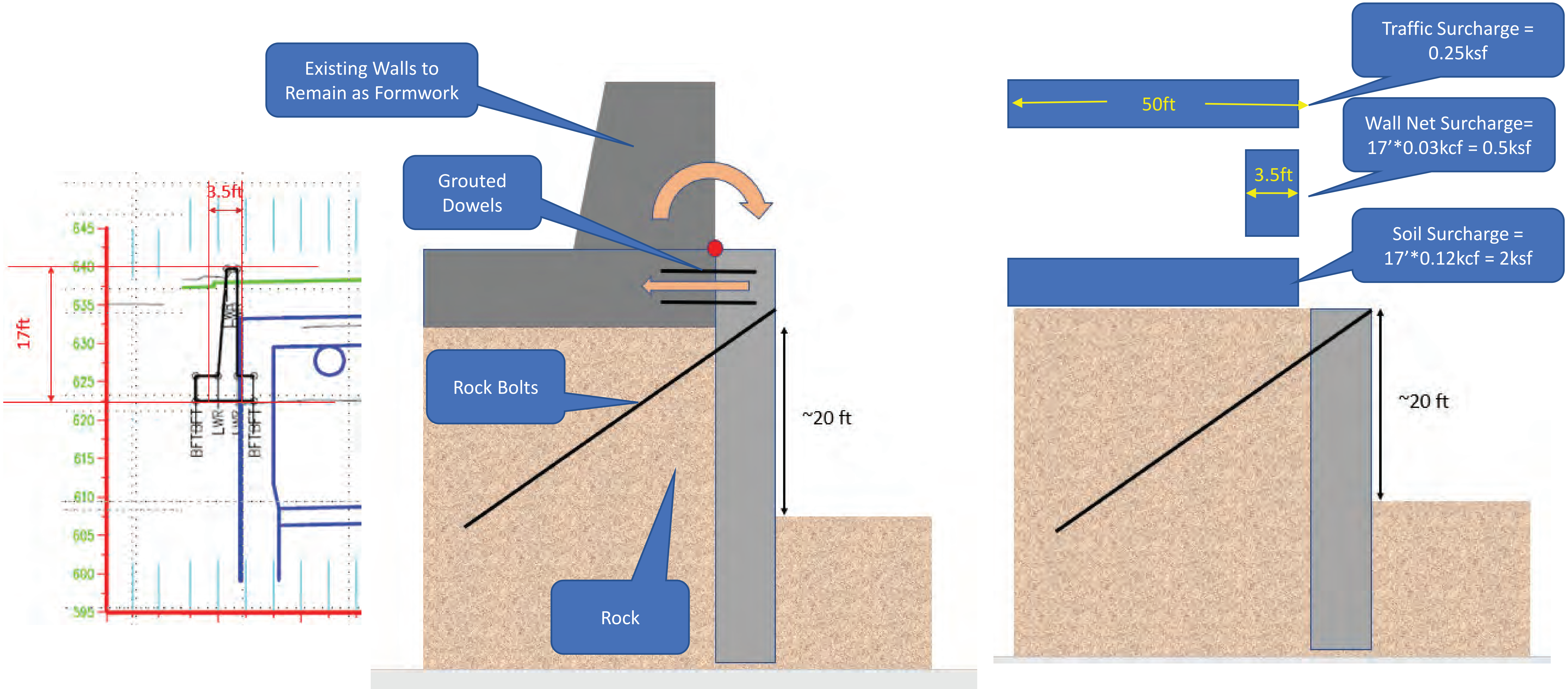
Underground Structure for Sta 115+00 (High Rock Area)

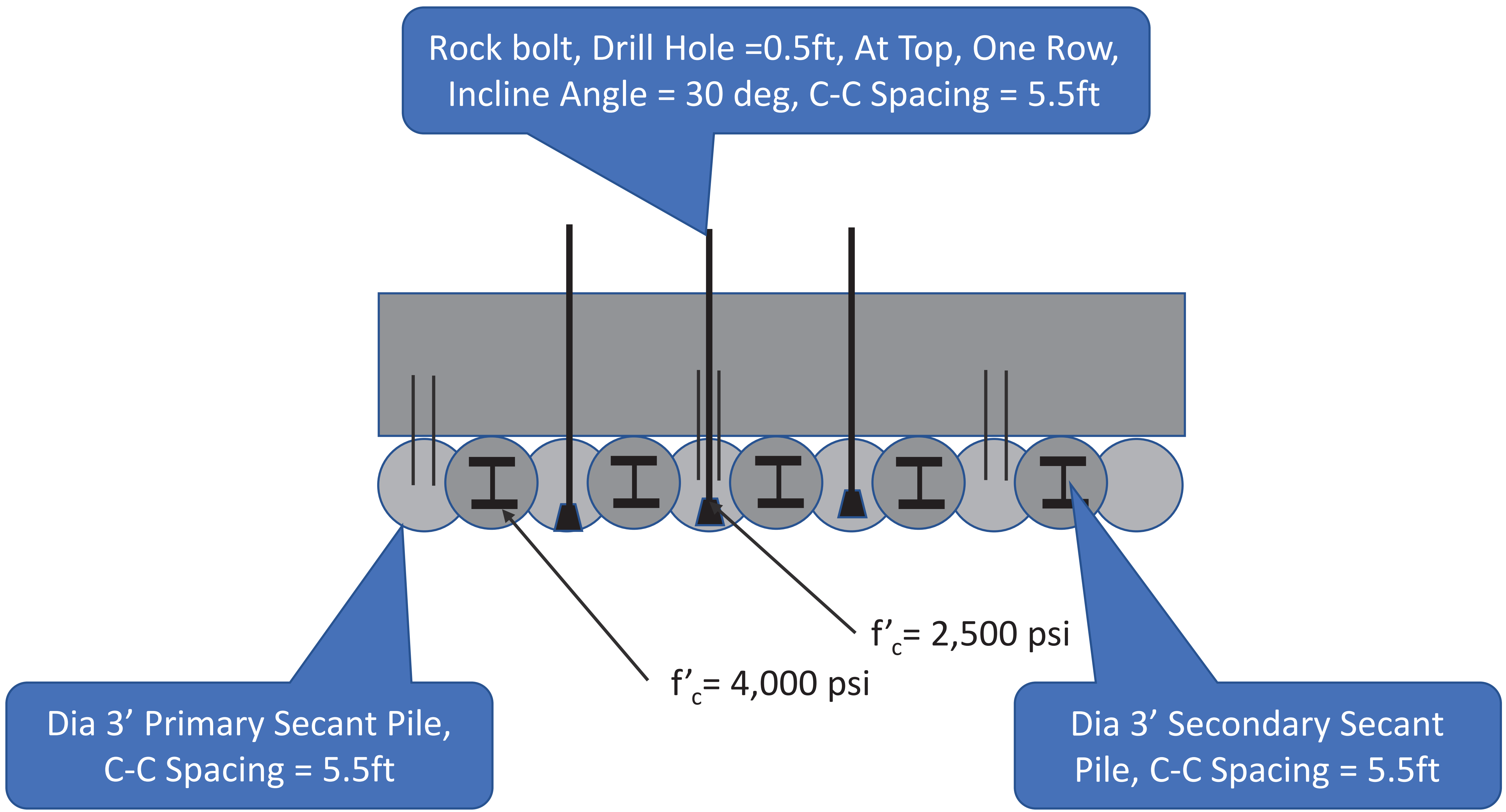


East Walls Replacement with Secant Piles and CIP Walls

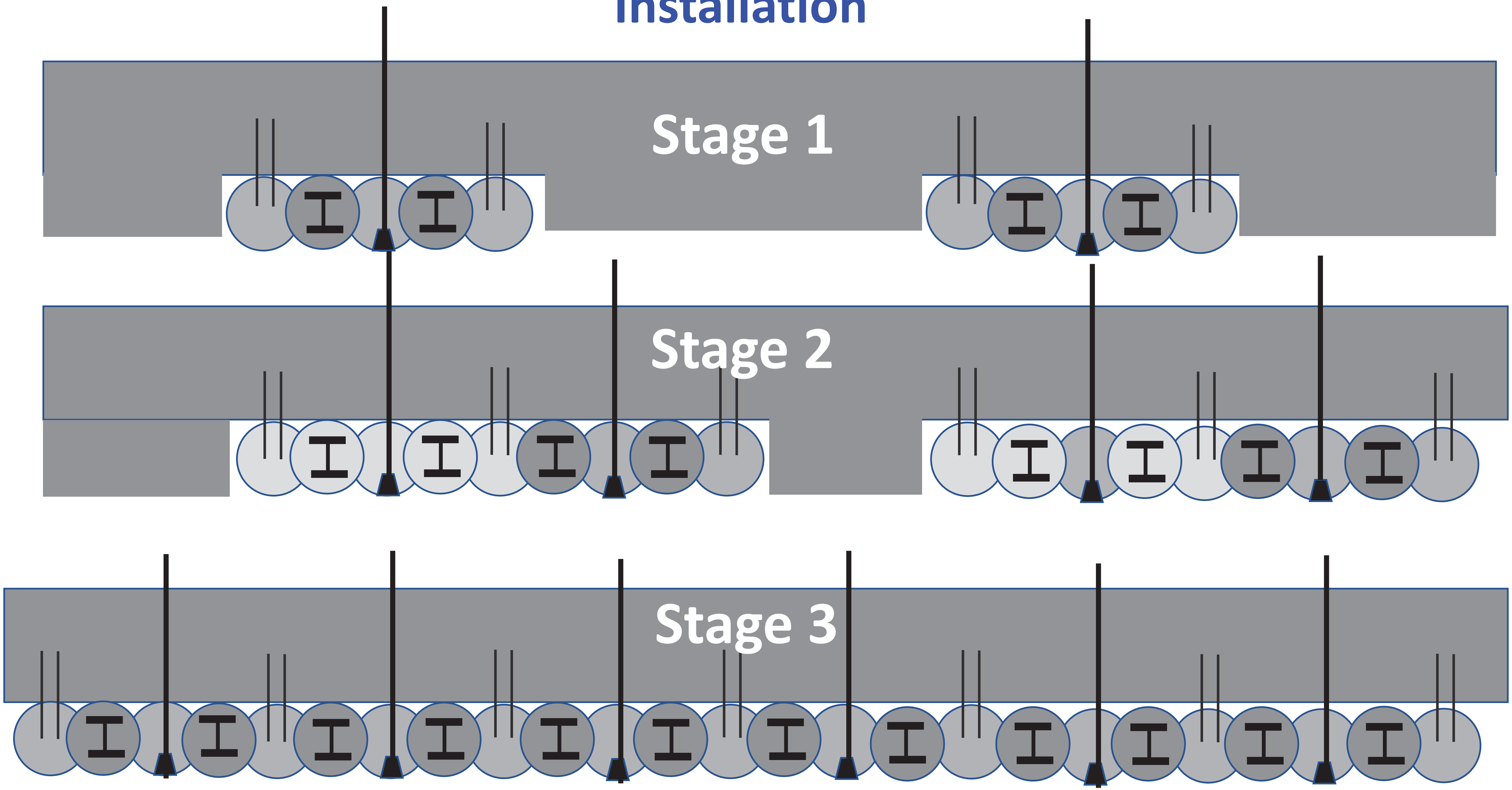


West Walls Underpinning with Secant Piles



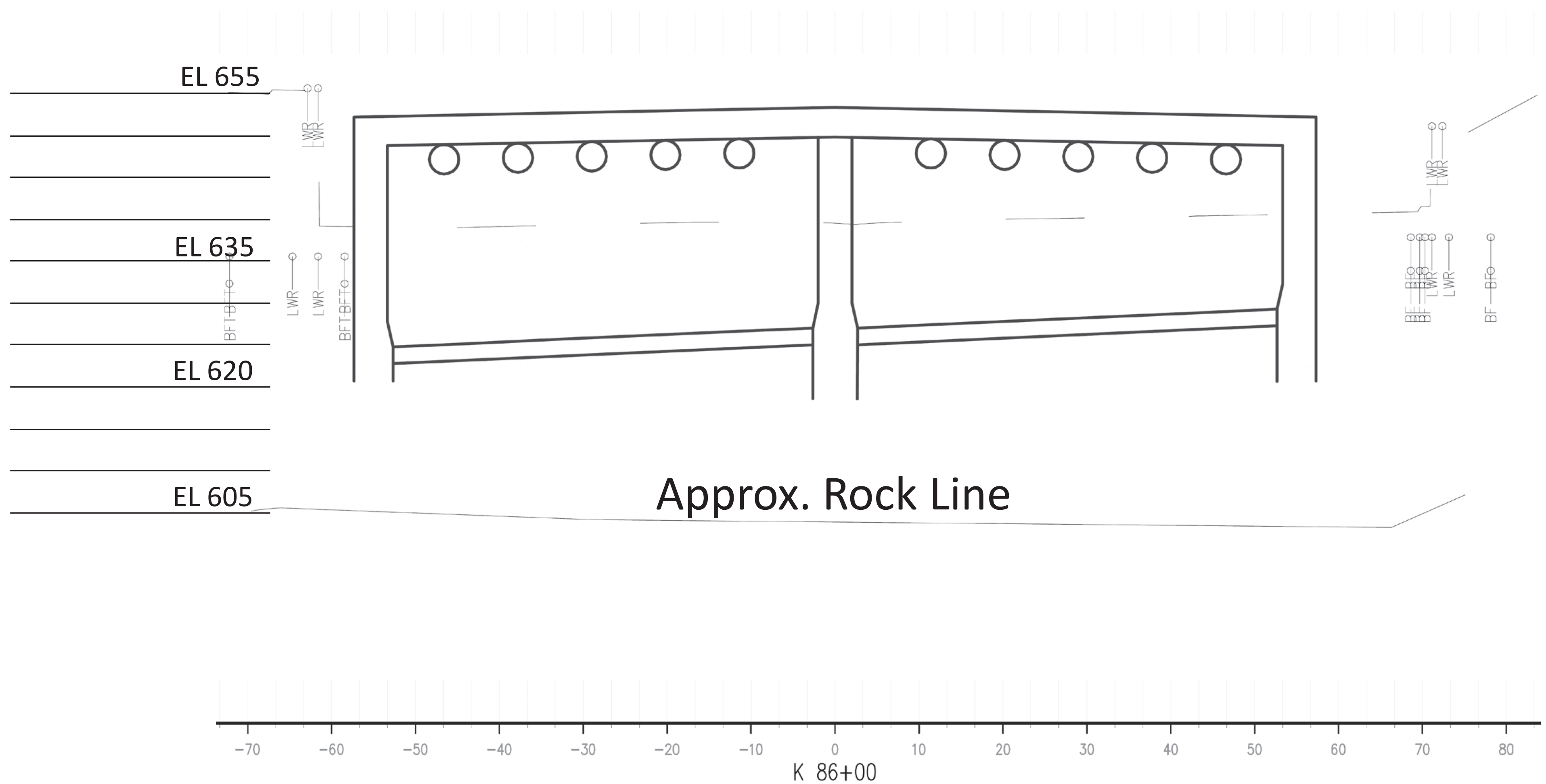

Rock bolt, Drill Hole =0.5ft, At Top, One Row, Incline Angle = 30 deg, C-C Spacing = 5.5ft
f'c= 2,500 psi
f'c= 4,000 psi
Dia 3' Primary Secant Pile, C-C Spacing = 5.5ft
Dia 3' Secondary Secant Pile, C-C Spacing = 5.5ft

Staged Removal of Existing Wall Toe and Secant Pile Installation

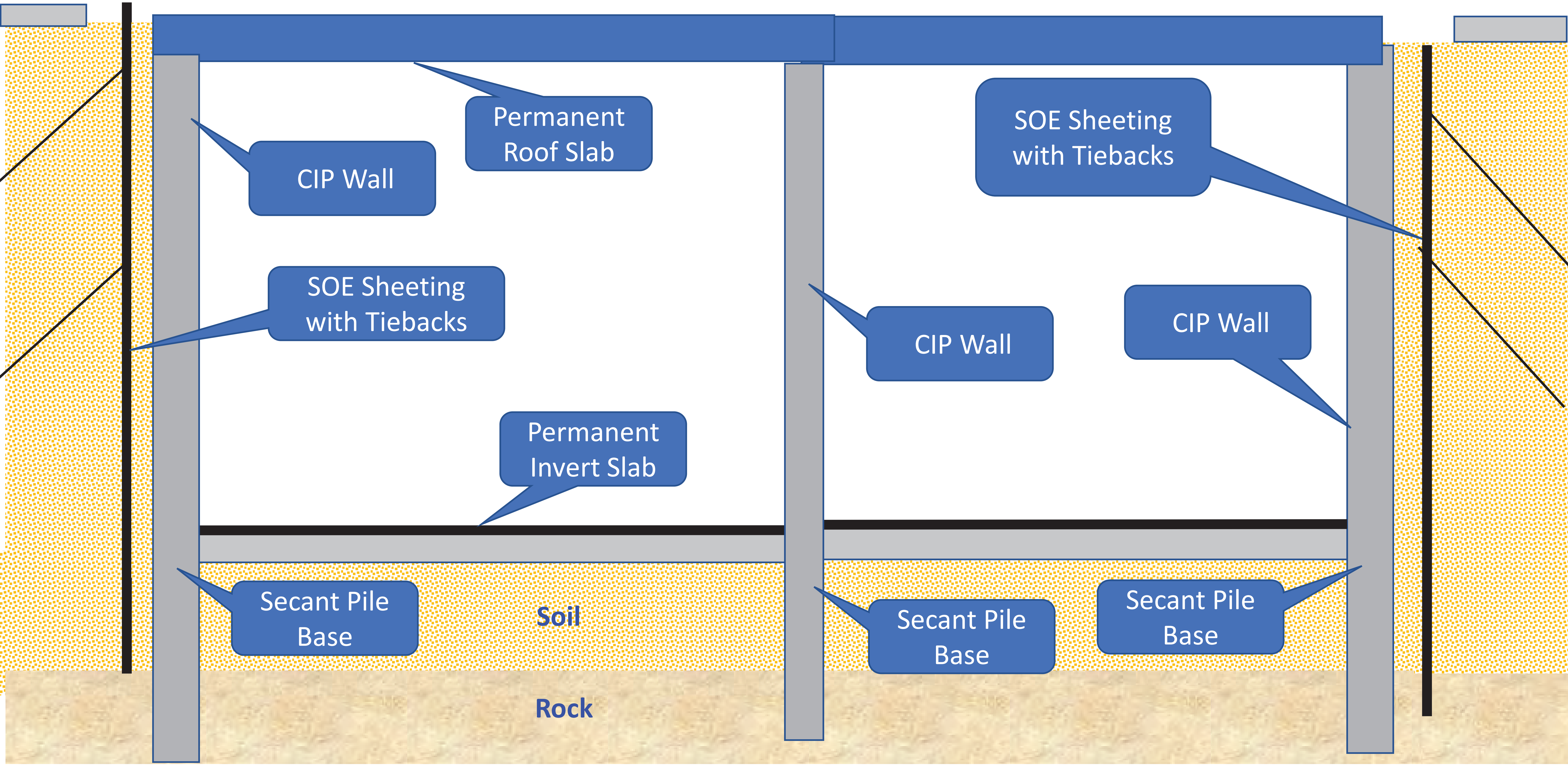


Appendix D:
Retaining Wall Concepts in Low Rock Areas

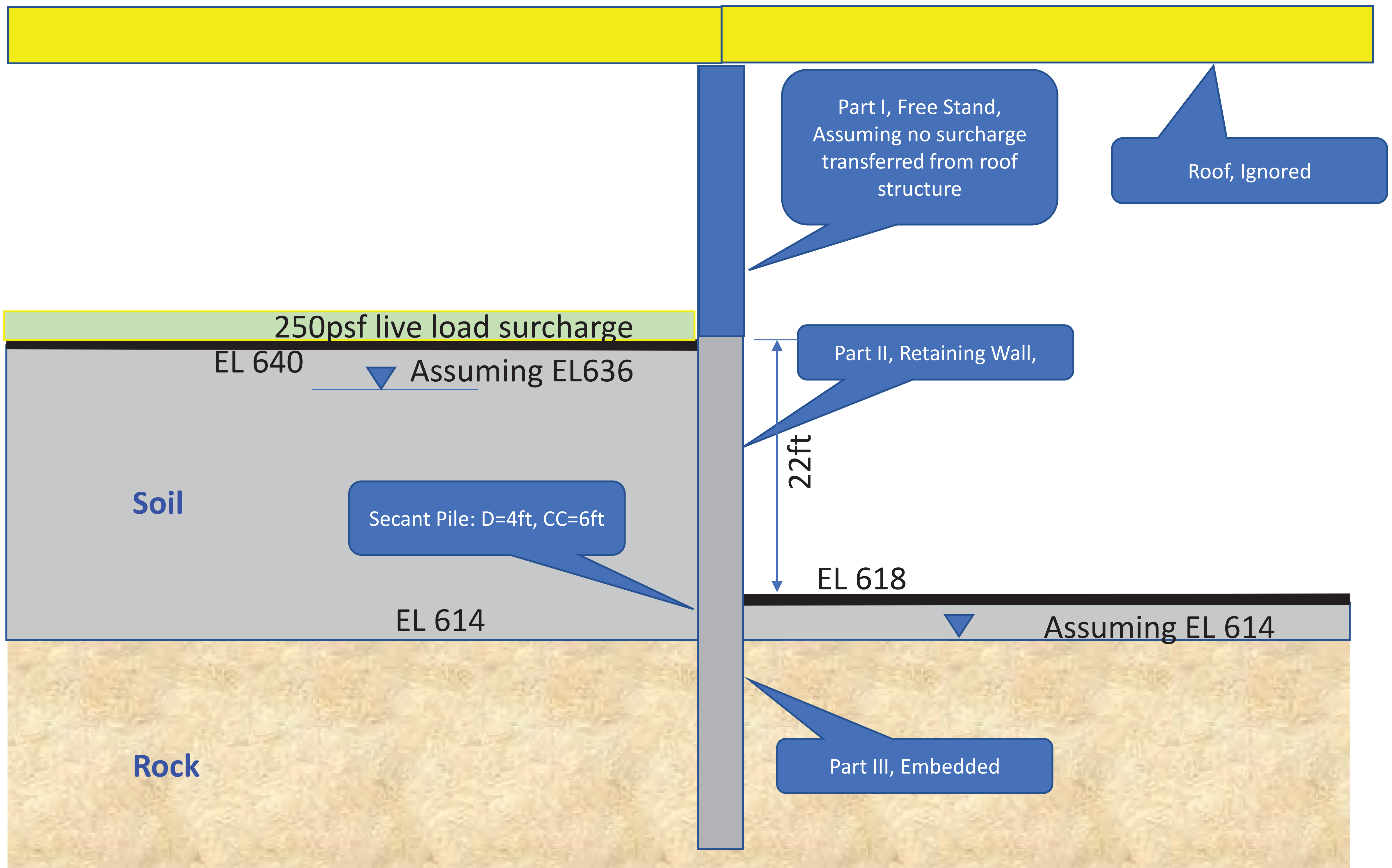
Underground Structure for Sta 86+00 (Low Rock Area)



Proposed Permanent and SOE for Low Rock Areas

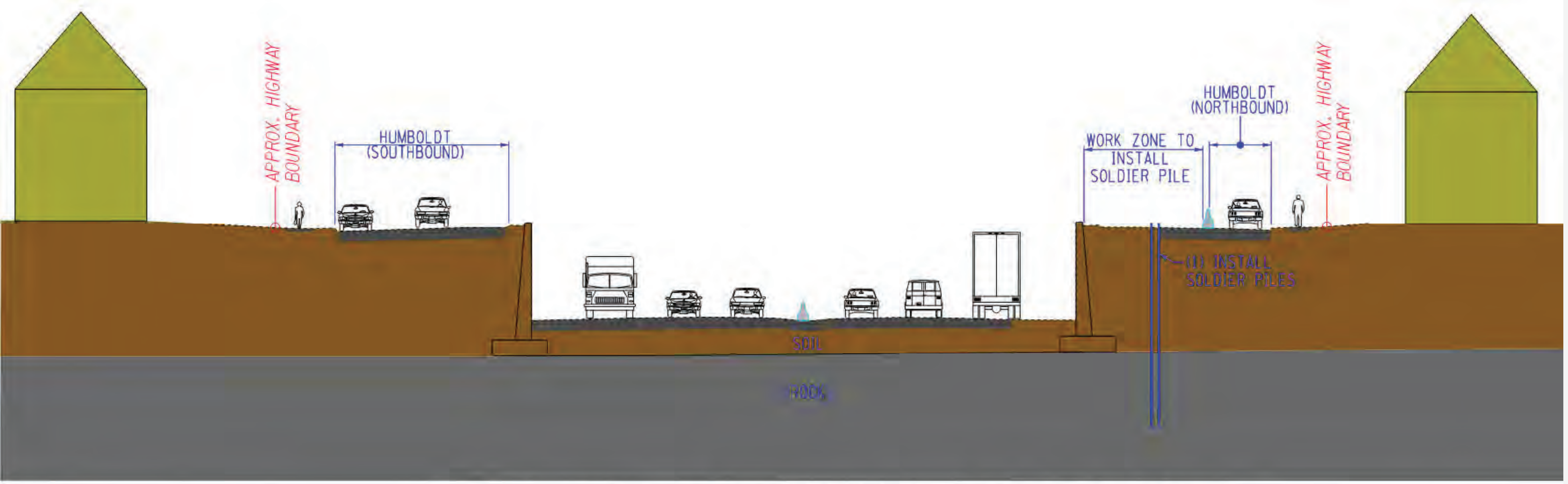


Center Wall – Temporary Load Case during Construction

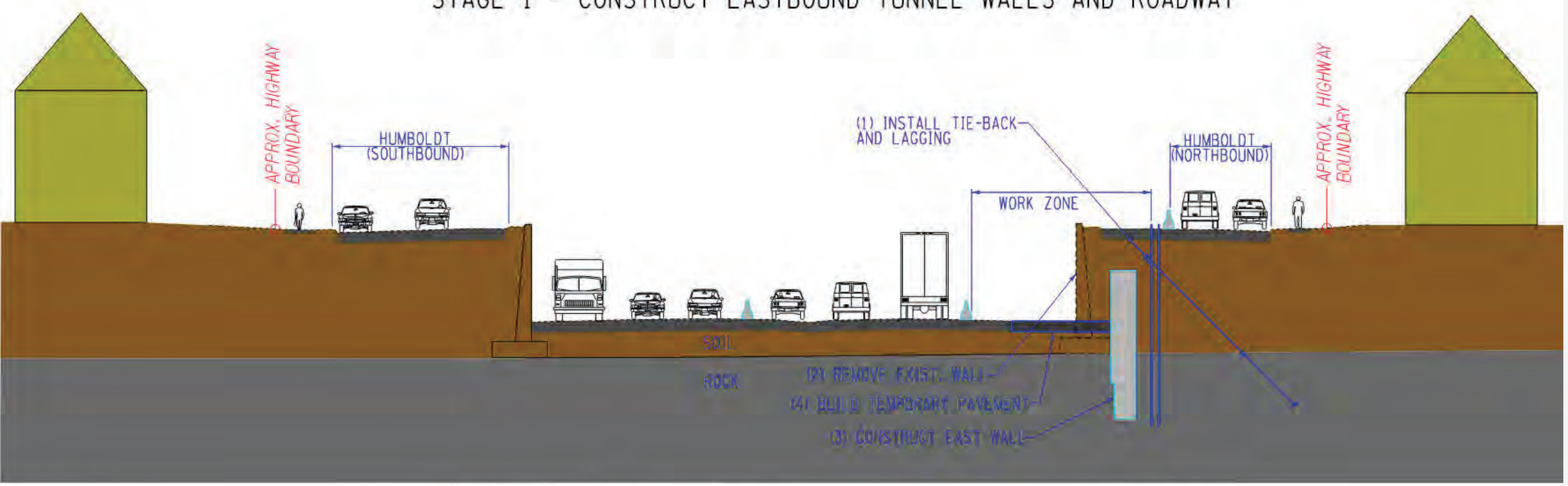


Appendix E:
Construction Staging Sections

Construction Staging Sections



STAGE 1 - CONSTRUCT EASTBOUND TUNNEL WALLS AND ROADWAY



STAGE 2 - CONSTRUCT EASTBOUND TUNNEL WALLS AND ROADWAY

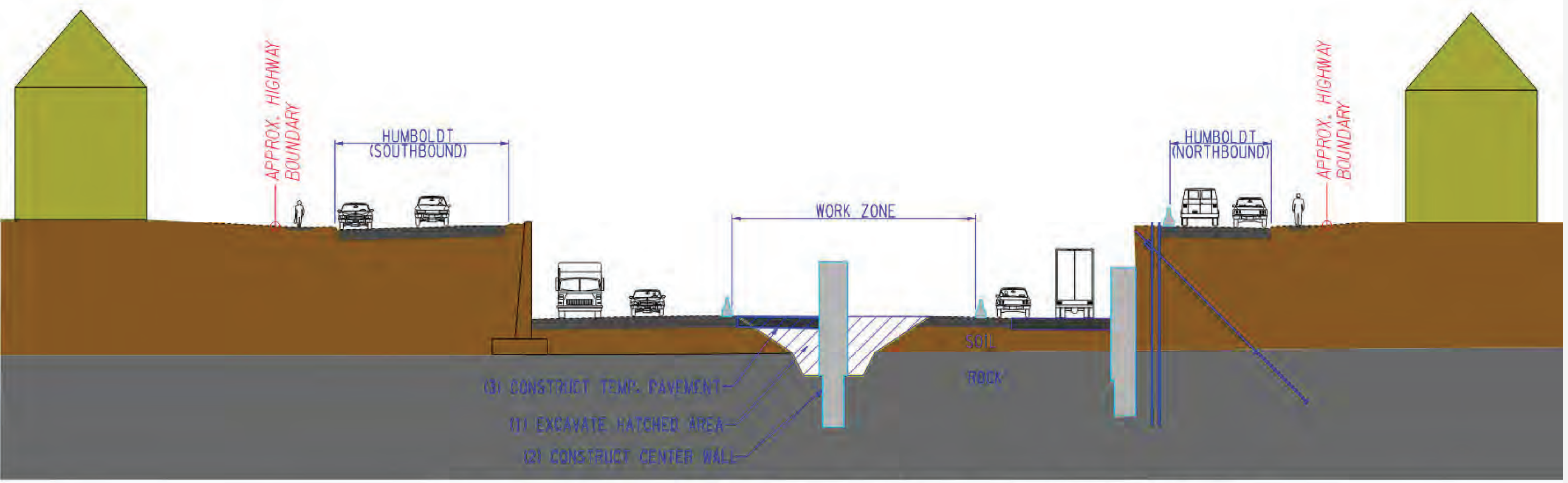
<u>Stage 1 traffic notes:</u>

1) Humboldt SB operates normally
2) Kensington operates normally
3) Humboldt NB temp. impacts

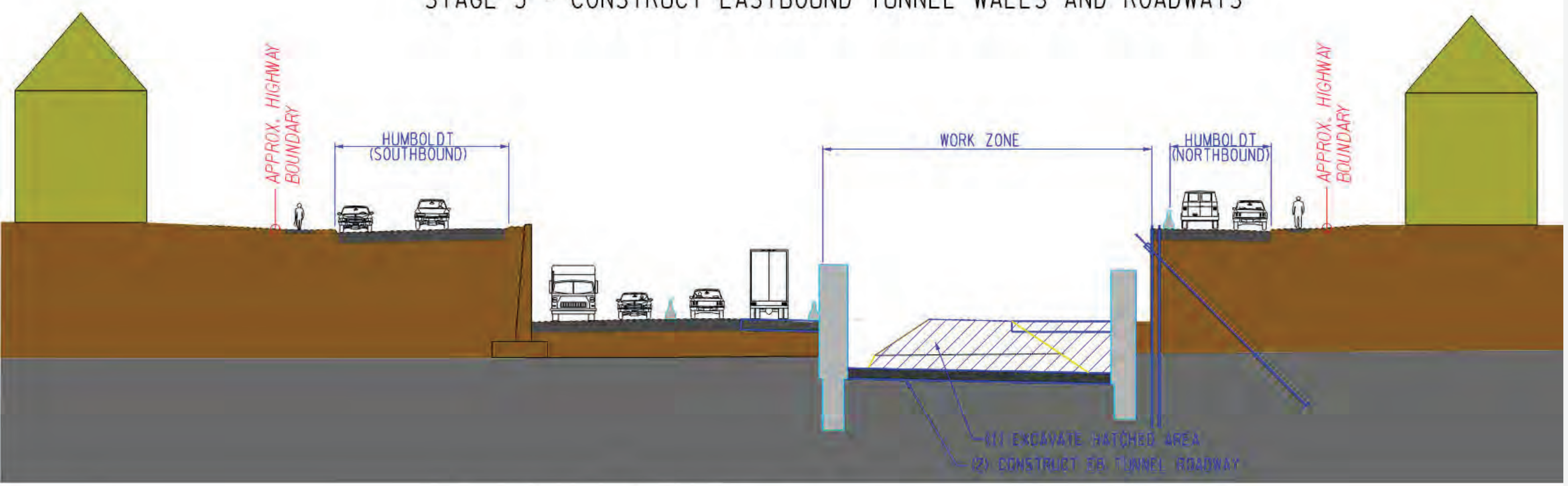
<u>Stage 2 traffic notes:</u>

1) Humboldt SB operates normally
2) Kensington operates normally (shifted, reduced shoulder)
3) Humboldt NB (reduced width)

Construction Staging Sections



STAGE 3 - CONSTRUCT EASTBOUND TUNNEL WALLS AND ROADWAYS



STAGE 4 - CONSTRUCT EASTBOUND TUNNEL WALLS AND ROADWAY

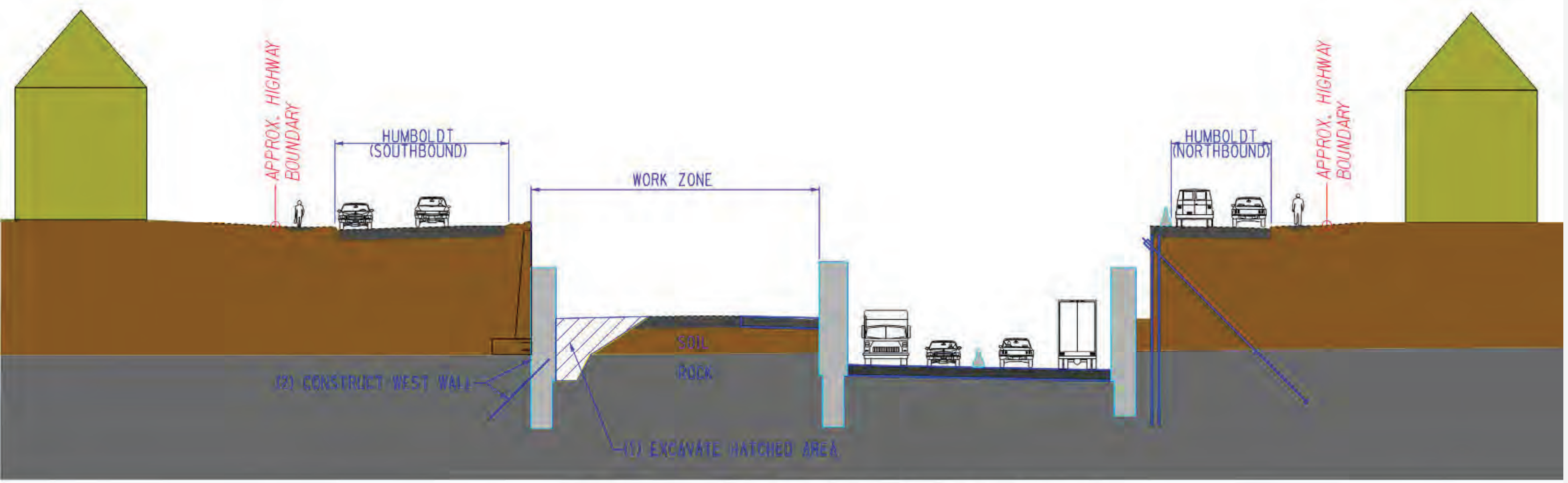
Stage 3 traffic notes:

1) Humboldt SB operates normally
2) Kensington 2-lanes each direction (as-shown)
3) Humboldt NB (reduced width)

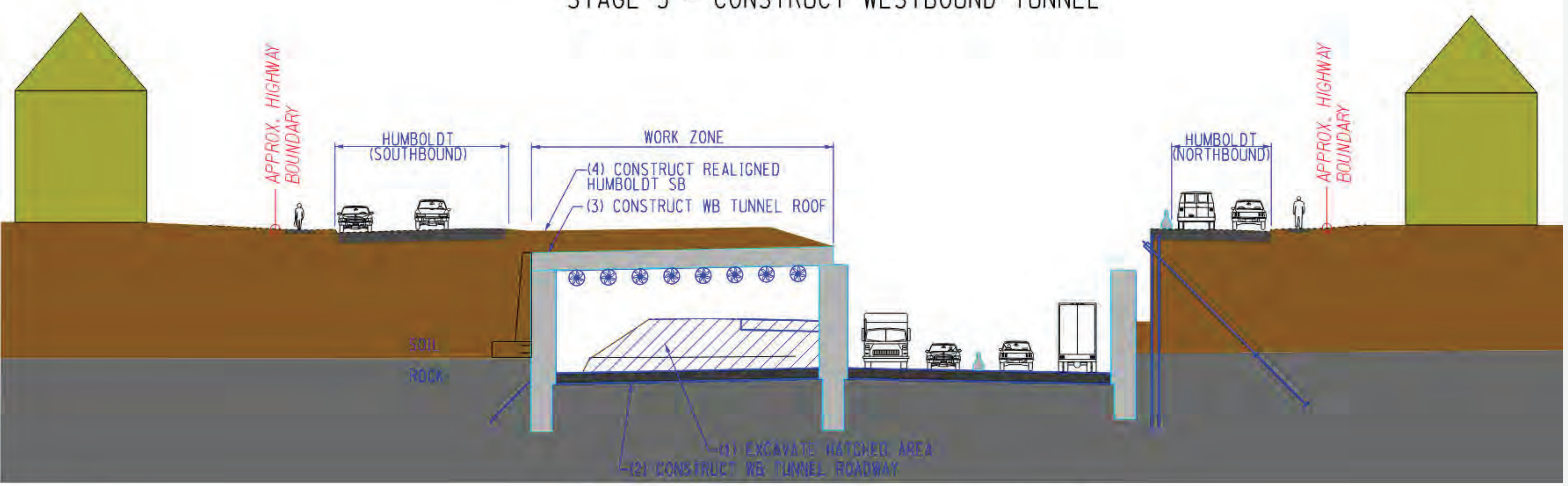
Stage 4 traffic notes:

1) Humboldt SB operates normally
2) Kensington 2-lanes each direction (as-shown)
3) Humboldt NB (reduced width)

Construction Staging Sections



STAGE 5 - CONSTRUCT WESTBOUND TUNNEL



STAGE 6 - CONSTRUCT WESTBOUND TUNNEL

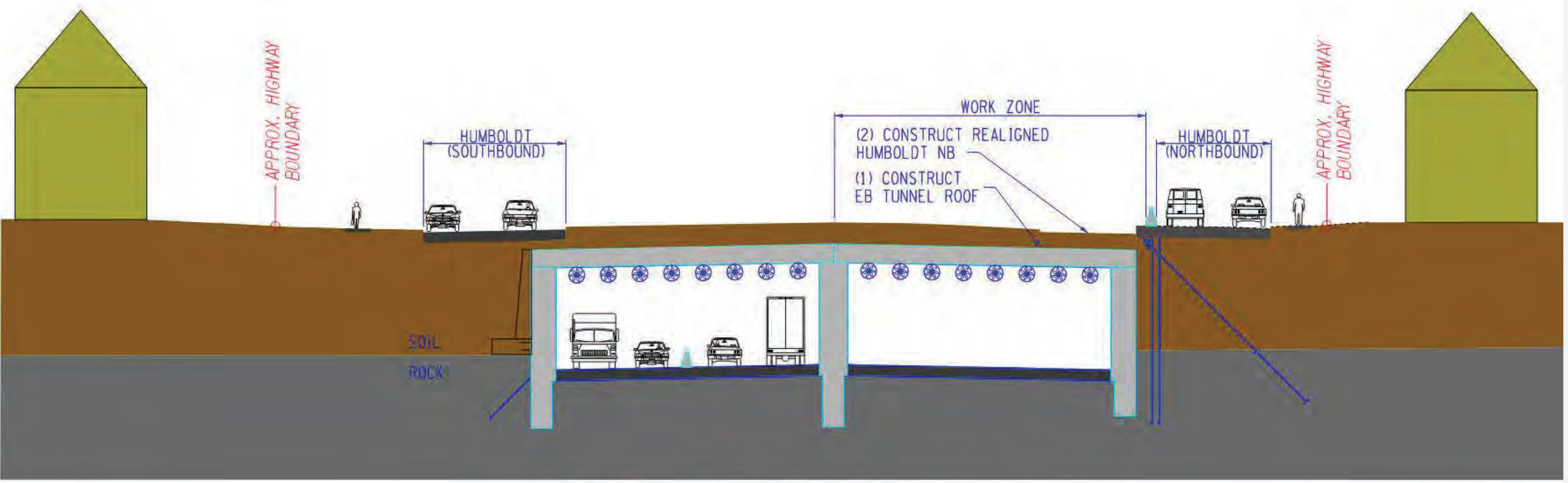
Stage 5 traffic notes:

1) Humboldt SB operates normally
2) Kensington 2-lanes each direction (as-shown)
3) Humboldt NB (reduced width)

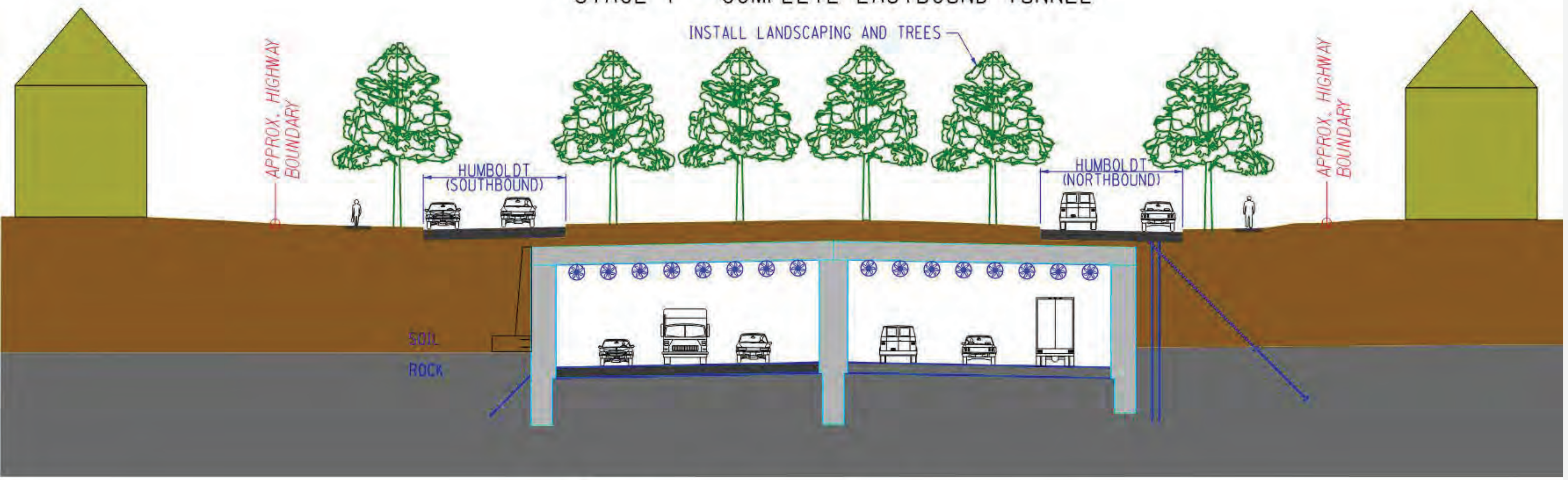
Stage 6 traffic notes:

1) Humboldt SB operates normally
2) Kensington 2-lanes each direction (as-shown)
3) Humboldt NB (reduced width)

Construction Staging Sections



STAGE 7 - COMPLETE EASTBOUND TUNNEL



STAGE 8 - COMPLETE TUNNEL CONSTRUCTION

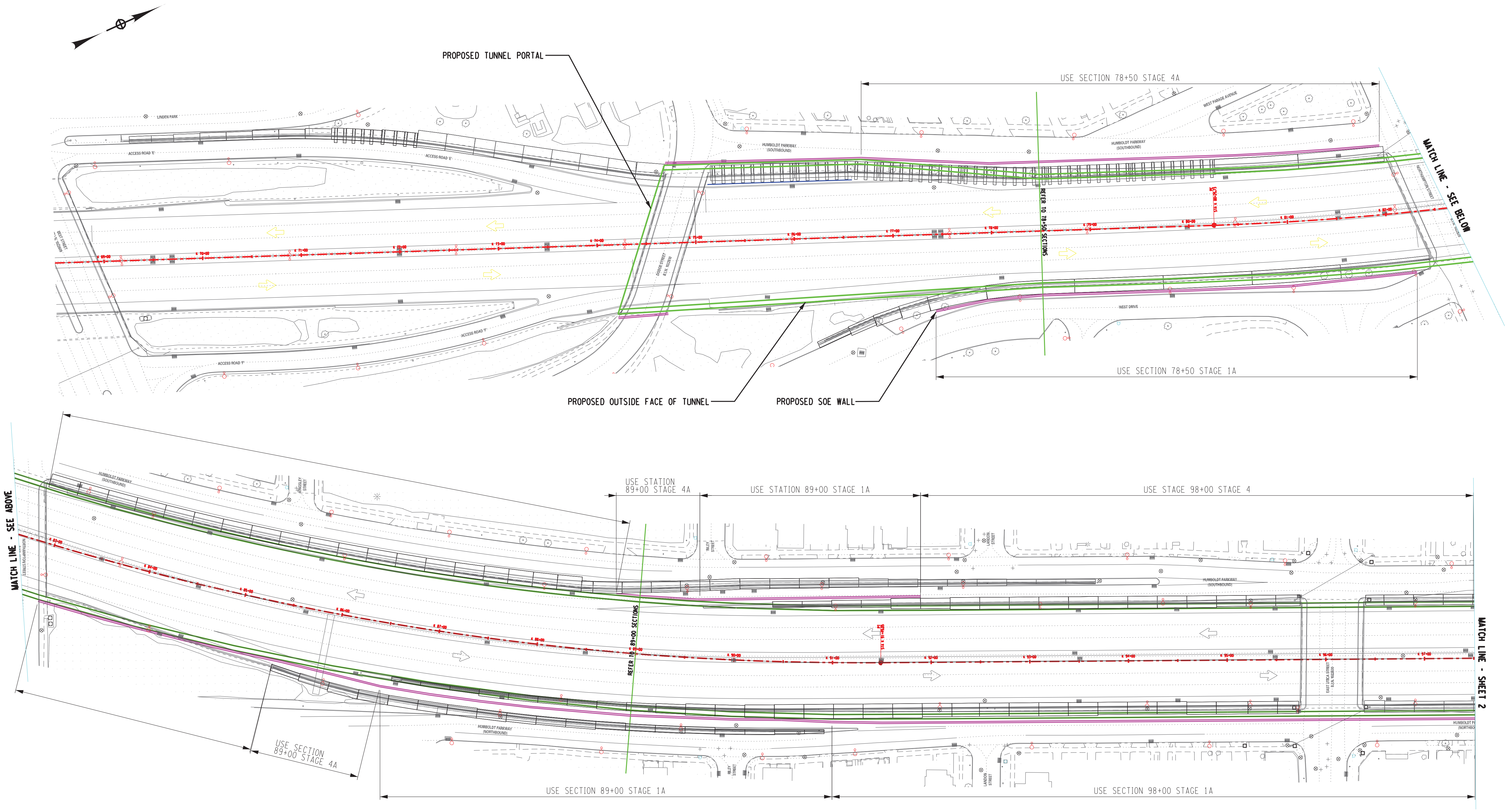
Stage 7 traffic notes:

1) Humboldt SB operates normally
2) Kensington 2-lanes each direction (as-shown)
3) Humboldt NB (reduced width)

Stage 8 traffic notes:

1) Humboldt SB operates normally
2) Kensington 3-lanes each direction (as-shown)
3) Humboldt NB operates normally

Appendix F:
Temporary and Permanent Retaining Wall Plans



PROPOSED TUNNEL WALLS

TEMPORARY SOE WALLS

SCALE: 1" = 100'

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AS-BUILT REVISIONS DESCRIPTION OF ALTERATIONS:		PIN	BRIDGES	CULVERTS	ALL DIMENSIONS IN ft UNLESS OTHERWISE NOTED	CONTRACT NUMBER
	COUNTY: REGION:				WZTC OVERVIEW PLANS	DRAWING NO. SHEET NO. 1 OF 2

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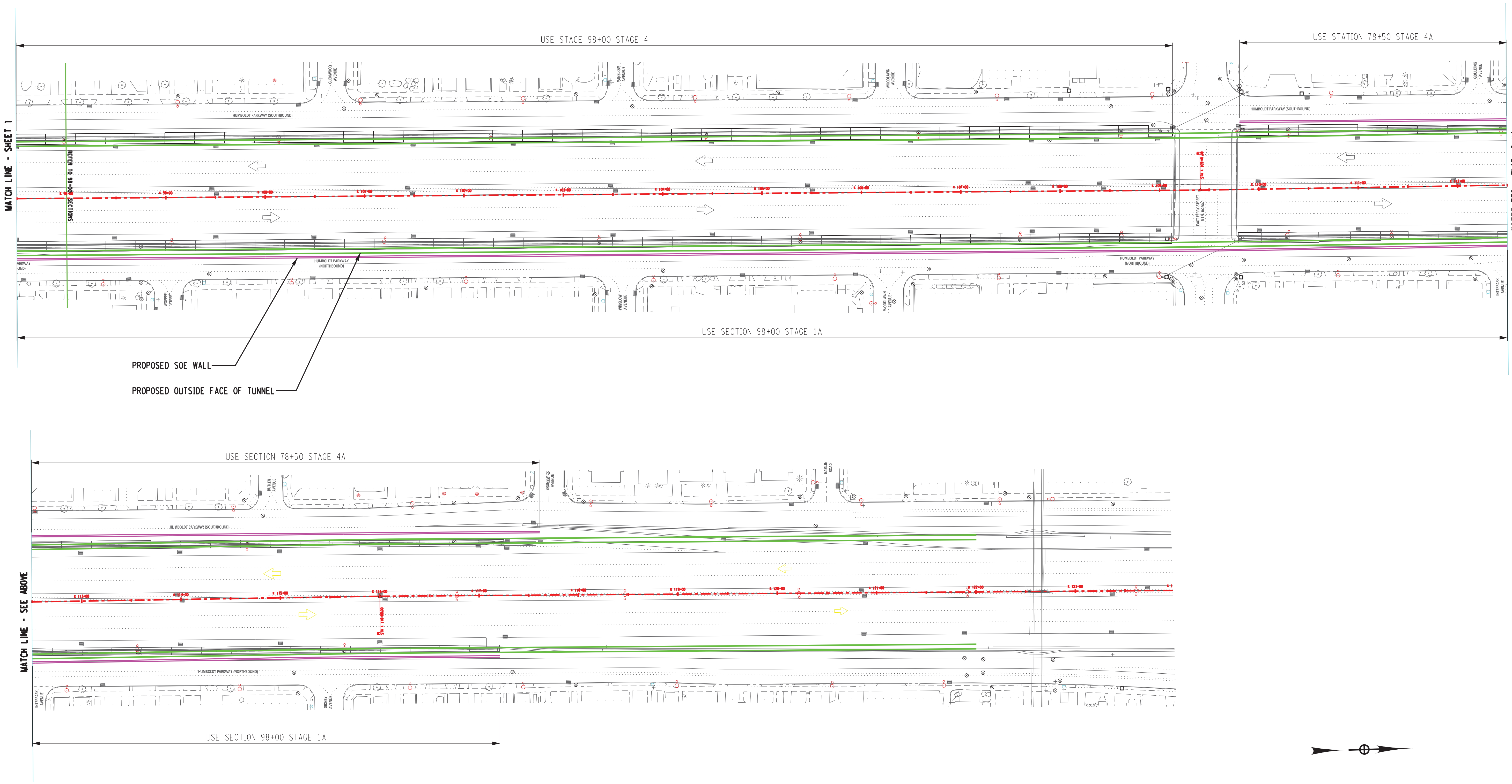

PROPOSED TUNNEL WALLS

TEMPORARY SOE WALLS

SCALE: 1" = 100'

AS-BUILT REVISIONS DESCRIPTION OF ALTERATIONS:	COUNTY: REGION:	PIN	BRIDGES	CULVERTS	ALL DIMENSIONS IN ft UNLESS OTHERWISE NOTED WZTC OVERVIEW PLANS	CONTRACT NUMBER DRAWING NO. SHEET NO. 2 OF 2

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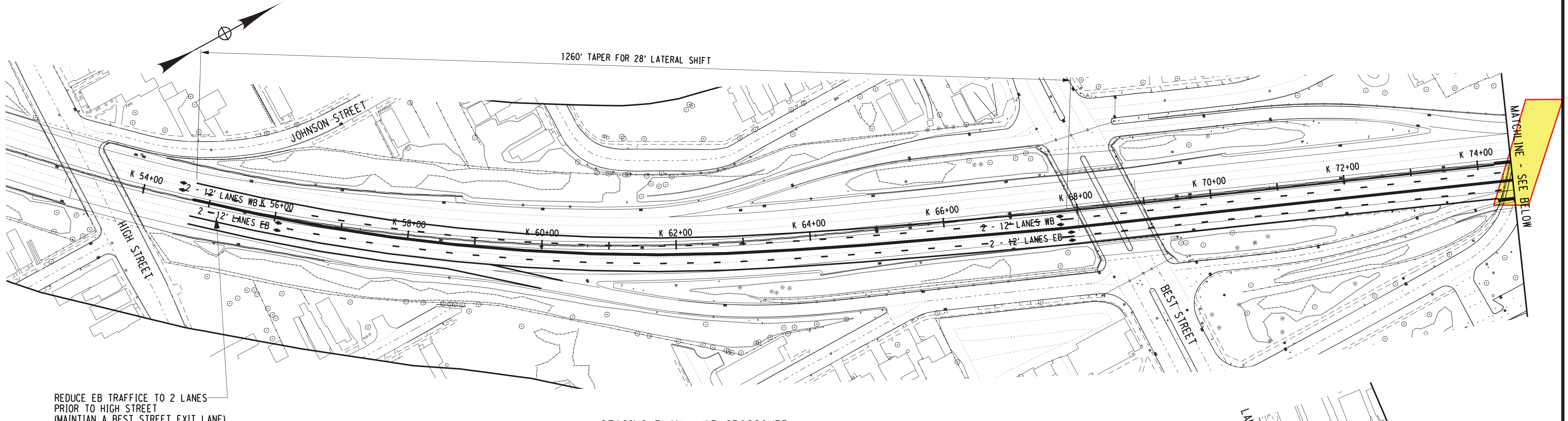
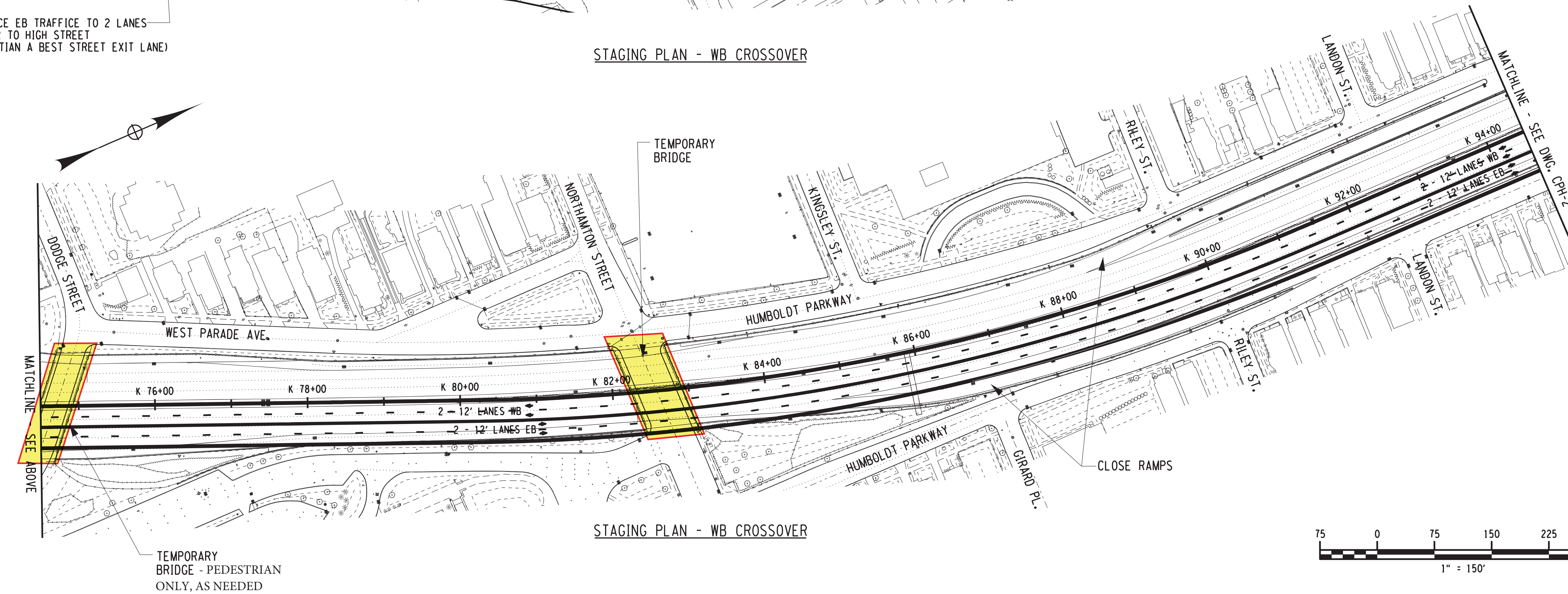
Appendix G:
Work Zone Traffic Control
Staging Crossover Plans



STAGING PLAN - WB CROSSOVER



STAGING PLAN - WB CROSSOVER

75 0 75 150 225 300'

1" = 150'

AS-BUILT REVISIONS DESCRIPTION OF ALTERATIONS:	COUNTY: REGION:	PIN	BRIDGES	CULVERTS	ALL DIMENSIONS IN ft UNLESS OTHERWISE NOTED	CONTRACT NUMBER
					NY 33 WB CROSSOVER PLAN (SHEET 1 OF 2)	DRAWING NO. CPH-1 SHEET NO.

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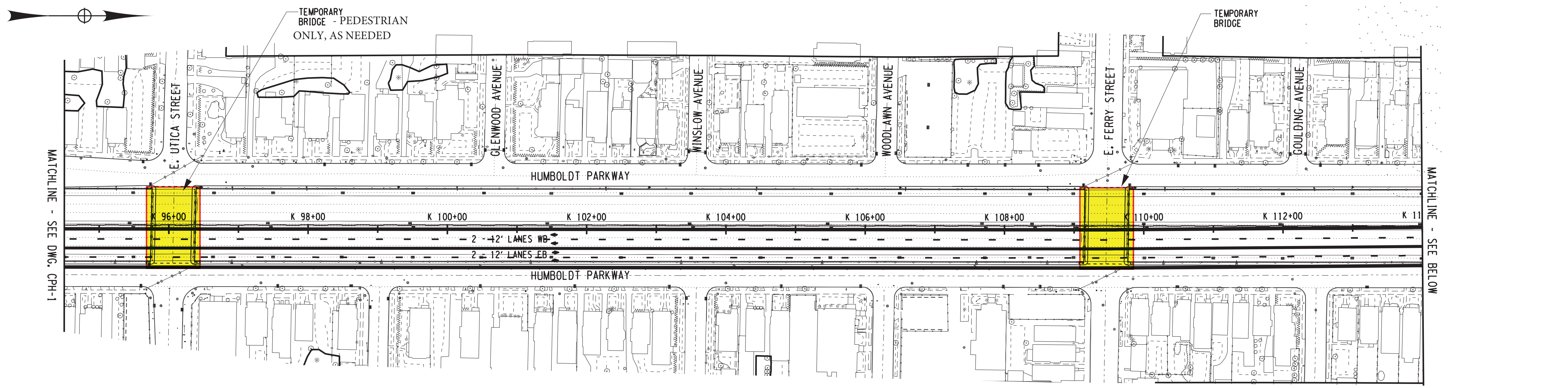
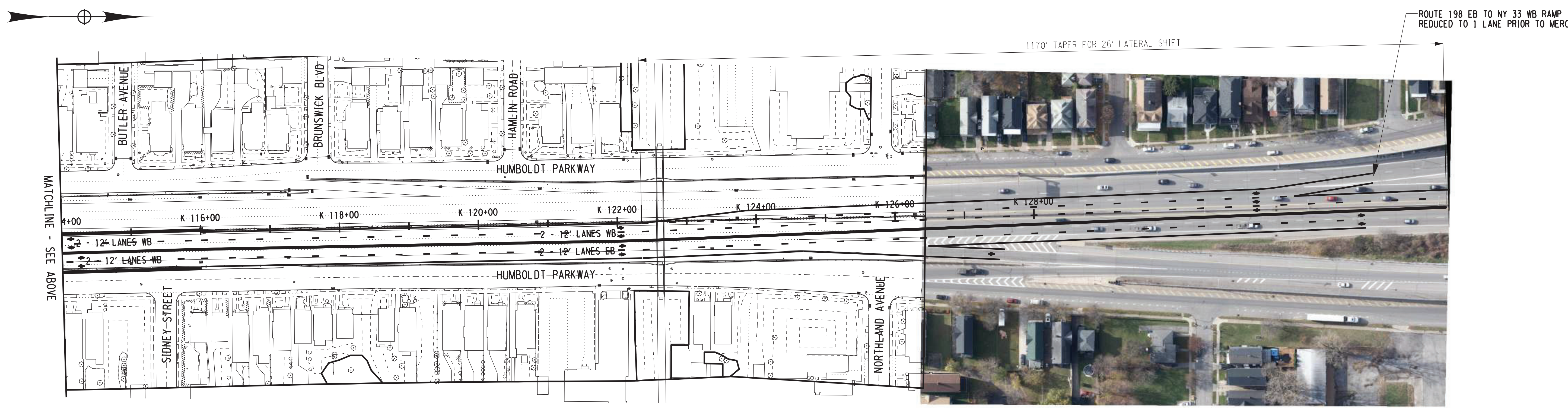


STAGING PLAN - WB CROSSOVER



STAGING PLAN - WB CROSSOVER

75 0 75 150 225 300'

1" = 150'

AS-BUILT REVISIONS DESCRIPTION OF ALTERATIONS:	COUNTY: REGION:	PIN	BRIDGES	CULVERTS	ALL DIMENSIONS IN ft UNLESS OTHERWISE NOTED NY 33 WB CROSSOVER PLAN (2 OF 2)	CONTRACT NUMBER DRAWING NO. CPH-2 SHEET NO.

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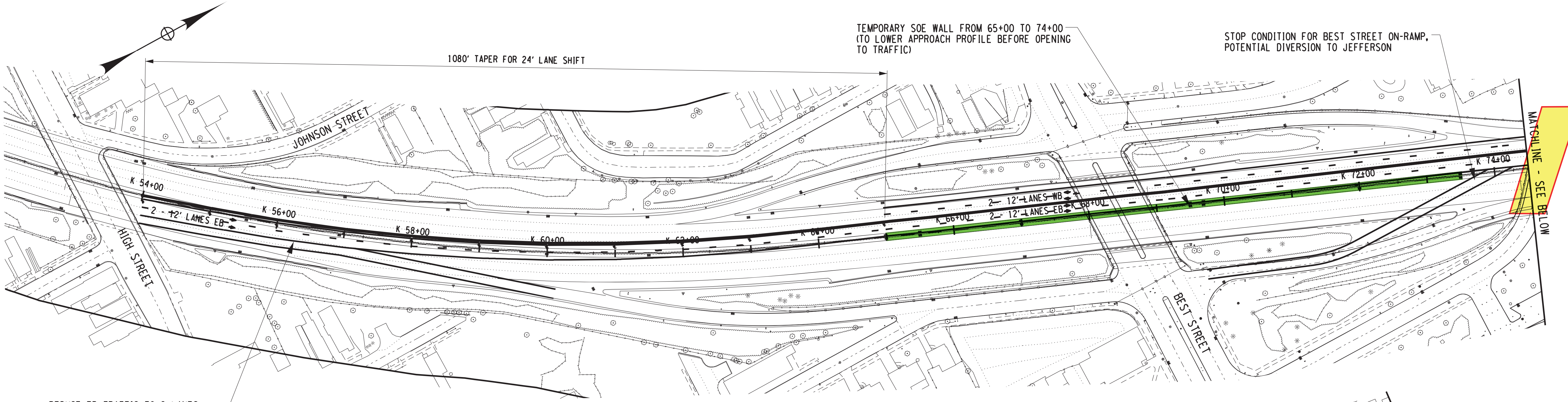
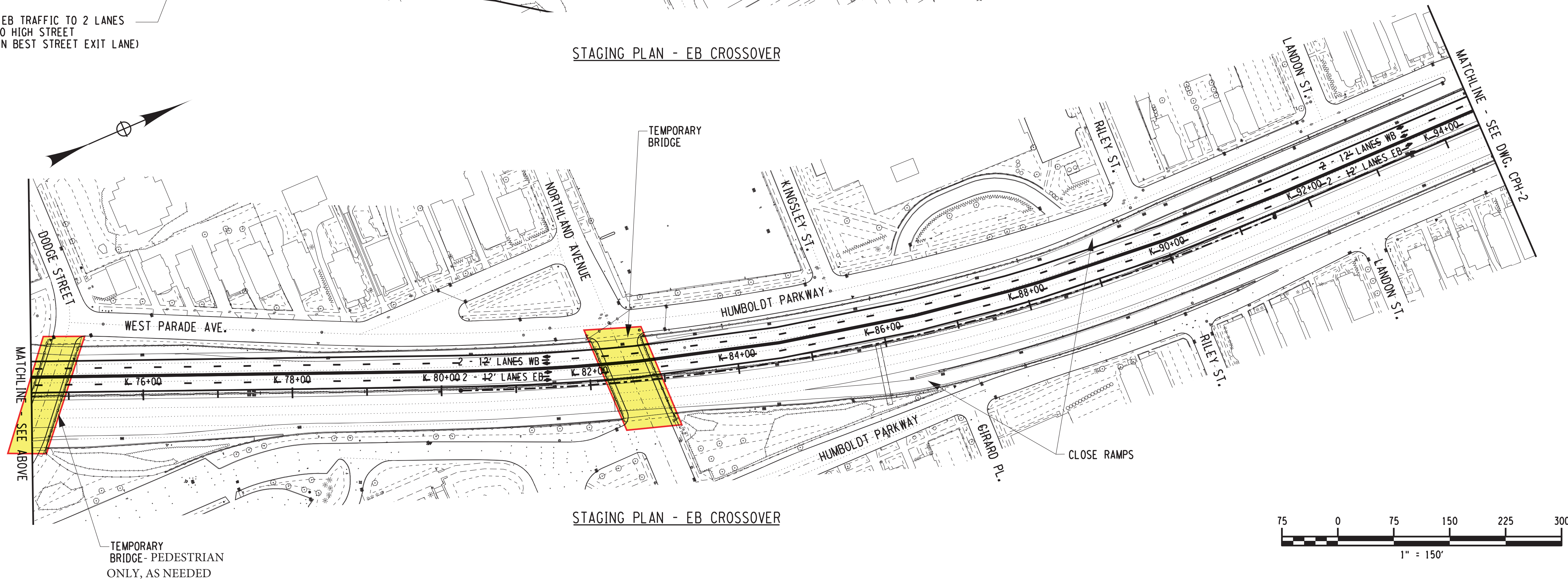


STAGING PLAN - EB CROSSOVER



STAGING PLAN - EB CROSSOVER

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PIN | BRIDGES | CULVERTS

ALL DIMENSIONS IN ft UNLESS OTHERWISE NOTED

NY 33 EB CROSSOVER PLAN
(1 OF 2)

CONTRACT NUMBER

DRAWING NO. CPH-1

SHEET NO.

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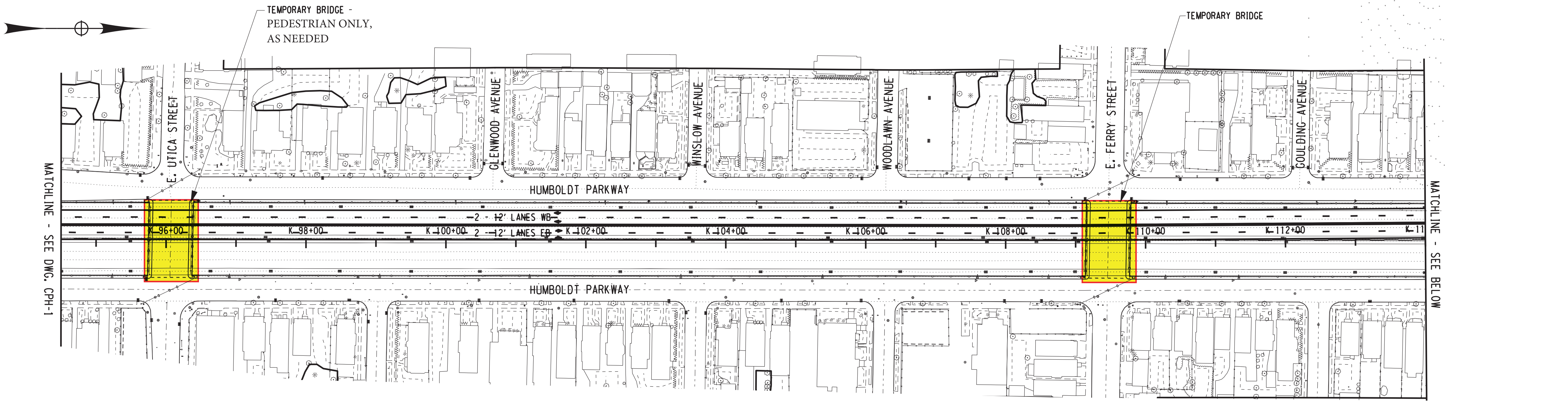
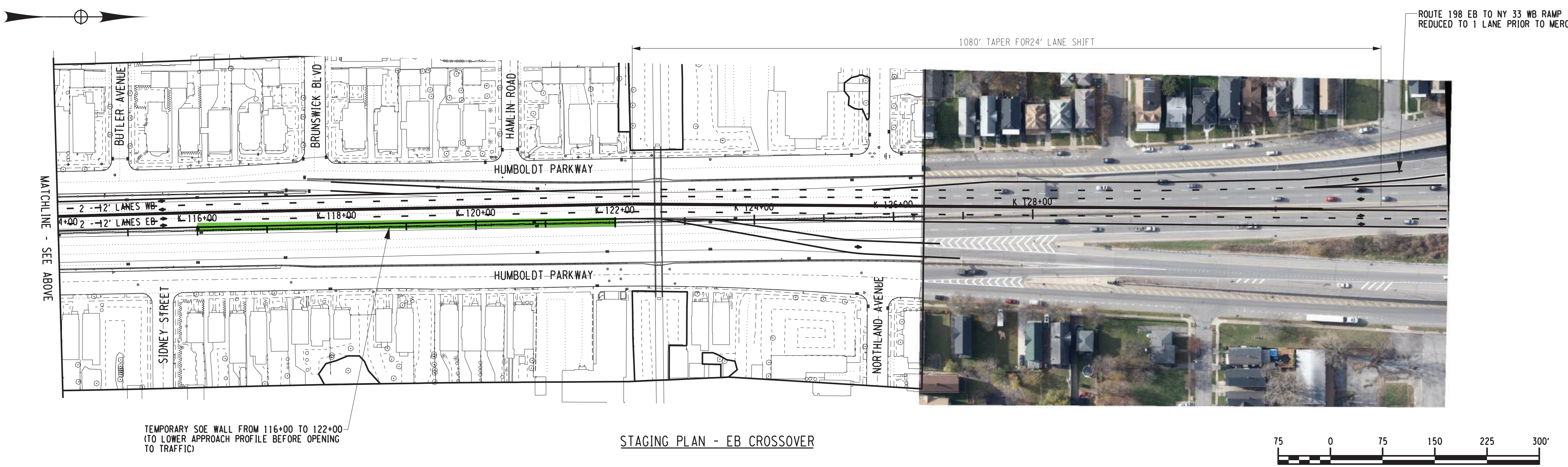


STAGING PLAN - EB CROSSOVER



STAGING PLAN - EB CROSSOVER

75 0 75 150 225 300'

1" = 150'

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AS-BUILT REVISIONS DESCRIPTION OF ALTERATIONS:	COUNTY: REGION:	PIN	BRIDGES	CULVERTS	ALL DIMENSIONS IN ft UNLESS OTHERWISE NOTED NY 33 EB CROSSOVER PLAN (2 OF 2)	CONTRACT NUMBER DRAWING NO. CPH-2 SHEET NO.

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