A Systematic Approach for the Protection of Structures Adjacent to Bored and Cut-and-Cover Tunnels for the Regional Connector Transit Project

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ABSTRACT: The Regional Connector Transit Corridor Project will connect the existing Blue, Gold and Expo Lines which serve Santa Monica, Pasadena, Long Beach and the Eastside to downtown, but do not currently interconnect. Along the alignment three underground stations, one crossover mined cavern, and four cross-passages will be constructed. As the alignment runs through the heart of downtown Los Angeles, a number of existing buildings, structures, and utilities are located within the potential influence zone caused by the project's underground construction. During the advanced preliminary engineering phase, a building protection program was carried out to identify potential risks to the adjacent structures and suitable measures to mitigate the anticipated impacts. The paper discusses a systematic approach during advanced preliminary design to ensure protection of adjacent and buildings and structures along the alignment and a summary of the results and recommended mitigation measures.

INTRODUCTION

The Regional Connector Transit Corridor (Regional Connector) project consists of approximately 580 m (1,900 ft) of cut-and-cover tunnel; two sections of twin-bored tunnels totaling approximately 1,460 m (4,800 ft) in length; three underground stations near the intersections of 2nd and Hope Streets, 2nd and Broadway, and 1st and Central Streets; one mined crossover cavern of approximately 90 m (300 ft) in length; and four cross-passages. Figure 1 schematically shows the project alignment and major components. The preliminary engineering assessment of potential impacts on adjacent buildings and structures was performed by the Connector Partnership Joint Venture that includes AECOM and Parsons Brinckerhoff, and their subconsultants.

GEOLOGIC CONDITIONS

The main geological formations along the tunnel alignment consist of Younger Alluvium, Older Alluvium, and Fernando Formation. The Younger Alluvium consists primarily of medium dense silty, fine- to medium-grained, poorly graded to well-graded sand with some gravels and medium stiff to stiff silts and clays. The Older Alluvium consists of dense to very dense, poorly to well-graded sand with variable gravel and cobble contents. The Fernando Formation consists predominantly of extremely weak to very weak, massive, clayey siltstone with rare interbeds of well cemented, medium strong to strong, fined-grained sandstone. The clayey siltstone is generally moderately to highly weathered at shallow depths and slightly weathered to fresh at greater depths below the contact with the overlying alluviums.

The majority of bored tunnels will be excavated completely within the Fernando Formation, except for a stretch of approximately 300 m (1,000 ft) on the eastern end where they will be excavated in a full face of Older Alluvium or a mixed face of Fernando Formation and Older Alluvium. Cut-and-cover excavations will encounter these soil and rock strata at variable depths. The groundwater level varies from one meter (a couple of feet) below to about 18 m (60 ft) above the tunnel crown.

EXISTING STRUCTURES AND UTILITIES ALONG TUNNEL ALIGNMENT

A total of 53 buildings along the entire tunnel alignment were determined to be in close proximity to the alignment and could be affected by the tunnel and station construction. These include 30 buildings adjacent to the bored tunnels, 3 buildings adjacent to the cavern, and 20 buildings adjacent to the cut-and-cover excavations. The adjacent buildings consist primarily of high-rise office buildings with underground basements along Flower Street and part of 2nd Street west of Main Street; and of one- to six-story retail and office buildings, parking structures, and one ten-story building east of Main Street.

The main adjacent underground structures and utilities include the 2nd Street Tunnel running along

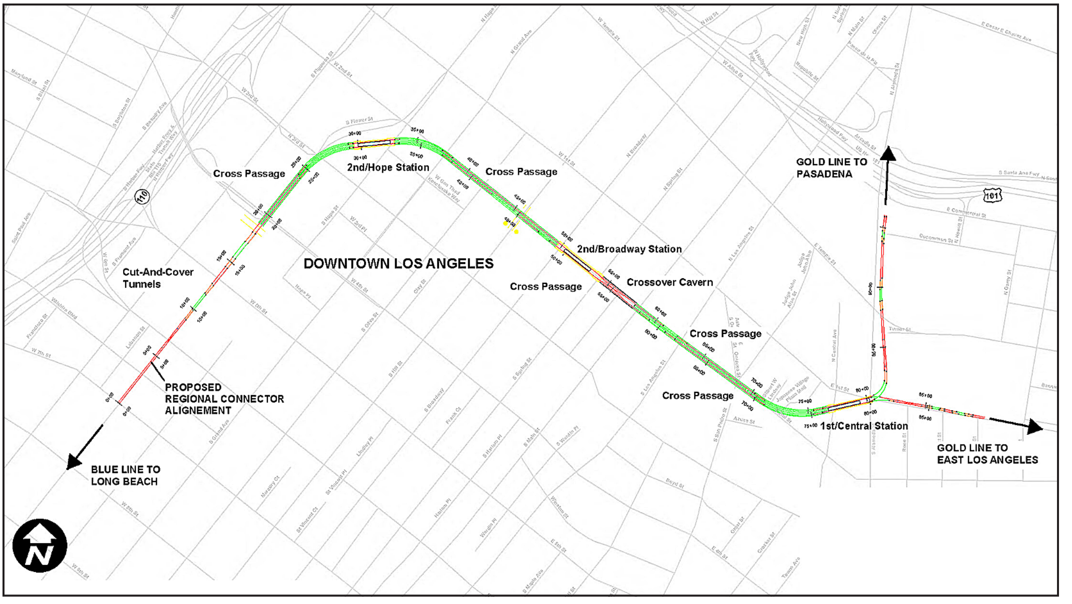

Figure 1. Regional connector project alignment

the Regional Connector alignment between Hill and Hope Streets; the Red Line Tunnels cross over the proposed alignment at Hill Street; foundations of the 4th Street Bridge and adjacent ramp; and the Grand Avenue Bridge piers located on both sides of the bored tunnels at Grand Avenue. In addition, there are a number of existing underground utilities of variable size and age located within the potential influence zone, including storm drains, sewer lines, water lines, gas lines, and telecommunication lines, and especially four major utilities including the Flower Street Storm Drain, Bunker Hill Central Plant Piping, Los Angeles County Storm Drain, and Alameda Storm Drain.

The information on existing conditions of the adjacent buildings, structures and utilities was obtained from various sources, including the prior studies, records available at the City of Los Angeles Department of Building and Safety, building walk-throughs (for selected buildings), and especially records from the property owners where the majority of building information was collected. During the building walk-throughs, photos were taken (where practical) to document the building existing conditions.

IMPACTS ON ADJACENT BUILDINGS AND STRUCTURES

Assessment Methodology

In an attempt to better identify and characterize potential risks associated with the critical adjacent buildings and structures, the potential impacts were evaluated in two stages: preliminary assessment and second stage assessment.

The preliminary stage involves the estimation of free-field settlements induced by the underground construction without considering the presence of the existing structures, and the screening of the existing buildings to be evaluated in the subsequent stage. The preliminary stage uses conservative screening criteria to assure that the buildings that are not considered in the second stage assessment will not be subject to damage levels more severe than "Negligible." A maximum absolute settlement of 6 mm (0.25 inch) and maximum settlement trough slope of 1/600 were adopted as screening thresholds for this project. All buildings and structures that have either absolute settlement or slope exceeding the above thresholds are evaluated in the second assessment stage.

The second stage focuses on evaluation of structural response to the estimated ground movements and severity of the possible damage, and to determine which buildings or structures are potentially at risk of being damaged, requiring mitigation or repair. The second stage assessment is more rigorous as the buildings' properties and structural behaviors are taken into account. The buildings adjacent to bored tunnels were evaluated using the Boscardin & Cording (1989) method while those adjacent to cut-and-cover excavations were evaluated using the Son & Cording (2005) method.

The Boscardin and Cording method is an empirical method that predicts potential damage to existing buildings and structures based on the critical tensile strains estimated using a deep beam model, which are a function of the building angular distortion and horizontal tensile strain. Depending on the location of the building relative to the tunnel excavations, different portions of the building can lie in a hogging or sagging zone which is separated from each other by the point of inflection of the settlement trough. Since the building portions in each zone experience different structural responses to the settlement and ground horizontal strains, they are considered separately, as recommended by Mair et al. (1996) and illustrated in Figure 2.

For the building portion located in a hogging zone, the neutral axis of the beam is assumed to be at the lower edge of the beam, and the maximum angular distortion is calculated using the equation recommended by Boscardin and Cording (1989); while in the sagging zone, the beam neutral axis is assumed to be at mid-height, and the angular distortion is calculated using the equation recommended by Walhs (1981).

Settlement calculations following the Boscardin and Cording method are performed using MathCAD software. In order to estimate the magnitude of expected damage to the structure, the calculated maximum values of angular distortion (β_{max}) and horizontal strain ($\varepsilon_{h,max}$) for each building are compared to limiting strain values by plotting in the chart illustrated in Figure 3 and correlating with the visual building damage classification presented in Table 1.

Ground Movements Caused by Bored Tunnels

Ground movements induced by tunneling consist of both vertical and lateral movements in directions transverse and parallel to tunnel alignment. Ground movements transverse to the tunnel centerline are more critical to the adjacent buildings and utilities. The ground movements parallel to the tunnel excavation are considered less critical to the buildings and structures because the impact of longitudinal settlement is typically transitory, leveling off as the tunnel passes.

The induced settlements transverse to the proposed tunnels are estimated using the semi-empirical method that was originally proposed by Peck (1969), and subsequently updated by O'Reilly and New (1982 and 1992) and others. This method assumes that the shape of the settlement trough above a single tunnel follows a Gaussian distribution and that the volume of the settlement trough is equal to the total volume of lost ground during tunneling. The total settlements caused by two tunnels are the sum of the settlements caused by each individual tunnel, assuming superposition.

The shape of the settlement trough over a single tunnel is characterized by three main parameters: depth to the tunnel springline (z), the ground loss (V_l), and horizontal distance from the tunnel centerline to the point of inflection of the settlement profile curve (i). In this study, the depth z is the vertical distance from the building or structure's foundation

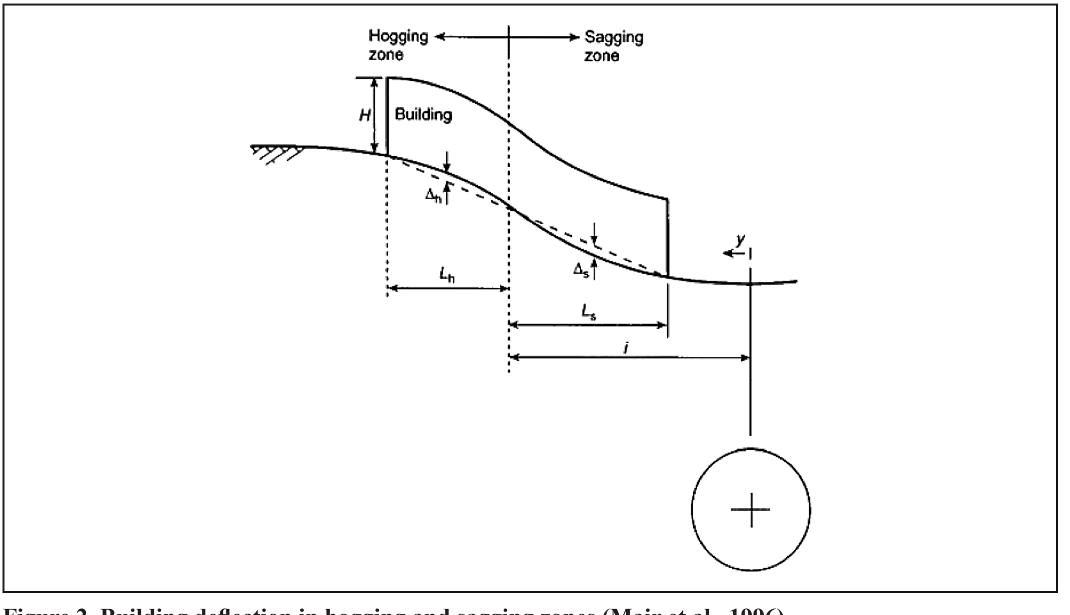

Figure 2. Building deflection in hogging and sagging zones (Mair et al., 1996)

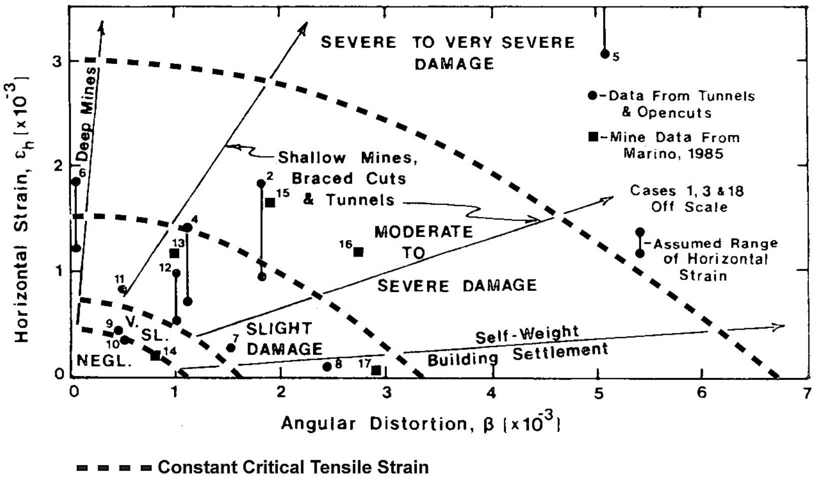

Figure 3. Relationship of damage to angular distortion and horizontal strain (Boscardin and Cording, 1989)

Table 1. Classification of visible damage (Boscardin and Cording, 1989)

Damage Level	Description of Damage*	Approximate Width of Cracks,† mm
Negligible	Hairline cracks	<0.1
Very slight	Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building. Cracks in exterior brickwork visible upon close inspection.	<1
Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible, some re-pointing may be required for weather tightness. Doors and windows may stick slightly.	<5
Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Tuck-pointing and possibly replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility service may be interrupted. Weather tightness often impaired.	5 to 15, or several cracks > 3 mm
Severe	Extensive repair involving removal and replacement of sections of walls, especially over doors and windows required. Windows and door frames distorted, floor slopes noticeably. Walls lean or bulge noticeably, some loss of bearing in beams. Utility service disrupted.	15 to 25, also depends on number of cracks
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 >25, depends on number of cracks

* Location of damage in the building or structure must be considered when classifying degree of damage.
† Crack width is only one aspect of damage and should not be used alone as a direct measure of it.

bottom (or pile tip if buildings are founded on piles or caissons), or utility springline, to the proposed tunnel springline at the location of the structure under consideration. The settlements caused by a single tunnel excavation are predicted using the following equations:

$$S_{z(x)} = S_{z,\max} * e^{\left(-\frac{x^2}{2i^2}\right)}$$

$$S_{z,\max} = 0.313 * V_l * \frac{D^2}{i}$$

$$i = K * z$$

where:

$S_{z(x)}$ = settlement at location x from tunnel centerline

x = horizontal distance from tunnel centerline

z = vertical distance from tunnel springline to point of analysis
i = distance from tunnel centerline to point of inflection on settlement profile curve
D = excavated tunnel diameter
V_l = average ground loss
K = trough width factor

The ground loss, V_l, and settlement trough width factor, K, are two important input parameters that need careful evaluations. Ground loss is the factor that has the most significant effects on the tunneling-induced ground movements. Limiting ground losses into tunnel excavations is the primary method to limit ground movements. Table 2 summarizes monitored ground losses associated with the use of EPBMs from recently completed tunnel projects worldwide. In all of these tunnel projects, the reported ground losses were typically achieved with a good control of face pressure, bentonite slurry injection in the annular gap around the TBMs, and tail-skin grouting to limit ground movement into the tunnel excavation. Higher ground losses were reported along a learning curve, or in tunnel sections where tail-skin grouting was not implemented or inadequate face pressures or slurry injection pressures were applied.

Based on the above reported ground loss and taking into account the mixed face conditions of alluvium and Fernando Formation on the eastern end of the alignment, a typical ground loss of 1.0% was assumed for the tunnel excavation in alluvium or mixed face conditions. For the tunnel excavation in Fernando Formation, which consists primarily of massive, weakly cemented, very weak to weak clayey siltstone, a typical ground loss of 0.5% is expected without bentonite injection around the shield.

The transverse distance from the tunnel centerline to the inflection point, ($i = K^*z$), is characterized by the depth to the tunnel springline, z, and a trough width factor K, which is a function of ground type. Table 3 shows the K values of different soil types

Table 2. Monitored ground losses of recent tunnel projects

Project Names		Exc. Dia., m (ft)	Year	TBM	Geologic Conditions	Typical Ground Losses, %	References
Sao Paulo Metro Line 4—Lot 1, Sao Paulo, Brazil		9.50 (31.2)	2009	EPBM	Three formations: soil derived from the alteration of gneiss; interbedded high to medium plasticity clay and sandy clay with gravel; and interbedded medium stiff to hard clay with fine to coarse sands	< 0.4	Pellegrini and Perruzza, 2009
Barcelona Metro Line 9, Barcelona, Spain	Mas Blau to San Cosme Segment	9.40 (30.8)	2008	EBPM	Submerged fine silty sands and clayey silts	0.4 to 0.8	Mignini et al., 2008
	Segment IV-B (San Adria)	11.95 (39.2)	2007	EPBM	Sands, clay, and silts overlying gravels with sands	0.7 to 1.0	Della Valle, 2007
	Segment IV-C (Trajana)	11.95 (39.2)	2007	EPBM	Mixed face of silts and sands or gravels in a sandy clay matrix overlying highly to completely weathered granodiorite	0.2 to 0.6	Della Valle, 2007
	Fira to Park Logistic Segment	9.4 (30.8)	2006	EPBM	Silty sands with sandy silts, silts and silty clays	0.3 to 0.4	Orfila et al., 2007, Della Valle, 2007
Madrid South Bypass M-30 Tunnels, Madrid, Spain		15.0 (50.0)	2007	EPBM	Mixed face of sandy clay overlying hard clay with gypsum	0.1 to 0.4	Universidade da Coruña, 2008.
MTA Gold Line Eastside Extension, LA, USA		6.52 (21.4)	2007	EPBM	Mix of stiff to hard silt, lean clay, sandy clay, and loose to very dense sand and gravel	< 0.3	Choueiry et al., 2007
Channel Tunnel Rail Link, London, UK[(1)]		8.14 (36.7)	2004	EPBM	London clay: Stiff to hard clay	0.3 to 0.8	Bowers et al., 2005; Mair and Borghi, 2008.
					Fine and medium silty sand	0.3 to 0 8	

Table 3. Settlement trough width factors, *K*

Soil Types	*K* Value
Artificial fill	0.3
Younger alluvium	0.3
Older alluvium—above groundwater table	0.2
Older alluvium—below groundwater table	0.6
Fernando formation	0.4

selected for this preliminary engineering study as recommended by O'Reilly and New (1982) and Peck (1969). The composite trough width parameter i of N soil layers above the tunnel springline, each of thickness z_N, is calculated using the following equation recommended by O'Reilly and New (1992).

$$i = K_1 z_1 + K_2 z_2 + \ldots + K_N z_N$$

Ground Movements Caused by Cut-and-Cover Excavations

The first practical approach for estimating ground movements caused by deep excavations was proposed by Peck (1969). Peck compiled data on ground settlement adjacent to temporary braced sheet pile and soldier pile walls and developed a chart that gave the ground settlement as a function of distance from excavation and type of soil. Since the publication of Peck's paper, other empirical and semi-empirical methods have been proposed to estimate ground movements caused by deep excavations. In this study, the vertical ground movement (settlement), horizontal ground movement, and settlement envelope are estimated following the empirical method proposed by Clough and O'Rourke (1990). Knowing the maximum settlement, the surface settlement envelope can be estimated using the dimensionless diagrams shown in Figure 4. In stiff clays, residual soils, and sands, maximum lateral wall movements and settlements of the retained soil average about 0.2% to 0.3% H, with a scattering of case history data up to 0.5% H.

Unmitigated Effects of Ground Movements to Buildings and Structures

A total of 33 buildings adjacent to the bored tunnels and caverns are assessed with the preliminary analysis. Sixteen buildings are screened out of the second stage assessment and these buildings are considered as not being affected or negligibly affected by the tunneling-induced ground movements.

Seventeen buildings having maximum total settlement and slope that exceeded the above criteria were evaluated in the second stage assessment using the Boscardin and Cording method. Of these, nine buildings have a maximum anticipated damage level ranging from "Negligible" to "Very Slight"; one building has a maximum anticipated damage level as "Slight"; four buildings have a maximum anticipated damage level as "Moderate"; and three buildings have a maximum anticipated damage level ranging from "Severe" to "Very Severe."

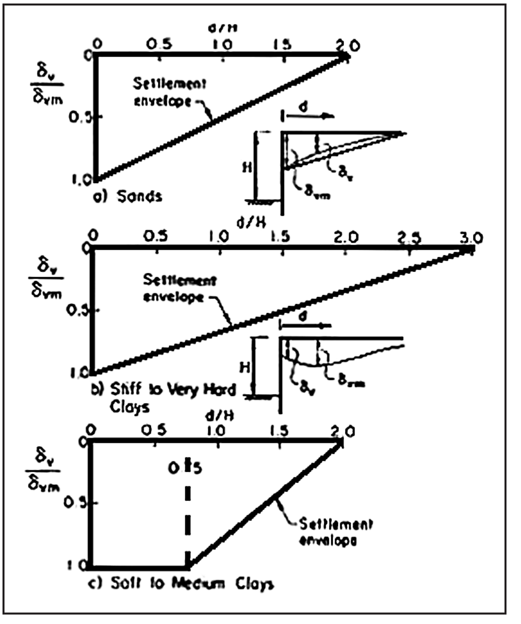

Figure 4. Recommended dimensionless settlement profiles adjacent to excavations (after Clough and O'Rourke, 1990)

Additional analyses were performed for pile foundations located in the pile influence zone to evaluate the additional pile loads caused by tunneling-induced ground movements. This zone is defined by Jacobz et al. (2001) as a soil prism above the tunnel spring line that is limited by a 1:1 (45 degrees) upslope line on each side of the tunnel. A review of the buildings on piles or caissons along the proposed alignment indicates that the piles of the Angelus Plaza Parking Structure are more critical than the remaining piles and caissons. These piles were evaluated for the anticipated additional load caused by the lateral ground movements using the LPILE V5.0 program (Ensoft, Inc., 2005). The results from this pile analysis indicated that the internal forces in the pile caused by the lateral ground movements and vertical loads from the above structure are well below the pile capacity.

Twenty buildings adjacent to the cut-and-cover excavations were identified during the preliminary assessment stage. Five of these buildings were determined to be outside and fifteen buildings were within the approximate limits of the settlement trough. Of these fifteen buildings, nine buildings have maximum estimated settlements below the thresholds and were not analyzed further. The remaining six buildings having the maximum estimated settlements above the thresholds were subsequently analyzed in the second stage using numerical modeling (as discussed subsequently) to determine the potential damage levels.

Numerical Modeling

Subsequent to the analyses using empirical methods, numerical modeling using PLAXIS computer program was performed for several buildings and structures that are either structurally critical or located adjacent to complex excavations. These include the Higgins Buildings, 2nd Street Tunnel, Redline Tunnels, 4th Street Bridge and Ramps, buildings adjacent to crossover cavern, Bunker Hill Central Plant piping, and the six buildings adjacent to cut-and-cover excavations that have maximum estimated settlements above the specified thresholds.

Numerical analysis allows different excavation sequences and initial ground support schemes to be modeled and the ground movements in each case to be determined. Since the numerical modeling procedures are based on a case-by-case basis, these numerical analyses are not presented in detail in this paper. Some details can be found in the papers previously published by Navid et al. (2012) and Bergeson et al. (2012).

The settlement, angular distortion and horizontal strain are calculated at the foundation level of the buildings and structures and used to assess the level of possible damage according to Boscardin and Cording's method. Results from these numerical analyses indicated that the maximum anticipated damage levels of the Higgins Building, 2nd Street Tunnel, Redline Tunnels and 4th Street Bridge and Ramps are "Negligible." Among the six buildings adjacent to cut-and-cover that were analyzed with numerical modeling, five buildings have the expected damage level of "Negligible" and one building has "Very Slight." Based on the results of the above analyses, the buildings and structures that are anticipated at higher risks and require mitigation measures are flagged for protection as illustrated in Figure 5.

IMPACTS ON UTILITIES

Settlement impacts on buried pipeline utilities are typically caused by one or more of the following effects, as summarized in O'Rourke and Trautman (1982): (1) tensile pull-apart at joints; (2) opening of joints between pipe segments, θ, due to relative rotation between two pipe segments; and (3) straining of pipe caused by flexural deformations, ε_b, and lateral deformations, ε_h, that lead to rupture or intolerable deformation.

The first two effects primarily occur at well-defined joints and would be more likely to occur for fairly rigid, jointed pipes, such as concrete pipes or vitrified clay pipes (VCP). The third type of effect is caused by differential settlements and lateral ground movements, and is most likely to occur in flexible pipelines with well-designed rigid joints that can take significant rotation, such as welded steel pipelines or small (less than 20 cm in diameter) cast iron pipes (CI) and ductile iron pipes (DIP) (O'Rourke and Trautman, 1982). Schematics of each of these three modes of failure are shown in Figure 6.

The tensile pull-apart at the joints is typically only a factor if one end of the utility is fixed to some rigid object (i.e., building, manhole, etc.), or if joints are particularly sensitive, such as in cast iron pipes. O'Rourke and Trautmann (1982) reported an allowable axial joint slip of 25 mm (1.0 inch) for buried CI pipe. Attewell et al. (1986) also reported a range of allowable axial joint slip of 10 to 25 mm (0.4 to 1.0 inch) for sound CI water and gas mains with different joint packing materials (cited by Bracegirdle et al. 1996). Considering the potential existing joint deformations, a reasonable lower allowable limit of 10 mm (0.4 inch) was used for CI and other types of jointed pipes, such as concrete, VCP, or DIP, in this study.

Joint rotation failure will occur for rigid utilities with joints, or for any utility that has joints that allow rotation. For utilities transverse to a single tunnel excavation, the critical joint rotation point is directly above the sagging point of the settlement trough.

Tensile strains in utilities are estimated following procedures proposed by Attewell et al. (1986) that are summarized in the paper by Bracegirdle et al. (1996). In this study, the smaller utilities were approximately assumed to follow closely the ground settlement trough; thus, the utility bending tensile strain is calculated directly from the settlement trough hogging curvature. However, larger utilities and structures such as the Los Angeles County Storm Drain and Red Line Tunnels have a significant relative stiffness compared to the surrounding ground, hence modifying the settlement trough curvature. Consequently, the utility-ground interaction was considered by following the procedure outlined by Yeates (1985).

The utility additional strains caused by ground movements should be limited so that the utility total tensile strains are kept below the limiting strains at cracking. For cast iron pipes, an elastic tensile strain of 400 micro-strain can be derived from the design stress specified by codes (Attewell et al., 1986). For brick and ductile iron/steel utilities, limiting additional tensile strain of 150 and 600 micro-strain were assumed, respectively.

Figure 5. Adjacent buildings impacted by bored tunnels and cut-and-cover excavations

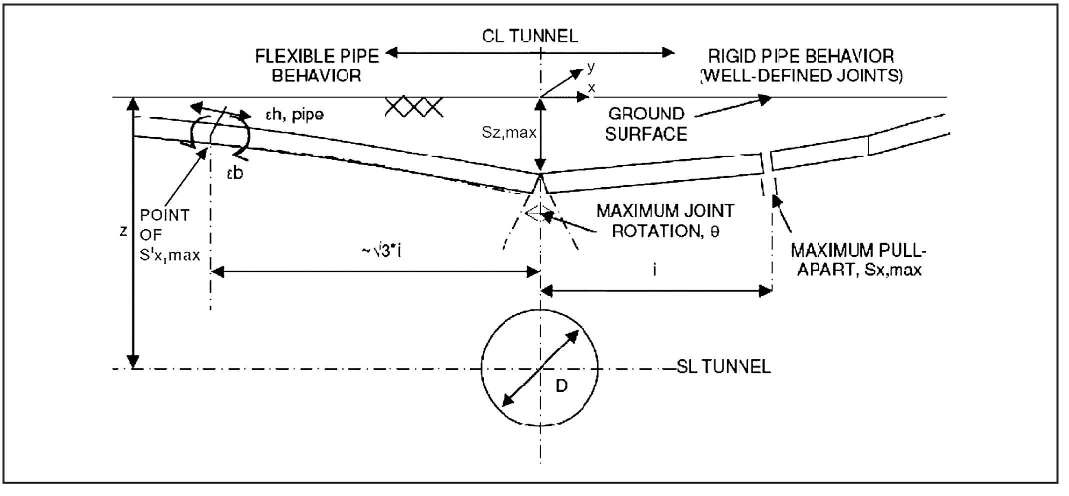

Figure 6. Utility impacts from tunneling induced ground movements

For each failure mode, a ratio that compares capacity to demand is calculated to estimate the impact of anticipated settlements on the utilities. This ratio can be interpreted as follows:

- Ratios < 1.0: The utility would likely be adversely impacted by construction and pro-active measures should be taken to prevent damage.
- 1.0 ≤ Ratios < 1.5: Significant impacts are not expected but the utility may be affected by construction. Specific geotechnical instrumentation and surveys may be warranted to monitor soil and utility deformations.

Table 4. Analytical results for utilities

No.	Utilities	Depth to Centerline (m)	Dimension (cm)	Orientation Relative to Tunnels	Minimum Capacity-to-Demand Ratio
		2nd and LA Street Intersection			
1	Storm drain, RCP	2.7 (9 ft)	46 ID (18 in)	Transverse	3.3
2	Sewer, VCP	4.5 (15 ft)	40 ID (16 in)	Transverse	2.9
3	Sewer, RCP	9 (31 ft)	76 ID (30 in)	Transverse	1.5
4	Gas, CI	1.2 (4 ft)	15 ID (6 in)	Transverse	1.9
5	Water, CI	1.2 (4 ft)	30 ID (12 in)	Transverse	1.8
6	BP&L, URC	1.2 (4 ft)	100 by 64 (40 by 25-in)	Transverse	1.6
		2nd Between LA St and San Pedro St			
7	Storm drain, Brick (R:67+00)	1.5 (5 ft)	78 ID (31 in)	Parallel	1.1
		2nd and San Pedro Street Intersection			
8	Storm drain, RCP	2.4 (8 ft)	46 ID (18 in)	Transverse	1.0/2.0*
9	Gas, CI	0.6 (2 ft)	7.5 ID (3 in)	Transverse	0.9/1.8*
10	Water, CI	1.8 (6 ft)	30 ID (12 in)	Transverse	0.6/1.2*
11	Water, CI	1.5 (5 ft)	20 ID (8 in)	Parallel	0.6/1.2*
12	Sewer, VCP	3 (10 ft)	20 ID (8 in)	Parallel	1.9/3.8*
13	Electric ducts, URC	0.9 (3 ft)	53 by 53 (21 by 21-in)	Parallel	0.8/1.6*

* First and second values based on values of ground volume loss of 1.0% and 0.5% respectively.

- Ratios ≥ 1.5: No adverse impacts are expected and no specific geotechnical instrumentation or monitoring will be required.

The results from analyses performed for the typical utilities located at the intersections of 2nd Street with Los Angeles Street and San Pedro Street are presented in Table 4.

MITIGATION MEASURES AND ANTICIPATED EFFECTS

The analytical results indicate that the majority of the adjacent buildings, structures, and utilities have the anticipated damage levels of "Very Slight" or less severe, which require continuous monitoring only. The buildings and structures that have anticipated damage levels of "Moderate" or more severe require mitigation measures in advance. These include five buildings in the Little Tokyo area, the Bunker Hill Central Plant pipes crossing Flower Street, the Los Angeles County Storm Drain and some small utilities in the mixed-face tunneling zone on the eastern end of the alignment.

Mitigation measures recommended for this project consist of (1) controlling TBM ground loss with the advanced TBM technology and (2) grouting technology including permeation grouting, jet grouting, compaction grouting, and compensation grouting.

Ground loss into the tunnel excavation is the most important factor contributing to ground movements around tunnels. Ground loss is generally caused by a combination of three sources: over-excavation of unsupported, unstable ground at the face; intrusion of surrounding material into the annular space caused by the cutterhead overcut and shield conicity; and intrusion of surrounding material into the annular space between the outside skin of the shield and the outside surface of the primary support. The advanced TBM technology allows effective control of these three sources of ground loss through applying positive face pressures, shield bentonite injection, and tail-skin grouting.

Pressurized closed-face TBMs apply a positive pressure to the tunnel face, counterbalancing external earth and hydrostatic pressures; hence being able to limit ground loss at tunnel face to minimal amounts.

Shield Bentonite Injection: Monitored settlement data indicate that 40–50% of total volume loss occurs along the shield (Leca et al., 2006). A system of injection lines is incorporated in new EPBMs to allow a controllable slurry injection, leading to an immediate support of the surrounding ground.

Tail-skin Grouting: The annular gap between the excavated face and the extrados of the lining contributes 30–40% to total volume loss around a tunnel excavation (Leca et al., 2006). This annulus can be effectively filled with grout as the shield advances. In current TBM design, a system of grouting pipes is incorporated that allows continuous grout injection through the tail shield, providing immediate support.

The results from the analyses performed indicate that successfully controlling the ground loss below 0.5% will protect the Redline Tunnels, Broad Museum, and the utilities in the mixed-face zone from considerable damages.

Due to the sensitivity of the Redline Tunnels and the Broad Museum, mitigation measure in form of controlling TBM ground loss with the advanced TBM technology is required even though the anticipated damage levels from the analyses are "Very Slight."

Grouting was recommended as mitigation measures for the Little Tokyo buildings, the Bunker Hill Central Plant piping, and the utilities in the mixed-face zone. The grouting program includes performing permeation grouting or jet grouting prior to tunneling to create a supported zone around the tunnels, hence reducing ground loss due to tunneling. In addition, compensation grout pipes are also installed in advance underneath the building foundation in order to correct the buildings' excessive settlement when detected.

GEOTECHNICAL INSTRUMENTATION

A geotechnical instrumentation and monitoring program is required to provide warning of potentially damaging settlements to existing buildings, structures, and utilities along the proposed alignment. Recommended geotechnical instrumentation for this project consists of the following.

Multiple position borehole extensometers (MPBXs): Each MPBX would be installed with at least 3 anchors; the deepest to be located about 1.5 m (5 ft) above the tunnel excavation and the other anchors would be located approximately at 3 to 4.5 m (10- to 15-ft) intervals above the lowest anchor.

Deep Benchmarks: Deep benchmarks are installed to provide a reference elevation for comparison of potential elevation changes measured by the MPBX, ground surface points, and building points. They must be installed with the tip at an elevation below the tunnel elevation in order to provide a stable reference that is not affected by tunneling or other near surface influences, such as temperature or moisture changes.

Groundwater Monitoring Wells or Piezometers: Including a standpipe piezometer or pressure transducer for monitoring groundwater elevations. These are required at certain locations along the alignment corridor to track the extent to which groundwater level lowering may occur.

Inclinometers: Inclinometers are required to monitor lateral ground movements due to station excavations. An inclinometer consists of a casing, probe, and readout indicator. The casing is installed within about 1.5 m (3 ft) of the excavation walls, extends below the excavation bottom level, and is grouted to allow the same lateral movements as the surrounding ground.

Ground Surface Settlement Points: These reference points may be installed in arrays that are perpendicular to the tunnel so as to help evaluate the extent of settlement associated with tunneling activities.

Building Monitoring Points: These are survey points usually installed on faces of critical buildings and structures or structures where damaging settlement are anticipated. The monitoring data can be recorded with conventional optical survey equipment or with real-time automated motorized total stations (AMTS).

Crackmeters: In certain cases, crackmeters can be used to monitor construction-related changes to existing cracks. These sensors may be manual or electronic (i.e., vibrating wire crack gauges).

Tiltmeters: In certain cases, tiltmeters can be installed on main structural components of buildings and structures to monitor structure tilt due to ground movements.

Convergence Monitoring Points: Convergence monitoring points are required at key locations to monitor for possible convergence of the bored tunnels, mined cross passages and the mined cavern for the cross-over structure.

Instrumentation Zone: An instrumentation zone is defined as a portion of a cut-and-cover excavation that contains equipment to monitor loads in support elements. For braced excavations, an instrumentation zone includes strain gauges installed on a specified number of struts. For a tie-back excavation, load cells would typically be installed on a group of tie-back anchors on opposing sides of the excavation. In either case, the load monitoring sensors would be read electronically using switch boxes, data loggers, and other associated equipment so that readings can be obtained in near "real time."

CCTV: Preconstruction survey of selected utilities can be performed by closed-circuit TV (CCTV) to examine the existing internal conditions of the utilities. Based on the results of the surveillance, additional mitigation or protection measures could be considered.

INSTRUMENTATION MONITORING

Two phases of instrument readings are needed as described below.

Pre-construction readings: These readings are conducted to document proper operation of the instruments and document baseline readings for critical buildings and utilities.

Readings during construction: These readings are conducted to confirm proper operation of the instruments and to establish baseline (i.e., pre-construction) conditions. Sufficient measurements are needed to document stable and reliable readings.

Instrumentation monitoring requirements during construction will depend on the progress of the excavation work, the type of instrument, and the characteristics of the structures located nearby. Typically, load monitoring instruments, settlement monitoring instruments, and convergence monitoring instruments are monitored at least daily when excavation is occurring. As noted above, some of these instruments will be required to be monitored automatically, using data loggers.

Other instruments, such as inclinometers and piezometers typically monitor conditions that change less rapidly than the load and settlement instruments, and therefore are typically monitored less frequently. Typical monitoring frequency for these instruments is weekly. However, more frequent monitoring may be required in certain circumstances.

CONCLUSION

A systematic approach was employed for the evaluation of potential risks associated with the ground movements caused by tunneling and cut-and-cover excavations of the Regional Connector project. The two stage evaluation proves to be an effective approach that allows elimination of non-critical structures and allows in depth assessment of critical structures. A careful review of settlement data of the previous tunneling projects in the similar geological conditions allows a more reasonable estimation of ground loss due to tunneling. Numerical modeling is necessary to estimate the ground movements caused by complex excavations or to assess the potential risks of the structures that are more sensitive to ground movements. This paper presents the work performed during the preliminary engineering phase. As the design-build delivery format is used for this project, the wining team will be responsible for the final assessment of potential damages to adjacent buildings and structures and the corresponding building protection program. It is believed that a combination of proper mitigation measures and a complete geotechnical instrumentation and monitoring program would effectively mitigate the potential risks due to tunneling-induced ground movements.

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