

Peanuts and Tunnel Stuffing – The Brightwater Conveyance System East Contract

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ABSTRACT

The Brightwater conveyance system is located near Seattle, Washington in the north-western United States. The East Contract portion of the system consists of a 4.3 km long, 5.9 m diameter tunnel in soft ground and two large shafts constructed in challenging soil conditions. The two shafts are situated side-by-side in an urban valley. One will house an influent structure to direct incoming sewage and outgoing treated effluent, while the other will house a 645 megalitre per day (ML/d) influent pump station.

The tunnel contains four pipelines: two force mains to convey untreated wastewater eastward from the new pump station to a new treatment plant, one gravity-pressure pipeline to convey treated effluent westward to a new outfall in Puget Sound, and one high-pressure pipeline to convey reclaimed water from the treatment plant to downstream distribution points. The shafts and the tunnel presented many design challenges.

Groundwater is near the surface in the area surrounding the shafts, and compressible peat and clay soils preclude dewatering during construction. These and other challenges were encountered during design of the pump station shaft and multiple-pipe tunnel. The challenges were addressed by a design that includes a dual intersecting circular cell shaft (referred to as 'the peanut') and a single tunnel into which stacked pipes will be 'stuffed'. This paper discusses the design approach and design details for both elements.

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INTRODUCTION

King County is the wastewater conveyance and treatment authority for the Seattle, Washington (US) metropolitan area. In 1999, after six years of study, the county decided to construct a new wastewater treatment system to reduce the pressures of population growth on its existing conveyance system and two existing wastewater treatment plants. Located in the rapidly growing area north of the city of Seattle, the proposed Brightwater treatment system includes a new membrane bioreactor treatment plant designed to produce effluent water of a quality that meets regulatory permit requirements.

The conveyance elements of the Brightwater system will include approximately 21 km of pipeline (influent and effluent) in four major tunnels, a marine outfall pipeline discharging into Puget Sound, four shafts, a new influent pump station (IPS), several microtunnels and two odour control systems. To minimise the construction impacts on the largely residential areas along the pipeline alignments, King County elected to construct the conveyance system elements primarily by soft ground tunnelling. When complete, the conveyance portion of the system will have the capacity to deliver up to 645 ML/d of wastewater to and from the plant. It will also convey reclaimed water to distribution points, from which it can be routed to potential users. The conveyance system will span east to west across northern King County, as indicated on Figure 1.

The East Tunnel Contract (ETC) portion of the conveyance system connects the IPS to the plant, which is located at the eastern edge of the project. The two shafts, the influent structure (IS) and IPS, are located adjacent to each other on the North Creek Portal (NCP) site at the west end of the project. The IS shaft is the starting point for all tunnelling activities, and the IPS

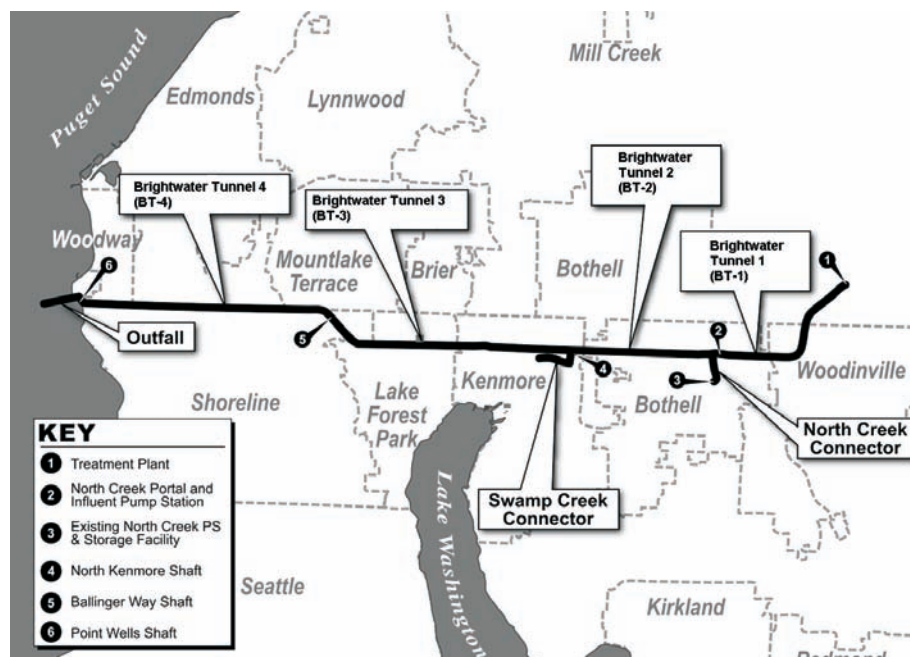


FIG 1 - Brightwater conveyance system.

shaft will ultimately house the new 645 ML/d influent pump station. The NCP site is connected to the plant by Brightwater Tunnel No 1 (BT-1).

NORTH CREEK PORTAL SHAFTS

Geology

The ground surface in the Brightwater project area was shaped by the advancing Vashon Stade glacial ice, which created generally north-south trending drumlinoid topography. This topography consists of ridges or upland areas separated by modern stream valleys and the troughs of the larger lakes and waterways of the Puget Sound region. As well as being shaped by glacial activity, the project area is cut by several linear north-west-south-east and north-south trending modern stream valleys.

Geotechnical exploration for the entire Brightwater project encountered subsurface deposits from at least three glacial cycles and three non-glacial cycles, resulting in a stratigraphy with widely varying properties. Due to all this glacial activity, zones of non-glacial soils, glacial tills, lacustrine clays, silts and outwash sands and gravels are present, in both overridden and non-overridden deposits. Non-overridden deposits, also referred to as recent alluvium and recessional outwash, tend to be loose to medium dense sands and silty sands. The overridden or advanced deposits tend to be dense to very dense silty sands and clays.

The NCP site is located within one of the north-south trending modern stream valleys. Soils consist of loose sands and silty sands, and low-strength unconsolidated organic deposits. While not present directly at the shafts, unconsolidated organic deposits are present at depths from 2 m to 10 m throughout the east end of the NCP site and the surrounding area. In addition, groundwater at and surrounding the site is within about a metre of the ground surface.

Shaft layout

Before the layouts of the shafts at the NCP site were determined, both the construction and operational requirements for the site were evaluated. During this evaluation, different shaft shapes, such as circular or rectangular, were considered, as were single and dual shaft arrangements. Due to schedule constraints and

ground conditions, construction needs proved to be just as important as operational needs.

During construction, a shaft is needed on the NCP site for tunnelling activities associated with the ETC as well as the Central Tunnel Contract (CTC). The ETC activities include launching and staging the BT-1 tunnel drive and launching the first drive of the microtunnel that connects the Brightwater system to the existing wastewater system. All of these activities launch from the IS shaft. CTC activities also require a shaft on the NCP site to serve as an end point for the Brightwater Tunnel No 2 (BT-2) tunnel boring machine (TBM). To meet project schedule constraints, the shaft arrangement must allow the IPS contractor to begin construction of the IPS within the shaft before BT-1 is complete, without hindering the ETC contractor. Another complicating factor is that the NCP site is relatively small, with about 11 000 m² of useable space. As such, the shaft(s) must occupy as little space as possible to allow both contractors adequate working room.

Operationally, the NCP site is the low point in the conveyance system for influent flows. Influent flows travel to the site by gravity from two separate directions, are combined, and are then pumped uphill to the plant for treatment. To accomplish this, the shaft(s) on the NCP site must be able to receive and combine influent flows and house the new approximately 32 m by 18.3 m IPS. Within these requirements, a major hydraulic design criterion is that the influent flow combination point has to be separated from the IPS wet well by 25 - 30 m to ensure laminar flow. Also, a portion of the shaft has to act as a surge chamber to temporarily hold excess influent flows during peak periods without flooding the IPS.

After reviewing the construction and operational constraints it was determined that an arrangement of two separate shafts would be best (Figure 2).

The IS shaft will stage all tunnelling activities during construction. During final operation, the southern half of the IS will collect influent flows and direct them toward the IPS by gravity. The southern half of the IS will also act as a surge chamber for influent flows. The northern half of the IS will serve as a pipe gallery for the force main pipes coming out of the IPS and into BT-1. The IS contains an isolation gate that can be used to isolate the IPS if it must be taken off-line. Also housed in the IS is an effluent system drain that will allow treated effluent to be drained from the effluent pipeline.

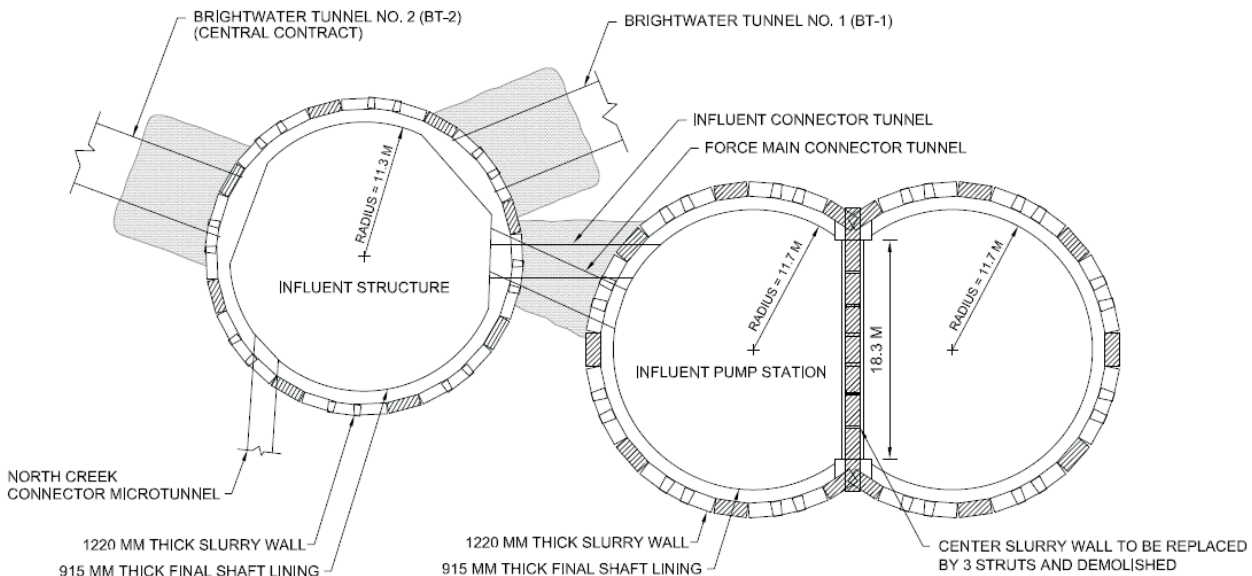


FIG 2 - Influent structure (IS) and influent pump station (IPS) plan view.

The IPS shaft, located just east of the IS shaft, houses the new IPS. The separation of the shafts provides the necessary distance between where the influent flows combine in the IS and where they enter the wet well in the IPS. To facilitate movement of influent between the structures, the shafts are connected by two tunnels with diameters of approximately 4.4 m. These tunnels are to be hand-excavated through ground improved by jet grouting. Both connector tunnels will be finished by the ETC contractor with 3.7 m diameter steel casings. Final outfitting will be completed by the IPS contractor. The lower connector tunnel conveys influent by gravity from the IS into the IPS wet well, while the upper tunnel contains two force main pipelines (1220 mm and 1675 mm) to convey pumped influent back into the IS and up through BT-1 to the plant (Figure 3).

Shaft support and construction

The soft soils and high groundwater levels at the NCP site, and nearby office structures, roadways and underground utilities, dictate that shafts on the NCP site be constructed with minimal dewatering, and be essentially watertight when complete, in order to avoid any dewatering-induced settlement in the surrounding area. Several shoring systems were initially considered for the shafts, including interlocking steel sheet piles, concrete caissons and slurry walls. However, as the overall conveyance system configuration developed, the required shaft depths at the NCP site became too great for sheet piles, and hydrostatic uplift loads became too great for caissons without the use of tie-down anchors. As a result, slurry walls were chosen as the best shoring system for the NCP shafts. They provide a relatively watertight shaft wall, and can be extended significantly below the shaft invert to provide additional weight for resisting the large hydrostatic uplift forces.

Excavation method and shaft invert placement are also dictated by site soil and groundwater conditions. The soil at the NCP site does not include a fine-grained layer at depth into which the shaft walls can be terminated to limit groundwater infiltration into the shaft during excavation. Therefore, in order to control risks associated with base heave and piping within the shaft, and ground subsidence outside the shaft due to dewatering, it is required that the majority of the excavation for both shafts be done in the wet. This technique consists of excavating soil from within the slurry walls while the water elevation in the shaft is kept near the groundwater elevation outside the shaft. At the NCP site it was determined that the top approximately 10 m of the shafts could be excavated conventionally with minor dewatering, while the remainder had to be excavated in the wet to avoid impacting groundwater around the site.

In order to create a relatively watertight excavation, both shaft inverts are formed by cast-in-place reinforced plugs placed by the tremie method. Water levels within the shafts can be lowered temporarily during invert placement to reduce working pressures for the divers placing the reinforcement that ties the invert plugs to the slurry walls. Following curing of the invert plugs, the water within the shafts is removed, leaving dry and unobstructed excavations.

Influent structure shaft

A circular shape was deemed best for the IS shaft, because in a circular shaft all of the soil and groundwater loads are carried in hoop compression, allowing for an unbraced open excavation. An unbraced excavation is necessary due to the requirement for wet excavation and placement of the invert plug in the wet. Internal bracing would obstruct the excavation equipment, be extremely difficult to install underwater, and obstruct placement of the invert plug reinforcing mats. The unbraced excavation also allows better access for tunnel construction and final build-out of the shaft by the pump station contractor.

The IS shaft has an internal diameter of 24.4 m, and a depth to the top of the invert plug of 22.6 m. Shaft diameter was determined by tunnelling needs and final operational requirements, while depth was determined by a combination of topography and the invert elevations of the incoming influent pipes. The slurry walls are 1220 mm thick and 39.6 m in depth, with the extra depth below the invert being provided for additional weight to resist the hydrostatic uplift forces. The invert plug is 4.1 m thick and structurally connected to the slurry walls. Prior to tunnelling, the excavated shaft receives a 915 mm thick, cast-in-place reinforced concrete final lining, bringing the finished diameter of the shaft to 22.6 m. This final lining is designed to carry the specialised loads generated at each of the tunnel penetrations, as well as all long-term seismic and groundwater loads. The slurry walls carry the long-term soil loads.

Influent pump station shaft

The size and depth of the IPS are dictated by operational requirements. The invert depth of the IPS is a function of the slope of the influent channel carrying wastewater from the IS shaft into the IPS wet well, and was determined to be 25.3 m, about 2.7 m deeper than the IS shaft. The excavation itself has to be large enough to house a pump station with approximate outside dimensions of 32 m × 18.3 m. The design of the IPS calls for six pumps (two 75 ML/d and four 170 ML/d) to provide

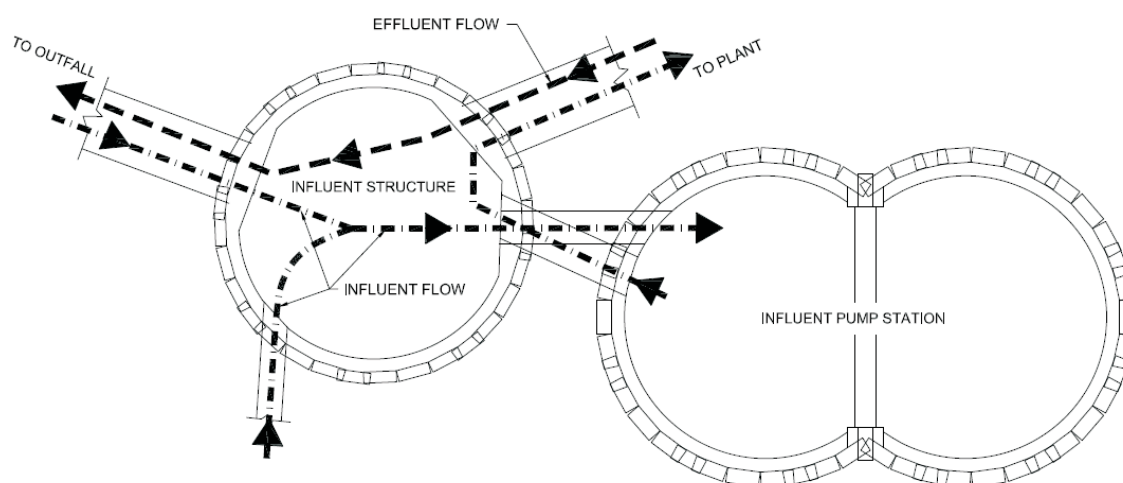


FIG 3 - Influent structure (IS) and influent pump station (IPS) flow arrangement.

capacity for the range of expected flows and to provide sufficient redundancy for potential failures. The pumps are arranged in a linear fashion parallel with the influent flow in conformance with Hydraulic Institute standards. This pump arrangement and the requirement for a 'self-cleaning' wet well result in a rectangular footprint for the pump station wet well.

A rectangular shape was initially considered for the IPS shaft as well, but was eliminated because it would require large amounts of internal bracing, an infeasible requirement for a shaft excavated mostly in the wet with a tremie-placed invert plug. A circular shaft similar to the IS was preferred, but the rectangular pump station footprint meant the diameter of the shaft would be very large at approximately 37 m and would leave a large amount of unused space in the four quadrants of the circle. More importantly, however, a shaft of that size would not fit on the NCP site and still allow the ETC contractor room to complete its contract work.

To address these difficulties, a 'peanut' shaft, consisting of two adjacent and intersecting circular shafts, was designed. The amount of circle overlap was determined by the minimum width required for the pump station (18.3 m), while the cell diameters were chosen to provide the necessary pump station length (32 m). The final layout is two intersecting 25.3 m diameter cells, with a distance of 18.3 m between the cell intersection points. The only required internal bracing is between the two cell intersection points; the rest of the loads are carried in hoop compression. Initially, a solid centre wall with access holes was envisioned to span the intersection points, but as pump station design progressed it became clear that larger unobstructed access ways between the cells were needed to provide a continuous pump station layout. In the final configuration, the solid centre wall is replaced by three large, rectangular, reinforced concrete beams, which allow unobstructed movement of ceiling cranes from one shaft cell to the other. The dimensions of the beams from top to bottom are 1220 mm × 1675 mm, 1375 mm × 1675 mm and 1830 mm × 1980 mm. Their tops are located flush with floor levels within the pump station to minimise the amount of usable space they occupy.

As with the IS shaft, the slurry walls are 1220 mm thick, but were increased from 39.6 m to 48.4 m in depth to provide additional weight to resist the hydrostatic uplift forces. The invert plug thickness was increased from 4.1 m in the IS to 4.6 m in the IPS and is also structurally connected to the slurry walls.

Shaft design

Jacobs Associates is the Engineer of Record for both the slurry walls (early works) and the final lining of the IS and IPS shafts. Consequently, Jacobs Associates was able to take advantage of the early works when designing the final lining (ie semi-composite structure). As stated previously, existing infrastructure around the shaft site eliminated the option to dewater during shaft excavation. With the large footprint and depth of the inverts, the major design issue was managing the high uplift forces due to hydrostatic pressure.

Slurry walls

The slurry walls are designed for short-term and long-term loads in accordance with American Concrete Institute (ACI) Code 318-02 and relevant local building codes. During early works, the slurry walls were designed to resist temporary ground, hydrostatic, surface surcharge and short-term earthquake loads. The north-west region of the US is categorised as a highly active seismic region.

For the permanent condition the walls are designed for the long-term ground loads and surface surcharge. The long-term earthquake surcharge ($1.6 \times D$, where D = shaft diameter) was taken up by the final cast-in-place lining and not the slurry walls. In addition to the typical load case previously mentioned, a

special load case, assuming a heavy soil for the upper 15 m of liquefiable soil, was developed and checked. This load case did not control the overall design and therefore did not add to the wall thickness or require additional reinforcement in the walls.

A three pinned arch method similar to the one shown in Puller (2003) was used to analyse the slurry walls. An assumed slurry wall panel verticality tolerance of 1:200, or 0.5 per cent was used in the design and included in the project specifications. The assumed verticality tolerance affects the required (effective) overall panel thickness because adjacent panels are assumed to deviate vertically in opposite directions (one toward the shaft centre and one away) during construction, representing the worst-case condition as the panels are installed. Using this design philosophy, an effective thickness is determined. This requires the panels to be thicker than necessary to ensure the effective thickness at depth is adequate to resist the loads. The effective slurry panel thickness of 610 mm for both the IS and IPS shafts was based on the aforementioned loadings and verticality tolerance. The as-built walls are 1220 mm.

Invert slabs

As previously mentioned, the invert slabs for both shafts are structurally connected to the slurry walls, allowing the entire shaft structure to act as a single unit to resist the large hydrostatic uplift loads. This structural connection is formed through a series of block-outs formed in the individual slurry wall panels. The block-outs are removed following shaft excavation, but prior to placement of the invert concrete. Prior to placing the invert plug reinforcing mats, 2.1 m long N36 steel dowels are connected into the block-outs using form savers placed in the slurry panel reinforcing cages. When installed, these dowels are located just below the final position of the invert plug reinforcement mats, and extend toward the shaft centre. Placement of the invert concrete fills these block-outs and engages the dowels and invert plug reinforcement, allowing the invert to resist the uplift loads and remain in place through concrete shear and engagement of the edge dowels.

The invert slab for the IS shaft is 4.1 m thick and is designed to act as a two-way slab (two mats of N43s each way), with the invert slab and slurry wall being connected around the entire shaft perimeter. The overall factor of safety against uplift for the completed structure in the temporary condition is just over 1.1. The dead weight (slurry walls and invert slab) alone is not heavy enough to achieve this factor of safety, so side friction acting on the outside of the slurry walls is used. Only the soil below 15 m to the bottom of the slurry walls is considered, as the upper 15 m of soil outside the shaft is considered potentially liquefiable. A factor of safety of two is applied to the frictional force. Due to conservative side friction assumptions, and the assumption of the groundwater being at the ground surface, it is believed the aforementioned factor of safety of 1.1 is much closer to 1.3 or better. In the final condition of the shaft, side friction is no longer needed to achieve the desired minimum factor of safety of 1.5 against uplift.

There are two invert slabs for the IPS shaft, one for each cell. Both are designed similarly to the IS slab with regard to side friction and factor of safety against uplift. However, the IPS is deeper than the IS shaft, and higher hydrostatic forces dictate an increased invert thicknesses of 4.6 m. Another difference is that both IPS inverts are designed to act as one-way slabs, with the action being parallel to the wall connecting the cell intersection points. This is because there will be little lateral soil pressure acting on the sides of the centre slurry wall after shaft excavation. The lack of lateral pressure combined with the reduced weight following demolition of the upper portion of the wall, limit the ability of the centre wall to aid the invert slabs in resisting the uplift forces. As a result, the invert slabs are not structurally tied to the centre slurry wall, just tied to the perimeters of the two shaft cells.

Final linings

The cast-in-place (CIP) reinforced concrete final linings of the IS and IPS shafts are designed to take advantage of the shaft early works (ie slurry walls). The final CIP linings resist the full hydrostatic pressure and long-term earthquake surcharge loads. It is assumed that the slurry walls will leak at some point during the 100-year service life and, as such, hydrostatic loads were transferred from the slurry walls to the final lining. To transfer the earthquake loads from the slurry walls to the CIP lining, 25 mm dowels spaced at 1.5 m by 1.5 m were designed. These dowels were designed to act in shear to keep the lining in contact with the slurry walls during a seismic event. The contractor opted to install 25 mm high-strength steel coil rods spaced at approximately 1.5 m by 1.5 m to facilitate the installation of the false work for pouring the CIP final linings. At the contractor's suggestion the strength and positioning of the steel rods was analysed to confirm that they were adequate to permanently connect the final lining to the slurry walls. This eliminated the need for the 25 mm dowels as originally designed, thereby saving the contractor and King County time and money.

Certain portions of both shafts are required to contain wastewater. A watertight final lining is required for these sections, which required designing to a stricter ACI code (ACI 350-01) than was required for the slurry walls (ACI 318-02). Rather than designing to two separate codes, it was decided to use ACI 350 for the entire final lining, as the possible reduction in steel was not significant. The decision to use ACI 350 for the entire CIP lining also reduced the risk of quality control issues during construction.

Empirical and numerical methods were used to analyse the lining. The numerical models were completed using the structural program STAAD.Pro (build 1002.US REL). The seismic load cases were performed in the STAAD models and an empirical check was done using methods shown in Wang (1993) and Hashash *et al* (2001).

Beam-spring models were used within STAAD to analyse the behaviour of the final lining for the various load cases. A beam-spring model consists of a series of beam elements (final lining) and elastic springs. Elastic springs are used to replicate the soil interaction between the final lining and the ground. The soil spring stiffnesses are calculated using the estimated ground properties. Initially, the slurry walls were considered in the STAAD models, but eventually the walls were neglected and the softer, less stiff, ground springs were used. The CIP lining, being circular in shape, develops most of the load in axial thrust. Thus, outward displacements (ovaling) due to unbalanced loads are minor. The softer, less stiff ground springs produced slightly higher forces, but comparatively the difference was insignificant when related to the necessary detailing of the CIP lining.

For the IS structure, two-dimensional plane-strain models were used. For the more complex IPS shaft geometry, a full three-dimensional model was used to better understand the behaviour of the structure during various load cases. Figure 4 shows the output stresses of one three-dimensional model used during analysis.

Influent pump station (IPS) cross struts

The three cross struts that connect the cell intersection points in the peanut will eventually act as heavily loaded beams for the final pump station layout. Before the final internal pump station is in place, the struts will undergo three main load stages. The first is the temporary stage before the final lining is poured. At this point, the struts will have unbraced lengths (in both directions) of about 18.3 m. The second main loading stage occurs after the final lining is in place but before the internal pump station is complete. The third and final loading stage occurs when the internal pump station is complete. The final loading did not control the design of the struts as floor

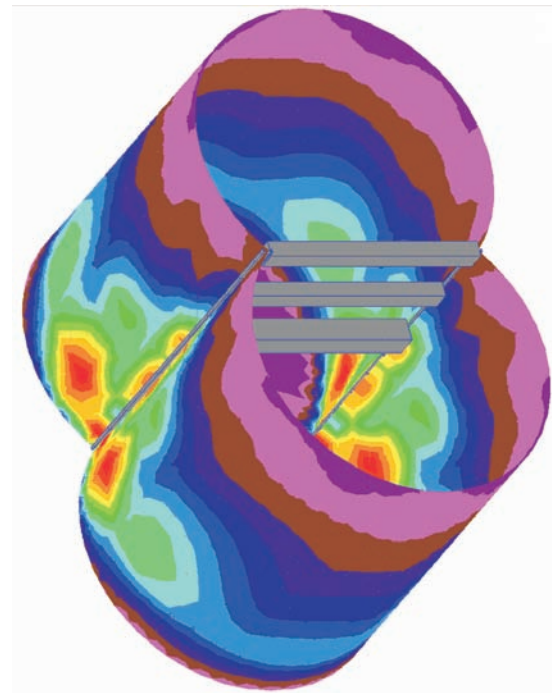


FIG 4 - STAAD plot of stresses within the influent pump station (IPS) final lining.

slabs/beams. A main central column was added to the struts, thus reducing the unbraced lengths to 50 per cent in the y-y axis and to the beam spacing in the x-x axis.

The maximum factored designed strut loads are 3750 kN for the top strut, 23 705 kN for the middle strut and 62 760 kN for the lowest strut. With these high loads, especially for the lower strut (425 cm² main steel), a large amount of reinforcing steel is required. To achieve a practical reinforcement spacing within the struts, the concrete strength is increased to 40 MPa.

Figure 5 shows the as-built wye connection detail used to connect the cross struts to the slurry walls and final lining. The end block connection is detailed to act in shear to transfer the thrust loads from the final lining. The wye is also designed to act as a beam between the struts and invert slab to distribute the large slurry wall thrust loads.

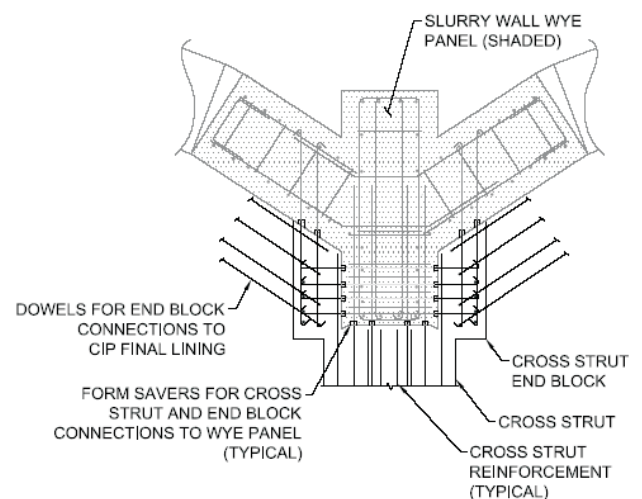


FIG 5 - Influent pump station (IPS) wye panel cross strut connection.

BRIGHTWATER TUNNEL ONE (BT-1)

BT-1 connects the IPS at the NCP site to the new treatment plant. The tunnel is approximately 4.2 km long and is being constructed with an earth pressure balance (EPB) TBM, which has an excavated diameter of about 5.9 m. Initial support of the tunnel is provided by a 255 mm thick, fibre reinforced concrete bolted and gasketed segmental lining system. The internal diameter of the finished tunnel is about 5.08 m. BT-1 will convey influent toward the plant, treated effluent away from the plant to the new outfall in Puget Sound and reclaimed water to distribution points where it can be routed to potential users.

Tunnel layout

The original plan for conveying wastewater to and from the plant involved two 4.3 m diameter tunnels, one gravity flow influent, and one pressurised gravity flow effluent acting as an inverted siphon. The plant was to have the IPS on site to bring the influent to the surface for treatment. However, in this arrangement the influent tunnel would be very deep by the time it reached the treatment plant, due to the topography of the area and the gravity flow arrangement. This would have led to a large and very deep (approximately 120 m) IPS excavation and subjected the tunnel to very large hydrostatic forces, which could be risky during construction. As a result, the IPS was relocated to the NCP site in one of the low spots along the alignment. The revised location of the IPS shaft is much shallower, but at this location the influent has to be pumped up to the treatment plant.

The twin tunnel arrangement was also replaced with a single tunnel layout early in the design process. A larger single tunnel was deemed to be less costly and involve less construction risk than the original twin tunnel plan. Also, due to its smaller easement width, the larger single tunnel crosses beneath fewer private parcels, significantly lowering the amount of private easements King County was required to obtain. Eliminating private easements was a primary design criterion to reduce the acquisition risk for the tunnel horizontal alignment.

Pipeline requirements

BT-1 will simultaneously carry influent toward the plant and effluent away from the plant, requiring separate pipelines in the finished tunnel. Hydraulic analyses revealed that twin influent pipelines, sized at 1220 mm and 1675 mm, were necessary to make the IPS work efficiently. The 1220 mm pipeline will carry flows from 35 ML/d to 245 ML/d, and the 1675 mm pipeline will carry flows from 245 ML/d to 490 ML/d. Both pipelines will be operational when flows range from 490 ML/d up to the IPS maximum of 645 ML/d. Basically, the smaller influent pipe will be the duty pump early in the project life, with the larger used for occasional high flows. As flows increase, the larger pipeline will be used more extensively. Between the years 2030 and 2040, as the treatment plant flow capability is expanded, both pipes will be required more often to convey the flows.

The effluent pipe is operationally simpler than the influent pipes, because it acts as an inverted siphon. As such, the effluent pipe was sized for the maximum possible flows from the plant in 2040 and treated as a gravity-flow pressure pipe. Due to the large number of pipes being placed within BT-1 and their effect on the overall tunnel size, the effluent pipe was sized as small as possible between the plant and the NCP. This resulted in a slightly larger effluent pipe west of the NCP, but a relatively slim 2135 mm ID pipe for BT-1, keeping BT-1 to a smaller size.

In addition to the ability to carry influent and effluent, King County requested that the tunnel be capable of conveying reuse water from the plant, necessitating an additional pipeline. This pipeline is sized at 685 mm, and brings the total number of pipes in the tunnel to four. Three small fibre optic cables will also be placed in BT-1 to enable communication among the various facilities of the Brightwater system.

The tunnel is backfilled with cellular grout, which helps stiffen the composite section of the tunnel and pipes, providing additional resistance to seismic strains. The mix design characteristics specified for the backfill were carefully selected, as the backfill needs to be fluid enough to be pumped over long distances to fill the annular spaces, but stiff enough to provide the additional seismic resistance and low enough in cement content that it does not boil the water in the mix as it hydrates. It also cannot damage the three fibre optic cables in the tunnel, which are temperature-sensitive. Figure 6 shows a cross-section of the completed BT-1. The decision to backfill the tunnel with cellular grout instead of leaving it open and securing the pipes mechanically was based on seismic concerns. Anchoring completely full pipelines of this size to resist the design level earthquake would not be cost effective given the size of the tunnel.

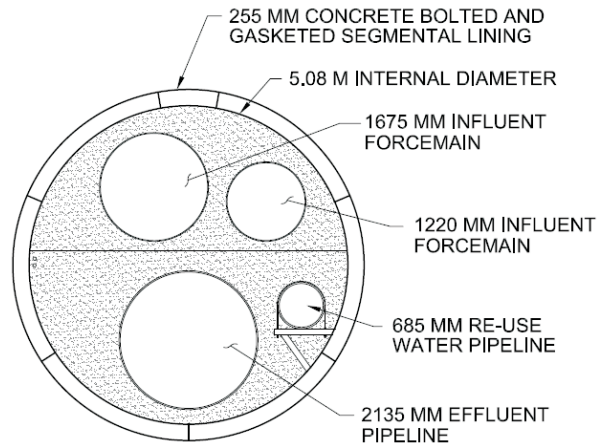


FIG 6 - Brightwater tunnel one (BT-1) section.

Pipeline design

All four pipelines are designed to remain elastic under all operating conditions, including internal operating pressures, internal transient pressures and external buckling stresses due to external groundwater pressure in an empty pipe condition. The pipes are also designed to resist the stresses imposed during the design earthquake and remain in service. A limited amount of damage or yielding may occur at the joints during the design earthquake, but should be repairable. Table 1 lists the operating conditions for which each pipeline has been designed.

TABLE 1
Pipeline pressures.

Pipeline size (mm)	Internal operating pressure (MPa)	Internal transient pressure (MPa)	External buckling pressure (MPa)
1220 (influent)	0.83	1.56	0.35
1675 (influent)	0.83	1.56	0.35
2135 (effluent)	0.66	Not applicable	0.35
685 (reuse)	1.31	1.65	0.35

Due to the relatively high internal pressures, the contractor is only allowed to construct the influent and effluent pipelines from either lined steel pipe or fibreglass reinforced polymer pipe (FRPP) within the majority of BT-1. The even higher pressures associated with the reuse pipeline require it to be constructed of either steel or ductile iron. As discussed in more detail below, pipe material requirements within the designated thrust restraint zones (located at the ends of BT-1) are more stringent. The contractor has chosen steel pipe for all four of the pipelines.

Outside of the thrust restraint zones, both restrained joints and unrestrained slip joints were allowed, depending on the operational loading conditions of the pipe and certain construction considerations. Table 2 lists joint types specified for each pipeline.

TABLE 2
Allowable steel pipe joints.

Pipeline size (mm)	Allowable joints
1220 (influent)	Carnegie rubber gasket, or rolled groove rubber gasket
1675 (influent)	Carnegie rubber gasket
2135 (effluent)	Butt welded, or lap welded, or Carnegie rubber gasket
685 (reuse)	Carnegie rubber gasket, or rolled groove rubber gasket

Unrestrained slip joints were allowed because: the tunnel is completely backfilled, they provide flexibility for the pipelines during a seismic event and they are easier to install than restrained joints (ie butt-welded joints). The contractor chose to use either Carnegie rubber gasket or rolled groove rubber gasket joints for each pipeline. The use of slip joints is anticipated to greatly decrease the amount of time necessary to install the pipes within the tunnel.

Since steel pipe was chosen by the contractor instead of FRPP, an interior lining is required within each pipeline to prevent corrosion. As with the pipe material and joint types, several lining types were allowed in the specifications for each pipeline, based on the operational condition of the pipe. Table 3 lists the linings specified as allowable for each pipeline.

TABLE 3
Allowable steel pipe linings.

Pipeline size (mm)	Allowable lining
1220 (influent)	Polyurethane
1675 (influent)	Cement mortar, or polyurethane
2135 (effluent)	Cement mortar, or epoxy
685 (reuse)	Cement mortar, or epoxy

Polyurethane was specified for the 1220 mm influent pipeline because it is expected to be subjected to increased usage and abrasion. At least two options were given for the remainder of the pipelines. However, polyurethane was proposed for all of the pipelines by the contractor. Polyurethane was originally not allowed in the 2135 mm pipeline due to the difficulty of field-repairing polyurethane linings. This was a concern because the polyurethane lining is shop-applied, and backfill concrete equipment will move through the effluent pipeline during pipe installation within the tunnel, damaging the lining and necessitating repairs. It was believed that cement mortar or epoxy would be easier to repair in the field. However, after reviewing the product data and repair procedures proposed by the contractor, and meeting with the material manufacturer, polyurethane lining has been allowed for all four pipelines.

The design also addresses pipe thrust. High pressures in the four pipelines, and associated bends in the pipelines, resulted in a large amount of thrust. This isn't a concern for the middle section of the tunnel, where unrestrained slip joints are allowed because the pipes are completely encased with backfill to prevent movement. However, thrust presented a problem at either end of BT-1 where the pipes exit the tunnel and lose their confinement, particularly at the west end in the IS. As a result, thrust restraint zones were designed for each end of the tunnel. Pipes within

these zones are specified as steel, and pipe joints are specified as butt-welded so the pipe acts as a single unit. Full-circumference steel rings, resembling penstock seepage rings, were designed for each of the pipes to engage the backfill concrete and resist the thrust forces. Backfill within the zones was specified as structural concrete instead of the cellular concrete used in the majority of the tunnel. Initially, friction between the backfill concrete and segmental lining was projected to resist the overall thrust forces of the four pipes. However, this resulted in overly long thrust restraint zones, requiring large amounts of structural concrete backfill and several butt-welded joints for each pipe. Realising that butt-welded joints are time consuming and expensive, the decision was made to limit the thrust restraint zones to 12 m, a reasonable length of a single stick of pipe. To account for this shorter length, N8 steel dowels are placed into the tunnel segments within the thrust restraint zones to engage the structural backfill around the pipelines. The combination of butt welded pipe joints (if necessary), steel rings around the pipes, structural concrete backfill and dowels in the segmental lining, allows the entire tunnel structure within the thrust restraint zones to act as a single unit and provide the necessary thrust resistance to keep everything in place.

Pipeline layout and minimum tunnel size

The minimum ID of BT-1 is based on optimising the installation and arrangement of the four pipelines inside the tunnel. Complicating matters is the restriction that all of the pipes must be installed from the uphill (plant) side of the tunnel because the IPS contractor is occupying the NCP site soon after completion of BT-1 excavation. The ETC contractor is only given a small area on the NCP site to access the IS shaft for ventilation and emergency tunnel evacuation.

Several options to install the pipes were considered, ranging from sequentially hanging the pipes from transverse beams mounted in the tunnel, to mounting one section of each pipe into a large frame and transporting entire frame assemblies into the tunnel one at a time. After many discussions with contractors experienced in placing pipe in tunnels, a more conventional approach was assumed for design. The effluent and reuse pipes will be installed in the tunnel first, beginning at the NCP end and moving east out of the tunnel. The reuse pipe will likely be mounted along the tunnel wall on a support system while the effluent pipe will be placed in the invert of the tunnel on blocking. These pipes will be transported into the tunnel on rail and placed along the entire tunnel length, with lifts of cellular concrete backfill being placed as necessary in the lower half of the tunnel. Rail equipment will then be installed on top of the backfill and the twin influent pipes will be installed in the upper half of the tunnel, with backfill placed in the same fashion.

Because flow within all of the pipelines is pressurised, specific pipe locations are not important in most of the tunnel. Accordingly, specific pipe locations and inverts are only specified at either end of BT-1, where connections with other contracts are necessary. Pipe locations along the 4.2 km length may vary as deemed necessary by the contractor, so long as the relative pipe positioning remains the same (ie the influent pipes remain above the effluent pipe). The relative positioning requirement is necessary to eliminate drastic changes in the influent pipe gradients, which could lead to very large surge pressures within the pipelines.

Concurrently with determining how the pipes would be arranged, attention was being focused on how large the tunnel needed to be to house the pipelines. The following tolerances were considered, all of which are included in the final project specifications:

- 150 mm minimum backfill around each individual pipeline at all times. All of the pipelines are designed against single lobe buckling (Jacobsen) due to external pressure. This design

methodology assumes competent backfill being present around the entire pipe to help resist buckling and limit the mode of possible failure. This requirement is necessary to assure design assumptions in this area are valid.

- 255 mm minimum backfill thickness between the outside of any pipeline and the wall of the segmental lining. This criterion was added to augment the above criterion to allow some additional access around the outside of the pipes if needed during installation. Because of the small size of the reuse pipeline, and the assumption that it would be mounted to the wall, it was exempt from this tolerance.
- 100 mm tolerance on the TBM line and grade. This is a requirement of the TBM and was considered during this analysis to ensure that variability in the tunnel alignment wouldn't encroach on the room necessary for the final pipeline arrangement.
- 25 mm segment roundness tolerance (on radius). This is a requirement for the segmental lining system and was added to ensure that potential loss of space within the tunnel was considered.
- 25 mm line and grade of secondary lining. This tolerance requires that line and grade of the pipelines within the tunnel match those of the finished tunnel.

Based on the sizes of the four pipelines within BT-1, and the above tolerances, it was determined that a minimum 5.08 m tunnel ID was necessary to adequately house the pipelines as designed. This minimum tunnel diameter is specified on the contract drawings. However, since pipeline installation methods are assumed and not specified, and specific pipeline placement within the tunnel is not specified, a specific ID for BT-1 is not specified. The contractor was free to choose a larger tunnel ID if deemed necessary.

As it turns out, the contractor has chosen to build a 5.08 m ID tunnel, the construction of which began in September 2007. As of the writing of this paper, the contractor has not submitted plans for installation of pipes within BT-1; however, discussions have indicated installation will likely be similar to the methods assumed and described here.

Excavation of BT-1 is anticipated to be complete in November 2008 (Figure 7). Pipe 'stuffing' will begin immediately following completion of mining and is expected to last approximately one year.



FIG 7 - Brightwater tunnel one (BT-1) tunnel boring machine (TBM) launch from influent structure (IS) shaft.

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