Modelling of the seawater intake pump station of the Southern Seawater Desalination Plant, Western Australia: benefits and lessons learnt.

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ABSTRACT

As Australia faces a growing population and increasing episodes of drought, the availability of a clean and reliable water resource is an important issue. Water supplied from desalination plants has recently become an option to complement the limited available fresh water resources. In the south west of Western Australia, a second desalination plant was recently commissioned to supply the Perth metropolitan area. The seawater intake pump station is a critical infrastructure component of this desalination plant as it forms a single interface between the main plant facilities and the marine seawater intake and brine outfall pipe network. Despite software limitations using the three dimensional finite element geotechnical analysis software Plaxis 3D (V2.2), the complete soil structure interaction analysis was embarked on to model the complex geometry, top-down construction, excavation and dewatering procedure, buoyancy effects and tunnelling soft eyes and jacking forces. The paper discusses how some of the limitations in the software were addressed to enable the full structural and geotechnical designs to be completed successfully. It allowed a conservative evaluation of the displacements, bending moments, and water pressures, as well as axial and shear forces applied on the walls and slabs. Although only partially able to accommodate refinement of the local geology, as well as changes in the construction sequence and layout, the model was used as an on-going tool for structural design and allowed a better understanding of the behaviour of the complex 3D soil-structure interaction. It highlighted the importance of constructing a robust model from the beginning while anticipating future modifications and refinements leading to greater complexity.

*Keywords: d*iaphragm wall, excavation, pump station, model*,* soil-structure interaction

1 INTRODUCTION

The Southern Seawater Desalination Plant is located on the south western coast of Western Australia, 130 km south of Perth between Binningup and Myalup. It is owned by Water Corporation which is the main provider of water, wastewater and drainage services in Western Australia. Phase 1 of the project was commissioned in late 2011 and is producing 50 GL of potable water per year by seawater reverse osmosis supplied to the Perth metropolitan area. This makes the plant the second major seawater desalination plant in Western Australia and one of the largest plants of this nature in the world.

A critical component of the plant is the SeaWater Intake Pump Station (SWIPS). Located 350 m inland from the high level water mark behind the coastal dune system, the SWIPS is a combined seawater intake and brine outfall structure that forms a single interface between the main plant facilities and the marine seawater intake and brine outfall pipe network.

The Plaxis 3D (Version 2.2) software was used to build a 3-Dimensional (3D) half-model of the SWIPS. The model was used extensively to support both geotechnical and structural design, although it was difficult to adapt the model to the complexity of the structure, the variations in the construction sequence and the incoming geotechnical field data.

2 THE SEAWATER INTAKE PUMP STATION – A CRITICAL INFRASTRUCTURE

The Seawater Intake Pump Station (SWIPS) is a 33.3 m wide by 37.7 m long symmetrical structure comprising three units: two seawater intake chambers and one centrally placed brine outfall chamber. Each of the seawater intake unit includes two sub-compartments or chambers: a Marine Tunnel Chamber and a Pump Room as shown in Figure 1.

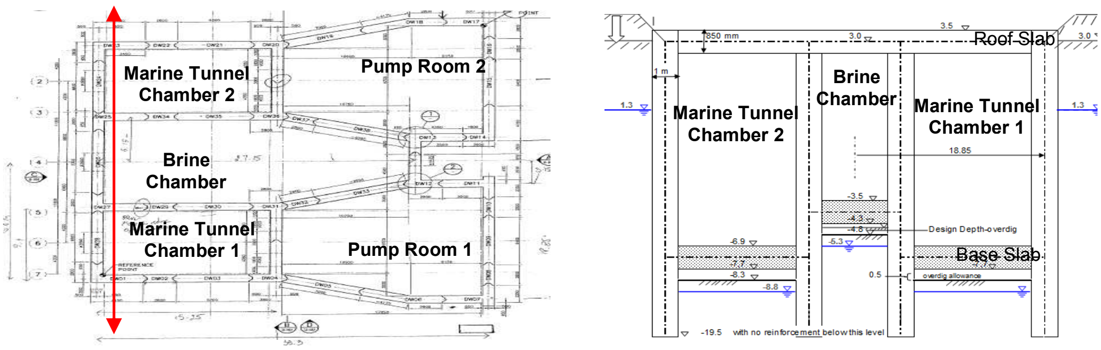

Figure 1. SWIPS diaphragm walls layout (left) and section (right)

Phase 1 comprised the construction of the brine unit and only one of the two intake units for production of 50 GL of potable water per year.

The diaphragm wall (D-wall) technique was selected for construction of the structural walls and the reinforced groundwater cut off walls of the SWIPS which has a design life of 100 years. The design was initially based on 1.0 m thick D-walls with 22.5 m deep structural walls with slurry walls going as deep as 31.0 m to reach a suitable level for water cut off. Internal D-walls were constructed between the pump rooms and tunnel chambers to facilitate early excavation of the tunnelling pit as well as to provide reaction points to distribute the large jacking forces during tunnelling operations.

The maximum design groundwater level is RL 1.3 m or 1.7 m below ground level at its peak. Finished floor levels of approximately 10.0 m and 7.0 m below ground level for the intake units and the brine unit respectively resulted in significant excavation; dewatering and uplift pressures (see Figure 1).

3 CONSTRUCTION OF A 3-DIMENSIONAL MODEL

3.1 Benefits

Preliminary design was undertaken using the Plaxis 2D (Version 9.02) finite element software on specific cross-sections across the SWIPS that resembled plane strain conditions. As can be seen in Figure 1 above only short sections of the D-wall could be modelled in this manner. In addition, the openings in the roof slab on both tunnelling and brine chambers had to be sized to lower the Tunnel Boring Machine (TBM) into the chambers. This meant that outstand beams were required to prop the top of the D-walls to enable the top-down construction to proceed. As different stages of the construction were occurring concurrently, the 3D effects of differential excavation and dewatering required careful consideration. The construction programme also dictated that soil be removed from the pump rooms before the large jacking forces were applied to the internal D-walls so passive soil resistance could not be relied on to distribute the large loads. The 3D model assisted in a more realistic approach for modelling of the distribution of these large forces constrained by the weak D-wall joints. It also allowed extraction of displacements, bending moments, water pressures, axial and shear forces at any stage and any location of the SWIPS for use in the structural design.

3.2 Geological Profile

Preliminary information was limited to one borehole across the SWIPS. During detailed design, five additional boreholes were drilled and the soil profile refined.

Typically, the geological profile comprised:

- A variable thickness of Safety Bay Sand from the surficial dune system;
- A thin band (approximately 1 m thick) of lagoonal sediments, generally comprising dark grey to black sandy silt;
- The Spearwood Dune System, which comprises Tamala Sand, overlying variably cemented calcarenite of the Tamala Limestone; and
- The Leederville Formation, comprising generally sand, sandstone, siltstone and shale. It is noted with an interbedded sequence of Leederville sand and Leederville sandstone. The Leederville Sandstone was initially assumed to be continuous across

the footprint providing a seal layer that allows water cut off. Later investigation using percussion drilling proved this layer to be discontinuous lenses over short distances beneath the structure. This resulted in removal of the slurry wall and detailed piping failure checks to ensure passive resistance was not lost during de-watering.

3.3 Constitutive Models and Modelling Parameters

The constitutive models and their associated critical parameters used in the analysis are presented below.

Table 1: Summary of Models and Geotechnical Parameters used in the Analysis

Model	Type	Geological Units	Critical Modelling Parameters
Mohr Coulomb	Drained	Lagoonal Sediments Cemented Tamala Limestone	Young's Modulus (kN/m^2) Poisson's Ratio v Cohesion c (kPa) Friction Angle Φ (°) Dilatancy Angle (°) Unit weight (kN/m^3)
Mohr Coulomb	Non Porous	Leederville Sandstone	
Elasto-Plastic Hardening Soil	Drained	Safety Bay Sand Variably Cemented Tamala Limestone Tamala Sand Leederville Sand	Cohesion c (kPa) Friction Angle Φ (°) Dilatancy Angle (°) Unit weight (kN/m^3) Secant stiffness E^{ref}_{50} Tangent Stiffness E^{ref}_{oed} Unloading/reloading Stiffness E^{ref}_{ur} Reference Stress for Stiffness p^{ref} (kN/m^2) Permeability k_x, k_y and k_z (m/day)

The drained behaviour was input to take into consideration the long term interaction of the soil against the structure. The Elasto-Plastic Hardening Soil model allowing the increase of soil stiffness's with pressure, understanding of the initial condition and history of the site was fundamental.

3.4 Construction Sequence

3.4.1 General

The construction schedule required the tunnelling for the intake pipe to start as soon as possible. Hence the first marine chamber into which the Tunnel Boring Machine (TBM) was to be lowered had to be excavated in priority over the pump room and the brine chambers at the SWIPS. The voided roof slabs were then cast immediately in the same order. Each chamber was dewatered and excavated following which the base slab was cast to prop and seal the chamber. Dewatering was stopped once the base slab had cured. Tunnelling started immediately.

Although the construction sequence was regularly subject to change in response to the results obtained from the model and also to accommodate priorities in the construction as they evolved, the typical sequence modelled for each chamber can be summarised as follows:

- Installation of the D-walls;
- Casting of voided roof slab with a 20 kPa load (slab had holes large to drop in TBM);
- Excavation and dewatering to the defined base slab level;
- Casting of base slab with a load of 10 kPa for lighter equipment and machinery;
- Start of tunnelling (at the Marine Tunnel chamber); and
- Stop dewatering allowing uplift pressure on structure and floor slabs.

3.4.2 Installation of the Diaphragm Walls - Pinned and Fixed Joints

Location of Pinned and Fixed Joints

As with all D-walls construction, the alternating construction of panels results in vertical joints between panels that can transmit shear, axial force but not moment. Modelling of the joints to allow shear but not moment was one of the main challenges in modelling the SWIPS as the version 2.2 of Plaxis 3D did not allow connections between adjacent panels to be pinned. While the connection between the roof plates was considered fixed, it was conservatively modelled as pinned at the following locations:

- The vertical joints between the D-walls panels, named “Shear Keys” (Modelling of the shear keys is detailed below);
- A 0.15 m vertical section at the top of the D-walls across the whole SWIPS; and
- The outer 500 mm (outside ring) of all floor slabs.

Modelling of the Panel Joints (Shear Keys)

The SWIPS consisted of a total of 41 panel joints. A water-stop was cast into the D-walls panel joints to achieve a watertight seal between the panels. The joints, referred as “shear keys”, determined the behaviour of the D-walls as they provide a preferential line for shear with no moment being transferred. Shear keys were modelled using a 0.3 m wide plate which is the shortest length possible that allows a fine meshing in the software.

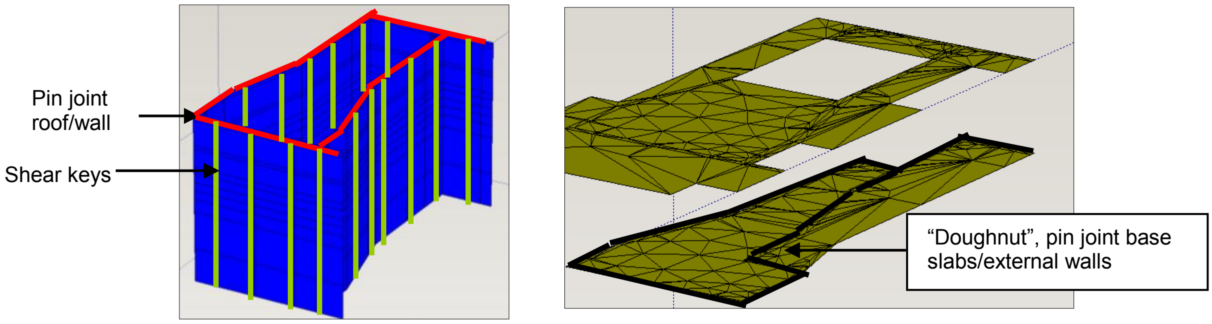

Figure 2. Modelling of Shear Keys within the D-walls and slabs in Plaxis 3D (2.2) – Half SWIPS

According to the relationships between structural forces strain and stresses used in Plaxis 3D, properties for the shear keys were modelled by correcting the input by dividing the thickness by 10, multiplying the Young’s Modulus in all direction by 10, and multiplying all the Shear Modulus by 10. The bending moments become then divided by 100 at the shear keys while all others outputs remain unchanged. This methodology was applied to all the pinned joints in the models of the SWIPS in Plaxis 3D.

3.4.3 Modelling of Tunnelling

The construction of the SWIPS was the first step towards the construction of the two marine seawater intake pipes (only one for Phase 1) with the central brine outfall pipe, the intakes being 3.06 m diameter and the outfall 2.90 m diameter. The TBM was lowered down the Marine Tunnel chamber and started tunnelling by boring through a “soft eye” area of the D-wall panel with glass fibre reinforcement. This results in a re-distribution of the forces across the surrounding panels.

The soft eyes were modelled square for more simplicity and necessity due to software limitations, by two 3.06 m and 2.90 m part of D-wall associated with two horizontal work planes in Plaxis corresponding to the top and bottom of the soft eyes. A thin 50 mm thick plate was installed at the position of the soft eye to avoid the need to model the full tunnel and lining.

Tunnelling jacking forces could potentially generate up to of 12,000 kN of lateral thrust. Four jacks distributed the load (i.e. 3,000 kN each) on a thrust block constructed against the internal D-wall. Then, ten diagonal steel props on the other side of the wall (in the pump room) were used to temporarily support the load.

As diagonal props could not be modelled in the version of Plaxis used at the time, modelling of the forces in the diagonal props resulting from the horizontal jacking forces applied on the opposite side of the D-wall was solved by an association of springs on the D-wall panel and diagonal forces on the base slab. The load to apply for each of the prop depended of the 3D behaviour of the structure and was determined by a preliminary run assuming that 50% of the total load was transferred to the wall (6,000 kN) and the other 50% through the D-wall into the base of the structure.

4 LIMITATIONS AND LESSONS LEARNT

4.1 Evolution of the SWIPS Model

As the project was progressing, the construction sequence and SWIPS layout was regularly revised by both the client and the contractors to suit schedule, improve costs and in response to the results of the geotechnical and structural models.

As such, the following changes and variations were analysed:

- Order and levels of excavation of the different chambers;
- Depth of the diaphragm walls;
- Props and anchors options;
- Presence of a crane beam at the Marine Tunnel chamber;
- Slab thicknesses;
- Location of openings in the roof slabs;
- Loading options on the SWIPS roof and bases slabs;
- Fixity / Pinning of the diaphragm walls / panels;
- Addition of barrettes to support the roof slab;
- Application of pipe work forces to the D-wall panels; and
- Replacement of the capping beam / slab on one side with anchors due to late casting of the roof slab.

All these variations resulted in adding elements to the model, updating work plans and revising the model geometry which involved significant modelling and running time. Indeed, the 3D complexity of the geometry of the SWIPS resulted in issues with meshing, re-definition of all the construction sequence stages (for each element and cluster) and up to 15 hours calculation. Significant time was lost if any of the stages was not input properly or an element forgotten. In summary, running a 3D model of this size is very time consuming and it should only be used when it is absolutely necessary as was the case here.

4.2 Comparison of Plaxis 2D and 3D Models

The comparison was made using the Plaxis 2D and Plaxis 3D models at the location of the marine tunnel to verify the design and also to be able to used 2-Dimensional modelling as much as possible as it is quicker to run. The same structural, geotechnical profile and parameters as well as the groundwater were used in both models.

The Plaxis 3D model only allowed hydrostatic or interpolated water pressure distributions whereas the Plaxis 2D model was able to perform steady state flow calculations. Hydrostatic calculation in the 2D analysis results in an overestimation of the differential pressures on the wall if flow was possible around the toe of the wall as was first assumed when the sandstone layer was assumed continuous across the entire base of the structure. Subsequent percussion drilling provided the sandstone discontinuous and flow calculations and piping checks were then implemented. The Leederville sandstone layers were thus replaced by Leederville sand (20 m/day permeability as assumed in Plaxis 3D) and a steady state flow calculation used to obtain the water pressures on the wall.

The similarities in the results of the 3D Plaxis and the 2D Plaxis models at locations where the walls were effectively one way spanning confirmed the 3D model was performing adequately.

4.3 Refinement of the Geology

More geotechnical data became available during the design, comprising five drilled boreholes, four cone penetration tests and percussion drilling to verify the extent of the Leederville sandstone layer. Review of the soil profile at the SWIPS indicated a thicker layer of Safety Bay Sand overlying the Lagoonal Deposits. The succession of layers of Tamala Sand and variably cemented Tamala Limestone was then shifted deeper with the Leederville Sand observed 4.0 m deeper than the initial borehole 25m away indicated.

The effect of the changes on the profile was determined on Plaxis 2D models in order to avoid unnecessary 3D interactions in the Plaxis 3D model that would complicate the analysis and for time efficiency. A sensitivity analysis was also run on the effect of changes in the permeability of the Leederville sand (1, 5 and 10 m/day).

Table 2: Comparison between initial and revised soil profiles

Permeability Leederville Sand (m/day)	Pore Pressures* (kPa)		Maximum Displacements (mm)		Maximum Bending Moments (kN.m/m)	
Soil Profile	Initial	Revised	Initial	Revised	Initial	Revised
1	165	140	19.5	23.0	1450	1650
5	165	145	19.8	25.0	1450	1750
10	165	150	20.0	28.0	1450	1900

*Pore pressures values inside the SWIPS at RL19.0m in the Leederville Sand for both profiles.

As shown in Table 2, it was found that at low permeability the updated geotechnical data had a smaller effect on the displacements and bending moments. However, at high permeability, the sensitivity analysis indicated that permeability was critical for the design as passive pressure loss significantly affected the bending moments and deflections of the wall. In addition, lack of continuity of the sandstone layer was accounted for by modelling the layer as a dense permeable sand.

4.4 Monitoring Observations

Eight points were monitored regularly during excavation of the marine tunnel chamber at two different depths. The total maximum displacements obtained ranged from 1.7 to 2.5mm where the Plaxis 3D model showed values of approximately 6 to 8mm. It should be noted that the first measurements were taken when the chamber was half excavated. Theoretical values extracted from Plaxis 3D considered the deflection between no excavation and a full excavation. Therefore, it is difficult to make a direct comparison between the factual and predicted values. Even if the range of displacements is coherent with the results of the monitoring, this highlight the importance that monitoring has to be done appropriately in order to use the results.

5 CONCLUSIONS

The modelling of the SWIPS was a significant exercise that allowed a good understanding of the behaviour of the structure at each stage of its construction. It highlighted the sensitivity of the SWIPS to the soil profile and particularly to the presence of the Leederville sand layer and its permeability. The assumptions made to get through 3D version 2.2 software limitations, such as modelling of the joints and the tunnelling, appeared to be successful even if results of the monitoring program could not be directly compared to the prediction.

The Plaxis models, particularly the 3D version 2.2 although being time consuming, were regularly revised as new information was made available (ground conditions, modification in the construction sequence, etc.). These models provided the structural designer’s stage construction input for the structural design to proceed successfully as borne out by the safe and timely completion of the construction.

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