Stormwater tank construction at Blackpool

N. Wharmby & B. Kiernan
Bachy Soletanche Ltd, Lancashire UK
L. Duffy
Bechtel Water Technologies Ltd, Warrington UK
D. Puller
WS Atkins Consultants Ltd, Epsom UK

Abstract

The North West Water Bathing Waters Programme II, valued at approx. £100m, includes a number of co-ordinated contracts targeting coastal water improvements along the Fylde coast. The purpose of the programme in the Blackpool area is to reduce the volume of storm water effluent entering the bathing waters by limiting the number of discharges to an average of three per season per outfall.

Discharges will be limited by the inclusion of 90,000m³ of storage tank capacity in the wastewater network. The storage capacity serves to limit peak flows through the treatment works to design levels. During storms the effluent is discharged into the tanks by gravity. When the flows in the wastewater network subside the storm water is pumped from the tanks back into the network for subsequent treatment.

To provide a 60,000m³ storage capacity at the Blackpool site two 36m internal diameter tanks extending to a depth of 40m were constructed under Bloomfield Road car park. These tanks were formed using 1.5m thick and 45m deep diaphragm walls excavated through very soft organic alluvial deposits, Glacial Till and Mercia Mudstone using both conventional grabs and Hydrofraise cutters.

The paper covers the design development and critical design and construction aspects with particular regard to the prevailing ground conditions, construction tolerances, slab connection and interaction with the diaphragm wall, stability during excavation and long-term buoyancy.

Background

In response to an EU Bathing Water Directive, North West Water (NWW) launched an ambitious clean – up initiative titled ‘Sea Change’, a £500million programme of work aimed at improving the quality of discharges to the North West coastal waters. The objective was to assist the Environment Agency (EA) in delivering the necessary bathing waters quality standards. Wide ranges of projects were constructed under ‘Sea Change’ along the Fylde coast. This work included:

- A 14km interceptor tunnel along the seafront from Blackpool’s Central Pier to Fleetwood to collect and transport flows to Fleetwood WwTW
- A new WwTW at Fleetwood, capable of treating 2200ls⁻¹
- An additional stage of treatment at the Clifton Marsh WwTW, which serves the Preston and South Fylde areas

Although each scheme was completed on time in 1996 and achieved its requirements in terms of discharge consents and spill frequency, the resulting improvements to the bathing waters were not sufficient to meet the required quality standard. As a result of the bathing water quality failures in 1997, NWW working under the direction of the EA, undertook a fast-track programme of further investment. The main thrust of these projects was to further reduce storm discharges to an average of three per Bathing Season per outfall and to provide further treatment at existing WwTWs.

This programme of work commenced in March 1998, with a tight deadline of completion by the Bathing Season of 1999.
Modelling of the Wastewater Network

In order to reduce storm discharges to an acceptable level, the need to construct additional storage within the wastewater network was established. To determine how much storage and to optimise its location, sophisticated and detailed modelling work was undertaken of the network by Bechtel, NWW’s Engineering Services provider.

An extensive flow survey and data acquisition exercise was carried out. The latest modelling software (Simpol continuous simulation version, Hydroworks RTC and Simulink) were used to create a model of the network. Historical rainfall data was used to simulate a long-term, continuous period of rainfall. An iteration process was then performed to determine the additional storage required, and at which location, to limit the required average 3 spills per bathing season per outfall.

The existing tunnel and wastewater network had a storage capacity of 80,000m$^3$ and 20,000m$^3$ respectively. For the Fylde Coast Scheme, an additional 90,000m$^3$ of storage was identified to achieve the 3 spills per bathing season criterion.

Storage Location

As initial estimates for the additional storage volumes became apparent, concerns relating to the suitable locations for the storage arose. It was necessary to locate 60,000m$^3$ in the centre of Blackpool, in a heavily populated urban environment. The remaining 30,000m$^3$ could be located at the new Fleetwood WwTW.

A number of potential options and configurations were explored. The final choice of location was based on many factors, principally construction costs, impact on the local community, controlability for a given partition of storage at different sites, available access for construction, ease of connection to the existing sewer system and underlying ground conditions.

The extensive surface car parking area at Bloomfield Road in Blackpool was selected as the most appropriate location for the 60,000m$^3$ Blackpool tanks. The final outline of the overall Fylde Coast Scheme is as shown in Fig. 1.

Site Investigation

In view of the extremely tight timescale for the project site investigations were commenced for the Blackpool tanks in April 1998, in tandem with the modelling work. A series of 17 boreholes were undertaken for the tanks and associated works to allow connection to the existing wastewater network to be made. The boreholes varied in depth from 15m to 44m and were bored using cable tool percussive techniques at casing sizes commencing at 300mm to 150mm diameter. Eight boreholes were extended to depths of up to 98m by rotary coring at SWF, PWF and 412 size, using polymer mud in brine as flushing medium. Geophysical logging was carried out within the rock at four borehole locations using calliper, temperature, natural gamma and acoustic logging techniques.
Ground Conditions

The geological profile (ref. Table 1) from published geological maps of the area and associated memoir was confirmed by the ground investigation although the Lower Boulder Clay proved to be absent at the location of the tanks.

<table>
<thead>
<tr>
<th>DRIFT</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Post Glacial Deposits</td>
<td>Made Ground</td>
</tr>
<tr>
<td></td>
<td>Blown Sand</td>
</tr>
<tr>
<td></td>
<td>Peat</td>
</tr>
<tr>
<td></td>
<td>Older Marine and Estuarine Alluvium</td>
</tr>
<tr>
<td>Glacial Deposits</td>
<td>Upper Boulder Clay</td>
</tr>
<tr>
<td></td>
<td>Middle Sand</td>
</tr>
<tr>
<td></td>
<td>Lower Boulder Clay</td>
</tr>
<tr>
<td>SOLID</td>
<td></td>
</tr>
<tr>
<td>Triassic Deposits</td>
<td>Singleton Mudstone of the Mercia Mudstone Group (incorporating a zone of Mythop Salt / Halite)</td>
</tr>
</tbody>
</table>

The key geological features to have a significant impact on the design and construction of the tanks may be summarised as follows:

1. Soft peat deposits were confirmed at shallow depth, with very soft Alluvial Clay extending up to 6.50m below ground level.
2. Upper Boulder Clay comprised stiff to very stiff fissured clay, laminated in parts.
3. The Middle Sands proved to be more extensive, up to 23m thick and comprising medium dense to very dense sand to well graded sand and gravel, with cobbles and boulders with some clay layers present. At depth significant difficulties were experienced in penetrating this deposit, due to its coarse and dense nature.
4. The Singleton Mudstones were located at depths of between 37.8m and 42.7m. These were typically red brown thinly to medium horizontally bedded very silty mudstone, with beds of siltstone and gypsum.
5. The Halite (Rock Salt) was located at depths of between 54.0m and 60.4m and proven to depths of 98.0m with hydrostatic water pressures.
6. Water levels in the Middle Sands, the principle aquifer, were at a depth of 1.7m bgl.

Underground Storage Tank Design

The design & build main contract was awarded to Bachy Soletanche Ltd. The overall concept design for the tanks was driven by the requirement for 60,000m$^3$ of storage, the available plan area and the geology. Numerous options were considered but all required relatively deep excavations that meant the ground water pressures would have a significant influence on the final design adopted both in the temporary and permanent conditions.

During the construction phase, the base of the excavation must be stable. This could have been readily achieved using dewatering. However, this requires the abstraction of ground water and generates ground water flows; this could cause dissolution and the formation of cavities in the Halite. The formation of solution features below the tanks or in the surrounding area was an unacceptable risk.

In the permanent condition with the tanks empty, the large hydrostatic uplift pressures on the base slab have to be resisted. The use of ground anchors was not favoured due to limited depth of Mercia Mudstone and the presence of the Halite and associated corrosive saline ground water. The mass of the structure combined with friction on the shaft walls was required to provide overall stability.

The final overall design was a fine balance of tank diameter and depth to provide the required volume with adequate factors of safety for both temporary and permanent stability. The solution required the construction of two 36.0m internal diameter tanks with the base slab founded on the Mercia Mudstone (see Fig.2). A tunnel connects the two tanks.
Diaphragm Wall Design

The diaphragm wall was required to withstand soil and water pressures to enable the excavation to formation level and facilitate subsequent completion of the tanks. As a circular structure subjected to a relatively uniform loading, the walls are self-supporting provided they are both continuous and of sufficient strength to withstand hoop compression forces (ref. Fig.3).

Based upon the ground conditions, water pressure and maximum depth of excavation, the hoop compression load was calculated to be 20MNm\(^{-1}\); this required a structural wall thickness of 1200mm.

It is essential that the design of the diaphragm wall considered the constructional tolerances achievable particularly as the excavation extended to depths of over 40.0m. The following tolerances were considered in the design:

- Guidewall setting out = +/- 15mm
- Guidewall spacing = + 25mm
- Verticality = 1 in 300 i.e. 133mm offset for each panel over a depth of 40.0m

Based upon the above a 1500mm thick wall was required and the use of conventional grabs over the full depth was not possible due to the verticality tolerance to be achieved. As this is required only towards the base of the excavation consideration was given to the use of internally cast rings as the excavation proceeded, however, the programme was too tight to accommodate such delays in the bulk excavation.
Diaphragm Wall Shaft Interaction Analysis

Due to the close proximity of the tanks and the programme driven requirement to excavate the tanks at different times / rates, the soil loading and stiffness would not be uniform around the tanks as had been assumed in the design (ref. Fig.4). WS Atkins carried out the interaction analysis using FLAC program. FLAC (Fast Lagrangian Analysis of Continua) version 3.3 is a two-dimensional finite difference program particularly suitable for the analysis of soil structure interaction. Materials are represented by elements that form a grid that can be adjusted to the shape of the object modelled. Structures of various geometry and properties can be modelled together with their interaction. The overall objectives of the interaction analysis were:

- To estimate the induced additional movements of the diaphragm walls at Serviceability Limit State.
- To estimate additional structural forces and bending moments in the diaphragm walls at Serviceability and Ultimate Limit States.
- To check the design of the diaphragm wall reinforcement under the additional structural forces and bending moments according to the requirements of BS 8110.

### Geotechnical Parameters for FLAC Analyses

<table>
<thead>
<tr>
<th>Ground conditions</th>
<th>Thickness (m)</th>
<th>Bulk density (kNm⁻³)</th>
<th>Drained Young’s modulus (MPa)</th>
<th>Poisson’s ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Ave</td>
<td>Worst credible</td>
<td>Moderately conservative</td>
</tr>
<tr>
<td>Made ground</td>
<td>0.8 - 1.8</td>
<td>1.2</td>
<td>17</td>
<td>3</td>
</tr>
<tr>
<td>Peat</td>
<td>0.9 - 2.3</td>
<td>1.7</td>
<td>12</td>
<td>0.2</td>
</tr>
<tr>
<td>Soft clay</td>
<td>1.6 - 3.4</td>
<td>2.4</td>
<td>19</td>
<td>3</td>
</tr>
<tr>
<td>Glacial Till (firm to stiff clay)</td>
<td>9.2 - 13.8</td>
<td>10.6</td>
<td>21</td>
<td>21 + 2.3 z¹</td>
</tr>
<tr>
<td>Glacial Till (medium dense to dense sand)</td>
<td>19.7 - 26.8</td>
<td>23.7</td>
<td>21</td>
<td>45 + 1.6 z¹</td>
</tr>
<tr>
<td>Completely to highly weathered Mudstone</td>
<td>0.4 - 9.7</td>
<td>2.3</td>
<td>22</td>
<td>30⁺</td>
</tr>
<tr>
<td>Moderately weathered Siltstone</td>
<td>0.0 - 7.7</td>
<td>2.4</td>
<td>22</td>
<td>1000</td>
</tr>
<tr>
<td>Moderately weathered Mudstone</td>
<td>4.1 - 17.8</td>
<td>12.3</td>
<td>22</td>
<td>1000</td>
</tr>
<tr>
<td>Slightly to moderately weathered Halite</td>
<td>N/A</td>
<td>N/A</td>
<td>22</td>
<td>2000</td>
</tr>
</tbody>
</table>

(¹): z is the depth (m) below top of layer
(²): Below tank base slab

Table 2 : Geotechnical Parameters for FLAC Analyses

**Analytical Approach**

The interaction between the two diaphragm walls was analysed for a section located at a depth of 41.0m with ground water taken at 2.0m below ground level. The geotechnical parameters are given in Table 2. However, for the interaction analysis an average bulk density, \( \gamma \), of 20kNm⁻³ and an effective density, \( \gamma’ \), of 10kNm⁻³ was assumed to determine the total and effective vertical stresses. An earth pressure coefficient, \( K \), equal to 1 was considered for the prediction of the horizontal earth pressures. The following construction sequence was adopted for the analysis:
Stage 1: Initial equilibrium stage
Stage 2: Construction of diaphragm wall for tank 1
Stage 3: Construction of diaphragm wall for tank 2
Stage 4: Excavation of tank 1 to a depth of 40m
Stage 5: Staged excavation of tank 2 to depths of 5m, 10m, 15m, 20m, 25m, 30m, 35m and 42m

Results of the Interaction Analysis

For the Serviceability Limit State, at the end of Stage 4 the diaphragm wall in Tank 1 was found to displace uniformly towards the inside of the tank under the applied nett pressure by approximately 5.9mm. The axial load was found to be constant around the perimeter of the tank and to be approximately equal to 14.1MNm$^{-1}$. This result could be validated using simple theoretical solutions:

Axial load = Net pressure × Tank inside radius, \( a = (836 - 38) \times 18 / 1000 = 14.4\text{MNm}^{-1} \)

The corresponding radial displacement, \( w \), of the diaphragm wall of thickness \( h \) is given by:

\[
w = \frac{(P \times a^2)}{(E \times h)} \quad \text{where} \quad P = \text{applied net radial pressure;}
\]

\[
E = \text{Young’s modulus of Diaphragm Wall}
\]

The calculated value shows good agreement with the value obtained from FLAC. The results of the analysis carried out to model the progressive excavation of Tank 2 are given in table 3.

<table>
<thead>
<tr>
<th>Excavation depth in tank 2</th>
<th>5 m</th>
<th>10 m</th>
<th>15 m</th>
<th>20 m</th>
<th>25 m</th>
<th>30 m</th>
<th>35 m</th>
<th>42 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. disp. (mm)</td>
<td>6.0</td>
<td>6.2</td>
<td>6.4</td>
<td>6.5</td>
<td>6.7</td>
<td>6.9</td>
<td>7.1</td>
<td>7.3</td>
</tr>
<tr>
<td>Min. disp. (mm)</td>
<td>4.3</td>
<td>3.9</td>
<td>3.5</td>
<td>3.1</td>
<td>2.7</td>
<td>2.3</td>
<td>1.9</td>
<td>1.4</td>
</tr>
<tr>
<td>Max. Tangent. Bending Moment (kN.m$^{-1}$)</td>
<td>42</td>
<td>53</td>
<td>71</td>
<td>88</td>
<td>108</td>
<td>126</td>
<td>145</td>
<td>169</td>
</tr>
<tr>
<td>Max. Axial Load (MNm$^{-1}$)</td>
<td>14.0</td>
<td>14.1</td>
<td>14.1</td>
<td>14.1</td>
<td>14.1</td>
<td>14.1</td>
<td>14.1</td>
<td>14.1</td>
</tr>
<tr>
<td>Tank 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. disp. (mm)</td>
<td>2.2</td>
<td>2.8</td>
<td>3.4</td>
<td>4.1</td>
<td>4.8</td>
<td>5.5</td>
<td>6.2</td>
<td>7.2</td>
</tr>
<tr>
<td>Min. disp. (mm)</td>
<td>-2.1</td>
<td>-1.5</td>
<td>-0.9</td>
<td>-0.3</td>
<td>0.4</td>
<td>1.0</td>
<td>1.6</td>
<td>2.5</td>
</tr>
<tr>
<td>Max. Tangent. Bending Moment (kN.m$^{-1}$)</td>
<td>156</td>
<td>156</td>
<td>153</td>
<td>154</td>
<td>154</td>
<td>156</td>
<td>157</td>
<td>160</td>
</tr>
<tr>
<td>Max. Axial Load (MNm$^{-1}$)</td>
<td>1.86</td>
<td>3.63</td>
<td>5.38</td>
<td>7.14</td>
<td>8.90</td>
<td>10.6</td>
<td>12.4</td>
<td>14.8</td>
</tr>
</tbody>
</table>

A negative displacement indicates a movement towards the outside of the tank.

Table 3: Summary of results from Interaction Analyses

For the Ultimate Limit State a partial factor of 1.4 was applied to both earth and water pressures. Under these loading conditions, the maximum axial load in the diaphragm wall, at a depth of 42.0m, was found to be 19.8MNm$^{-1}$ for Tank 1 and 20.8MNm$^{-1}$ for Tank 2. The tangential bending moment induced by the interaction between both tanks was respectively 235kNm$^{-1}$ for Tank 1 and 223kNm$^{-1}$ for Tank 2. Additional vertical and horizontal reinforcement was provided in the affected panels to cater for the additional tangential and longitudinal bending moments in the diaphragm walls.

Base Slab Design and Interaction with the Diaphragm Wall

In the permanent condition the base slab is required to withstand nett water pressures of approx. 400KPa and distribute the resulting total load of 400MN (3.6MNm$^{-1}$ of wall) to the perimeter of the slab and into the diaphragm wall. A number of connection details were considered at the interface between the base slab and the diaphragm walls including the following:
- pinned connection (no structural continuity from slab to wall)
- ‘fully fixed’ connection (via bend-out bars or couplers in the diaphragm wall)
- corbel / bearing connection

The dual considerations of achieving a practical waterproofing detail as well as adequate structural connection at the base slab/wall joint were paramount. Due to the large clear span and the nett water pressure acting on the slab, a so-called fully fixed connection would have resulted in enormous bending moments being transferred into the diaphragm walls. Such high bending moments would have been larger than the moment capacity available from a 1500mm thick wall. Alternatively a pinned connection would have presented waterproofing difficulties and it would have resulted in the greatest mid-span bending moment for the base slab.

Based on the foregoing considerations and the results of some simple calculations obtained from closed form solutions a corbel connection arrangement was adopted. The corbel dimensions were 800mm width by 1500mm height with a 400mm wide bearing zone (see Fig 5). Grade 50N concrete was utilised in order to accommodate the high bearing stresses.

The design and detailing must take account of the achievable construction tolerances. A good example of this being the consideration of the vertical positional tolerance of the couplers embedded in the diaphragm wall based upon:

1. The cage construction accuracy (cage building and splicing / joining).
2. Positional accuracy after concreting.

Based upon the connection detail both the base slab and the effect of the moment applied to the diaphragm wall were further investigated using a FLAC axisymmetric analysis.

**Base Slab Analysis**

The different soil layers were modelled as linear elastic materials with the assumed worst credible (WC) or moderately conservative (MC) parameters listed in Table 2. An adopted wastewater density of 10kNm$^{-3}$ was used. The base slab was modelled as a linear elastic material with the following properties:

- Young’s modulus, $E_{c28} = 31$GPa
- Poisson’s ratio, $\nu = 0.2$
- Density, $\rho = 24$ kNm$^{-3}$

The construction sequence analysed is summarised below:

- **Stage 1:** Initial equilibrium stage
- **Stage 2:** Construction of diaphragm wall, excavation of Tank 1 to final level and construction of corbel and base slab
- **Stage 3:** Application of wastewater and uplift pressures with gap forming between the base slab and corbel/diaphragm wall
To investigate the effect of the connection, sensitivity studies using numerical models considering three different slab end conditions were analysed:

1. Free-ended, i.e. no restraint in any direction
2. “Supported” in the vertical direction with a nominal frictional upward force applied at the perimeter of the base slab and equal to 10% of the downward resultant force
3. “Connected” with diaphragm walls. In model 3, this connection is modelled as a fully bonded link consisting of elasto-plastic elements whose compressive and tensile strengths are 40MPa and 1MPa, respectively. Therefore, model 3 is thought to represent the extreme case.

The available output data from each stage of the FLAC analysis included the distribution of radial and tangential normal stresses ($\sigma_{xx}$ and $\sigma_{zz}$, respectively), distribution of shear stresses ($\tau_{xy}$) and displacements of all the nodes. The structural forces and bending moments in the slab were calculated from the corresponding stresses. The base slab reinforcement was designed and detailed in accordance with the requirements of BS 8110.

In order to achieve structural efficiency a variable thickness slab was selected. However, this presented reinforcement detailing difficulties that were compounded by the sumps, gullies and pits located in the upper surface of the base slab. In order to resist the considerable peak mid span hogging moments a combined grid of tangential and radial reinforcement was provided with up to 3 layers of T40s in each direction on the top face (ref. Fig.6).

**Applied Moment to the Diaphragm Wall**

As a compression structure the diaphragm wall shaft would require little moment capacity apart from that required to cater for constructional tolerances. However, with the high base slab edge loads applied eccentrically via the corbel it was necessary to consider the possibility that the tension could locally occur across the diaphragm wall panel joints.

Based upon the load cases tabulated in Table 4 some tension on the outer face of the diaphragm wall was found to result from the introduction of a tangential bending moment in the diaphragm wall.

<table>
<thead>
<tr>
<th>Load case</th>
<th>Factor applied to Uplift pressure below slab</th>
<th>Factor applied to Water pressure on diaphragm wall</th>
<th>Factor applied to Earth pressure on diaphragm wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.4</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
</tr>
</tbody>
</table>

**Table 4 Ultimate Limit State Load Cases**

The axisymmetric FLAC sensitivity analysis carried out for a single tank with a corbel contact width of 400mm showed that a tensile stress of approximately 0.53MPa resulted at SLS in the outer face of the wall at the top of the base slab. However, the section was found to remain globally in compression with an axial load of around 1.6MNm$^{-1}$. At the top of the corbel, the outer face of the wall was found to be in compression (1MPa).
Diaphragm Wall Details

The presence of up to 6.5m of very soft organic clays coupled with heavy plant operating adjacent to the trench and the need to achieve tight tolerances lead to the use of cement-bentonite barrettes to support the guidewalls and maintain trench stability. The barrettes were arranged parallel to the diaphragm wall and aligned with the inside face; they also served to limit concrete overbreak.

The overall tolerance requirements and prevailing strata meant the use of the Hydrofraise cutter or mill would be needed. With this technique the use of a cut concrete joint can be adopted which is particularly suitable for compression structures as it allows optimum control and inter-panel contact. Based upon this system a three “bite” Primary and alternate single “bite” Secondary panel layout was adopted with a nominal 300mm wide cut area. (Ref. Fig. 7). Reinforcement detailing was carried out to suit.

At the cage detailing stage the weight of the cage and the requirement for lifting and stiffening steel must be considered. In this particular case the reinforcement was nominal except for the lower zone that was connected to the corbel and was subject to the uplift induced moments. The Primary panels were reinforced with two separate cages made up of three sections spliced over the trench as they were lowered in.

Diaphragm Wall Construction

The excavation of the panels was not suited to a single method. By virtue of its operation the Hydrofraise requires an initial excavation and is not ideally suited to high plasticity clays. For this reason rope suspended grabs were used to excavate down to the dense Glacial Sand and the Hydrofraise completed the diaphragm wall panel excavation. To achieve the required verticality tolerance the Hydrofraise machine (Evolution 2) was equipped with instrumentation that enabled 3D real time positional monitoring.

The fundamental advantage of this instrumentation and method of construction is that alignment correction is a continuous operation. With other methods the excavation can be significantly out of line before it is identified and subsequent correction is difficult. With the Hydrofraise alignment correction is achieved by adjusting the individual cutter drum speeds or the angle of the cutter drums themselves as indicated in Fig. 8.
Fig. 9 details the instrumentation used to provide real time monitoring for the experienced operator to work with and be guided by. The system and methods adopted were proven when the shaft was excavated. The actual worst case verticality tolerance measured was 1 in 500 or 80mm at an excavation depth of 40m.

Fig. 9: Hydrofraise “Evolution 2” Real Time Positional Monitoring

Bulk Excavation of the Tanks

Immediately upon completion of the Tank 1 diaphragm wall, bulk excavation for the tank commenced concurrent with the completion of diaphragm walling to Tank 2. Initially due to the large diameter of the tank, conventional excavators and dump trucks were employed for removal of spoil via a ramp constructed as the excavation descended (see Plate 1). The efficiency of this method diminished at approximately 18m below ground level, and excavation continued using two 16t excavators lowered into the tank which then loaded spoil into skips capable of carrying 12t from the excavation each cycle. The skips were hoisted to the surface using hydraulic cranes of 80t capacity and the material was stockpiled on site prior to being carted off site. Using this method to a depth of up to 45m production rates of up to 1000m³ were achieved each day out of each tank.

The quality of the diaphragm wall construction was evident as the bulk excavation proceeded and demonstrated by virtually no overbreak, only minor damp patches at joints and in particular the impressive verticality recorded as 1:1000 on the majority of panels.

As the excavation proceeded through the non-cohesive Glacial Till there was evidence that water was entering both of the tanks from an unknown source. The probability of a defect in the diaphragm wall below the excavation level was considered low based upon both the records and the observed quality of construction. The source was eventually identified as being from abandoned site investigation boreholes centrally located in each shaft that extended down through the Mercia Mudstone layers into the underlying Halite layer over 54m below ground level. A re-appraisal of the risks associated with both the construction and operation of the tanks resulted in some key changes to the contract. There were two significant problems identified:
1. How to safely and expeditiously construct the works with potentially high inflows of water 45m underground?
2. Was there dissolution of the Halite taking place that could lead to instability immediately under the tanks or further afield?

Initially it was considered prudent to make attempts to seal the source of the leaks. In Tank 1, the borehole could be located whilst the excavation was still at a relatively high level in Glacial Till. An over-drilling, casing and pressure grouting solution sealed the borehole. However, in Tank 2 the source was not identifiable until the excavation had reached the Mudstone layers and similar attempts were fruitless. The integrity of the Mudstone forming a plug above the Halite was critical to the stability of the excavation; disturbance whether by drilling and or grouting was a concern.

Plate 1: Bulk Excavation of the Shafts

The conclusion reached in Tank 2 was that the remaining works would have to be constructed around the water ingress problem, and subsequently seal the ingress once the tank was in a stable condition. This was carried out by installing drainage systems below base slab level and suction pumping from chambers as the slab was being constructed. In addition to pumping pipe work, further valves and pipes were accommodated within the densely reinforced base slab to enable the delicate operation of sealing the leak from the Halite. The fears over the potential dissolution of the Halite added a further urgency to the situation and acceleration measures were employed.

It was an essential part of these revised works to carry out detailed monitoring of the situation and formulate detailed risk management plans. Risk control and mitigation measures were the focus of the design and construction teams especially as the flow rates entering the tank over the period of works fluctuated from a daily average of 18ls$^{-1}$ to in excess of 100ls$^{-1}$ for short periods of time. Pumping systems were put in place, with standby and backup pumps, to deal with the varied flow rates and thus ensure that the base slab could be constructed “in the dry”. As part of the acceleration measures, the main part of tank base slab was cast in one continuous pour lasting 36 hours.
Once the base slab and corbel had been successfully cast and cured, the decision was taken to turn off the suction pumps, close the valve arrangement in the base slab and allow full water pressure to be taken on the underside of the base slab in a controlled manner. Due to concerns over the integrity of both the Mudstone and the Glacial Till, and the reliance of the diaphragm wall design on skin friction, it was considered prudent to monitor Tank 2 for any signs of uplift. Using Engineers from Soldata Ltd, the “Cyclops” automatic real time monitoring system was installed and the tank monitored; this confirmed negligible tank movements.

Further to risk studies it became evident that the tunnel connection between Tanks 1 and 2 would have to be re assessed. In view of the tight project deadlines, the safest and most practical solution in this instance was to instigate ground freezing using nitrogen (ref.Plate2). This 1.8m diameter tunnel had lain in the horizon between the Mudstone and the Glacial Till so it was therefore decided to carry out a redesign to enable tunnel to be excavated purely in the gravels due to concerns over the Mudstone.

Finally it was important to establish what if any effect there had been on the integrity of the Halite and whether dissolution had occurred. Boreholes up to 100m deep were constructed around the tank and a series of geophysical investigations carried out at various horizons. After analysis it was concluded that there was not a significant risk of voids below the structure and therefore the tanks could be safely commissioned.

Concurrent with the works 45m below ground level, works to the roof slab were carried out. This comprised 2No. precast concrete box beams 27m long and weighing 85t with 21No. precast concrete “T” beams spanning between and a cast in-situ deck using permanent GRP formwork. As the car park was to be reinstated, the roof was designed to Highways standards.

**Conclusion**

The project had an extremely tight programme from the initial flow modelling through to its final commissioning and required the construction of the largest underground storage tanks of their type in the United Kingdom. The construction was carried out in an urban environment and the design and construction team faced many challenges but in the spirit of co-operation managed to bring the project in on time and contribute significantly to the beach at Blackpool obtaining the bathing waters “Blue Flag”.

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Plate 2: Connection Tunnel

[Image of tunnel and construction site]