Rehabilitation of the 80-Year-Old North Outfall Sewer (NOS) Tunnel

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ABSTRACT: In 1990, a residential area above the North Outfall Sewer (NOS) tunnel constructed in 1924, experienced subsidence. After sewage was diverted into the North Outfall Replacement Sewer (NORS), soil probing from within the tunnel was conducted, soil-improvement options were developed, and a preferred grouting scheme was analyzed by numerical modeling to evaluate the stability of the 80-year+ old tunnel liner under grouting-induced loading conditions. This paper discusses the initial field investigation and the development and analysis of a grouting scheme originally intended to be implemented before relining of the tunnel. The scheme was later modified, and the new liner was installed before the final probing and grouting work was done.

INTRODUCTION

The North Outfall Sewer (NOS) was constructed in 1924 to carry raw sewage to an ocean outfall near Playa Del Rey. A plot plan and typical cross section of the lower portion of NOS, from I-405 to the beach, are presented in Figures 1 and 2. The western and eastern tunnel segments shown in Figure 1 were hand-mined with timber sets and lagging for initial support (Figure 3), with forepoling used to minimize caving ahead of the tunnel face in the generally medium dense to dense, poorly graded fine sands (Figure 4) underlying this area. The annular space between the final unreinforced-concrete liner and the temporary timber support, and occasional cave-ins, reportedly were backfilled with excavated soil or crushed rock. A typical tunnel profile and section with subsurface conditions along the western tunnel segment are shown in Figure 5. The NOS segment between the western and eastern mined tunnels was constructed by cut-and-cover as shown in Figure 6.

In late 1993, an 8 × 11 m surface depression was observed on Zitola Terrace above the western tunnel segment, which eventually grew to 15 × 34 m and 0.3 m deep. Damage to surface structures included cracked driveway pavements, hardscape, and houses that settled up to 10 cm. Subsequent exploratory borings drilled from the surface along the western tunnel alignment found very loose soils and voids 2 to 3 tunnel diameters above the tunnel

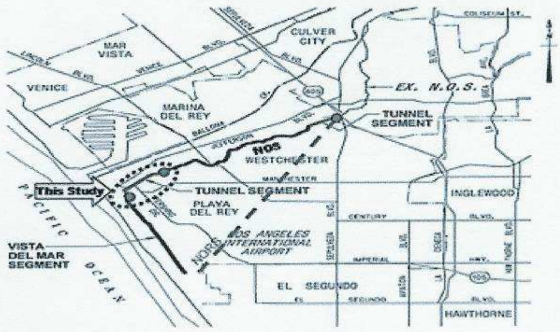


Figure 1. Location map showing study area

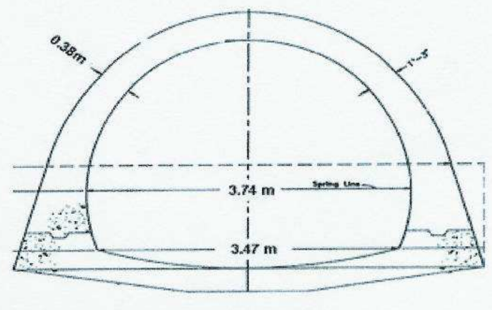


Figure 2. Typical cross section of North Outfall Sewer (NOS)

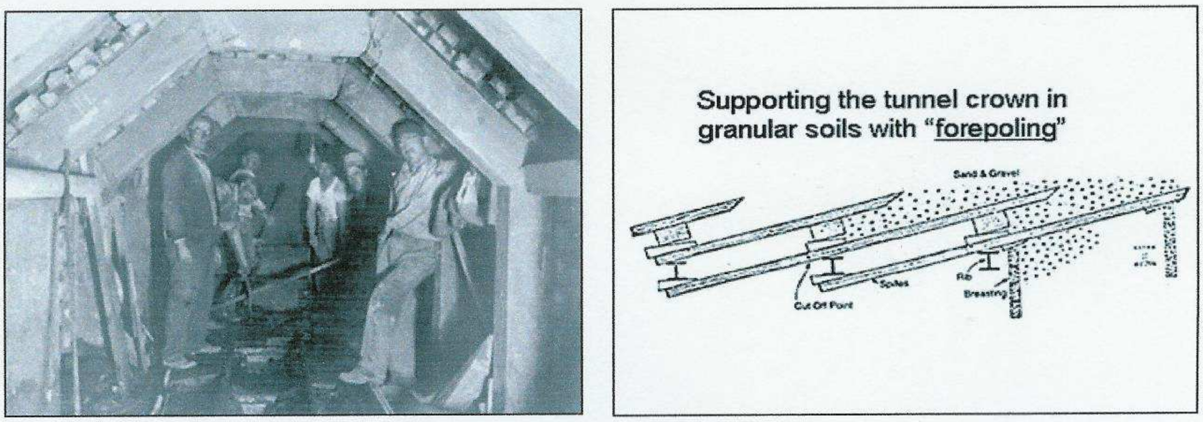


Figure 3. **Tunneling in the 1920s with timber sets and lagging**

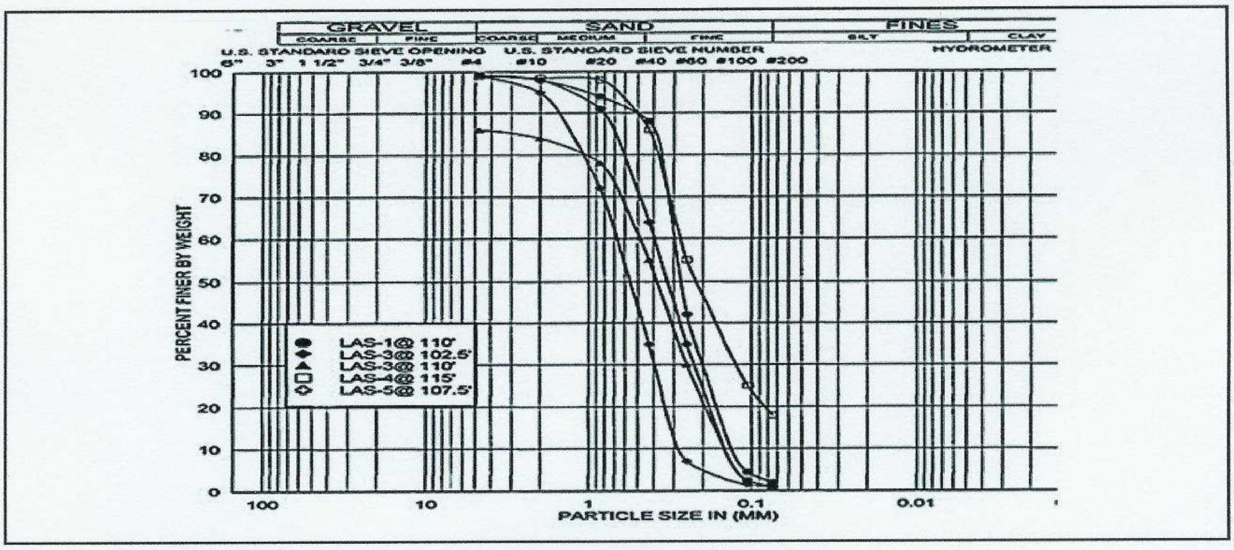


Figure 4. **Typical grain-size curves of dune sands along NOS tunnel**

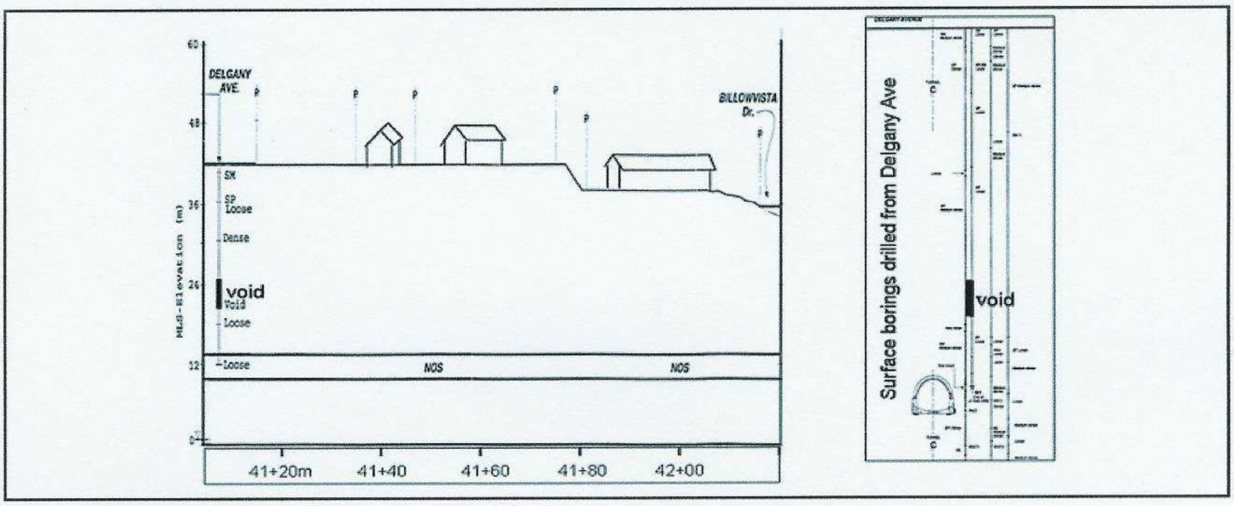


Figure 5. **Subsurface conditions of NOS western tunnel segment**

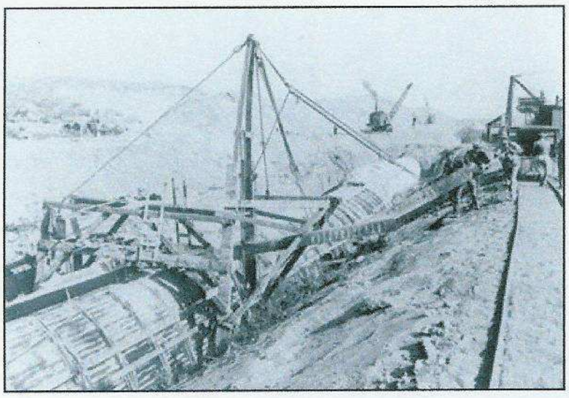


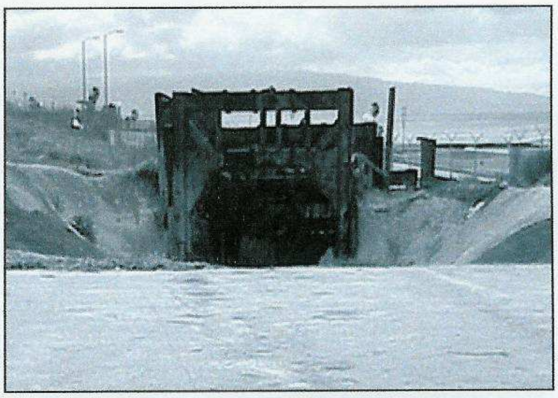
Figure 6. Cut-and-cover construction of NOS in the 1920s

crown (Figure 5). These conditions were suspected to have been caused by voids created around the tunnel during the original construction, and then slowly migrating upward driven by decades of seasonal rainwater seeping into the ground.

After sewage from NOS was diverted into the North Outfall Replacement Sewer (NORS) in the mid-1990s, the City initiated a major rehabilitation program of NOS. This program includes initial soil probing from inside the tunnel, installation of a new structural tunnel liner (relining), and final soil probing and grouting from inside the tunnel to fill voids and strengthen overburden soils. This paper discusses the initial soil probing, the development of soil-improvement alternatives, and the numerical analyses carried out to assess the stability of the 80-year+ old tunnel liner under grouting-induced loading conditions. The final probing and grouting program, originally intended to be completed before relining the old tunnel, was eventually postponed until after the new liner was installed. The implementation of this grouting program, which currently is close to completion, will be the subject of another paper.

INITIAL PROBING FROM INSIDE THE TUNNEL

In 1999 initial soil probing was performed from inside the 1.2 km long western tunnel segment of NOS. It included coring and testing 22 samples of the concrete liner and performing 22 Cone Penetration Tests (CPT) through the cored holes in the tunnel crown at 200-foot intervals. Accessing the tunnel through the Playa Vista portal near the beach (Figure 7) a modified rubber-tire loader (Figure 8a) was used as a mobile work station and to transport equipment and personal. Ventilation was provided by a fan located downstream from the access portal blowing fresh air through the full cross section of the tunnel.



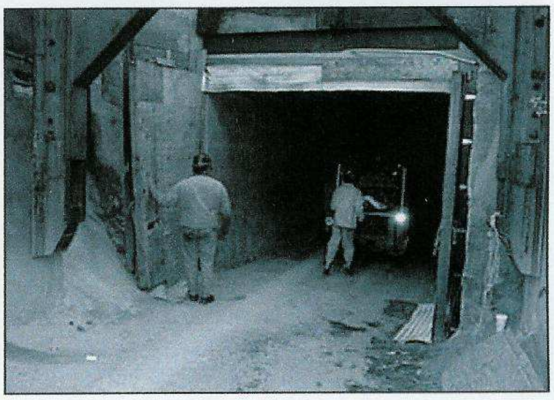


Figure 7. Access through Playa Vista portal

Concrete Coring and CPT Probing

Concrete coring was accomplished using an electric drill with a 76-mm diameter core barrel, which was advanced manually along a guide rail braced against the tunnel invert and the crown. CPT probing was performed with a 35-mm diameter steel cone attached to 1.2 m long sections of coupled steel rods pushed up through the cored holes with a hydraulic jack attached to the bucket of the tunnel loader. The cone was advanced up to 5 m above the tunnel crown, at a penetration rate of 25 cm per minute, while recording the oil pressure in the hydraulic jack using a computerized data acquisition system (Figure 8b). A trial survey with Ground Penetrating Radar did not produce conclusive results and was discontinued.

Initial Findings and Recommendations

The thickness of the tunnel liner at the 22 cored locations was found to be highly variable, ranging from 15 to 77 cm. Unconfined compression strength of the cored samples ranged from 20 to 45 MPa; and the elastic modulus ranged from 12,000 to 50,000 MPa.

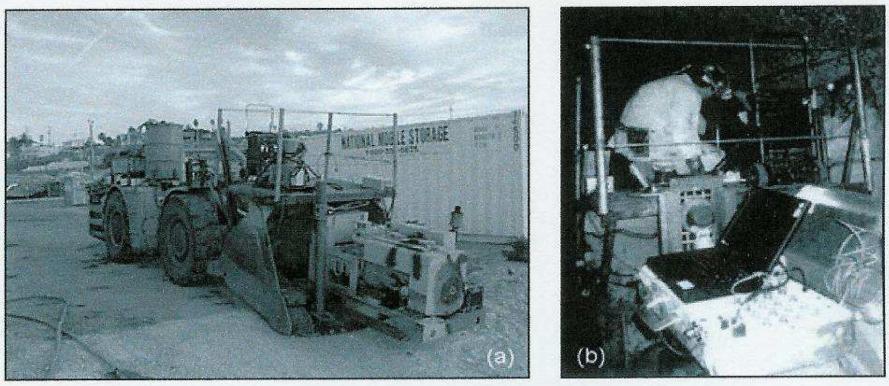


Figure 8. (a) Modified loader with CPT equipment; (b) CPT data acquisition

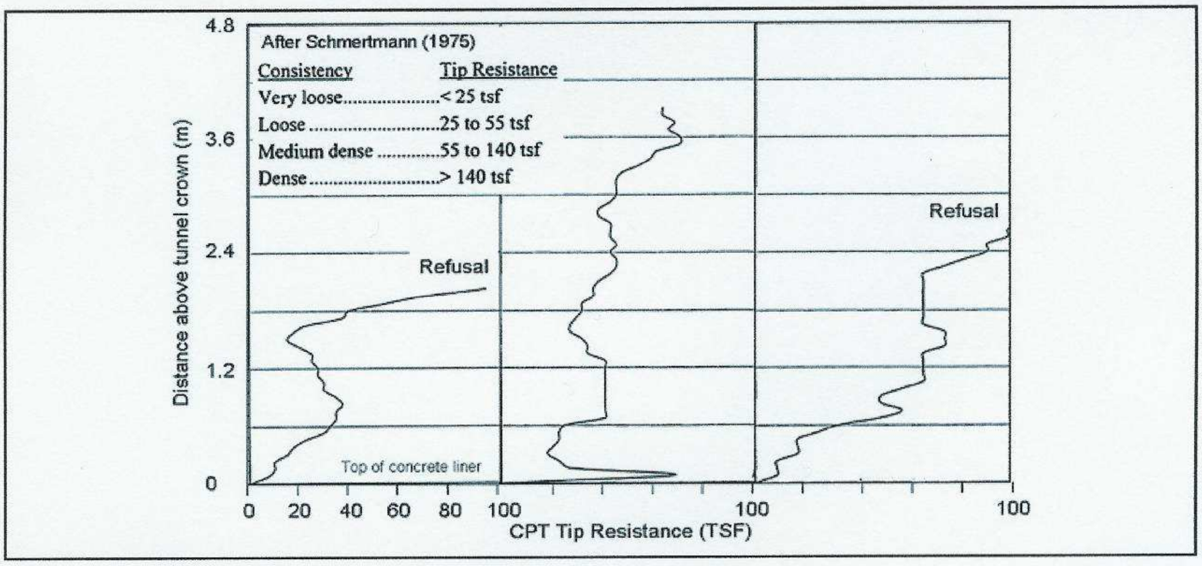


Figure 9. Typical CPT tip-resistance profiles

Typical profiles of CPT tip resistance are shown in Figure 9, indicating the presence of very loose soil zones within approximately 1 to 3 m above the tunnel crown. These zones likely originated from initial voids created around the tunnel liner during the original construction by way of poorly backfilled annular space and/or cave-ins during mining, as well as rotting of the timber cribbing and temporary supports left in place. Such voids then slowly migrated upwards, eventually causing surface subsidence. This process can take decades in fine to medium-grained sands, which temporarily lose their capillary-tension based (apparent) cohesion when seasonal rainwater seeps into the ground. During such episodes, the leading edge (roof) of the upward-migrating void (chimney) gradually unravels filling the tail end of the chimneys with loose debris.

Given the limited scope of the initial investigation, it was recommended that additional probing at a finer grid pattern be performed to be able to draw more definite conclusions about the state of the soil and structural capacity of the NOS tunnel. However, because of the cost-intensive nature of such investigations in the confined space of the tunnel, it was decided to combine the additional soil probing with the remediation-grouting program aimed at filling existing voids and strengthening the overburden soils above the tunnel.

DEVELOPMENT OF GROUTING OPTIONS

Input from Specialty Contractors

Looking for fresh ideas on ways to combine additional soil probing with soil-remediation measures,

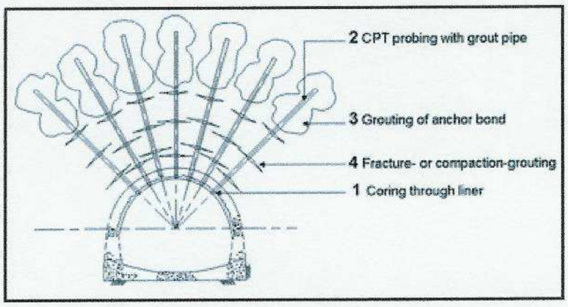


Figure 10. Probing and grouting sequence (before re-lining)

specialty contractors were invited to participate in brainstorming sessions. These sessions were held separately with each of four pre-qualified contractors, with the understanding that all would bid on whichever scheme would be selected in the end. Candidate schemes had to work around two basic requirements: (1) soil improvement was to be completed before relining the tunnel; and (2) there had to be a way to verify the effectiveness of the improvement measures.

Since any remedial measure would involve pumping grout under pressure into the soils above the tunnel, the resulting additional load on the old, partially deteriorated tunnel liner was a major concern. Hence, it was understood at the outset that the liner had to be supported during grouting operations. Installing temporary support inside the tunnel, either stationary or mounted on a jumbo, would be costly and take up valuable working space. Furthermore, even though temporary support would protect the workers inside the tunnel during grouting operations, locked-in residual stresses in the soil above the tunnel could damage the liner when the support is removed. One scheme proposed in the brainstorming sessions held the promise for solving both of these problems using permanent soil anchors. This innovative idea called for multi-purpose steel pipes to be used for all three functions in the following sequence: (1) soil probing, (2) liner anchoring, and (3) soil grouting. This scheme was eventually selected for further development and analysis as described below.

Preferred Probing and Grouting Scheme

The preferred scheme shown schematically in Figure 10 was to be implemented using the following construction sequence:

1. Coring through tunnel liner.
2. CPT probing to refusal with a sacrificial cone mounted on a steel pipe with sleeved grout ports.
3. Phase-1 (anchor grouting): grouting of sleeve ports within the competent soils at the far end of the pipe, to establish an anchor-bonding zone.
4. Phase-2 (remedial grouting): contact-, fracture-, and/or permeation grouting of the voids and loose soils above the tunnel.

Tunnel-liner deflection, anchor load, and surface structure movements were to be closely monitored during construction. In order to avoid interference with surface structures, probing and grouting was to stop within 7 m of the ground surface.

NUMERICAL ANALYSIS OF LINER STABILITY

To evaluate the stability of the tunnel liner during grouting a parametric study was performed analyzing the effect of grouting sequence and maximum pressures applied. The specific purpose of this exercise was to investigate trends and establish an order-of-magnitude range of grouting pressure which could be applied without endangering the stability of the tunnel liner. Considering the uncertainties about the nature and extent of voids around the tunnel, as well as questions about the structural integrity of the 75-year old concrete liner, it was concluded from the outset that extensive monitoring should be performed during construction. Hence, another important objective of the analysis was to investigate which parameters should be monitored during construction, and what range of measured values should be expected.

General Modeling Approach

The analysis was performed with the finite-difference program FLAC, Version 3.4 (Itasca, 1998). FLAC offers a wide range of capabilities to solve complex problems in geomechanics, including non-linear static and dynamic stress-strain analysis of soil continua, soil-structure interaction, and ground-water flow. The program has been thoroughly verified against closed-form solutions, physical models, and case histories in the field (Roth, et al. 1993, 1996, 1997, 2001).

The tunnel was modeled for an average invert depth of 65 feet below ground surface, and assuming the tunnel liner to be of unreinforced concrete. Because previous investigations found significant variations of the liner thickness along the alignment, three models where analyzed with liner thicknesses of 15, 45, and 76 cm at the tunnel crown (Figure 11). The soil was modeled with as elasto-plastic Mohr-Coulomb material with shear strength defined by friction angle and cohesion, and elastic behavior governed by E-modulus, E, and Poisson's ratio, ν.

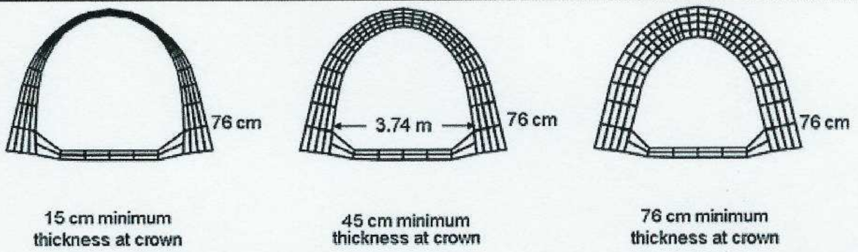


Figure 11. Model meshes for tunnel liner

Table 1. Soil properties used in FLAC analysis

Soil Type	Total Unit Weight (kg/m^3)	Friction Angle (degrees)	Cohesion (kPa)	Poisson's Ratio (-)	Modulus (E) Parameters κ	α
Dense (In-Situ) Sands	1830	38	0	0.35	885	0.87
Loosened Sand after ground loss during mining	1680	33	0	0.22	660	0.40
Grouted sand	1830	38	70	0.35	—	—

Soil Properties

Subsurface conditions along the NOS alignment before tunnel construction were assumed to consist of medium dense to dense, poorly graded fine sands (SP) with a groundwater level below tunnel invert. In an attempt to simulate the loosening of soils due to ground loss during mining, strength and stiffness of the soil above the tunnel were reduced before the temporary support was introduced in the model. The soil properties used in the analysis are listed in Table 1.

Young's modulus, E, of ungrouted sand was varied with confining pressure after Duncan and Chen (Duncan, et al., 1980) according to the following equation:

$$E = \kappa \times p_a \times \left(\frac{\sigma_3}{p_a}\right)^{0.5} \times \alpha$$

where p_a is the atmospheric pressure, σ_3 is the minimum confining pressure, and κ and α are empirical factors obtained from matching the results of triaxial compression tests at different confining pressures.

Structural Properties

The tunnel liner was modeled as a region of 4 by 29 continuum elements for the arch, and 2 by 6 continuum elements for the invert slab. Three different models were analyzed, with minimum crown thicknesses of 15, 45, and 76 cm as shown in Figure 11. The liner at the springline was assumed to be 76 cm thick for all models. The unreinforced concrete was modeled as a Mohr-Coulomb material with zero friction, with unconfined compressive strength of 20 MPa, Young's modulus, E=8600 MPa, and zero tensile strength.

Anchor elements consisted of 19 mm diameter steel bars or cables. The anchors were unbonded for the first 3 m behind the tunnel liner and had 1.5 m of bonded length in competent material. They were aligned perpendicular to the tunnel liner oriented at 45, 60, 75, and 90 degrees from horizontal.

Modeling Sequence

The modeling sequence depicted in Figure 12 consisted of the following analysis steps:

Step 1: Pre-tunneling in-situ stresses with soils consisting of unsaturated, medium dense to dense, cohesionless sand.

Step 2: Simulation of ground loss during mining by allowing the tunnel crown to sag 8 cm before installing beam elements of the temporary support.

Step 3: Replacement of temporary support with final concrete liner composed of continuum elements.

Step 4: Phase-1 grouting: application of grouting pressure within the 1.5 m long bonded zone.

Step 5: Insertion of 4.5 m long anchor elements with 1.5 m bonded zone at the far end; and pre-stressing of anchors with a nominal load of 1.5 kN.

Step 6: Phase-2 grouting: application of grouting pressure within the zone between tunnel liner and bonded zone of anchors.

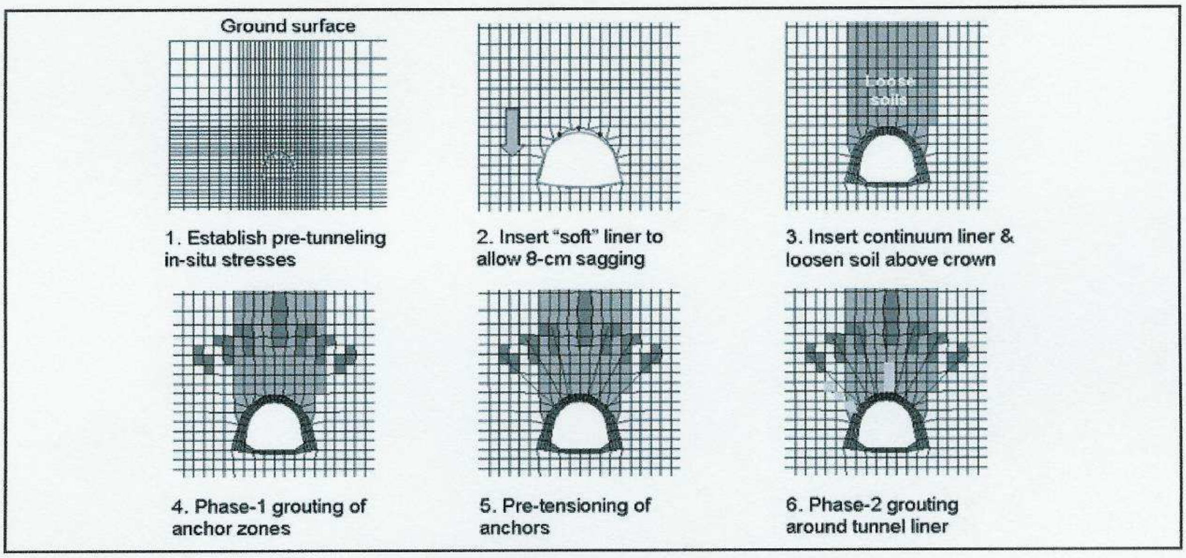


Figure 12. Modeling sequence simulating Phase-1 and Phase-2 grouting

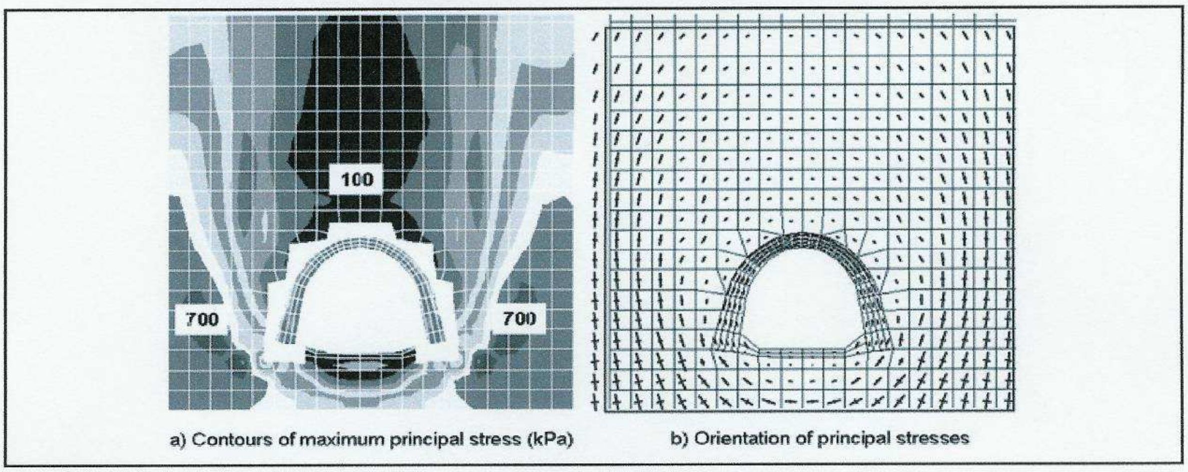


Figure 13. Soil arching after crown is allowed to sag 8 cm (Analysis Step 2)

Soil Arching Above Tunnel Crown

Existing subsurface information clearly indicated the presence of voids or loose soils above the tunnel crown. Thus, the tunnel crown was allowed to sag approximately 8 cm by way of temporarily relaxing the stiffness of the tunnel liner. The resulting transfer of overburden load to the soil around the tunnel (arching) caused a significant reduction of compressive stress in the tunnel liner from 2000 kPa to 200 kPa. Figure 13 shows contours and orientation of principal stresses due to soil arching.

Grouting Simulation

Grouting was simulated by incrementally increasing the normal stress within the soil zones being grouted. A routine was developed in which the target grouting pressure and pressure increments were specified for up to 4 individual soil zones. During "grouting" the soil strength and stiffness within each grout zone was greatly reduced to allow the zone to expand without internal resistance. After grouting, the zone properties were again modified to reflect the strength increase due to cementation of the grouted soil.

Because Phase-1 grouting involved the establishment of bonded lengths for the soil anchors, this grouting phase was applied before the anchors were ready to support the tunnel liner. Therefore, this grouting phase was considered to be the most critical construction stage. After all anchors were installed and their bonded lengths grouted during Phase 1, nominal prestressing of 1.5 kN per anchor

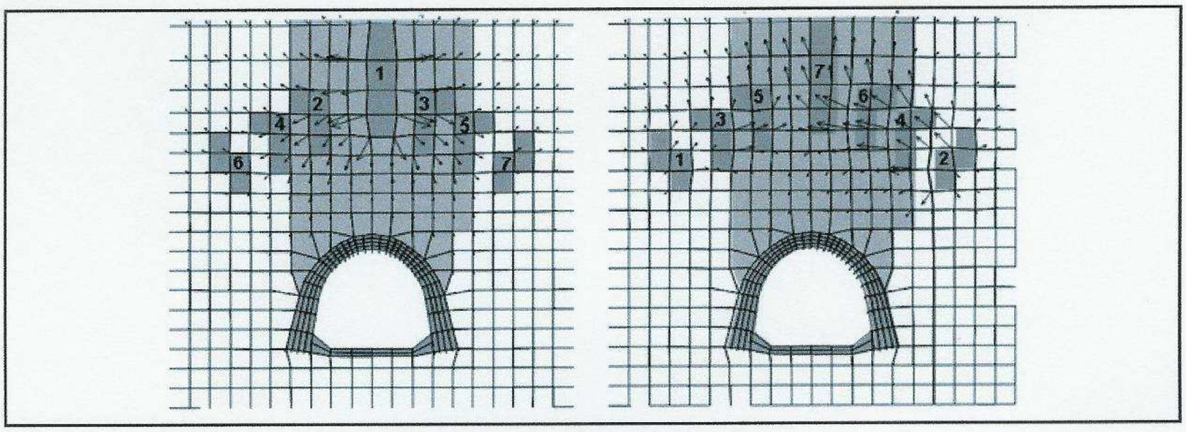


Figure 14. Comparison of Phase-1 grouting sequences (Analysis Step 4)

was applied before the next grouting phase was applied. Phase-2 grouting would then be performed in soil zones between the tunnel liner and the bonded lengths of the anchors.

Analysis Results

Structural performance criteria were established with the objective of defining the limits of tolerable liner response to grouting. Cracking and deformation of the tunnel liner were considered to be the two governing factors in this context. Performance criteria used to establish maximum allowable grouting pressures consisted of limiting grouting-induced tensile strains (i.e., cracking) and radial deformation of the tunnel liner to values of 0.1% and 25 mm, respectively.

Anchor Grouting (Phase 1)

Because the tunnel liner would not yet be supported during Phase-1 anchor grouting, this grouting stage was most critical for the stability of the liner. To investigate whether the sequence of Phase-1 grouting would have any effect on liner stability, the following scenarios were simulated:

- Bottom-Up Grouting sequence: 45°L, 45°R, 60°L, 60°R, 75°L, 75°R, 90°; and
- Top-Down Grouting Sequence: 90°, 75°L, 75°R, 60°L, 60°R, 45°L, 45°R,

where the angles indicate anchor orientation with respect to horizontal on the left (L) and right (R) side of the tunnel section shown in Figure 14. This figure compares grouting-induced soil and liner displacements of the two grouting sequences with an applied grouting pressure of 700 kPa. The more symmetric loading condition of Top-Down grouting resulted in less distortion and cracking of the liner than Bottom-Up grouting. Because Top-Down also resulted in less soil heave above the tunnel crown, it was adopted as optimum grouting sequence for all subsequent analyses.

Contours of plastic tensile strains (%) in the tunnel liner and displacement vectors are shown in Figures 15 and 16, for 15 cm and 76 cm thick tunnel liners in response to 700 and 1400 kPa grouting pressures, respectively. Figure 17 summarizes the results of the Phase-1 grouting analyses by plotting plastic tensile strain (indicating cracking) and crown displacements versus the maximum grouting pressure applied during Phase-1 grouting.

Applying the structural performance criteria discussed above, the analysis results indicated that the old tunnel liner would tolerate the following maximum grouting pressures during Phase-1 grouting without significant loss of structural integrity (Table 2).

Remedial Grouting (Phase 2)

Model runs were also performed to evaluate the effect of Phase-2 remedial grouting on the tunnel liner. Grouting pressures were applied in 3 m long soil zones along the anchors, between the tunnel liner and the anchor bonded lengths. Within these zones, grouting was performed in individual elements at various distances from the liner at the 45- and 90-degree tieback locations. Maximum grout pressures were increased until the deflection or tensile-strain criteria for the tunnel liner were exceeded. An example of computed displacement vectors for a 45 cm thick liner subjected to 1250 kPa of Phase-2 grouting pressure is presented in Figure 18.

The analysis results indicated that Phase-2 grouting could be applied at maximum pressures of 120 kPa for the 45 and 76 cm tunnel liners, and 1000 kPa for the 15 cm liner. Potential failure modes in all cases appeared to be localized punching through the concrete liner or anchor pullout.

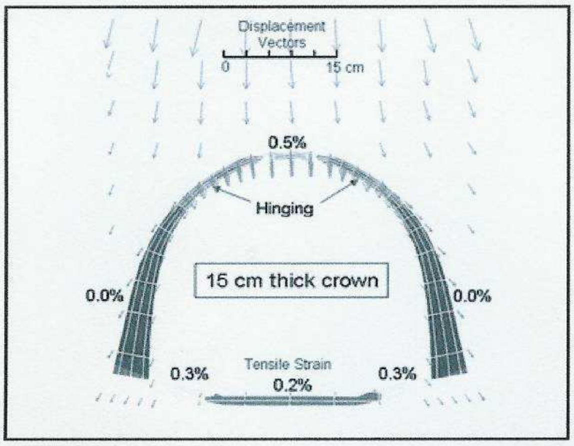


Figure 15. Displacements and tensile strains Phase-1 grouting at 700 kPa

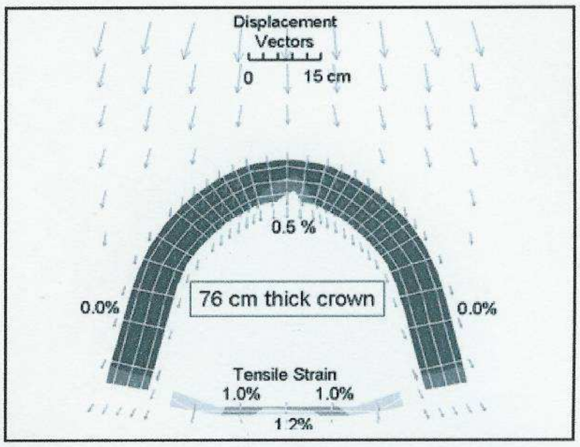


Figure 16. Displacements and tensile strains Phase-1 grouting at 1400 kPa

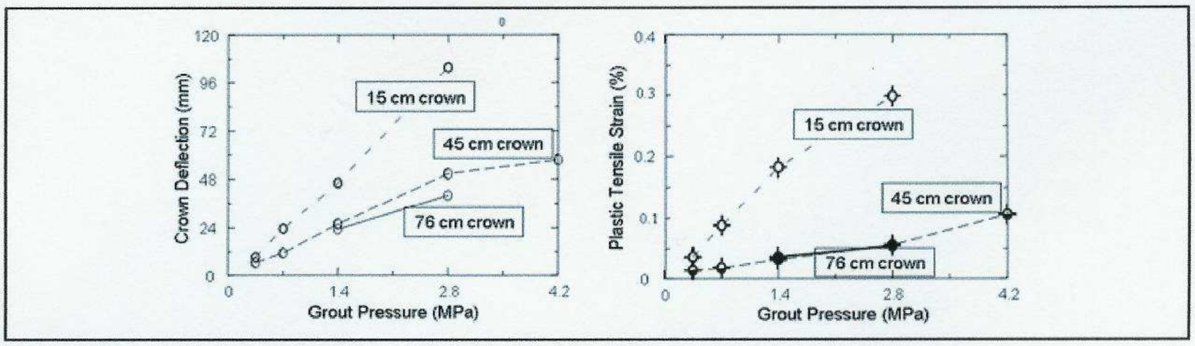


Figure 17. Crown deflection and liner tensile strain induced by Phase-1 grouting

Table 2. Allowable grouting pressure for Phase 1

Liner thickness at Crown (cm)	Allowable Pressure Applied 3 m Above Crown (kPa)
15	400
45	1250
76	1500

ONGOING PROBING AND GROUTING IMPLEMENTATION

As with most projects, final changes were made prior to going to bid in early 2005. The most significant change was that a new structural liner was installed before the probing and grouting work was done, thereby negating the need for the Phase-1 grouting of the anchors. With the new liner in place, we were able to increase the applied pressures for the Phase-2 remedial grouting. The increased grouting pressure allowed the work to proceed more quickly, but also required more sensitive monitoring of the surface for potential heave.

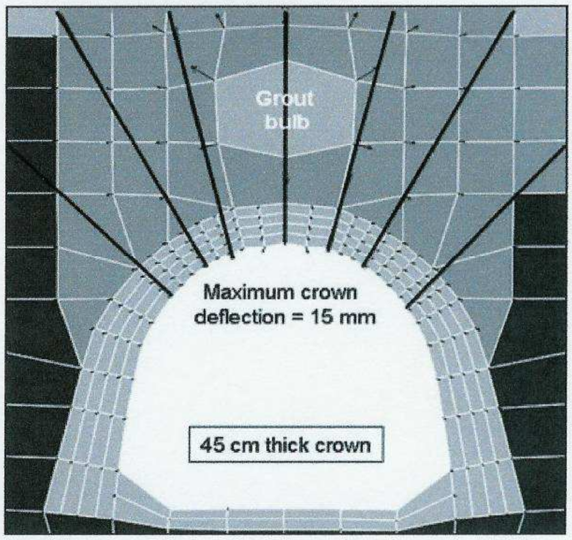


Figure 18. Phase-2 grouting with 1250 kPa pressure.

Figure 19. Sleeved grouting pipes with sacrificial cone

The rest of the original program was left largely unchanged with the primary emphasis on soil probing. This was done by reading the backpressure of the jacks that installed the sleeve port grout pipes shown in Figure 19, which had sacrificial cone tips slightly larger in diameter than the standard ASTM D5778 CPT. At the outset of the program, tip-resistance data from these modified cones were correlated in side-by-side testing with standard CPT equipment.

The data gathered during probing and grouting, and a more detailed explanation of the overall program, will be discussed in a follow-up paper. In summary, this work was conducted along 1500 m of the NOS tunnel. Fracture grouting was performed after filling gaps around the tunnel liner with contact grout. In addition, permeation grouting with microfine cement was performed in tertiary passes to strengthen the overburden soils as an added insurance policy.

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