Chapter 40

THE SOUTH BAY TUNNEL OUTFALL PROJECT
SAN DIEGO, CALIFORNIA

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ABSTRACT

Sewage presently being discharged into the Tijuana River Valley pollutes the valley as well as the beaches in the South Bay of San Diego, making the beaches unusable nearly year round. The SBTO will convey flows down a 8.5 m (28 ft.) by 58 m (190 ft.) deep drop shaft, through a 3.35 m (11 ft.) by 5.8 km (3.6 mile) long tunnel, and up a 46 m by 2.7 m (150 ft. by 9 ft.) riser shaft connecting to a 1.5 km (0.95 mile) of seafloor pipeline and 1.2 km (0.76 mile) of diffusers on the ocean floor.

The drop shaft will be sunk through 38 m (125 ft.) of saturated sands and gravels and 20 m (65 ft.) into the dense fine sands of the San Diego Formation. The tunnel liner has been designed as a five piece, single pass, gasketed, bolted, reinforced concrete liner designed to withstand an external head of 76 m (250 ft.) of water and internal operating head of up to 27 m (89 ft.) and to accommodate movement at the numerous fault crossings. The tunnel is contemplated to be driven with a state-of-the-art Earth Pressure Balance Machine (EPBM) or slurry shield machine through a wide range of conditions from clay to gravel with boulders.

PROJECT DESCRIPTION

The South Bay International Wastewater Treatment Plant (SBIWTP) will treat raw sewage originating in Tijuana, Mexico (Figure 1). The design of the SBIWTP, which is proceeding in parallel with the design of the South Bay Tunnel Outfall (SBTO), (Figure 2) is based on the

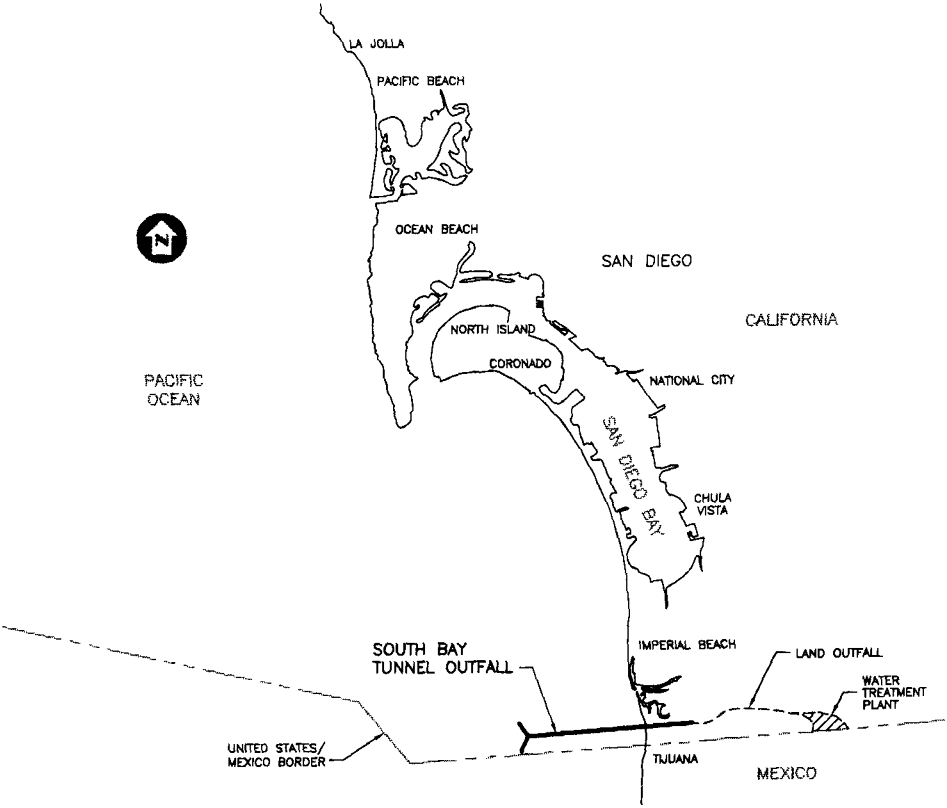


FIGURE 1. SITE LOCATION AND GENERAL PROJECT ARRANGEMENT

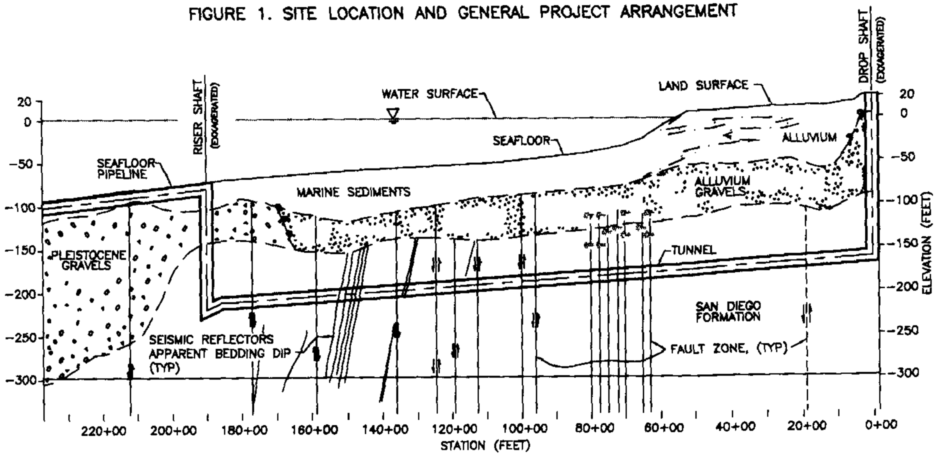


FIGURE 2. GENERALIZED GEOLOGICAL PROFILE OF TUNNEL ALIGNMENT

provision of wastewater treatment to the advanced primary level in the first phase of operation. Treated effluent from the SBIWTP will pass through an Effluent Distribution Structure and an Energy Dissipation Structure before entering the existing South Bay Land Outfall (SBLO or "Big Pipe"), a 3660 mm (144-inch) diameter, 3,750 m (12,300-ft.) pipeline. An Anti-Intrusion Structure will be constructed at the end of the SBLO and a Drop Shaft will convey effluent form the SBLO to an elevation of about 50 m (-165-ft.) mean lower low water (MLLW). At that point, a tunnel will be constructed with an internal diameter of 3353 mm (121 in.) and an overall length of approximately 5,800 m (19,000 ft.). The tunnel will terminate about 4,270 m (14,000 ft.) offshore where it will connect to a Riser, where effluent will be conveyed vertically to a 3050 mm (120-in.) pipeline on the seabed. A 1,520 m (5000 ft.) long sea floor pipeline will convey the effluent to a Wye where it will be discharged along two, 610 m (2000 ft) diffuser legs on the seabed. The total cost of the SBIWTP, SBLO, and SBTO is approximately $400 million dollars, and is expected to be completed by 1998. The SBTO schedule is 34 months.

The Tunnel is unique in that it will carry effluent under an internal operating heads of 1.1 to 2.7 bar (16 to 39 psi), and it will be constructed under the ocean with potential for high external groundwater pressure up to 7 bars (100 psi), averaging 6 bars (86 psi). The liner design criteria require design for the internal and high external heads and soil cover, while the Tunnel Boring Machine (TBM) must be designed to withstand the high external head. The tunnel liner design incorporates a five-piece segmented, single-pass liner with continuous hoop steel to carry the internal pressure

REGIONAL GEOLOGY AND SEISMICITY

The SBTO site is located in a coastal plain subprovince characterized by Quaternary to Tertiary aged sedimentary deposits within the Peninsular Ranges Geomorphic Province of Southern California. The topography along the alignment of the underground structures ranges from an elevation of about +7.6 m (+25-ft.) (MSL) at the Drop Shaft to about 23 m (-75 ft.) on the seafloor above the Riser Connection. The onshore portion of the SBTO alignment marks the southern margin of a broad lowland area of the Tijuana River Valley. The offshore portion of the SBTO alignment is located on an inner shelf of the continental borderland, which has been strongly influenced by the rise and fall in sea level over the last 20,000 years. The geologic formations and units encountered during the subsurface investigation are flat lying to westerly dipping (i.e. 2 to 15 degrees) and include the following (as shown in Figure 2): fill; alluvium; marine sediments; alluvial gravels; Pleistocene gravels; and the Plio-Pleistocene-aged, San Diego Formation, consisting of overconsolidated marine clays, silts, sands, gravels, cobbles, and boulders.

Numerous faults, both active and dormant, many of which cross the SBTO alignment, are located within the South Bay of San Diego County. These faults form a broad and diffuse set of northwest-trending, right (east) stepping, predominantly right lateral strike-slip splay faults, which merge to the better defined Rose Canyon fault to the north in the central portion of San Diego County. Even further to the north in Los Angeles, the Rose Canyon fault zone becomes part of the well defined Newport-Inglewood fault. The key seismic sources are the Rose Canyon fault as well as the Coronado Banks fault, which is about 10 km (6 miles) west of the Riser. These faults are capable of producing a Magnitude of 7-1/4 and 7-1/2 earthquake, respectively.

A probabilistic seismic hazard analysis (PSHA) and a probabilistic fault displacement analysis (PFDA) were performed and the results are in Table 1.

Table 1. Design Earthquakes, Peak Ground Accelerations, and Fault Displacements

Average Return Period	Probability of Exceedance	PSHA Peak Ground Acceleration	PFDA Fault Displacement
110 years	50% in 75 years	0.22g	~0
710 years	10% in 75 years	0.63g	76 mm (3 inches)

GEOTECHNICAL CONSIDERATIONS

An extensive geotechnical/geological investigation was embarked upon and included 5 onshore and 8 offshore borings. There are three geologic and/or soil units, and various soil subunits, which will be encountered in the underground excavations including: (1) Fill, (2a) Alluvium and (2b) Alluvial Gravels, and (3) the San Diego Formation (as shown in Figure 2). The alignment starts at the Drop Shaft where it will be constructed through Fill, Alluvium, Alluvial Gravels (basal alluvial layer), and the San Diego Formation. The Tunnel will be entirely in the San Diego Formation. The Tunnel profile averages approximately 61 m (200 ft.) below sea level; overburden/cover over the tunnel ranges from about 55 m (180 ft.) onshore to about 43 m (140 ft.) offshore. The overlying unconsolidated materials and the San Diego Formation are saturated responding to the sea level offshore, and the groundwater table rising to an elevation of about 3 m (10 ft.) onshore.

San Diego Formation

For the most part, the San Diego Formation is an indistinctly bedded, poorly indurated, fossiliferous marine siltstone. From a tunneling standpoint, the material will behave as "soft ground". Therefore, this extremely weak, friable, rock/strong soil-like assemblage of sediments is described according to the Unified Soil Classification System (USCS). Many of the fine grained materials are micaceous and behave like a strong soil and as a massive unit. The Formation consists of very stiff to hard clays; very dense silts; very dense, fine silty sands; clean sands; clayey, silty and sandy gravels/cobbles; and well-cemented concretions. The Formation may also contain some boulders up to 1 m (3 ft.) diameter. The Formation is unweathered or in a "reduced" state as opposed to the weathered or "oxidized" state observed in the nearby quarry and along highway cuts and cliffs. Consequently, the Formation appears to lack the characteristic cementation, buff color, and stone-like qualities of its claystone, siltstone, sandstone or conglomerate portions exposed above the water table. The San Diego Formation is relatively flat lying to gently west dipping.

The San Diego Formation exhibits a range of properties and behavior based on material type. The ground conditions based on laboratory and field investigations may be classified into 5 material types: (1) Clays and Cohesive Silts (CL, CH, ML); (2) Sandy Silts (ML); (3) Silty Sands (SM, SP); (4) Gravels, Cobbles, and Boulders or GCB (GC, GW, GP); and (5) Fault Zones.

Fault Zones may consist of any of the other material types, in combination or alone. A summary table showing the range and average of material properties is provided in Table 2. Gradations of materials from borings are summarized in Figure 3. Insitu Stress State.

Table 2 Summary of Geotechnical Properties Based on Laboratory Tests

Parameter	Unit	Clays (CL & CH) and Cohesive Silts (ML)	Sandy Silts and Silts with Sand (ML)	Silty Sands (SM)	GCB Gravelly Soils
Moisture Content	%	29.1 (23 - 41)	28.5 (19-35)	29.7(27-35)	22 (12-32)
Total Density	pcf	118.4 (108 - 124)	117.3 (116 - 119)	122.0 (119 - 125)	
Liquid Limit		43.6 (33 - 51)			(65[1])
Plastic Limit		27.5 (22 - 42)			(30[1])
Plasticity Index		16.1 (4 - 26)			(35[1])
Grain Size, D60	mm	0.037 (0.025 - 0.06)	0.071 (0.038 - 0.17)	0.27 (0.085 - 1.00)	21.1 (5.5 - 40)
Grain Size, D10	mm	0.0019 (0.0010 - 0.0040)	0.007 (0.002 - 0.01)	0.05 (0.007 - 0.25)	11.2 (0.002 - 23)
U, D60/D10		20.8 (11.1 - 50.0)	13.0 (6.5 - 32.0)	7.9 (3.3 - 15.0)	690 (1.5 - 2750)
% Passing #4 Sieve	%	100 (96 - 100)	99 (88 - 100)	98 (81 - 100)	18 (0 - 53)
% Passing #200 Sieve	%	88 (67 - 97)	68 (50 - 83)	27 (4 - 50)	11 (0 - 42)
Unconfined Strength	psf	6,073(1,670 - 13,750)	3,770 (2,040 - 5,500)	(1,380)	
Hydraulic Cond.	cm/s	($2.5x10^{-4}$ - $2x10^{-7}$)	($1.6x10^{-4}$ - $3.7x10^{-5}$)	($1.5x10^{-3}$ - $1.5x10^{-4}$)	($1.0x10^{-2}$ - $3.0x10^{-4}$)
Inflow Rate	gpm	(0.005 - 0.06)	(0.9 - 3.8)	(3.5 - 35)	(7.0 - 235[4])
Total Friction, ϕ	deg	34.9 (22 - 44)			
Total Cohesion, C	psi	9.9 (1.56 - 26.5)			
Effective Friction, ϕ'	deg	28.7 (26 - 31)			
Effective Cohesion, C'	psi	13.7 (7.9 - 18.6)			

1. Atterberg limits based on fines in one sample of clayey gravels.
2. Numbers in parentheses indicate range.

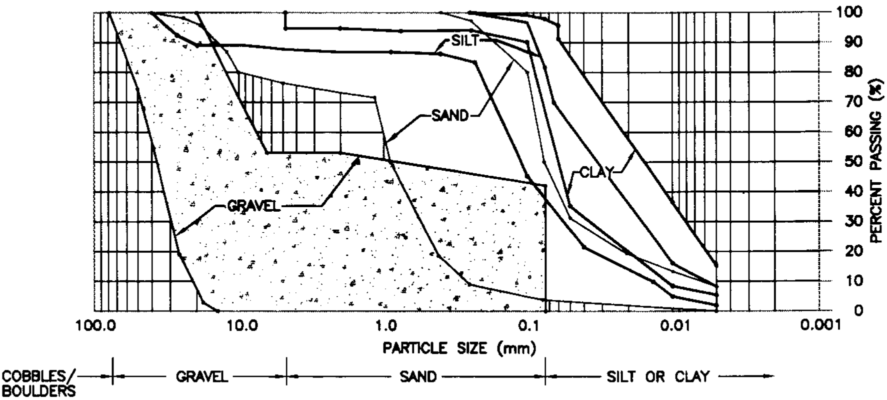


FIGURE 3. GRADATION CURVE ENVELOPES FOR CLAY, SILT, SAND AND GRAVEL FROM BOREHOLES

The total and effective vertical stresses are a maximum and the groundwater pressure is a minimum at the Drop Shaft. Over the land portion of the tunnel the total and effective vertical stresses are nearly constant at about 1150 kPa (165 psi) and 620 kPa (89 psi). As the tunnel moves offshore, the decreasing ground cover results in a slight drop in total and effective stresses while the groundwater pressure steadily increases with increasing tunnel depth below the sea. For the offshore portion of the alignment, the effective stress is nearly constant at 490 kPa (71 psi), and both the groundwater pressure, 680 kPa (99 psi), and the total stress, 1130 kPa (165 psi), are a maximum at the Riser.

The horizontal *in-situ* stresses were not measured, but can be inferred by a number of methods. Based on material properties (average effective friction angle of 30 degrees), the at-rest lateral earth pressure ratio, K_o, for a cohesionless material is 0.5. Based on the regional tectonics and the style of faulting (right lateral strike slip and normal), the maximum horizontal stress should be directed nearly perpendicular to the tunnel axis or within about 20 to 30 degrees.

Two of the water pressure tests in boring OB-1 went to a high enough pressure to cause apparent hydraulic fracturing and jacking of the soil surrounding the drillhole. Taking the pressure at maximum flow, after breakdown, as the minimum stress or shut-in pressure (may be high), the minimum stress appears to be equal to the vertical stress or $K_o = 1.0$. Taking this one step further and assuming that a vertical fracture was formed in the stiff clay (a bold assumption), the maximum horizontal stress is on the order of $K_o = 1.4$-1.6 for no material tensile strength and $K_o = 1.5$-1.7 for an upper bound strength of 140 kPa (20 psi). In summary, while the minimum horizontal to vertical stress ratio could range from 0.5 to 1.0, a value close to 1.0 is likely, directed near parallel to the tunnel axis. The maximum stress ratio could range from 1.0 to 1.7, but the value is probably less than 1.5.

Ground Conditions/Behavior

Ground conditions and potential for high external groundwater pressure require the use of a specially-designed, closed-face, fully-shielded Earth Pressure Balance Machine (EPBM) or slurry machine capable of operating in the wide range of soil conditions. For the range of materials expected, the stand-up time of an unsupported excavation below the water table would range from several hours to no standup time. Ground conditions could vary from firm, to slow and fast raveling, to cohesive running, to running, to flowing and to squeezing ground of various degrees. Competent clays and cohesive silts, those with greater than 200 to 350 kPa (30-50 psi) compressive strength, would be expected to behave as firm to slow raveling ground with moderate stand-up time (hours) and tunneling could be accomplished in an open heading provided support is installed immediately. Less competent clays and cohesive silts, with less than 200 to 350 kPa strength, would be expected to exhibit fast raveling or running behavior and/or considerable squeezing behavior to create very difficult tunneling conditions. In granular materials, on the other hand, well cemented ground or ground having a strong matrix or binder may exhibit behavior similar to competent clays or cohesive silts. Poorly cemented or cohesionless materials will flow into the tunnel heading, and create a condition characterized by flowing ground. Finally, it should be noted that the San Diego Formation may be slightly overconsolidated. It is estimated that the preconsolidation pressure; i.e. the amount of overburden the material has experienced in the past is on the order of 1000 to 1500 kPa (140 to 210 psi). Consequently, the soils may exhibit some swelling behavior.

For the clays and cohesive materials, the stability number ($N= P_z - P_a/S_u$) may be correlated to the ground behavior classification as discussed by Peck (1969) and Heuer (1974), as shown in Table 3. Stability numbers (N) were calculated based on the total vertical stress at springline, (P_z) unconfined compressive strength of clays and cohesive silts ($S_u = q_u/2$), and an internal compressed air pressure, P_a, of 3 bars (3 atmospheres, 44 psi); the minimum pressure for which the TBM is designed to use when servicing the cutterhead. The stability numbers without compressed air range from 3.5 to 29 while the stability numbers with compressed air range from 2.5 to 21.

It should be noted that the undrained shear strength was estimated from unconfined compressive strengths where sample disturbance may have contributed to loss of strength. In addition, most of the direct shear and triaxial tests in clays and cohesive silts derive significant shear strength from friction, on the order of ϕ'= 30 degrees. Using the average friction of 29 degrees and average cohesion of 100 kPa (14 psi) from the triaxial tests results, and assuming an internal confining compressed air pressure, P_a, of 3 bars acts on drained soil at the face, the strength of the soil would be on the order of 400 kPa (57 psi). Using the range of total vertical stress which may be expected in the tunnel of 1050 to 1150 (150 to 165 psi), the stability number, N, would range from 2.6 to 2.9. The external pressure from the ocean, however, could be as high as 700 kPa (100 psi). If the compressed air pressure is exceeded by the pore pressure, the strength of the soil would drop to as low as 230 kPa (33 psi). The corresponding stability number, N, would range from 4.5 to 5. In other words, the stability problem is transient, and depends upon the seepage, head loss between the ocean and the tunnel, and the permeability of the material in the heading. In addition at some point, the seepage pressure in the cohesive silt may be high enough that the stability number criterion is not valid and the silt will behave as cohesive running to flowing ground.

Table 3 - Criterion for Stability in Plastic Clays at Depths Greater Than Two Diameters

Value of N	Effect on Tunneling*			
7	Squeezing	General shear failures and ground movement around tunnel heading cause shield control to become difficult; shield tends to dive.		
6	Squeezing	Shear failure ahead of tunnel causes ground movements into the face even in shield-tunneling.		
5	Squeezing	Clay may squeeze rapidly into shield void.		Squeeze loads on tunnel supports must be considered.
4	Ravelling		Tunneling without unusual difficulties.	
1	Ravelling to Firm	Rate of squeeze does not present a problem.		

* The analysis may be applied to cohesive silts only if their properties are adequately defined by their undrained shear strengths.

Modified from McCusker (1982) after Peck (1969) and Heuer (1974).

The average permeability of materials as estimated from field and laboratory testing by material type is provided in Table 1, and vary from 1.0 x 10^{-2} to 2 x 10^{-7} cm/sec, corresponding to flows of 900 and 0.02 l/min (235 and 0.005 gpm) at the tunnel face based on porous media flow through a homogeneous isotropic media (assuming an impermeable liner). It should be noted, however, that the permeability of the materials are a function of the gradation. Granular materials sampled in the subsurface boring investigation scalp off the coarse-grained materials larger than the sampler size and may wash out the cohesionless sands and silts. There are correlations to the percent passing the No. 200 mesh sieve (0.074 mm) as well as the d_{10} grain size [size of grain in mm of the -10 % fraction; i.e, $(d_{10})^2$ (mm) = k (cm/sec)]. Although these correlations may not be strictly applicable, if there are enough fines, the permeability of the granular materials may be closer to the low range. Taking permeabilities on the order of 10^{-3} to 10^{-5} cm/sec, the corresponding inflows computed would be from a few to several tens of gallons per minute of inflow.

It should be noted, however, the small inflows in permeable materials could be dangerous. Potential for piping may be great depending upon the material gradation. Pinhole dispersion tests indicate that the clays tend to be non-dispersive while the coarser materials including the silts will tend to pipe. In addition, there are other possible groundwater inflow behavior other than porous media flow. These include fracture flow or "flow channeling" as well as fracture or shatter zone flow whereby large quantities of water could flow into an unsupported heading through joints or at fault zones. Some materials behave as strong soil, and so the potential for fractures within the San Diego Formation cannot be completely ruled out. It is believed, however, that porous media flow through coarse-grained materials will dominate flow behavior and is of greater concern. The closed-face shield, EPBM or slurry machine were specified because of the above concerns.

Another important consideration is the interaction of the cutters with boulders in GCB ground. Simple (Terzaghi) bearing capacity analyses indicate the bearing capacity of any boulder in a tunnel face/matrix of cohesive material would be considerable. The boulder in such a matrix would be cut without getting plucked out of the face. Within the San Diego Formation, however, the presence of boulders in very cohesive materials is not likely. Boulders are likely be surrounded by cohesionless materials or "nested" with other boulders. The bearing capacity of a boulder in a matrix of cohesionless material would be based on friction and its embedment in the tunnel face. Simple bearing capacity analyses indicate a 30 cm (1-ft.) diameter boulder with an effective embedment of 15 cm (1/2-ft.) (assuming 1/2 of the diameter) would have an ultimate bearing capacity of about 190 and 1000 kPa (4 and 21 ksf) for assumed friction values of 35 and 45 degrees, respectively. A 9.1 metric ton (10 ton) cutter force applied normal to a 30 cm (1-ft.) diameter boulder would exert a pressure of 1230 kPa (25.5 ksf), exceeding the capacity of the matrix to hold the boulder. Cutters applying an inclined load would apply close to the same normal pressure, but the bearing capacity would be reduced by 25 to 30% or more due to the inclined load (of 5 to 10 degrees). A 60 cm (2-ft.) diameter boulder with an effective embedment of 30 cm (1-ft.) would have and ultimate bearing capacity of 390 to 2000 kPa (8 and 42 ksf) for assumed friction values of 35 and 45 degrees, respectively. A 9.1 metric ton (10 ton) cutter force applied normal to a 60 cm (2-ft.) diameter boulder would exert a pressure of 310 kPa (6.4 ksf). Cutters applying an inclined force to a 60 cm diameter may exceed the capacity of the matrix to hold the boulder. For a 1 m (3 ft.) diameter boulder, it appears that the bearing capacity of the matrix greatly exceeds the bearing pressure of 130 kPa (2.8 ksf). It should be noted that the above discussion neglected the benefits of the confinement (or surcharge, increased depth factor) provided by the closed face holding the adjacent matrix.

TUNNEL BORING MACHINE

There have been three TBM's identified worldwide that were designed for operation in closed mode (pressurized tunnel face) at groundwater pressures above 7 bars (230 ft.), the maximum pressure in the SBTO; and of these three, only one has successfully operated in saturated, unconsolidated, soft ground conditions similar to that expected in the SBTO. The Channel Tunnel TBM, on the French side, excavated fractured chalk (a weak rock) at up to 10 bars pressure (335 ft.) head using a piston discharger to isolate and remove pressurized muck from the cutterhead. This was successful for operation in the non-abrasive chalk, but resulted in a muck high in water content, poorly suited for transport by conveyor. The Storebaelt Project EPBM's were designed to operate in soil and alluvium at up to 8 bars (260 ft. head). Problems with the machines operation resulted in a large scale dewatering program designed to keep pressure below 3 bars (100 ft. head). Accordingly, the maximum pressure experienced by the Storebaelt machine was 3 to 4 bars. The only known EPBM to work in soft ground, in true EPB mode, at pressures at or above that expected in the SBTO was the machine used on the Nara Prefecture water tunnel in Japan (Kawai and Tanabe, 1988).

Constructed between 1984 and 1989, by a joint venture between the Obayashi and Okumura Corporations, the 3.95 m (13.0 ft) diameter shield section of tunnel was 1151 m (3775 ft) long. This successful drive encountered two materials: a poorly graded, cohesive gravel with 10-50 percent gravel by volume, comprised of strong clasts, with an average diameter of 13 cm (5 in.) up to maximum size of 30 cm (12 in.); and a sandy unit mixed with gravel. The EPBM was specially designed to withstand a maximum water head of 11 bars (360 ft) with ground cover of 135 m (440 ft) in gravel. Major designs issues were strength of the shield body and bulkhead, and the cutterhead seal and tail seal watertightness and durability. The tail seal was comprised of a four row wire brush seal. To tackle the high water pressure a series of three screw conveyors were used, each dropping the pressures 3 to 4 bars (45 to 60 psi). Cutters, both drag pick and roller type, were of an abrasion resistant design and based on trial wear calculations were to be replaced at least twice.

When operating under maximum cover, water pressure up to 11 bars was measured when the machine was at rest, dropping 2 to 3 bars when excavation was resumed. In areas where the content of fines decreased below 25 percent, mud was injected as required for "sand plug" formation. When very stiff soil was encountered, up to 2200 kPa (310 psi) compressive strength, excavation rates were reduced to between 1 to 2 cm/min (0.4-0.8 in/min.) despite changing cutters on three occasions.

The 150 mm (6 in.) tail void was filled with a fast setting (4-8 seconds), low strength (2500 kPa, 360 psi) grout mix of unknown composition. Grouting was performed at constant pressure equal to the water pressure plus 3 bar (maximum of 15 bar), and occasionally would find its way into the pressure chamber when operating in those areas with high water pressure. This was rectified by decreasing the grout set time. Grout placement volume was about 130 percent of theoretical.

Machine Type

In general, as compared to EPB type machines, slurry machines offer more security, especially when operating in coarse grained materials at high water pressure. The cutterhead torque requirements are less, the cutterhead and tool wear is less, access to the back of the cutterhead is easier and the machine is normally fitted with a compressed air lock which is required on the SBTO machine. While the backup for a slurry machine is simpler, the slurry transport and surface plant and equipment for slurry treatment and separation is more complicated and expensive to operate. The need for environmental treatment of the muck to reduce salt content may, however, mitigate some of these costs.

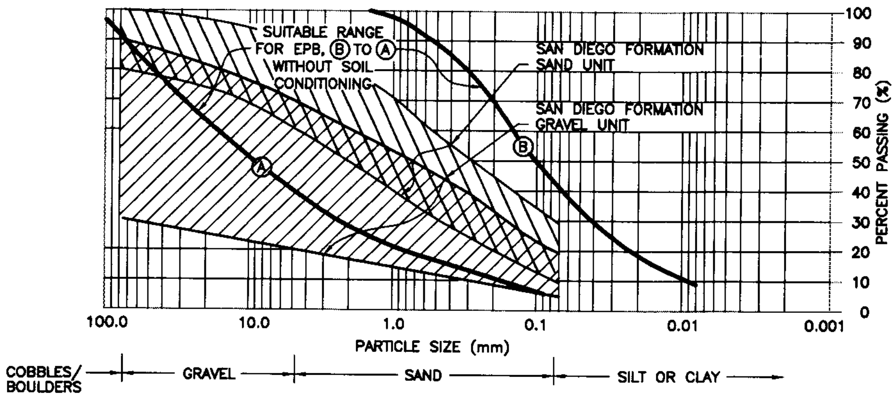
Given the length of tunnel drive in fine grained soils and the potential for operating in open mode within the stiff cohesive soils, the EPB type machine offers significant advantage over a slurry machine provided that modern foam/mud injection techniques are integrated into the system. In the end, the Contractor will select the machine type that offers the greatest security with the least number of interventions into the cutterhead for repair and/or maintenance.

Material Excavation

The particle size envelope for materials encountered in borings and from bulk samples are provided in Figures 3 and 4. Since gradation analyses from borehole samples may have limitations by scalping coarse-grained materials larger than the sampler size or washing out cohesionless materials, gradation ranges from bulk samples of the San Diego Foundation sandstone and coglomeratic unit are also included. Also displayed (on Figure 4) are material gradation limits from Hitachi Zosen (1981) between which EPB type machines can operate without soil conditioning. While other EPBM manufacturers may have different limits to define this range, the curves can be used to illustrate the expected material response to excavation while operating in EPB mode.

As indicated by the gradation curve envelope, the fine grained materials must be made adaptable for excavation by EPBM. The very stiff silty clays and cohesive silts that are finer than, the B curve in Figure 4 are likely to slow excavation if the machine cannot be run in open mode. Stiff cohesive soils cause blockages when the material adheres to the walls of the cutter pressure chamber, greatly increasing cutterhead torque requirements. Excavation cannot proceed until the soil is conditioned, either by water or foam injection to reduce the "stickiness". This is less of a problem when the machine can be operated in open mode where the pressure chamber and screw conveyors are not full of compacted material.

The use of foam to condition the muck has many advantages and is required if an EPB type machine is selected. When injected at the cutterhead, into the pressure chamber and the screw conveyors, the foam reduces the torque required to turn the cutterhead, reduces wear of soil contact surfaces and cutting tools, improves material flow while reducing adhesion, and aids in the formation of the "sand plug" or pressure drop zone within the screw conveyors. The injected foam/air concentrations at the cutterhead, pressure chamber and screw conveyors are adjusted for torque, head speed, excavation rate, chamber pressure/gradient and material characteristics. Because of the beneficial aspects of reduced wear, foam injection should be used at all times, even when not specifically required for material conditioning. Another major advantage of foam over the injection of other products, such as water and/or clay, is that the muck volume is largely unaffected by the injection of air. Use of clay additives for slurry machine operation, however, is cost effective as the material is recycled.



SEE FIGURE 3 FOR GRADATION OF FINE GRAINED AND OTHER UNITS FROM BOREHOLES

FIGURE 4. ADAPTABLE PARTICLE SIZE RANGE FOR EARTH PRESSURE BALANCE MACHINE AND SAN DIEGO FORMATION SAND AND GRAVEL UNIT FROM BULK SAMPLES

With slurry type machines the stiff materials can also be a problem, but for different reasons. Detached chunks of soil can block the suction ports, slowing excavation and causing pressure surges. The high percentage of fines in this material will markedly increase demands on the surface separation plant irrespective of the material strength.

The conglomerate unit of the San Diego Formation has a large percentage of gravel, cobbles and boulders with little fines. When the coarse material fraction with particle size greater than 2 mm (0.079 in.) exceeds 70 percent, or the fines are less than 7 percent, foam or clay/silt additive is injected into the cutterhead chamber to improve soil plasticity and face stability. If material friction is too high, the muck bulks in the cutterhead chamber and will not flow or produce a uniform slurry. Furthermore, when an EPBM is employed as opposed to a slurry machine, the "sand plug" needed for controlled discharge of muck from the screw conveyor will not form. Essentially, the excavated material must be modified to the gradation of curve A in Figure 4 for successful operations.

Face stability cannot be maintained unless the cutter chamber pressure equals the sum of the groundwater and active soil pressures. If the soil is too permeable, above the 10^{-2} cm/s, as is likely in the Gravel-Cobble-Boulders (GCB), the unconditioned slurry will not form a relatively impermeable layer on the tunnel face, but will flow uncontrolled from the face whenever the chamber pressure is raised above the groundwater pressure. By using a clay or clay/silt additive to increase slurry density an impermeable layer can be achieved. For EPB operation in the permeable GCB the use of foam also serves to reduce the permeability of the muck in the pressure chamber and screw conveyor through the surface tension of the bubble membrane, thus preventing uncontrolled water flow through the soil mass.

Very strong boulders up to 1 m (3 ft.) in diameter are expected on certain reaches. To cope with boulders of this size the EPBM or slurry machine is to be fitted with disc cutters. Fortunately, the number of boulders in the 30 to 100 cm (1 to 3 ft.) size range is not great. Because cobble and gravel size particles dominate the GCB material, the design of the EPBM requires ingestion of minimum 30 cm (12 in.) diameter clasts. Similarly, a slurry machine must be able to crush the cobble and boulder size material to that required for slurry transport.

Material Abrasivity And Cutterhead/Tool Wear

Because of the risks and delay associated with entry into the cutterhead for inspection and/or repair, every effort should be made to design the cutterhead wear surfaces and cutters for the maximum life possible. For example, the outer ring of the cutterhead should be equipped with hardened wear surfaces and/or auxiliary cutting bits. Disc cutters should be mounted so as to increase the life of drag bits when operating in GCB materials. Drag bit shanks should be hardened to reduce wear that often occurs on the back surface of the bit. All rotating surfaces in contact with muck should be hard faced.

Based on typical life-wear data for an EPBM, it is expected that a complete cutter change would be required at least once during operation in the sand and silt materials expected along the majority of the drive and a second change after completing the excavation of GCB material. Generally, the gravel materials are twice as abrasive as the silt/sand. As previously indicated, the foam acts as a lubricant to reduce friction and should be used not only when required for soil conditioning, but whenever possible to reduce tool/cutterhead wear.

Because the "better" ground, where inspection and/or repairs can be made with greater security, is not conveniently located along the alignment, the number of cutter replacements will be more than if the full life of each cutter was realized. There are mandatory inspection stops where ground conditions, with the aid of compressed air, are expected to be suitable for manned entry into the pressure chamber. All worn cutters would be replaced at each inspection Station.

Although it is not required, the cutterhead and cutters should be fitted with some type of abrasion detection device. Various methods are continuous measurement by sonic method or wear point warning by electrical resistance, hydraulic pressure or color dye injection methods. This would provide warning of excessive wear, before sever damage is done, that could occur between the inspection Stations. The ribbon/screw conveyor may also require replacement and/or repair, and should be designed for replacement within the tunnel.

Face Stability And Cutterhead Access

Based on available drillhole information, the heading stability under 3 bars (45 psi) of compressed air is adequate for short term access. Although there are reaches of tunnel where the heading should be stable without compressed air, it is difficult to predict the standup time and the use of air adds a degree of security that is warranted. Also, by requiring the compressed air lock, with a minimum of 3 bars, the range of ground conditions where interventions can be safely made is markedly increased.

There are, however, significant reaches of tunnel in GCB and cohesionless sand or silt where the compressed air lock is unlikely to prevent flowing ground, as the air pressure is too low to balance the water pressure. If access to the cutterhead is required in one of these areas for an unplanned inspection/repair, the only likely method by which this can be effected is through the use of ground freezing. For this reason, freeze ports are to be provided on the EPB or slurry machine. It is not planned to use freezing for access, but is required for an unplanned event.

In addition to the required inspections, the Contractor is likely to make many additional inspections as required to monitor cutterhead/cutter wear. To facilitate this work the EPBM will be fitted with a probe drill to aid in evaluating ground conditions ahead of the face. It is likely the Contractor would drill up to 6 to 10 m (20 or 30 ft.) ahead to the face before emptying the pressure chamber of muck. This procedure would also apply at the required inspection Stations.

The ability to run the EPBM in open mode in cohesive material depends on many factors. The strength and uniformity of the ground in the tunnel face, the permeability of the soil mass, and the rate of machine excavation will all influence face stability. If the strength is such that the stability number is less than 5 and the face is more or less uniform, an open mode should be possible. If, however, a layer of cohesiveness silt or sand under high water pressure with recharge is located within otherwise competent material, a mixed face condition, closed mode is required. The machine must be capable of switching to closed mode in an instant. If this is not possible, the EPBM must be worked in closed mode.

Machine Requirements

Other machine requirements not addressed above are related to seals and bearings. Because of the long drive the cutterhead bearing and seal must be replaceable from within the tunnel, despite being an unlikely event, as they are designed to perform for the duration of the tunnel drive. A spare main bearing, bull gear and cutterhead seal are to be deliverable to the site within 3 weeks notice.

Tail seals, at a minimum, are comprised of three rows of wire brushes of which the inner two are changeable from within the tunnel. Tail seal life is a function of the way the machine is operated, the concrete finish on the outside of the segments, the quality and quantity of grease used, the care with which the segments are installed, and grouting procedures. Accordingly, the Contractor is responsible for estimating tail seal life. For ease of installations and safety, all cutters are to be back loading from behind the cutterhead. Because of the unprecedented nature and length of the tunnel drive a used machine or modified used machine is not allowed.

TUNNEL LINER DESIGN

The basic precast segment ring is comprised of five 1.2 m (4 ft.) wide tapered segments. Segment design includes a left and right tapered ring for steering adjustments (Figure 5). By using both a left and right taper the key is kept above springline for ease of installation. The Contractor may set the amount of ring taper used, typically between 1.3 to 2.5 cm (0.5 and 1.0 inch). All the segments are rectangular except the trapezoidal segment which acts as a key and the two adjacent segments. All segments subtend a 72 degree angle to the tunnel axis, at the

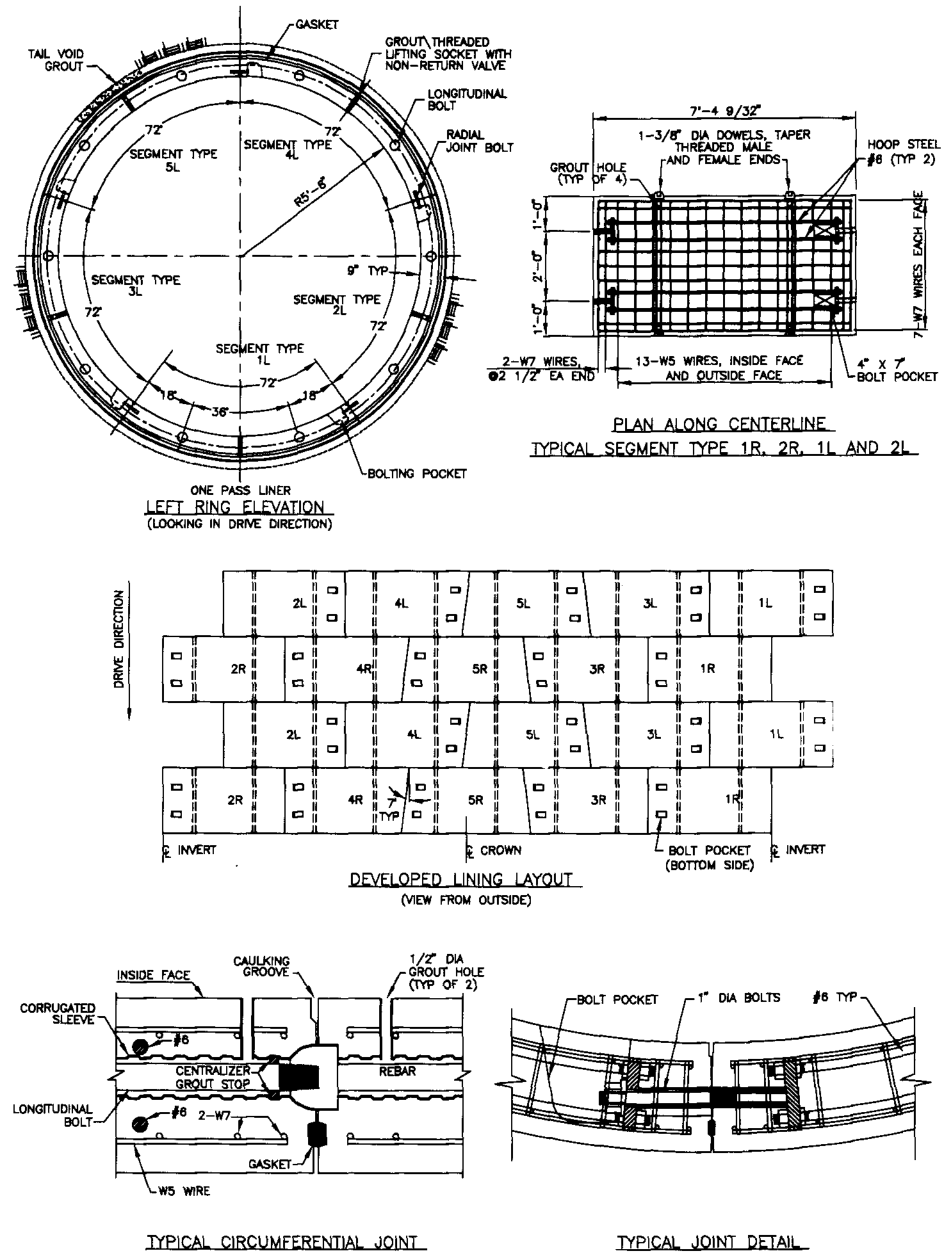


FIGURE 5. PRECAST SEGMENT LAYOUT

centerline of the ring. Segments are 229 mm (9 inches) thick to provide the minimum 38 mm (1.5 inch) cover on each face for corrosion protection of the reinforcement.

The tunnel liner is designed for full overburden and maximum external groundwater pressure for construction/inspection. For operation the liner is designed for a differential water pressure of 27 m (89 ft.) head, resulting in hoop tension reinforcement within each segment and tensioned radial bolts. The reinforcement steel meets ACI-224 crack control design with an 0.2 mm (0.008 inch) crack limit in direct tension for 12 m (40 ft.) head (normal operation load condition) and working stress at 60 percent of yield for 27 m (89 ft.) head (short term flushing load condition).

Circle joints are flat or plane, fitted with a 3.2 mm (1/8 inch) maximum thickness packer, designed for maximum thrust pressure of 12.6 mPa (1800 psi). The radial joints are articulated 2.5 m (8 ft - 2 in.) radius to accommodate high thrust, up to 2100 kN/m (144 kips/ft), and joint rotation upon liner ovalization or squat, up to 1.0 degree rotation or 0.80 percent diametric distortion. Radial joints do not use packers.

Circumferential bolts are spaced at 36 degrees, two per segment. Bolts are fitted with a spherical head which acts as both a shear key and alignment fixture to facilitate segment positioning on installation. The circumferential joint is designed for 35 mm diameter (No. 11) longitudinal bolts at all fault crossings (a total of 966 m of tunnel). At all other locations the Contractor may continue the use of smaller 19 mm diameter (No. 6) size bolts or change the design to use a minimum of two lock-up type bolts per segment. In any case, bolts must be used for the circumferential joints. Also, all longitudinal joints should be staggered in order to avoid cross joints at segment intersections.

Segment gaskets are required to form a watertight seal under a combination of gap and offset when exposed to a minimum water pressure equal to twice the maximum hydrostatic pressure of 14 bars (200 psi). Made from EPDM, the gasket profile has a minimum width of 25 mm (1.0 in.). The gap between the segments at the longitudinal joints must be fitted with a second gasket to prevent leakage past the tail seals. All other joints are fitted with a similar gasket to prevent grout migration into the joints.

Immediate grouting of the tail void is required to control liner distortion or ovalization. This is accomplished by grouting simultaneously with TBM advance, through pipes installed in the tail shield with ports immediately behind the wire brush seals. The grout mix must be sufficiently stiff so as to provide for the development of immediate passive reaction, to limit liner ovalization/float, in response to the external loads that develop after the ring leaves the tail shield. This is achieved by requiring a stiff or fast setting grout mix that provides normal stiffness through friction from a high sand content grout mix with additives for workability or the use of retarder/accelerator admixtures to gain rapid strength.

All bolt pockets are to be filled with specially cast concrete bolt pocket plugs to complete the smooth tunnel wall. Before the plugs are installed all segment bolts are retensioned to 22 kN (5 kips) minimum. All grout holes, lifting sockets and caulking groves are also patched to achieve a smooth tunnel wall. Because the precast concrete segments form the final tunnel liner, serviceable for the 75 year design life, the fabrication, installation, finishing and repair are rigorously controlled to achieve high standards.

CONCLUSIONS

Besides being of great benefit to the people of San Diego, the South Bay Tunnel Outfall will advance the standard-of-practice of tunneling. In this respect, the TBM will be required to handle a variety of ground conditions under high external head, as only a handfull of TBM's worldwide have been designed for such high heads. In addition, the design provides for a unique one pass liner system utilizing bolted, continuous hoop reinforcement for internal pressure.

ACKNOWLEDGMENTS

Acknowledgments are due to the Parsons Engineering Design Team for the SBTO including G. McBain, L. Meiorin, F. Collins, P. Amberg, W. Beuhring, C. White, D. Duprey, S. Palmer, S. Thukar, R. Cuartas, and B. Bucher. Thanks is also due to the Technical Review Committee: R. Trembath, T. Brekke, J. Wilton, B. Gerwick, and D.F. Fine. Special thanks to D. Schug of Woodward-Clyde Consultants and W. Ruth of the International Boundary and Water Commission.

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