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Final structures will consist of the following construction types:
- Retaining wall construction
- U-wall construction
- Cut-and-cover tunnel construction
- Mined (TBM-bored) tunnel construction

Retaining wall construction may consist of various types, such as cantilever retaining walls or retained earth retaining walls. Retaining wall construction and U-wall construction will be used for depressed roadway alternatives and for approaches to tunnel alternatives.

1.1 SPECIAL CONSIDERATIONS

The following special considerations will affect structural design and construction:
- Saline groundwater
- High snowfall accumulations and deicing salts
- Shale bedrock
- Potential high in-situ stress in rock.

1.1.1 SALINE GROUNDWATER

Saline groundwater apparently will be encountered during construction of both depressed roadway and underground roadway sections. The degree of salinity is unknown. Salinity can be qualitatively described as brackish (relatively low salinity) to briny (relatively high salinity). See the discussion re salinity above. Saline groundwater, irrespective of degree of salinity, will require treatment before discharge into municipal sewers, in accordance with NYSDEC requirements. The required level of treatment and the corresponding cost of treatment will be a function of the degree of salinity.

Therefore, dewatering should be minimized. Strict limits should be placed on allowable groundwater drawdown. This will require relatively impermeable Support of Excavation (SOE) walls with limited dewatering within the excavation for depressed roadway, tunnel approaches, and cut-and-cover tunnel construction. Such wall types would include slurry walls and secant pile walls. Saline groundwater conditions will affect the slurry used to stabilize trenches during the excavation stage of slurry wall construction, will influence concrete mix design, and will require use of anti-corrosion measures for all concrete structures, temporary and permanent.

1.1.2 HIGH SNOWFALL ACCUMULATIONS AND DEICING SALTS

According to AccuWeather, Syracuse is the snowiest major city in the US, with an annual average snowfall of approximately 124 inches. It has commensurately high use of road salt, which is applied in both crystalline and brine forms. Anti-corrosion measures would be required in the concrete structures to protect against deicing salts.

Depressed highways would be need to be cleared of snow accumulations. In addition, deicing would be required. This would likely be through the use of deicing salts. However, heating the road deck may be possible.

Tunnels are protected from snow, so snow is not a concern in covered sections. Ramps leading into tunnels are typically left open to the sky. However, a combination of gradient, ice, and snow accumulation could make covered approaches cost effective. Covered approaches can also improve the dispersal efficiency of air exhausted from ventilation systems.

Deicing salts are carried into tunnels by vehicles, and should be considered in the structural design.

1.1.3 SHALE BEDROCK

It is common practice to terminate both slurry wall construction and secant pile wall construction at or immediately below top of rock when soil is underlain by medium to high strength rock. For these conditions, it is important to know the top of rock location for evaluating the cost of slurry wall or secant pile wall construction. The top of rock elevation determined on the basis of the borings provided by NYSDOT has an unknown degree of certainty. Fortunately, the shale bedrock that will be encountered on this project is a weak rock, with an estimated unconfined compressive strength of between 2,000 psi to 5,000 psi. This means that it will be possible to economically install either a slurry wall or secant pile wall to the full depth of excavation. Theoretically, the wall then can be incorporated in the final construction. However, as discussed below, potential high in-situ stresses in the rock may preclude this potential cost-effective measure.

1.1.4 POTENTIAL HIGH IN-SITU STRESS IN ROCK

The sedimentary rocks in the Ontario Lowland are known to contain high levels of horizontal stress, related to their geologic history. The low strength, low modulus shales, such as those in the project area, often exhibit time-dependent deformation upon excavation. This results in lateral movement of excavation sidewalls into the excavation and buckling and heaving of the excavation invert. The lateral movement can persist for decades, but will reach an asymptote in a few months, based on measurements during tunnel construction in Rochester in 1980s. Such lateral movement has affected the alignment of turbines in electric power plants in Ontario, Canada.

In-situ stress measurements to determine the orientation and magnitude of high horizontal stress are routinely performed during the detailed design stage of a project. The magnitude of high horizontal stress may vary between measurement locations.

Standard practice in dealing with this phenomenon in foundation excavations includes the following procedures:
- Install compressible material against the trimmed rock face before constructing the wall.
- Remove buckled and heaved rock in invert and replace with lean concrete.
- Install permeable material to gradually stress the wall.

The pressures that the rock can apply to walls that restrain movement can be quite high and may result in wall failure. Thus, it may be inappropriate to extend slurry walls or secant pile walls through the rock to the bottom of the excavation.

For tunnel construction, the restraint against lateral movement of the tunnel sidewalls and buckling/heave of the invert will increase the load placed on precast concrete segmental lining sections.

1.2 SUPPORT OF EXCAVATION SYSTEMS

To excavate a depressed highway trench, or cut and cover tunnel, the adjacent soil and structures must be retained using a support of excavation (SOE) system. Methods that may be applicable include:
- Soldier pile and lagging
- Slurry walls (also known as diaphragm walls)
- Secant piles
- Jet grout infill panels
- Bracing and tie-backs

Sheet piles may also be applicable to shallow excavations, but as noted above, the fill materials contain obstructions that could interfere with installation.

1.2.1 SOLDIER PILE AND LAGGING

Soldier pile and lagging walls would require augering cased holes to the required depth, and installing steel H-piles at (typically) 5ft to 8ft centers. During excavation, wooden lagging would be placed between the flanges of the piles to retain the soil behind. The lagging boards are not watertight, so the technique would only be applicable in shallow areas above the groundwater table. The technique is particularly useful where utilities cross the excavation. The soldier piles can be placed on either side of the utilities, which are then supported across the excavation.

1.2.2 SLURRY WALLS

Slurry walls are constructed by excavating trench a few feet wide, while the trench is maintained full with a slurry. The slurry, containing either bentonite clay or polymer, stabilizes the soil while a trench panel is excavated. Excavation is performed using either a clamshell or a hydramill (a vacuum lift with a rotating cutting tool, also...
known as a hydroraise). A hydromill would likely be used on this project because it can excavate soil, weathered rock and weak unweathered rock (shale).

Alternate panels are excavated, followed by placement of reinforcement and then concreting. When concrete has attained sufficient strength, intervening panels will be excavated. The flexural strength of slurry walls is provided by either pre-tied reinforcing cages or multiple steel H-piles. The latter method is known as a Soldier Pile Tremie Concrete (SPTC) wall. This variant eliminates the requirement for a reinforcement cage.

Concrete is placed in the panel using the tremie method, which displaces the slurry. When all panels are complete, the wall is essentially watertight. Toe depths of 150-ft or more can be achieved.

1.2.3 SECANT PILE WALLS

Secant pile walls are constructed by auguring overlapping piles that are typically 3-ft to 5-ft in diameter. Temporary steel casing is typically used to support the ground during augering. Alternating pile – primary piles – are excavated first and filled with unreinforced concrete as the casing is withdrawn. The infill piles – secondary piles – are then constructed and include steel reinforcement cages or soldier-piles. A high degree of water tightness can be achieved, but leakage is generally higher than with slurry walls.

Toe depths of 90-ft or more can be achieved.

1.2.4 JET GROUT INFILL PANELS

Where obstructions such as utilities would cause a gap in an SOE walls, the ground can be strengthened and made watertight with grout. Jet grout is a common technique, in which a mixture of cement and water is injected into the ground through a nozzle. The nozzle is drilled to the required depth, and slowly withdrawn while rotating to create a column of ‘soilcrete’.

1.2.5 BRACING AND TIE-BACKS

As excavation proceeds, all the SOE systems noted above require a system of wales and either bracing or tiebacks to be installed to resist the lateral force of the soil and groundwater. No dewatering will occur outside the excavation. The slurry wall or secant pile wall will serve as a water barrier and will be designed for water pressure as well as lateral earth pressure.

Struts are placed between wales on each side of the excavation, typically at 10-ft to 20-ft centers. Wales run horizontally across multiple piles at vertical spacing of 10-ft to 20-ft and cause the walls to act monolithically rather than as individual piles. The space between struts must be sufficient for construction equipment to pass.

The location of struts can interfere with the permanent structure, so temporary works and permanent works must be designed to achieve good constructability. Tie-back anchors are installed in holes drilled through the support walls, and are grouted into soil or rock sufficiently far from the wall that stresses have no significant impact on the wall. Tie-backs are pre-tensioned and locked off at an anchor plate, which typically bears against a double channel wales.

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1.3 SUPPORT OF EXCAVATION CONSIDERATIONS FOR CUT AND COVER OR DEPRESSED HIGHWAY

Shallow excavations above the groundwater table for a depressed highway or tunnel open approaches can be supported by soldier pile and timber lagging SOE walls. Excavations extending below the groundwater table into saline water conditions should use either slurry wall or secant pile wall construction. Soldier Pile Tremie Concrete (SPTC) walls (a type of slurry wall) may be optimal. Compared with secant piles they can extend deeper, and reduce water leakage during construction.

A panel width of 10 feet could be applicable, with a panel thickness of 36 inches, and W36 soldier piles. These are common SPTC wall dimensions. Secant piles could be approximately 1000mm (approximately 39 inches) in diameter.

Using soldier piles rather than rebar cages would be beneficial as there is generally insufficient laydown area for large reinforcement cages. Figure 4 shows a rebar cage being tied, prior to installation.

It would be possible to construct a depressed roadway or cut-and-cover tunnel beneath the viaduct, but this would require low headroom excavation equipment. Low-headroom slurry wall rigs can typically achieve greater depths than secant pile rigs (Figure 5 and Figure 6). Limited headroom results in the rebar cage or soldier piles being lowered in sections and spliced together. Generally it is more cost effective to splice soldier piles, but both methods are slow and labor intensive. Limited headroom areas frequently have pile caps and other obstructions to be worked around. This can result in walls being constructed piecemeal, which tends to favor secant piling. Given that the cost of both methods is similar, the selection should generally be left to the contractor’s SOE designer, who can perform a detailed evaluation of conditions at each location.

The top of rock elevation varies throughout the various alternatives (see rock-line shown on alternative profiles in Appendix A). In areas where the top of rock is part-way up the tunnel or depressed highway walls, the SOE walls could either terminate a couple of feet into rock (Figure 7) or could be extended to beyond the bottom of the invert slab (Figure 8). The shale bedrock is relatively weak, which should permit full-depth wall installation using conventional equipment.

If the walls terminate at top of rock, the rock face created during bulk excavation would likely be reinforced with rock dowels and shotcrete. Typically, a small rock ledge is left at the toe of the SOE walls (Figure 7), but as shown in (Figure 3 – South Ferry) this is not always the case.

It may be necessary to place compressible material against the rock face to account for continuing time-dependent movement of the rock as a result of excavation-induced stress relief. Time-dependent movement may require termination of slurry walls or secant pile walls above top of rock, precluding their incorporation in the final structure.

The SOE walls will generally run parallel to the centerlines of the tunnel or depressed highway. However, local deviations around existing viaduct pile caps may be required.

A combination of tie-backs and bracing is likely to be cost effective (see Figure 7 and Figure 8). Tie backs would need to consider the piled foundations of adjacent structures, including the existing I-81 and I-690 viaducts.
1.4 POTENTIAL CONSTRUCTION METHODS

Depressed roadway alternatives and cut-and-cover tunnel alternatives constructed along the existing I-81 viaduct alignment will generally have a broader footprint than the viaduct, so that SOE walls will be located outside the viaduct footprint (Figure 9 – Stage 1). Existing on/off ramps and piles – especially raking piles – will require special considerations both for piling equipment and to ensure that the existing structure is not compromised while it is still in service. Traffic Management on Almond Street and other city streets would be necessary to minimize disruption, but the number of available travel lanes would be reduced during construction, and some closures would likely be necessary.

Once continuous SOE walls are in place, excavation under the viaduct could commence. However, excavation below the pile caps would remove confinement of the piles, which would decrease lateral and vertical capacity, and would ultimately result in piles buckling. While it may be possible to install bracing, a more cost effective method may be to install underpinning of the viaduct crossheads, as shown in Figure 9 Stages 2 and 3, and also in Figure 10 and Figure 11. The underpinning would be designed to transfer loads into the SOE system, or into new temporary foundations. The underpinning would be expensive, and given the low headroom under the existing viaduct (Figure 12) would preclude decoring over the cut to maintain traffic on Almond Street. However, it would potentially allow traffic on I-81 to be maintained during much of the tunnel or depressed highway construction.

Compared with the cost of underpinning, it may be more cost effective and less disruptive to city traffic to close I-81 at the beginning of construction, demolish it, and install decking throughout its length to provide a temporary ‘community grid spine road’ until the project is complete.

The I-690 viaduct is generally higher above grade, and does not have roads running continuously beneath. This makes underpinning of the I-690 viaduct more feasible while minimizing traffic impacts on city streets.

If surface traffic is to be maintained on streets that cross the cut-and-cover construction or depressed highway construction, a decking system would be required (Figure 13). A system of girders would span the excavation, from support wall to support wall. Intermediate ‘king posts’ could be placed to reduce spans, where necessary. Common decking girder configurations used in cut-and-cover construction include twin W36 sections or 60 inch welded plate girders. The former configuration often is used when utilities must be maintained within the limits of the excavation. Decking panels would span between girders. The decking panels would have a non-skid surface. Typical decking panels include 10 foot long X 5 foot wide X 1 foot thick precast concrete panels or 10 foot long X 5 foot wide by ½ inch thick steel plates with longitudinal W4 or W6 steel ribs at 1'-6” transverse spacing.

FIGURE 9: Potential Construction Sequence for Cut and Cover Tunnel

FIGURE 10: Support Trusses (100′L x 12′D), Midland Links, Birmingham UK

FIGURE 11: Plate Girder Support Beams (100′L x 7′D), Midland Links, Birmingham, UK

FIGURE 12: Low Headroom under Existing I-81 Viaduct

FIGURE 13: Pre-cast Street decking (top), Steel Street Decking (bottom) – 50th and 55th Streets Manhattan, NY
Instead of constructing a cut and cover highway directly under I-81, it is proposed to construct a stacked cut and cover tunnel along the northbound lanes of Almond Street, immediately east of the I-81 viaduct (Figure 14). The western SOE wall (left) is shown in a location where jet-grout infill panels would be used between other SOE methods (secant piles or slurry wall) where existing I-81 raking piles might conflict with an SOE wall. A stacked cut and cover arrangement would be used at the northern end of the Green Alternative, at the transition from a single-bore (stacked roadway) tunnel to cut and cover, north of Fayette Street.

The transition from a stacked tunnel to a side-by-side configuration at each end of the Green Alternative tunnel is shown on two drawings at the end of this appendix ("Green Alignment – North Sections", and “Green Alignment – South Sections”).

It is probable that earth excavation within the cut will be performed by front end loaders directly loading trucks inside the cut. Trucks will likely exit the cut en route to the disposal site using a ramp excavated as part of the depressed roadway/tunnel approach section. This would eliminate the need for cranes at street level to service the excavation. Decking panels along the curb line could be removed and the openings protected by barriers to provide ventilation of the cut. Alternatively, mechanical ventilation similar to mined tunnel ventilation can be installed beneath the decking.

Weathered rock and unweathered rock (shale) excavation probably can be done by ripping, because of the low strength and thin to moderate sub-horizontal bedding of the rock. Front end loaders will load the rock into trucks for transport to the disposal site. Work can proceed from both ends toward the middle for an all depressed roadway or an all cut-and-cover tunnel option.

Dewatering inside the excavation will be performed by eductors when excavating in cohesion-less soil (sand, non-plastic silt, gravel) and by pumping in cohesive soils (plastic silt, clay).

1.5 TUNNELING UNDER THE RAILROAD

All four preferred alignment alternatives pass under the New York, Susquehanna and Western Railway. This is a single railroad track that carries freight services, but no passengers. It is located close to the southern tunnel portal. Availability of track outages for tunnel and community grid viaduct construction would need to be determined. Alternative rail-freight routes may be possible, such as via currently out-of-service tracks through Utica. Tracks would be instrumented to monitor movements during tunneling, to ensure that any settlement stays within FRA safety thresholds.

On the Red Alternative, the alignment passes under the railroad close to Oakwood Avenue. An EPB tunnel boring machine is proposed, and the tunnel is expected to be in rock. Minimal settlement is anticipated, resulting in no impact to railroad operations.

On the Orange Alternative, the alignment passes under the railroad at Burt St, west of the existing I-81 viaduct (Figure 15). At this location, the railroad is elevated above grade. Bored tunnels are proposed, but the depth of piles supporting the existing railroad bridge over Burt Street would need to be determined to ensure no conflict.

On the Blue Alternative, the alignment passes under the railroad in three locations: near Townsend Street, near Onondaga Street, and near West Street. In each location the railroad is elevated, either on retained earth or on a bridge. An EPB tunnel boring machine is proposed, and the tunnel is expected to be in soil, or a mixed face of soil and rock. Pressurized face tunneling should result in minimal settlement, resulting in no impact to railroad operations. Foundations of the railroad structures would need to be investigated to ensure no conflict.

On the Green Alternative, the alignment passes under the railroad near Van Buren St, east of the existing I-81 viaduct (Figure 16). At this location the railroad is at grade. Bored tunneling is proposed, which should result in minimal settlement.

It may be possible to deliver materials (such as TBM segments) and to remove muck by rail. However, trucking is likely to be more cost effective and to provide better schedule flexibility.

Passenger services were provided on the line from 1994 to 2007. Ridership was low, so passenger services are unlikely to be reintroduced.
1.6 TUNNELING UNDER THE SYRACUSE UNIVERSITY STEAM PLANT

The tunnels of the Orange Alternative pass under the Syracuse University Steam Plant. The steam plant property is immediately north of the southern limit of the TBM section of tunnel. As shown on Figure 17, the tunnels pass below part of the property that is not currently developed but close to buildings on both sides. To the west is the Riley Steam Station that was constructed around 1950. The building includes a tall chimney, approximately 150-ft high. To the east is the Chilled Water Plant and the Steam Station Garage. The footprint under which the tunnels will pass was formerly occupied by a Cogeneration Plant, constructed around the year 1991. This was demolished between 2009 and 2011. Steam pipes, water pipes, electrical cables and other utilities cross the site at grade and elevated. Lightweight garage and storage structures also exist above the tunnel alignment.

The Chilled Water Plant had piled foundations, approximately half of which were timber, and the other half 12” steel pipes. The piles were located directly on the alignment of the tunnel. The delivered pile length was “35-ft to 45-ft”, but the installed length was permitted to be less provided specified criteria were met. Installed lengths are not known.

A rail spur trestle for coal deliveries historically crossed the tunnel alignment, but this had shallow footing.

The crown of the bored tunnels, based on the currently proposed profile, is approximately 45-ft below grade where they pass under the redundant piles. It is recommended to extend the 4.3% down grade as far as the railroad, which will increase the depth of the tunnel to approximately 60-ft under the steam plant. Available geotechnical information (from steam plant record drawings) indicates that top of rock (shale) is approximately 10-ft below grade close to Almond Street, but as deep as 45-ft closer to McBride Street. It is therefore possible that multiple piles, and potentially up to approximately 100 wooden piles and 90 steel piles could extend to 45-ft depth but it is considered unlikely that any would extend to 60-ft depth or into the path of the TBMs. The TBMs would likely not be able to mine through either steel or wooden piles. As seen on the Alaskan Way project (Appendix I), if a TBM encounters a steel pile it can have significant negative consequences for cost and schedule.

To minimize the risk of encountering a pile, additional investigations would be required, such as obtaining as-built records and excavating exploratory pits.
2 PERMANENT STRUCTURES: BORED TUNNEL, DEPRESSED ROADWAY AND CUT-AND-COVER TUNNEL

2.1 DEPRESSED ROADWAY & CUT AND COVER TUNNEL

Final construction of depressed roadway sections would probably consist of combinations of retained earth walls, conventional reinforced concrete retaining walls, permanently anchored retaining walls and U-walls (monolithic walls and invert slab) – Figure 18 and Figure 19. Cut-and-cover tunnel sections will consist of an invert slab, sidewalls, and roof slabs. Cut-and-cover sections would likely include an interior structural wall separating two, unidirectional roadways. This will improve safety during operation and will have the beneficial effect of reducing roof and invert slab thickness.

Buoyancy of the final structure will be a significant concern, and will likely result in a thicker invert that would be necessary to resist structural stresses. Heavyweight concrete that uses iron ore or other dense aggregate could be considered (per ACI 211.1). Concrete densities of up to 230 pounds per cubic foot (pcf) could be achieved, which would generally reduce the invert thickness by approximately 50%. However, this material has not previously been used for a transportation project on the scale of the I-81 Tunnel. The unit cost of the material is high, partly due to high transportation costs, but these could be moderated by bulk shipment via the Great Lakes to a port such as Oswego. Long-term performance would need to be evaluated, in particular the potential for alkali-silica reaction (ASR) and whether the magnetite/iron in the mix is susceptible to corrosion. Based on these concerns, heavy weight concrete is not currently recommended.

The weight of the SOE walls could be engaged to resist buoyancy, but the shear transfer would need to be designed to not compromise the waterproofing system. Also, the use of bentonite during construction of the SOE walls could reduce friction. Deflection of the SOE during excavation could reduce lateral loads (and therefore the frictional force), and the original in-situ loads may not recover prior to buoyant uplift forces occurring. It is possible to design physical interlock between NOE and a permanent structure, but this complicates waterproofing, corrosion control, rebar cages, and constructability. For these reasons the weight of the SOE has been neglected.

Permanent tie-downs could be used to resist buoyancy and thereby reduce the volume of concrete in the tunnel approach structures. Various projects have used this approach including two cut and cover stations on the North Shore Connector in Pittsburgh, PA; and stations in Berlin, Germany; Malmo, Sweden; and Thessaloniki, Greece. Corrosion resistance can be designed. However, due to the corrosive groundwater in Syracuse permanent tie-downs are not recommended. Other concerns include the necessary penetrations through the invert waterproofing system, and the difficulty/ inability to monitor/inspect the tension elements. If detailed design reveals a temporary condition where buoyancy is a concern (such as when the invert slab is placed, but prior to construction of the walls and roof) temporary tension elements (passive, or post-tensioned) could be considered.

Waterproofing will be applied to the interior face of the SOE wall or to the rock face prior to placement of reinforcement and forms and concrete placement. A smoothing layer of shotcrete (sprayed concrete) may be applied to the rock surface prior to placement of waterproofing. Waterproofing concrete additives may also be considered, but these are generally not effective for large volume applications.

The required internal widths of recessed roadways or tunnels can be affected by line-of-sight requirements for traffic driving around a curved segment of an alignment. A tunnel wall or bench that is on the inside of a horizontal curve can limit the length of the line of sight. The wall must be set back from the travel lanes sufficiently to maintain minimum AASHTO line of sight requirements for the project design speed. An extra wide shoulder on the inside of the curve may be needed.

Of the alternatives studied, the Green alignment has the tightest horizontal curve of R=926’ (located at the north portal near Fayette St which partially lies within the limits of a cut and cover structure) and will require a shoulder width of 18’11” in order to maintain the clear line of sight as traffic exits the north portal. This is in contrast to the internal width of the cut and cover and bored tunnel immediately south, with both structures being on a tangent alignment. In that location the shoulder widths required are reduced to follow the minimum [4 ft] suggested by the AASHTO Manual. All bored tunnels have been sized according to various tangent and horizontal curves.

2.1.1 TUNNEL WIDTH TO MAINTAIN LINE OF SIGHT AT CURVES

The required internal widths of recessed roadways or tunnels can be affected by line-of-sight requirements for traffic driving around a curved segment of an alignment. A tunnel wall or bench that is on the inside of a horizontal curve can limit the length of the line of sight. The wall must be set back from the travel lanes sufficiently to maintain minimum AASHTO line of sight requirements for the project design speed. An extra wide shoulder on the inside of the curve may be needed.

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The width of the tunnel is primarily determined by the number of lanes, the width of the lanes, the width of any shoulder, walkways (if used), any additional space for equipment mounted on the walls, and line-of-sight requirements (discussed below). The height of the tunnel is primarily determined by vehicular clearance required, and any additional height for signage, lights, ventilation and other equipment. These geometric requirements are discussed elsewhere in this report.

FIGURE 18: Depressed Highway with Cantilevered City Streets, Queens, NY

FIGURE 19: Depressed Highway with Cantilevered Almond Street

Design Speed = 50 MPH

GREEN NORTH ALIGNMENT

FIGURE 20: Line of Sight Requirements: 50 mph, 926’ radius curve.
2.2 MINED (TBM-BORED) TUNNEL CONSTRUCTION

2.2.1 TUNNEL CONFIGURATION

Bored Tunnels could be either a pair of parallel unidirectional tunnels with cross-passages (Figure 21) or a single tunnel with a stacked highway (Figure 22: Single Bi-Level Tunnel).

Case histories of previous large diameter tunnel projects are provided in Appendix N, including the Eurasia Tunnel in Istanbul, Turkey (Figure 23).

The minimum pillar width between two adjacent mined tunnels should be approximately half a tunnel diameter. This prevents overstressing of the soil in the pillar, which can lead to large plastic deformations. The pillar width can be reduced for favorable ground conditions or if ground treatment can be used effectively.

2.2.2 TUNNEL BORING MACHINE (TBM) SELECTION

A pressurized face tunnel boring machine (TBM) capable of excavating through soil and rock will be required for the mined tunnel portions of this project. There are two general types of pressurized face TBMs: the slurry shield and the earth pressure balance (EPB) TBM (Figure 24).

Slurry TBMs are generally used in cohesion-less soils (sand, non-plastic silt, gravel), while EPB TBMs generally are used in cohesive soils (plastic silts and clays). EPB machines often can be used in cohesion-less soils with the addition of conditioning agents. The effectiveness of conditioning agents may be affected by saline ground water. Slurry TBMs require more space at the ground surface to house a separation plant that removes bentonite slurry from the excavated material prior to disposal. This can be an important consideration in a confined urban site.

As described in Appendix D, ground conditions expected to be encountered by the TBM are variable. The soil is expected to comprise a mixture of sand, gravel, silt and clay. The rock is expected to be shale with a significant clay content, which may be interlayered with siltstone and sandstone. The shale is expected to be weak, with high horizontal stresses and the potential for producing hydrogen sulfide gas and methane. Groundwater will generally be close to the ground surface.

Earth pressure balance TBMs are considered more suitable for the geology. The anticipated high fines content of the soils should allow them to readily mix with water and conditioning agents within the TBM plenum to create a plastic material than can effectively maintain face pressures, and which can be removed through the TBM screw conveyor. In contrast, a slurry TBM would not be well suited to the high fines content, since the slurry separation plant would need to be large to remove the fines from the slurry circuit. The plant would require significant space at the surface, and fines removal could slow the advance rate of the TBM.

The shale is expected to be weak and to readily break down under the action of disc cutters mounted on the cutterhead. Disc cutters are also necessary in case boulders are encountered in the soil. Grille bars mounted on the cutterhead openings would prevent over-sized rock fragments from entering the plenum, which could subsequently clog the screw conveyor. Although it may, theoretically, be possible to mine the tunnel in open-mode, the limited rock cover above the crown of the tunnel could result in a high risk of groundwater inflows or weathered/fractured rock collapse. At least one diameter of rock cover is recommended in shale for working in open mode. It is assumed that that the EPB will be designed and operated only for closed mode. If, during final design supplemental geotechnical information and revised tunnel profiles result in more rock cover, some sections of the tunnel could be considered in open mode. In that case, a dual-mode TBM would be required that could be rapidly converted between modes. Probing ahead of the TBM cutterhead (up to 200-ft ahead) will be required to confirm whether the machine should operate in open mode or closed mode.

Wear protection will be required for the cutterhead and within the screw. Two of the tunnel alignments (Red and Orange) are anticipated to be entirely within shale, which would enable the TBMs to be optimized for mining rock. For the Blue and Green alternatives, the screw and other systems will need to be configured for mining through soil, mixed face and full face of rock.

Recently planned or constructed projects in shale that have used EPB TBMs include the Akron Ohio Canal Interceptor Tunnel, OH; and the West Trunk Sewer tunnel, Ontario.
The Akron Ohio TBM is 30-ft in diameter, and is convertible between open and closed modes.

### 2.2.3 Temporary Ventilation

The shales may contain hydrogen sulfide and methane gas, which are significant safety hazards in closed excavations and tunnels. Hydrogen sulfide, which is heavier than air, can accumulate at the bottom of excavations when air circulation is limited. The tunnel is expected to be classified per OSHA as “potentially gassy.” In a slurry machine, the closed slurry circuit would allow the gases to be released above ground at the separation plant. However, in an EPB machine, the gas would be released in the tunnel. Ventilation systems will be required to dilute the gases to safe levels. Gas monitoring will be required at multiple locations on the TBM and within the tunnel. If concentrations exceed allowable limits the TBM would be shut down until the gas concentrations are diluted. Life-safety systems will need to be non-sparking (intrinsically safe).

Hundreds of miles of tunnels have been constructed in gassy environments throughout the US without incident when OSHA requirements are observed. Representative locations include Rochester, Pittsburgh, Cleveland, Milwaukee, and Detroit. While there are recorded cases of explosions during underground construction in gassy ground, these typically occurred several decades ago, before modern safety protocols were routinely enforced. Today, the risk to workers is very low, and there is essentially no public safety risk.

### 2.2.4 Segmental Tunnel Lining

The mined tunnel will be lined with a precast concrete segmental lining, installed concurrently with advance of the TBM. For this study, a thickness of 2-ft has been assumed (for both tunnel diameters), based on precedent projects of similar size (Figure 25). If tanker trucks containing flammable liquids are permitted within the tunnel, additional protective concrete cover may be required over the interior reinforcement. Fires involving tanker trucks in the Mont Blanc and Tauron tunnels in Europe destroyed the tunnel linings. The World Road Association (PIARC) developed fire protection requirements for highway tunnels after these incidents. It would be preferable to ban such vehicles from the tunnel.

Conventional reinforcement is expected to be used in the tunnel segments. Steel fibers are typically used for smaller diameters tunnels (for subways and water management) but are not well suited to the larger stresses induced when handling larger segments. Segments would be designed to resist short-term load conditions including demoulding, stacking, handling, transportation, erection, TBM jacking force and backfill grout pressures. They would also be designed to resist long-term load conditions including load from soil, rock, high overburden, low overburden, high groundwater, low groundwater, fire, seismic events, traffic, internal structures, and supported equipment.

Special segments with increased reinforcement may be required near the portals, and at cross passage openings. Segments will be interconnected with bolts on the radial joints (between the segments in a ring) and either dowels or bolts on the circle joints (between rings).

As discussed below, two gaskets per segment – inner and outer – are recommended to resist groundwater and gas. Time dependent displacement in the shale is expected, which could result in high horizontal forces acting on the tunnel lining. To help counter this, a compressible backfill material may be required around the precast segments. The degree of compressibility and the thickness of the grout layer would need to be determined based on factors such as anticipated rock strain due to stress-relief, grout materials and the desired amount of outerhard overcut. Compressible grout was generally used in shale on the West Trunk Sewer tunnel project in the city of Mississauga, Ontario. The mix included Styrofoam beads to achieve the desired higher stiffness at low strains and lower stiffness at high strains. The backfill grout will help to resist ingress of water and gases, although the gaskets will be the primary method.

### 2.2.5 Buvancy Control

In downtown Syracuse the groundwater table is close to the surface, and the bored tunnels must be designed to resist buoyancy. Where the tunnels are in rock, buoyancy is not a concern. The Red and Orange alternatives are expected to be entirely in rock. However, the Green and Blue alternatives will have significant land in soil. In order to resist buoyancy, the minimum soil cover above the crown of the TBM should be approximately half the tunnel diameter.

The tunnel profiles and the start/and of the bored tunnels (in both soil and rock) have been selected to achieve a ground cover of at least half the TBM diameter (except at the north portal of the Green Alternative, as described below).

This amount of soil cover could be reduced if ground treatment were used. The effectiveness of ground treatment may be affected by groundwater salinity. On the Alaskan Way project the ground above the TBM was jet grouted to help resist buoyancy. In addition, that project used a 5-ft thick concrete slab tied into secant piles to hold down the TBM and the surrounding ground. This approach is proposed for the north portal of the Green Alternative, as shown on the drawing “Green Alignment – Bored Tunnel at Genesee Street” at the end of this Appendix. The secant piles would also act as settlement cutoff walls to protect the adjacent hotel. Other projects, including the Port of Miami, have placed fill above existing grade to resist buoyancy.

In the bi-level single bore alternative, the internal structure is limited, adding little to the overall weight of the tunnel. However, in the twin bored tunnel alternatives, most of the volume below the roadway is backfilled with lower strength concrete. This significantly improves the factor of safety in the long term condition. The temporary construction case becomes the critical condition for buoyancy. On the Groene Hart (Netherlands) project, backfill was placed within 20-ft of the back of the TBM shield to add weight in the temporary condition (Figure 26). The short length of tunnel ahead of the backfilling operation was held down by spanning between the TBM ahead, and the backfilled tunnel behind.

### 2.2.6 Internal Structures

In a twin bored tunnel arrangement, major internal elements are the backfill concrete (discussed above) and the bench/walkway (discussed in Appendix F).

A single bi-level tunnel requires more internal structures. Firstly the upper deck requires a reinforced concrete road deck that is generally attached at each end to the segmental tunnel lining. It is generally laterally free at one end to prevent point loads on the tunnel lining. The lower roadway can either be supported on a concrete deck, or on fill concrete. As shown in Figure 27, vertical reinforced concrete partition walls can provide separation for ventilation ducts, utility chases, emergency egress corridors and emergency egress stairways between decks.

As discussed in Appendix F, fireproofing is an important consideration. Concrete must be protected by a combination of fire-resistant panels, spray-on fireproofing, segment mix design, fire-suppression systems, and other means.

![Figure 25](image-url)  
**Figure 25:** Precast Segments for Eurasia Tunnel, Turkey

![Figure 26](image-url)  
**Figure 26:** Installing backfill and utility corridor in Groene Hart Tunnel, Netherlands

![Figure 27](image-url)  
**Figure 27:** Paris AB6 Tunnel – Installing Lower Deck
2.3 CROSS PASSAGES

2.3.1 CROSS PASSAGE GEOMETRY

Emergency egress/access will be required, in conformance to NFPA 502 recommendations. For twin bored tunnels cross passages will be required, for a single bi-level bored pressurized stairways between roadway decks will be needed.

Any tunnel alternative consisting of two, parallel unidirectional tunnels will require cross passages between tunnels to conform to the requirements of NFPA 502. Figure 28 shows a typical cross passage arrangement.

Figure 29 shows a typical cross-section passage. Each cross passage should have a minimum interior width of 17 feet to accommodate a 5 foot wide excavation walkway, a 3 foot wide utility space, two 3 foot wide conduit spaces and three 12"-in CMU wall partitions, as shown in the figure. The cross passage will have fire-rated doors at each end, to conform to NFPA 502 requirements. The selected dimension can accommodate cable or conduit racks along both sides of the cross passage, protected by a fire-rated partition wall.

For planning purposes, it is assumed that the cross passages will have a lining thickness of 18 inches, double reinforced. A thicker flat invert is proposed, to resist higher bending stresses and to accommodate conduits. The lining will resist loads from groundwater, rock and soil.

This results in a nominal minimum excavation width of 20 feet. The cross-passage floor will be at roadway elevation, as shown in the figure. The cross passage will have an arched roof.

The interface between the cross passage and main tunnel will require a cast-in-place concrete closure pour. The fire-rated door will be located in this closure section.

2.3.2 CROSS PASSAGE CONSTRUCTION IN ROCK (BETWEEN BORED TUNNELS)

The natural tendency of an excavated opening in thinly-bedded shale is to form a natural corbelled arch, as the shale progressively ravel. In Syracuse, this tendency would be enhanced by the anticipated high horizontal stress condition. The immediate installation of the precast concrete segmental lining at the rear of the Tunnel Boring Machine will prevent the development of this corbelled arch. However, removal of segments from a minimum of four consecutive rings within each tunnel will be required at each cross-passage location (20 linear feet of tunnel). The removal of the segments may result in immediate fallout of rock from above tunnel springline. The segment removal procedure must consider this possibility.

Once the segments have been removed, unshored rock reinforcement elements, such as Swellex bolts, should be installed in the exposed rock in combination with welded wire fabric (WWF). The rock reinforcement will tie the thinly-bedded rock together, to prevent stress-induced buckling and additional fallout and the welded wire fabric will minimize raveling of the rock surface between rock reinforcement elements. The rock reinforcement should be installed on a 5 foot X 5 foot pattern and the WWF opening is anticipated to be appropriate for these loads.

The cross-passage final lining should be designed for rock loads and for the theoretical water pressure at the cross-passage location. The assumed wall thickness of 18 inches is anticipated to be appropriate for these loads.

2.3.3 CROSS PASSAGE CONSTRUCTION IN SOFT GROUND (BETWEEN BORED TUNNELS)

Some alternatives may require cross passage construction in soft ground (soil). The soil formations along the alternative alignments are glacial outwash sands and gravels and glacial lake days and silts, which are described in Appendix D. Although the final cross passage geometry will be the same as for cross passages excavated in rock, the construction methodology required to achieve the end result will be different.

Excavation will require ground treatment to stabilize the soil units to prevent ground loss and possible surface subsidence associated with ground loss. Because of the variable soil conditions, inability to obtain surface access at some possible cross passage locations, and geometric constraints imposed by operating within the mainline tunnels, ground freezing may be the optimal methods for ground stabilization.

The ground freezing method requires low temperature brine to be circulated within freeze pipes, which results in freezing of the soil pore water. This results in a self-supporting mass of frozen ground that provides strength and groundwater cut-off. A series of horizontal freeze pipes will be drilled around the perimeter of the final cross-passage section from within the mainline tunnels. Brine will...
be circulated to the individual cross passage locations from a freeze plant located on the ground surface.

After effective freezing is completed, as determined by thermocouples installed in the ground and probing, excavation will be performed. It is likely that frozen ground will extend into the excavation cross-section. This frozen ground will be removed using mechanical excavation, as described for the cross passages in rock.

After excavation is completed, insulation will be applied to the frozen ground surface, followed by installation of waterproofing membrane, as described for the cross passages in rock. Placement of concrete will follow placement of the waterproofing membrane (Figure 30). Additional concrete will be placed over the exterior reinforcement of the lining to account for possible frost effects, even with the placement of insulation. Bar size and spacing in the cross passage lining can be varied to suit site-specific combination of groundwater and earth pressure.

An alternative to ground freezing is jet grouting. In the jet grouting method, the equipment drills to the required elevation, and the drill stems are withdrawn while injecting pressurized grout in a spiral manner. This results in overlapping columns of soil-mixed-with-cement that create a stable, watertight, zone. Grouting would be performed from the surface prior to the TBMs passing. It requires surface access, appropriate ground conditions (generally sands and gravels rather than silts and clays) and is limited to approximately 100-ft depth. In final design, the optimal ground treatment methods for each cross-passage would be evaluated. For this study, it is assumed that soft ground cross passages will use ground freezing.

2.3.4 EMERGENCY EXIT DOORS (BETWEEN CUT AND COVER TUNNELS)

Most cut and cover tunnels will be a single structure, where the northbound and southbound lanes will be separated by a dividing wall. Emergency exit doors will be provided in the dividing wall to facilitate access from the incipient tunnel to the ‘non-incident’ tunnel, and vice versa for emergency responders.

2.3.5 EGRESS STAIR SHAFTS

Access/egress shafts could be used instead of cross-passages, but are generally not recommended on grounds of cost, surface property impacts and being less usable by mobility impaired persons. Egress stairs may be suitable for within the cut and cover tunnels, especially on the Blue Alternate near West Street where the northbound cut and cover tunnel and southbound cut and cover tunnel are physically separate structures.

If used, stairs should have minimum interior dimensions of 12 feet x 20 feet to accommodate a scissor stair. A fire-rated door will be installed at the egress shaft access point at each roadway level. A hatch should be installed at street grade, flush with the sidewalk. The hatch should be clearly marked, with a sign forbidding placement of material on the hatch cover. The hatch doors should be designed to facilitate opening from the inside. Access to the shaft from the outside should require a key to prevent unauthorized access. First responder organizations should be provided with keys to the hatch doors. Hatch locations should be monitored by CCTV.

2.4 SPOIL (AKA “MUCK”)

Excavation of the cut and cover portion of the tunnels, the TBM mined tunnels and the cross passages will generate significant spoil (“waste”) material, often referred to as muck for tunneling operations. The tunneling options in this study would each generate large total volumes of spoil from 500,000 to 1,000,000 cubic yards or more, but the volume is spread out over the many months of tunneling operations. Efficiently handling, temporarily storing, removing and transporting from the site and disposing of the spoil (also referred to as “muck”) is key to successful tunneling operations. Tunneling operations will probably occur during both day and night shifts so much generation should be anticipated on all working shifts. As the material is excavated from the tunnel, the volume of material will expand or swell on the order of 1.3 to 1.5 times the in place volume (difference between bank cubic yards and loose cubic yards). This will require the project to store muck on-site during periods when hauling may not be permitted (e.g. during night time hours). At this early stage of the project, it is too early to identify a selected disposal site(s). However, this project site, with easy access to highways and with several landfills, quarries and sand and gravel operations within 30 miles suggest that there will be multiple options available for disposal sites. Often times on similar projects, disposal of the excavated material is left up to bidding contractors and market forces. The contractor with a good plan to deal with this issue—(e.g. "sell" the material as fill to another project)—will have a lower bid and the project benefits from that competition.
2.5 STRUCTURAL DURABILITY

2.5.1 ELEVATED RISK OF CORROSION

Chloride ions, when in solution with a supply of oxygen, can permeate through concrete cover and cause rusting of steel. This, in turn, can result in section loss and cracking. Reinforced concrete structures in Syracuse are at an elevated risk of such corrosion.

Groundwater is presumed to be saline, based on proximity to historical salt production from groundwater, and limited contemporary data (see Appendix D). Data from USGS (July 2000) shows that the salinity is around 5%. The external face of the tunnel will potentially be exposed to this condition.

The annual average snowfall of Syracuse is approximately 124 inches, or 10 feet per year. This requires large quantities of road salt to be applied. Any open approaches to the tunnel would require road salt, particularly due to gradients of up to 6% required to transition to/from the tunnel. These salts are likely to be tracked into the bored tunnels by vehicles.

Based on these two factors, reinforced concrete structures will need to be designed with specific measures to control corrosion. Such measures will also protect against other potential causes of corrosion such as carbonation and sulfate attack.

It is anticipated that the design life for the tunnel will be 100 years, prior to requiring significant repairs. Detailed analyses can be performed to determine appropriate protection measures, which can include reference to standard texts such as the Guide to Durable Concrete (ACI-201.2), Guide to Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structure (ACI-222.3R) as well as predictive software such as Life (ACI-304.2). Steel fiber concrete has better corrosion resistance than concrete with reinforcing bars. ACI 544.1R indicates that the depth of corrosion is typically limited to 0.10". It may be possible to design the segmental tunnel liners using steel fiber concrete. The concrete mix should be designed to have a very low permeability. A low water-cement ratio should be used, in addition to using pozzolanic additives such as blast furnace slag, fly ash or (less commonly) silica fume. Slag and fly ash do not significantly add to the cost of concrete. Corrosion inhibitors such as calcium nitrite can be added to the mix, and this is recommended. Alternatively, waterproofing additives such as manufactured by Kryton and Xypex can be used. These may be applicable in cross-passages in lieu of, or in addition to, a waterproofing membrane.

2.5.2 PROTECTIVE MEMBRANES AND GASKETS

Cut and cover structures and open cut structures should be protected from groundwater infiltration by including a waterproofing membrane between the support-of-excision system and the permanent structure. This significantly reduces the inflow of water, and the associated supply of salt solution. Some residual leakage may occur (even after corrective measures such as grouting), and evaporation of such leaks could result in a localized build-up of salt on the walls or roof. This would result in conditions that could accelerate corrosion. Additional measures are therefore required.

It is not possible to include such membranes around segmentally lined bored tunnels. At cross-passages the effectiveness (and cost effectiveness) of such membranes is questionable. In both cases, alternative measures are required.

Each bored tunnel segment will be surrounded by EPDM (synthetic rubber) gaskets. Cast-in gaskets are recommended for improved watertightness. Double gaskets (one near the inside face, one near the outside face) and interconnecting “ladder rung” gaskets are recommended. The second gasket provides added assurance against infiltration of saline water, methane and hydrogen sulfide. Higher strength dowels (or bolts) may be required between rings to maintain compression in the gaskets during ring installation.

2.5.3 STEEL FIBERS VS REINFORCING BARS (REBAR)

Steel fiber concrete has better corrosion resistance than concrete with reinforcing bars. ACI 544.1R indicates that the depth of corrosion is typically limited to 0.10". It may be possible to design the segmental tunnel liners using only steel fibers. This is confirmed by ACI 544.2. However, it is anticipated that due to the size of the precast segments and the stresses induced during demoulding and handling, rebar cages will be required.

2.5.4 CONCRETE MIX DESIGN:

The quality of concrete surrounding reinforcing bars is of primary importance in protecting the bars from corrosion. The concrete mix should be designed to have a very low permeability. A low water-cement ratio should be used, in addition to using pozzolanic additives such as blast furnace slag, fly ash or (less commonly) silica fume. Slag and fly ash do not significantly add to the cost of concrete. Corrosion inhibitors such as calcium nitrite can be added to the mix, and this is recommended. Alternatively, waterproofing additives such as manufactured by Kryton and Xypex can be used. These may be applicable in pass-through of seawater environment.

2.5.5 CONCRETE COVER

Sufficient concrete cover should be provided to achieve the desired design life. The minimum concrete cover required by AASHTO for protection of reinforcement is 4.0 inches for concrete with direct exposure to salt water, and 3.0 inches for concrete exposed to soil. Concrete mix should either 3.0 or 4.0 inches of cover is likely to be applicable, depending on other corrosion control measures and the degree of exposure. The Midtown Tunnel, VA, used 3 inches minimum cover in a seawater environment.

2.5.6 STRUCTURAL DESIGN

Structures should be designed to control crack widths by minimizing strain in tensile rebars. Typically this is achieved by designing as environmental concrete structures in accordance with ACI 350 which states that “Below-grade structures... which may be exposed to external groundwater pressures, generally are designed as environmental concrete structures”. If other corrosion control measures are used less conservatively designs may be permissible.

2.5.7 REBAR CORROSION CONTROL MEASURES

Epoxy coated rebar is frequently used to increase corrosion resistance. The epoxy coating is applied in a factory to the steel prior to shipping, so field bending is not possible. During handling and construction, coating defects may occur. These are frequently not observed and/or not repaired. Defects will lead to uneven corrosion through time, and contribute to the acceleration of corrosion. Although meriting further consideration, epoxy coated rebar is not currently recommended for the tunnels.

Stainless Steel typically has excellent durability, but its performance in saline environments is dependent on the specific stainless composition used. The grade supplied may not always meet the specification. The combination of high cost and questionable performance mean that stainless steel rebar is not recommended.

Galvanized reinforcing steel is hot-dipped prior to delivery. It can be bent on site, but the zinc coating may not always meet the specification. The combination of high cost and questionable performance mean that stainless steel rebar is not recommended.

Anodic protection is a technique to control the corrosion of rebar by attaching zinc ‘packs’, which corrode instead of the steel. The technique is commonly used during repairs of bridge decks exposed to deicing salts. The protection is unlikely to last for 100-years, and is not considered suitable for protecting the tunnels.

Impressed Current Cathodic Protection Systems (ICCP) use direct current (typically) to prevent the corrosion process from occurring. Some systems automatically adjust the current output to optimally protect the target structure. Typically, a protection system for a tunnel would be designed with multiple zones with separate cathodic protection transformer-rectifier circuits for each. Steel rebars within each zone would be welded for electrical continuity. In some tunnel structures, electrical continuity is provided during construction which enables installation of an impressed current at a later date should corrosion/potential difference measurement indicate this is required. During final design consideration should be given to either make provisions for a future system, or to install a fully operation system. The Midtown Tunnel, VA, used the preparatory approach in a seawater environment.
Potential Construction Staging Area - ALL Alternatives - South Portal
Potential Construction Staging Areas - Green Alternative - North Portal
Potential Construction Staging Areas - Blue Alternative - West Street
Potential Construction Staging Area - Blue Alternative - North Portal
Potential Construction Staging Area - Red Alternative - North Portal
Potential Construction Staging Areas - Orange Alternative - North Portal
Note: Red Alignment shown here is indicative and is for staging purposes only.
Proposed Red Alignment - Stage 1
*Current I-81 Viaduct to remain intact.
*Close off MLK and Renwick to traffic during construction.
*Soil to be flattened on the west embankment of viaduct to street elevation, set up Construction Staging Area.
*Retaining walls and soil embankment on the east of viaduct to be elevated to set up I-81 extension.
Proposed Red Alignment - Stage 2
*Construct 4 lane bi-directional traffic for temporary I-81 extension.
*Modify existing I-81 viaduct, removal of existing jersey barriers and replacement of new mediums and barriers.
Proposed Red Alignment - Stage 3
*Route SB and NB traffic over I-81 extension.
*Partial demolition of existing I-81 Viaduct in preparation of cut and cover excavation.
Proposed Red Alignment - Stage 4
*Begin Cut and cover tunnel and depressed roadway excavation and construction.
Proposed Red Alignment - Stage 5

*Re-deck SB viaduct over cut and cover tunnel/depressed roadway, install mediums/jersey barriers, and re-establish SB traffic.

*Launch TBM to north.
Proposed Red Alignment - Stage 6
*Construct ramp to depressed roadway
Proposed Red Alignment - Stage 7
*Open access to SB and NB traffic to tunnel.
*Reopen MLK/Renwick
Proposed Red Alignment - Stage 1
*Route SB traffic onto community grid-Genant Drive. Route NB traffic onto former SB lanes. Close off NB lanes and on ramp.
*Set up Construction Staging Area.
Proposed Red Alignment - Stage 2
*Install SOE of shaft and NB Cut and Cover Part-tunnel.
*Begin excavation and construction of TBM reception shaft (Assumed 1 shaft, Shape rectangular).
Proposed Red Alignment - Stage 3
*Prepare shaft and receive TBMs from south.
*Begin excavation and construction of NB Cut and Cover part-tunnel.
Proposed Red Alignment - Stage 4
*Deck over shaft.
*Back fill on top of NB cut and cover part-tunnel, widen I-81 NB roadway after Spencer Street Bridge and re-pave roadway.
Proposed Red Alignment - Stage 5
*Reroute NB traffic on the newly paved NB I-81 roadway grade.
*Demolish limited length of former SB I-81 roadway grade.
Proposed Red Alignment - Stage 6
*Install SOE of SB Cut and Cover Part-tunnel.
Proposed Red Alignment - Stage 7
*Begin excavation and construction of SB Cut and Cover part-tunnel, connecting to existing NB Cut and Cover.
Proposed Red Alignment - Stage 8
*Back fill on top of SB cut and cover part-tunnel, widen I-81 SB roadway after Spencer Street Bridge and re-pave roadway.
Proposed Red Alignment - Stage 9
*Reroute SB traffic on the newly paved SB I-81 roadway grade.
*Re-establish all on and off ramps.
LIMIT OF CROWNE PLAZA HOTEL

CAP SLAB

GROUND LEVEL
EL. 402.81

TENSION/SETTLEMENT CONTROL PILES

ASSUMED LIMIT OF PILE FOUNDATION

EXISTING I-81 OFF RAMP

EXISTING I-81 VIADUCT

ESTIMATED TOP OF ROCK
EL. 325.81

BORED TUNNEL - GREEN ALTERNATIVE
SECTION AT STA 116+00