Settlement Screening Analysis for the Baltimore Red Line

James Parkes[1], Harald Cordes[2], Jason Kotova[3], and Sotirios Vardakos[4]
[1]Parsons Brinckerhoff, Baltimore MD; [2]Parsons Brinckerhoff, Washington, D.C.; [3]Parsons Brinckerhoff, Baltimore, MD; [4]Parsons Brinckerhoff, Chicago, IL

ABSTRACT

The Baltimore Red Line is a proposed light rail transit line that includes a 3.4-mile Downtown Tunnel (DTT) segment through Baltimore City. The DTT consists of twin bored, 23-foot diameter tunnels, and five cut-and-cover underground stations. Construction of the DTT poses potential risk of excavation impacts to adjacent structures, which include masonry residential and commercial buildings, modern high rises, and historic structures. As part of a multi-stage evaluation, an initial screening assessment has been performed using analytical and empirical methods to estimate settlements and related impacts due to tunnel and station excavations. The screening assessment included detailed estimation of volume losses, comparison of the results of different methods, damage thresholds for buildings and utilities, and identification of shortcomings of the methods.

INTRODUCTION

The Baltimore Red Line is a proposed light rail transit line for the Maryland Transit Administration (MTA) that will run from East to West through Baltimore City and County. The project includes the Downtown Tunnel (DTT) segment consisting of 3.4 miles of twin running tunnels, two portals, and five underground stations. The tunnels have an approximate outside diameter of 23 feet and will be mined using pressurized face tunnel boring machines (TBMs). The stations have two or three levels underground and have approximate plan dimensions of 66-feet wide by 285-feet long. Portals and stations will be built using cut-and-cover techniques with slurry walls utilized for both temporary support of excavation (SOE) and permanent structural walls. The DTT passes through an urban area including the Central Business District near the Inner Harbor as well as the Harbor East and Fell's Point neighborhoods. The DTT alignment is shown in Figure 1. Existing structures along the alignment vary considerably and include modern steel and concrete framed low- to high-rise structures to 19th century brick row homes, commercial buildings, and warehouses, as well as a dense network of utilities.

Figure 1: DTT alignment location

GEOTECHNICAL CONDITIONS ALONG THE ALIGNMENT

The invert depth for the tunnels varies from 45 feet at the portals to up to 100 feet, with the majority of invert levels in the range of 65 to 85 feet in order to connect at the two- and three-level underground stations. Subsurface conditions along the DTT consist of variable fill and Coastal Plain sediments overlying crystalline rock of the Piedmont Plateau. Predominant rock types include igneous and metamorphic rocks consisting primarily of amphibolite and gneiss, with lesser amount of schist, marble, and other rock types.

Overlying the rock are Residual Soil and Transition Group materials, which consist of overburden derived from in-situ weathering of the parent bedrock. Residual Soil is completely decomposed and does not contain any relict rock characteristics. Residual Soils are only present in areas of limited extent. Transition Group materials are highly weathered and consist of soil-like materials that retain relict rock fabric and joints. These materials exhibit both soil and rock behavior. Residual Soil and Transition Group materials are very dense or hard based on SPT N-values.

Overlying the Residual Soil and Transition Group are Coastal Plain sediments consisting of Cretaceous and post-Cretaceous sediments. Cretaceous sediments are highly variable and include sand, gravel, and clay layers, although most of these sediments are granular with isolated interbedded clay layers. Cretaceous soils vary from clean to silty or clayey sands and gravels and are very dense.

The eastern portion of the DTT, from the Inner Harbor Station to the east portal, includes areas of in-filled marshes or reclaimed land. West of the Inner Harbor Station, the DTT runs through an upland area. Post-Cretaceous sediments are present in the eastern portion and are thicker in areas of reclaimed land. These sediments are similar to the Cretaceous sediments, although tend to be medium dense sands and gravels with occasional medium stiff to stiff interbedded clay. In areas of reclaimed land or former marsh, loose sands and soft organic silts are also present.

Fill overlies the post-Cretaceous sediments in the eastern section and the Cretaceous sediments in the western section of the DTT. Fill is highly variable in density and composition and includes brick, timbers, and other debris.

Groundwater levels are generally within 5 to 15 feet of the ground surface along the entire DTT. The Cretaceous soils are regional aquifers and a limited tidal influence is observed along the existing and former waterfront.

The DTT running tunnels encounter all of the materials described above except fill. The majority of the tunnels run through rock, Transition Group, and Cretaceous sediments. Mining conditions vary from full face conditions in rock, Transition Group, and Cretaceous soils, to mixed ground conditions consisting of a combination of these materials.

The station and portal excavations will encounter all of the materials described above. Some stations, such as Poppleton and Howard Street Stations, will encounter fill, Cretaceous soils, Transition Group, and rock. Other Stations, such as Inner Harbor and Fell’s Point, will encounter post-Cretaceous sediments but not rock.

EXCAVATION IMPACTS ON ADJACENT STRUCTURES STUDY

A settlement screening analysis has been performed for a Preliminary Engineering (PE) study on Excavation Impacts to Adjacent Structures (EIAS). The EIAS study was developed to assess ground deformations due to excavation of the tunnels and stations and the impact of those deformations on adjacent structures.

EIAS Background and Assumptions

The EIAS study involved analytical, empirical, and numerical analyses. The EIAS study pertained only to movements caused by excavations. Groundwater drawdown and blasting impacts are not addressed. Catastrophic events, such as tunnel blow-ins or SOE failure, are not considered either.

Staged Approach for Excavation Impact Assessments

The EIAS assessment follows a rational multi-stage process. This general process is applicable to both mined tunnel and cut-and-cover excavations. It consists of:

1. Perform a screening assessment using analytical or empirical methods to estimate the limits and magnitude of settlements. Damage assessment criteria are used to evaluate, whether adjacent structures may be at risk. The Stage 1 assessment is the focus of this paper.

2. Perform a detailed assessment of locations identified in Stage 1, including consideration for foundation loads. Numerical modeling is used for the Stage 2 analyses for the Red Line because it allows for a more detailed analysis and facilitates parametric studies. Results are used to further assess damage potential.

3. Perform detailed site specific analysis using soil-structure interaction and structural response. This considers the structure type, condition, and building stiffness to determine how much movement the structure can tolerate.

4. Development of mitigation strategies based on Stages 1 through 3. These may include changing the location of an underground structure, ground improvements, structural improvements, and instrumentation and monitoring.

Stages 1 through 4 are generally performed in order, although in some cases a stage may be omitted (Stage 2 may progress to Stage 4 to assess mitigation measures) or an iterative approach may be used (Stages 2 or 3 may be revisited to assess the results of Stage 4).

Building Damage Criteria. Stage 1 assessments are used as a screening tool to identify the zone of ground movements, estimate settlement contours, and classify structures into risk categories. Ground movements and deformation slopes using Stage 1 assessments are developed from analytical or empirical approaches, generally with spreadsheets, to allow quick assessment of a number of sections covering a large alignment area. Damage assessment criteria for Stage 1 assessments, presented in Figure 2 are simplified in order to assess a large volume of structures quickly. The damage assessment criteria are for masonry structures and are considered conservative for steel or concrete frames structures that can tolerate larger movements.

Generally, structures with “very slight” or “slight” risk did not warrant a Stage 2 analysis. Structures at “slight” damage risk will still require monitoring and may require minor repairs, such as crack filling and repointing, but are not anticipated to require significant mitigation measures. However, some judgment must be applied; for example, historic structures or structures in poor condition may be identified for subsequent Stage 2 analyses despite a low risk classification. All buildings located directly above the tunnels were considered to warrant a Stage 2 assessment because of the increased risk of mining directly underneath the structure. The results of Stage 2 assessments are evaluated using additional criteria, including horizontal strains and angular distortions.

Building damage classification After Burland (1995) and Mair et al (1996)					Approximately equivalent ground settlement and slopes (after Rankin 1988)	
1	2	3	4	5	6	7
Risk Category	Description of degree of damage	**Description of typical and likely forms of repair for typical masonry buildings**	**Approx. Crack width (in)**	**Max. tensile strain (%)**	Max slope of ground	Max settlement of building (in)
0	Negligible	Hairline cracks.	0.004	Less than 0.05		
1	Very slight	Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building. Cracks in exterior visible upon close inspection.	0.004 to 0.04	0.05 to 0.075	Less than 1:500	Less than 0.4
2	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible; some repainting may be required for weather-tightness. Doors and windows may stick slightly.	0.04 to 0.2	0.075 to 0.15	1:500 to 1:200	0.4 to 2.0
3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Brick pointing and possible replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weather tightness often impaired.	0.2 to 0.6, or a number of cracks greater than 0.12	0.15 to 0.3	1:200 to 1:50	2.0 to 3.0
4	Severe	Extensive crack repair involving removal and replacement of walls, especially over door and windows required. Window and door frames distorted. Floor slopes noticeably. Some loss of bearing in beams. Utility services disrupted.	0.6 to 1.0, but also depends on number of cracks	Greater than 0.3	1:200 to 1: 50	Greater than 3.0
5	Very severe	Major repair required involving partial or complete reconstruction. Beams lose bearing, walls lean badly and require shoring. Windows broken by distortion. Danger of instability.	Usually greater than 1.0 but depends on number of cracks.		Greater than 1:50	Greater than 3.0

Source: Adapted from Loganathan 2011

Figure 2: Analytical/Empirical Damage Assessment Criteria

Utility Damage Criteria. Deformation thresholds for buried utilities have been developed based on references (Attewell et. al. 1986; O'Rourke and Trautmann 1982), similar projects, and project-specific considerations. Deformation criteria are summarized in Table 1; values exceeding the thresholds indicate that a utility may be at-risk of damage. Both vertical and lateral ground movements are needed to assess utility damage potential.

Three modes of stress for buried pipelines are used to assess damage potential:

1. Straining of the pipe caused by flexural deformations that result in pipe rupture or intolerable deformation
2. Opening of joints due to rotation between pipe segments
3. Tensile pull-apart of joints caused by tensile axial movements along the pipeline

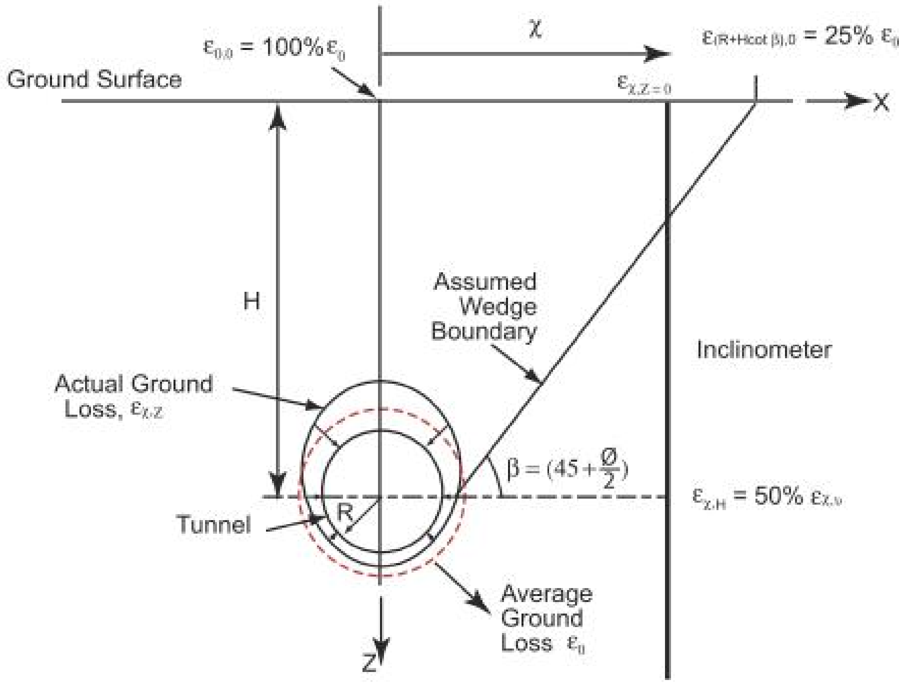
Deformation thresholds are reduced from published values for new pipelines to include consideration for pre-existing deformations/strains. Not all thresholds are applicable to all pipelines; joints opening and pull-apart are not considered for brick, cast-in-place concrete, or welded steel pipe. Joint pull apart is not considered for pipelines less than 8 inches in diameter because they behave like flexible pipelines (O'Rourke and Trautmann 1982). Cast iron pipes, which are generally water mains, are more critical because they provide water supply for fire fighters. Cast iron pipes are analyzed for joint deformation for diameters greater than 6 inches, and threshold values for cast iron pipes are very stringent because of the fire-life safety considerations. The threshold values in Table 1 are used for initial screening analyses; final design values may be revised based on evaluation by the utility owners.

Table 1: Utility Deformation Thresholds for Stage 1 Assessment

<table>
<tr><th>Utility Material</th><th>Dia. (in)</th><th>Allowable Joint Pull-Apart (in)</th><th>Allowable Joint Rotation (degrees)</th><th>Allowable Tensile Strain (microstrain)</th></tr>
<tr><td>Brick</td><td>All</td><td>N/A</td><td>N/A</td><td>150</td></tr>
<tr><td>Welded Steel Pipe (WSP)</td><td>All</td><td>N/A</td><td>N/A</td><td>600</td></tr>
<tr><td>Cast-in-Place Concrete (CIP)</td><td>All</td><td>N/A</td><td>N/A</td><td>300</td></tr>
<tr><td>Reinforced Concrete Pipe (RCP)</td><td>All</td><td>0.40</td><td>0.25</td><td>300</td></tr>
<tr><td>Terra Cottta Pipe (TCP)</td><td>All</td><td>0.40</td><td>0.25</td><td>150</td></tr>
<tr><td>Vitrified Clay Pipe (VCP)</td><td>All</td><td>0.40</td><td>0.25</td><td>300</td></tr>
<tr><td>Ductile Iron Pipe (DIP)</td><td>All</td><td>0.40</td><td>0.25</td><td>600</td></tr>
<tr><td rowspan="9">Cast Iron Pipe (CI)</td><td>6</td><td>0.08</td><td>1.10</td><td rowspan="9">400</td></tr>
<tr><td>8</td><td>0.08</td><td>0.90</td></tr>
<tr><td>10</td><td>0.08</td><td>0.80</td></tr>
<tr><td>12</td><td>0.08</td><td>0.70</td></tr>
<tr><td>16</td><td>0.08</td><td>0.50</td></tr>
<tr><td>20</td><td>0.07</td><td>0.40</td></tr>
<tr><td>24</td><td>0.06</td><td>0.30</td></tr>
<tr><td>30</td><td>0.06</td><td>0.20</td></tr>
<tr><td>40</td><td>0.05</td><td>0.15</td></tr>
</table>

STAGE 1 SCREENING ASSESSMENTS FOR MINED TUNNELS

The Stage 1 assessment for mined tunnels used an analytical approach developed by Loganathan (2011) to estimate ground movements. Ground deformations and closed form equations are presented in Figure 3.



Surface Settlement

$$U_{z=0} = \varepsilon_0 R^2 . \frac{4H(1-\nu)}{H^2 + x^2} . \exp\left\{ - \frac{1.38x^2}{(H\cot\beta + R)^2} \right\}$$

Subsurface Settlement

$$U_z = \varepsilon_0 R^2 \left(- \frac{z-H}{x^2 + (z-H)^2} + (3-4\nu) \frac{z+H}{x^2 + (z+H)^2} - \frac{2z[x^2 - (z+H)^2]}{[x^2 + (z+H)^2]^2} \right) \cdot \exp\left\{ - \left[\frac{1.38x^2}{(H\cot\beta + R)^2} + \frac{0.69z^2}{H^2} \right] \right\}$$

Lateral Deformation

$$U_x = -\varepsilon_0 R^2 x \left[\frac{1}{x^2 + (H-z)^2} + \frac{3-4\nu}{x^2 + (H+z)^2} - \frac{4z(z+H)}{(x^2 + (H+z)^2)^2} \right] \cdot \exp\left\{ - \left[\frac{1.38x^2}{(H\cot\beta + R)^2} + \frac{0.69z^2}{H^2} \right] \right\}$$

Where:

$U_{z=0}$ = Ground surface settlement at transverse distance to centerline
U_z = subsurface settlement at transverse distance to centerline
U_x = Lateral deformations
ε_o = average ground loss ratio
R= Radius of the tunnel
H = depth of the tunnel below ground at springline
z = depth below ground surface
x = lateral distance from tunnel center line
β = limit angle
ν = Poisson's ratio of soil

Source: Loganathan 2011

Figure 3: Ground Deformation Patterns and Equations for Closed Form Solution

The analytical approach is used to develop settlements for "green field" conditions. Limits and contours of settlements are used to assess damage risk using the criteria in Figure 2 and Table 1. This approach assumes that settlements result from volume losses due to tunneling. Therefore, volume loss estimates are key to the assessment.

Estimation of Volume Losses

Volume loss is the amount of soil excavated in addition to the theoretical tunnel volume, expressed as a percentage of the theoretical tunnel volume. Volume losses for TBM mined tunnels result from:

- Soils running, flowing, or collapsing into the face of an advancing tunnel.
- Closure of soils around the over-cut of the tunneling shield.
- Closure of the tail void space between the excavated diameter and the tunnel liner.

Volume losses vary by ground conditions and the alignment geometry. Higher losses occur when mining in multiple soil types and on curves, inclines, or areas with reduced tunnel separation (pillar). The DTT tunnel was subdivided into 60 sections, ranging in length from 75 to 1150 feet, based on mining conditions or alignment geometry.

Volume losses were estimated for "average" TBM control based on references, case histories, and engineering judgment. Volume loss estimates were intended to be reasonable yet conservative for the initial screening assessment. Estimates were initially developed for mining in full face conditions in one material type, then those values were adjusted for mining in mixed face conditions consisting of multiple material types. High and low values for "poor" and "good" TBM control were also developed for parametric study. Adjustment factors were developed for mining on curves, on inclines, and reduced pillar width. The resulting volume losses, summarized in Table 2, were used to estimate the amount of settlement for each section of the DTT.

Table 2: Summary of volume losses for Stage 1 Assessment of Red Line DTT tunnels

Ground Conditions at Tunnel Face	Volume Losses, straight tunnel			Adjustment factors for alignment conditions				
	"Average" TBM control	"Good" TBM control	"Poor" TBM control	Vertical curve	Inclined alignment	Horizontal Curve (R > 900')	Horizontal Curve (R < 900')	Reduced pillar width
Cretaceous	**0.65 %**	**0.45 %**	**0.85 %**	+ 0.15 %	+ 0.15 %	+ 0.10 %	+ 0.25 %	+ 0.40 %
Transition Group	**0.75 %**	**0.50 %**	**1.00 %**	+ 0.15 %	-	+ 0.10 %	+ 0.25 %	+ 0.40 %
Post-Cretaceous + Cretaceous	**0.80 %**	**0.60 %**	**1.00 %**	+ 0.15 %	+ 0.15 %	+ 0.10 %	+ 0.25 %	+ 0.40 %
Cretaceous + Transition Group	**0.90 %**	**0.65 %**	**1.15 %**	+ 0.15 %	+ 0.15 %	+ 0.10 %	+ 0.25 %	+ 0.40 %
Cretaceous + Transition Group + Rock	**1.00 %**	**0.75 %**	**1.50 %**	+ 0.20 %	+ 0.20 %	+ 0.15 %	+ 0.30 %	+ 0.50 %
Transition Group + Rock	**1.00 %**	**0.75 %**	**1.50 %**	+ 0.20 %	-	+ 0.15 %	+ 0.30 %	+ 0.50 %
Rock	**0.20 %**	**0.10 %**	**0.30 %**	+ 0.05 %	-	+ 0.05 %	+ 0.10 %	+ 0.10 %

The terms "good", "average", and "poor" are relative and subject to engineering judgment. "Average" workmanship relates to properly trained staff and equipment being properly used, such as timely adjustments to ground conditions, use of appropriate face pressures, monitoring of muck volumes versus theoretical excavation volume, proper tail void grouting, etc.

Estimation of Settlements and Zone of Ground Movements

A settlement trough was calculated for each DTT section. A limiting value of 0.05 times the maximum settlement was used to define the limits of the trough since the Gaussian function approaches, but never equals zero. A trough was developed for each tunnel. Superposition was used to estimate overall ground displacement pattern, from which contours of settlement were developed to assess differential settlement and angular distortion at building locations. An example of ground displacement estimates is shown in Figure 4.

As a reasonability check, the maximum settlement for a single tunnel was calculated using the empirical method by Mair (1993). This method is more simplified but is based on observed ground movements and is accepted in industry practice. Results of the empirical method were generally found to be within 10% of the analytical results.

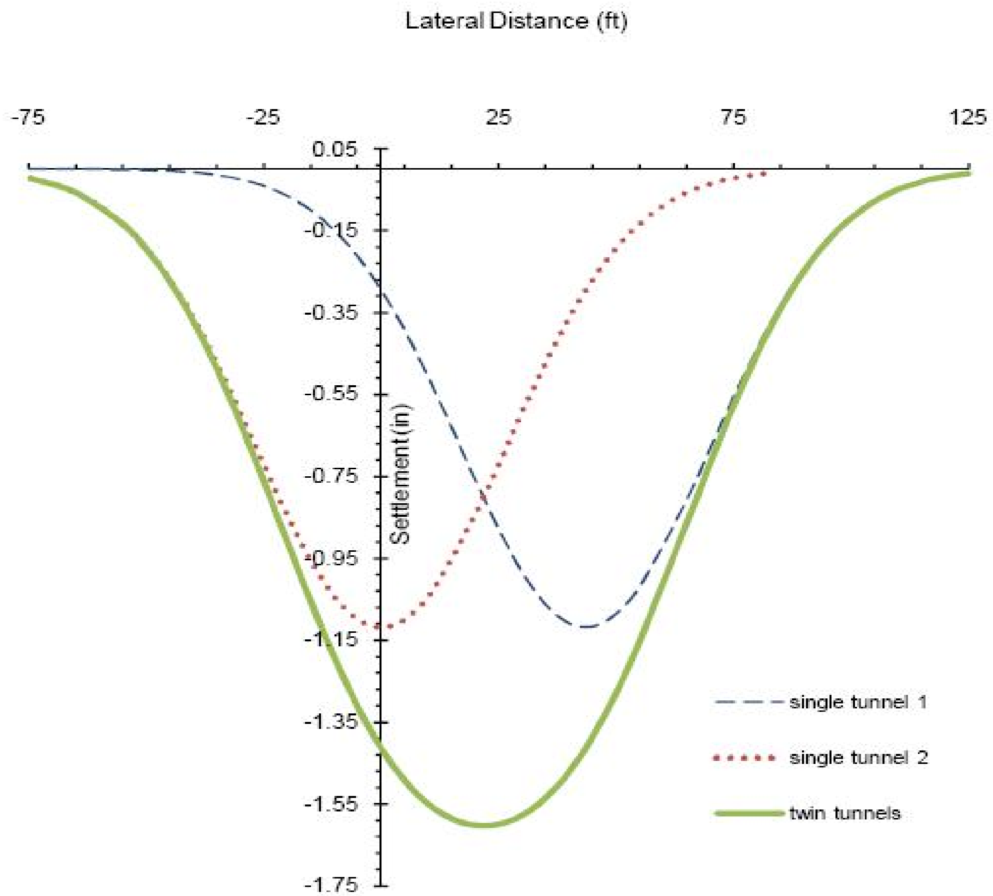


Figure 4: Example of Ground Displacements Estimated from Analytical Approach

Screening Assessment for Mined Tunnels

Settlement contours were developed and overlaid on the alignment plan as shown in Figure 5. Maximum and differential settlements were determined at each building within the zone of settlement and compared with the thresholds in Figure 2. This process was performed for the ground surface and one story below the surface to account for basements; "average" and "poor" TBM control levels were assessed. Structures at "moderate" risk or worse were identified for Stage 2 assessments. Results of the Stage 1 assessment are depicted in Figure 6.

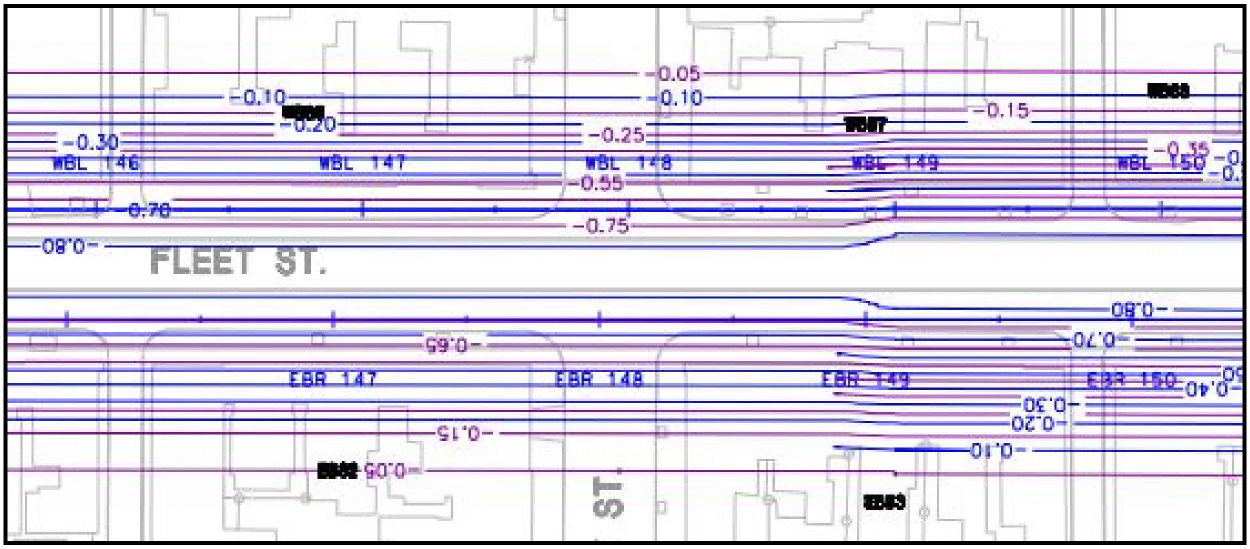


Figure 5: Example of Settlement Contours (inches) developed from analytical approach

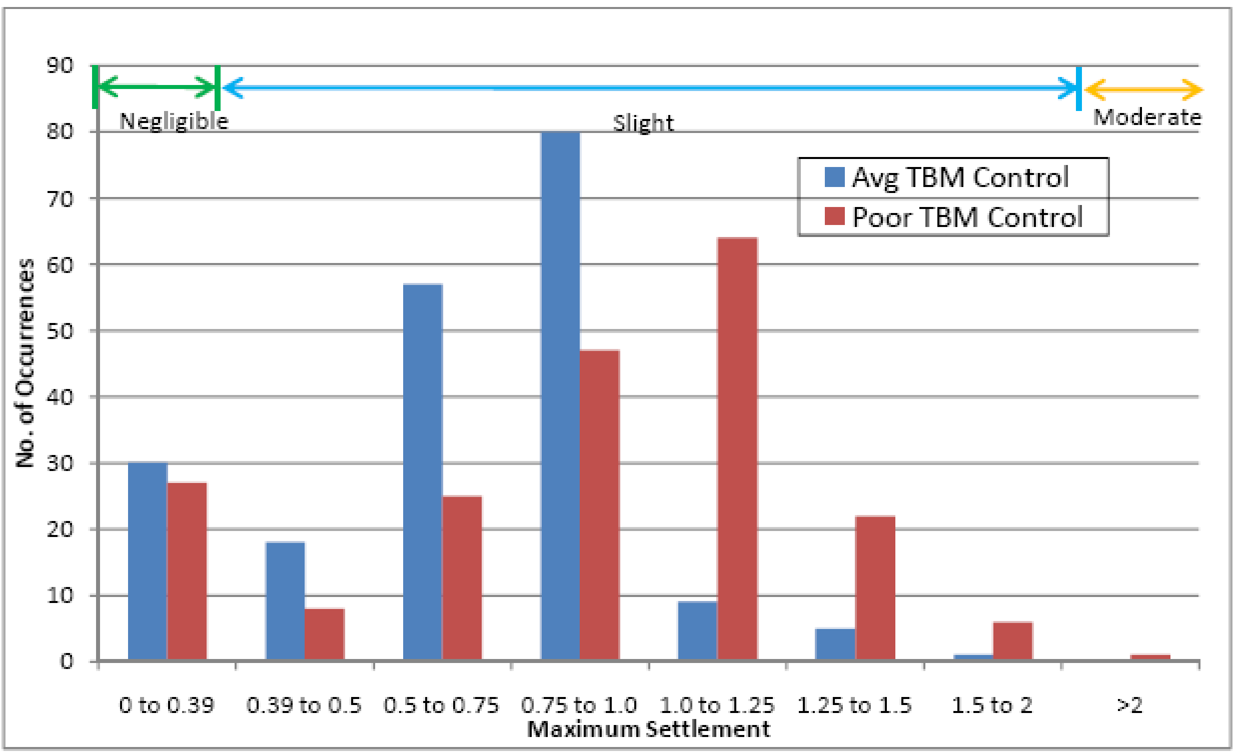


Figure 6: Histogram of Maximum Ground Settlement at Structures for the DTT Stage 1 Assessment

Historic structures and structures that the tunnels pass directly underneath were automatically identified for Stage 2 assessments. Structures that differ from the assumed structure type inherent to the published damage criteria (Figure 2) were also identified for Stage 2 assessments. The published thresholds in Figure 2 were developed for low to midrise shallow bearing structures, generally with length to height ratios (L/H) of 1 or more. Two examples of structures identified for Stage 2 assessments are a tower with an L/H ratio of 0.12 and an existing underground tunnel. A total of eight structures were identified for Stage 2 assessments.

Utilities were evaluated in a similar fashion. Results indicated that approximately 44 percent of the utility crossings were considered at-risk. This does not mean that these utilities will fail; but rather additional analyses and/or mitigation measures, including monitoring, are warranted. Only highly critical utilities were identified for Stage 2 assessments, for a total of 8 additional Stage 2 assessments. Other utilities will experience similar levels of deformation, so those Stage 2 assessments will be applicable to other utilities in similar ground conditions.

Limitations of Screening Assessment. The screening assessment relies on a number of simplifying assumptions. The following limitations are noted for the Stage 1 assessment:

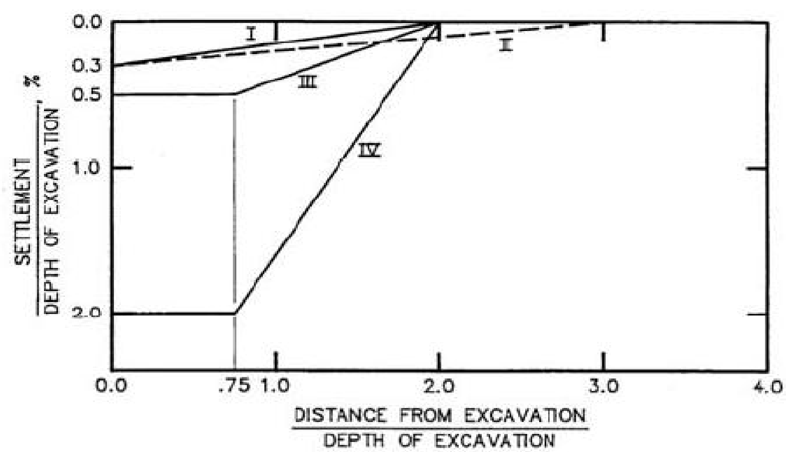
- Applies to a "green field" condition, no external structure loads are accounted for.
- Applies to soft ground conditions; values for rock were included for consistency and completeness.
- Settlement trough volume is equal to the volume losses; no soil arching or volume expansion is considered.
- Movements caused by each tunnel are assumed to be equal.
- No effect from mining of the first tunnel is considered on the second tunnel.
- Damage criteria pertain to masonry structures on shallow foundations with L/H ratios of 1 or more.

STAGE 1 SCREENING ASSESSMENTS FOR CUT-AND-COVER STRUCTURES

Similar to the mined tunnels, a Stage 1 screening assessment using empirical methods was performed to estimate ground movements due to excavation of the cut-and-cover stations and portals.

Stage 1 Estimation of Settlements for Cut-and-Cover Structures

Cut-and-cover structures were designed using AASHTO (2011) specifications, which includes an empirical method for estimating movements caused by multi-level braced SOE systems. This empirical method was developed by Peck (1969) and expanded by Clough and O'Rourke (1990). Settlements are estimated as percentage of the excavation depth as shown in Figure 7.

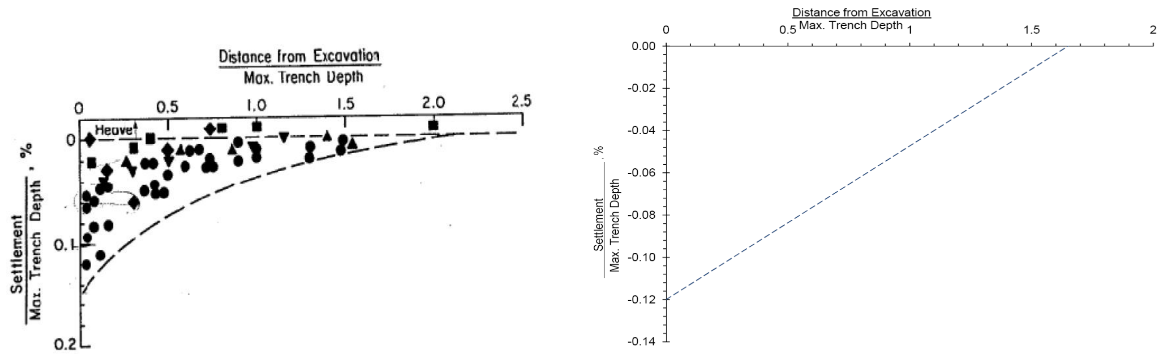


Curve I = Sand
Curve II = Stiff to very hard clay
Curve III = Soft to medium clay, $R_{BH} = 2.0$
Curve IV = Soft to medium clay, $R_{BH} = 1.2$

Source: AASHTO 2011

Figure 7: Empirical method for estimating settlements adjacent to braced walls.

Cut-and-cover structures will be built using slurry walls. Settlements will occur as a result of volume losses during slurry wall excavation. An empirical method, presented in Figure 8, was developed to estimate such settlements based on data presented in Clough and O'Rourke (1990). Total settlements at cut-and-cover excavations were estimated using superposition of the results from Figures 7 and 8.



(a) Clough and O'Rourke (1990) (b) Red Line Stage 1 assessment

Figure 8: Diagrams for Settlements Induced by Excavation of Slurry Wall Elements

Results of Stage 1 Assessment for Cut-and-Cover Structures

Settlements adjacent to portals and stations were estimated to be as high as 2.1 to 4.6 inches, resulting in damage risk classifications of "moderate" to "very severe." Therefore, all cut-and-cover excavations require Stage 2 assessments. It is emphasized that these settlements were not considered highly accurate, but rather were taken as an indication that Stage 2 assessments were required. As of the date of this writing, limited Stage 2 cut-and-cover numerical modeling has been performed and results for settlements are in the range of 0.7 to 1.8 inches adjacent to station excavations. These limited Stage 2 results suggest that the Stage 1 results are not highly accurate.

Limitations of Stage 1 Assessment for Cut-and-Cover Structures

The empirical screening for braced excavations is very simplified. The following limitations are noted:

- Settlements are based solely on depth of excavation, H. Deeper excavations result in greater settlements.
- Figure 7 was developed based on data for all wall types, including flexible walls such as sheetpiles and soldier piles and lagging. Slurry walls are stiffer and will experience less movement.
- Figure 7 indicates maximum settlement for granular soils and stiff clays of 0.3% of H. Clough and O'Rourke (1990) indicate that for excavations in sands, stiff clays, and residual soils (comparable with DTT soils) tend to average about 0.15% of H, which is half of the Figure 7 maximum settlement.
- Figure 7 is based on uniform soil conditions in which the wall toe may deflect laterally. Slurry walls for DTT stations will toe into rock, so wall deflections may be less.
- As a comparison, the Charles Center Station excavation for the existing Baltimore Metro was 66 feet deep and used slurry walls in similar conditions. The Stage 1 approach would predict settlements up to 3.5 inches. Actual observed settlements at adjacent buildings were 0.3 to 1.0 inches (Zeigler et. al. 1984).

SUMMARY AND CONCLUSIONS

A rational multi-stage approach has been developed for evaluating excavation impacts on adjacent structures for the Baltimore Red Line. The approach involves a Stage 1 analytical or empirical screening assessment to identify at-risk structures, a Stage 2 detailed analysis to refine ground movement estimates, a Stage 3 assessment of soil-structure interaction and structural response, and a Stage 4 development of mitigation measures. A Stage 1 screening assessment has been developed based on published procedures and judgment for mined tunnels and cut-and-cover structures. The Stage 1 procedure, results, and limitations have been presented.

ACKNOWLEDGEMENTS

The authors would like to acknowledge Dick Flanagan of the General Engineering Consultant team who provided overall technical guidance and review of the analyses and reports, as well as the Program Management Team who provided review of the design memoranda for the MTA.

REFERENCES

1. American Association of State Highway and Transportation Officials (AASHTO), 2011, *AASHTO LRFD Bridge Design Specifications*, Sixth Edition.
2. Attewell, P.B., Yeates, J., and Selby, A.R., 1986, “Soil Movements Induced by Tunneling and their Effects on Pipelines and Structures.”
3. Clough, G. W., and O’Rourke, T. D., 1990, “Construction Induced Movements of In-Situ Walls”, *Proceedings*, *ASCE Specialty Congress*, *Design and Performance of Earth Retaining Structures*, Cornell University, Ithaca, NY.
4. Loganathan, Nagan, 2011, *An Innovative Method For Assessing Tunneling-Induced Risks to Adjacent Structure*, PB 2009 William Barclay Parsons Fellowship, Monograph 25, Parsons Brinckerhoff, NY, NY.
5. Mair, R.J., 1993 “*Developments in Geotechnical Engineering Research: Application to Tunnels and Deep Excavations,”* Proceedings of Institution of Civil Engineers: Civil Engineering, 93. pp. 27-41.
6. O’Rourke, T.D. and Trautmann, C.H., 1982 “Buried Pipeline Response to Tunneling Ground Movements,” Europipe ‘82 Conference, Basel, Switzerland.
7. Peck, R. B., 1969, “Deep Excavations and Tunneling in Soft Ground,” Proceedings from the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City.
8. Zeigler, E.J., Wirth, J. L., Miller, J. T., 1984, “Slurry Trench Wall Replaces Structure Underpinning,” *International Conference on Case Histories in Geotechnical Engineering*, Vol. III, University of Missouri-Rolla, Rolla, Missouri.