The British Dam Society

Springwell Service Reservoir, managing and effectively mitigating ground risks in design and construction.

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SYNOPSIS Northumbrian Water Group appointed Mott MacDonald Bentley to design and construct a new 43ML service reservoir in Springwell, Gateshead to increase network capacity and resilience. The structure is 100m by 75m by 9m deep adopting a semi-precast concrete solution. It is founded largely on competent incompressible sandstone however, the southernmost third will encroach over significantly more compressible weathered rock. This presents a risk of early and long-term differential settlement that could impact reservoir structural integrity and potential safety if not adequately managed in design and construction.

Concept and Definition design by WSP involved extensive intrusive ground investigation work. Modern 3D digital geotechnical design tools (Leapfrog Works and Settle 3) have allowed designers to fully predict both immediate and future settlements of the structure. Initial assessments, based on first interpretation of borehole data, were beyond tolerable limits for practical and sustainable design of the structure requiring either alternative foundation solutions or ground improvement. To mitigate this risk a simple surcharging solution comprising temporary construction of an 8m high monitored surcharge bund, formed from site won materials, represented the most cost effective and sustainable solution.

Discussed are the geotechnical design processes and outputs through key design and construction phases: development of a representative 3D ground model; iterative 3D settlement analyses in collaboration with structural designers; design, implementation and monitoring of surcharging; back analysis of surcharge monitoring data to establish representative ground stiffness parameters for structural design; and validation of assumptions during construction.

INTRODUCTION

Northumbrian Water ('the client') identified a need for a new 43ML service reservoir (SR) to increase wholesome water supply network resilience in the South Tyneside area. The proposed site in Springwell, near Gateshead, Northumberland comprised a open grassed sloping field with an approximate 1 in 10 fall from north to south. Initial optioneering, outline design and early investigations and surveys were undertaken by WSP ('concept designer'). Following a competitive tendering process Mott MacDonald Bentley ('contractor') was appointed to undertake detailed design and construction of the project.

The contractor elected to design and construct a semi-precast concrete structure to allow construction completion within a very constrained delivery programme. Detailed structural design and supply of the structure was undertaken by FLI Precast Solutions ('sub-contractor'). Detailed design commenced in early 2023 with site construction commencing in May 2023.

The structure required a substantial temporary excavation of the sloping site. On completion the structure is to be fully landscaped to reduce visual impact on the local community. The initial investigation was up to 11m deep at its deepest to the north and was largely within competent Sandstone bedrock. The southern most third of the excavation however was within largely weathered Sandstone generally recovered as a residual Sand and Gravel.

The weathered extent of the formation strata presented distinct geotechnical design challenges due to the distinct relative differences in formation stiffness across the structure footprint. Namely, the greatest risk to the structure was short- and long-term differential settlement. This paper discusses the design approach by the contractor in close collaboration with the sub-contractor alongside the success of solutions implemented to reduce risks to tolerable structural design limits.

PROPOSED SERVICE RESERVOIR STRUCTURE AND GROUND LOADING

Form of Structure

The SR was to be a semi pre-cast DfMA solution comprising a combination of precast wall units, columns and roof beams with wall infills, base slab, and roof screed cast in-situ. Learnings from the contractor's previous experiences of similar structures were taken account of throughout this project (Aujla et al, 2021). The SR is split into two compartments (east and west) and has a total storage capacity of 43ML of wholesome water ready for customer supply. The key parameters of each compartment are listed in Table 1 below.

Table 1. SR Compartment Parameters

Parameter		Parameter	
Compartment Size (Internal)	49.0 x 72.5m	**Top of Roof Level**	141.70 – 140.98mAOD
Height of Wall Panels	8.75m	**Formation Level**	133.22 – 132.50mAOD
Internal Clear Height of SR	8.2m	**Base / Roof Slope**	1 in 100
No. of Compartments	2	**Top Water Level (TWL)**	139.00mAOD

Due to the method of construction adopted for the structure it inherently has a significant sensitivity to settlement and more critically differential settlement.

Structural Loads

Early initial structural design established the following loads would be applied to the ground during key loading stages through initial construction, testing and completion phases of the project:

- Wall loading following initial wall panel placement = 56kPa.
- General loading with SR cells full under leak testing = 100kPa
- Maximum finished loading to SR perimeter on completion of landscaping = 175kPa.

GROUND MODEL

Three phases of ground investigation (GI) were commissioned by the concept designer during outline design. The first phase of GI provided a broad understanding of the site's geology and identified a potential for Mudstone with a higher degree of weathering in the SW corner of the site. Two further phases of GI followed to focus on the weathering profile over the southern third of the SR. The difference in weathering profile is attributed to the site's natural topography and the relative difference in depth of excavation required to achieve the formation level for the SR.

A detailed 3D ground model was developed in detailed design based on all the available GI data across the site. The volume of available input data gave confidence that the ground model would be representative and reliable. Interrogation of the ground model identified that the SR formation strata would approximately comprise competent Sandstone over the northern most two thirds and largely Sandstone weathered to a residual Sand and Gravel to the southern third. Figure 1 illustrates a horizontal section cut at the structure formation level showing the general transition from competent to weathered rock with contours illustrating the general thickening of weathering to the South.

A band of weathered Mudstone (identified as a residual clay on borehole logs) was identified to underly both the weathered Sandstone and a thin band of competent Sandstone in the SW corner. It is considered that the Mudstone encountered that was logged as a residual Clay was significantly influenced by drilling with water flush opposed to natural in-situ weathering processes. This is interpretation is explored further in subsequent sections of this paper.

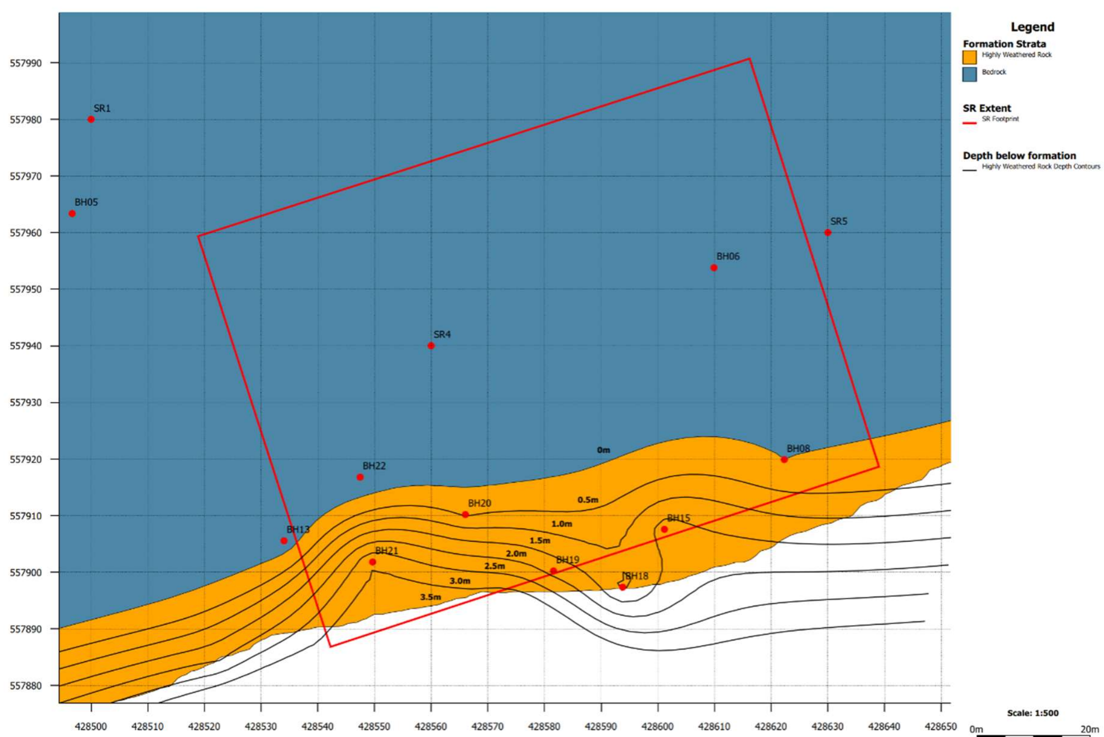

Figure 1: Formation strata; distribution of competent and weathered rock alongside interpreted thickness of weathered Sandstone

Table 2 summarises the initial geotechnical design parameters derived from the available in-situ and laboratory testing available from the various phases GI undertaken.

Table 2: Geotechnical Parameters

Geological Stratum	Geotechnical Parameter	Characteristic Design Value
Weathered Mudstone Firm to very stiff CLAY with low cobble content**.**	Unit weight, γ (kN/m^3)	18
	Drained Modulus of Elasticity (kPa)	10,000
	Coefficient of Volume Compressibility (m^2/MN)	0.10
Weathered Sandstone Medium dense SAND & GRAVEL.	Unit weight, γ (kN/m^3)	19
	Drained Modulus of Elasticity (kPa)	20,000
Mudstone Very weak to weak MUDSTONE.	Unit weight, γ (kN/m^3)	23
	Intact Rock – Young's Modulus (GPa)	1.7
	Rock Mass – Young's Modulus (kPa)	340,000
Sandstone Weak to medium strong SANDSTONE.	Unit weight, γ (kN/m^3)	23
	Intact Rock - Young's Modulus (GPa)	5
	Rock Mass – Young's Modulus (GPa)	1

INITIAL SETTLEMENT ANALYSES

Hand Calculations

Hand calculations were first undertaken to gain a basic understanding of the potential total and differential settlement of the SR. It was anticipated that settlement of the structure founded directly over competent rock would be minimal; however, settlement over weathered strata could exceed 55mm. Such potential differential settlement was generally considered intolerable for a semi-precast structure that was required to be watertight with a limiting crack width of 0.2mm. Due to the criticality of differential settlement more complex 3D settlement analyses were undertaken utilising Settle3 settlement design software.

3D Settlement Analyses

The 3D ground model developed was transposed into specialist 3D settlement analysis software adopting geotechnical design parameters as summarised in Table 2. This facilitated more complex and critical analyses of potential settlements across the structure based on the variability of the underlying ground conditions. This approach allowed the soil-structure interaction to be iteratively assessed. The approach established a representative worst credible output for which any appropriate mitigation measures that may be required could be considered.

Three key loading stages through to asset in service were established for analyses:

- Initial loading from precast wall units when placed on setting-out strips (temporary foundations).
- First filling of reservoir cells during water testing.
- Construction of landscape fill with reservoir fully loaded and in service.

Initial analyses indicated that settlements over the competent Sandstone could be in the order of 1-5mm whilst over the weathered rock to the southernmost extent of the structure could exceed 50mm (Figure 2). Given the very defined zone of weathered rock to the south, this would result in a very concentrated change of deflection and settlement. The potentially sudden transition may result in an abrupt angular distortion of settlement within the structure that was intolerable for structural design. This therefore required a different foundation solution, or ground improvement was required.

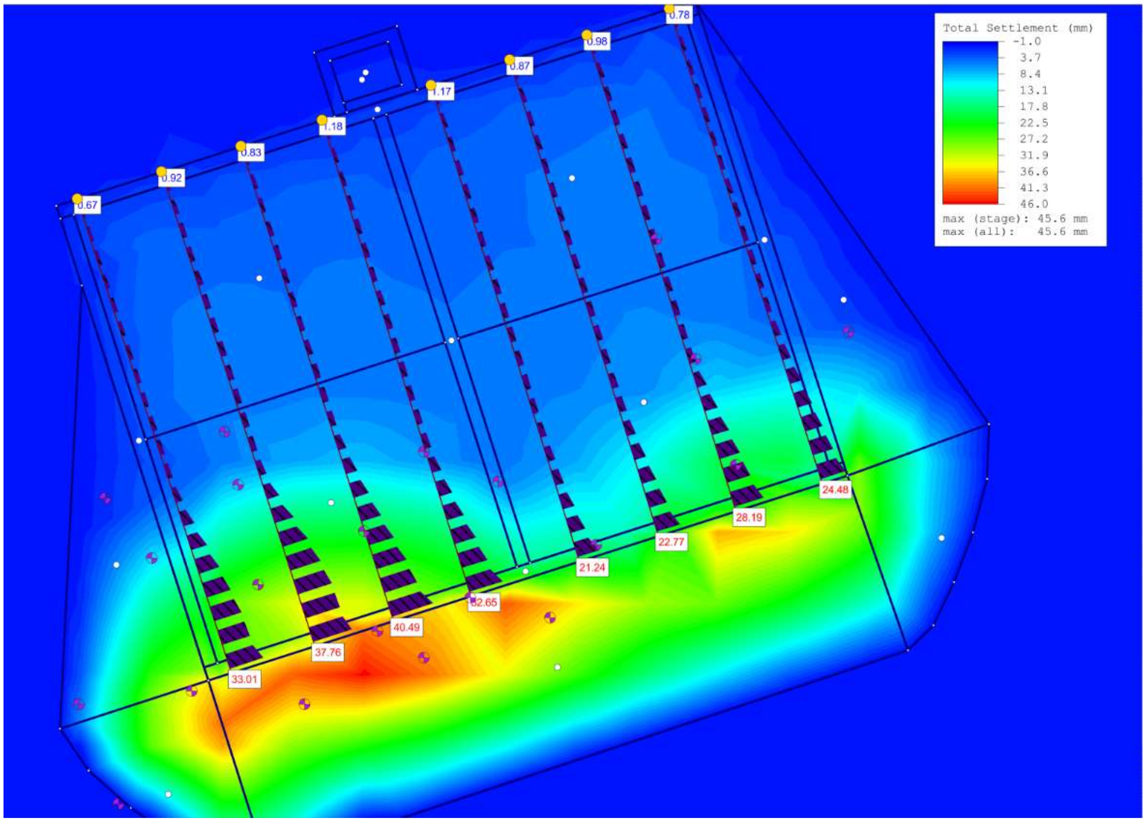

Figure 2: Settle3 preliminary settlement assessment

SETTLEMENT MITIGATION OPTIONS

Options to mitigate potentially excessive settlements included: a) excavate and replace with known compacted fill to competent rock; b) excavate and replace with mass fill concrete; c) piled foundations or d) ground improvement.

Of the options considered an opportunity was identified in the construction programme allowing a simple surcharging solution negating the need for alternative deep soil improvement techniques. A surcharging solution was pursued with the advantage this also returned the lowest embodied carbon option of those under consideration; the solution was implemented utilising freely available site won arisings.

GROUND IMPROVEMENT BY SURCHARGING

Surcharge requirements by analysis

On site there was a significant volume of available arisings to be excavated to achieve formation level of the structure, this meant there was an abundance of excess material available to consider a simple surcharging option. It was established that following initial excavation works there was a three-month period, prior to first delivery of structural elements, to facilitate surcharging of the site.

The established 3D settlement model was used to design the extent and size of surcharge bund required to reduce future ground settlements to a tolerable level for the structure. The design compared variations in the required surcharge bund height versus available surcharge timescales. It was established that application of a surcharge load of 144kPa (equating to an equivalent bund height of 8m) for a period of three months would induce a similar magnitude of settlement to that of the permanent in-service structure over its design life. Figure 3 illustrates predicted settlements resultant from the surcharge bund over the southernmost extent of the structure with settlement in the range of 29-50mm. Surcharging would remove a significant proportion of the likely settlement prior to construction of the SR. The magnitude of potential differential settlement would be reduced to tolerable structural design limits and minimise structural reinforcement requirements.

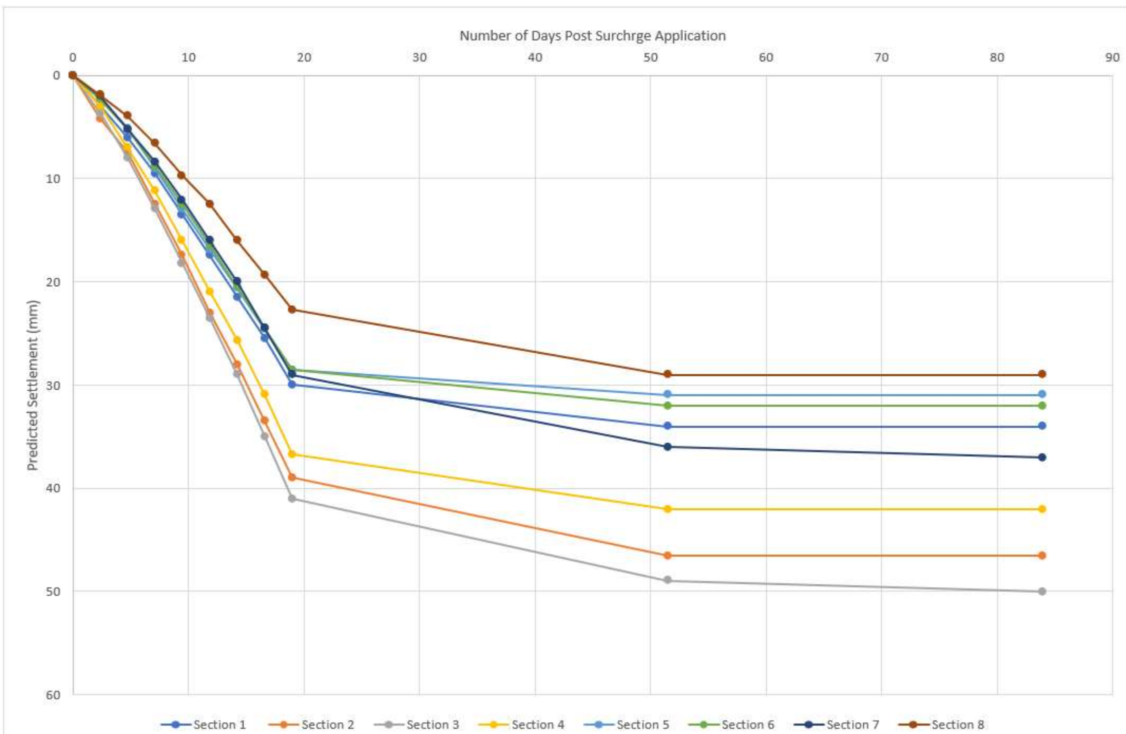

Figure 3: Expected Settlement Induced from Surcharge Bund (Section 1 = West, Section 8 = East)

Surcharge Bund Construction

The surcharge bund was constructed using as dug material comprising a combination of Glacial Till and weathered Sandstone to a height of 8m (Figure 4). The top of the surcharge bund extended 16m into the SR footprint on the western wall and 2m on the eastern wall to apply loading to the full extent of weathered strata. The approximate extent of the surcharge bund is shown in blue in Figure 5.

Figure 4 Constructed surcharge bund

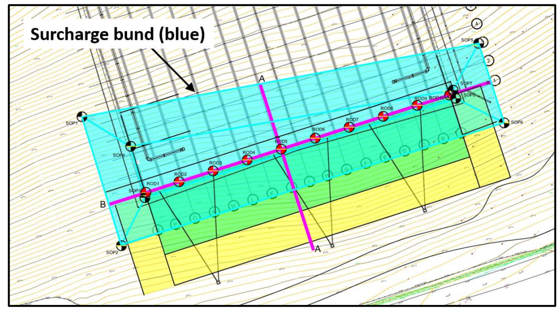

Figure 5: Extent of surcharge bund

Settlement Monitoring of Surcharge Bund

To facilitate settlement monitoring isolated to the underside of the surcharge bund, excluding potential consolidation settlement within the bund itself, eight rod settlements gauges (RSGs) were installed prior to bund construction. The RSGs were aligned with the southern wall of the structure (Figure 5). The RSGs comprised a $300mm^2$ base plate, 1m steel vertical extension rods and a protective plastic surround to isolate monitored movement to the base plate. RSGs were embedded in a fine sand surround to mitigate potential for disturbance during construction. The sand surround measure, however, was of limited success on this occasion and loss of verticality was observed, largely due to the significant plant size adopted during construction; this is discussed further below.

During bund construction and throughout the planned surcharge period RSG elevation readings were taken by site engineers. The cumulative change compared to initial baseline readings were continuously reviewed by design engineers.

Observed Settlement

Figure 6 presents observed settlement for each of the RSG's. Notably, there is an apparent 'rebound' in the readings following initial construction at day 20. This apparent rebound is a result of mathematical adjustment of settlement data to account for loss of RSG verticality that was induced by heavy construction plant constructing the bund.

Figure 6 illustrates a pronounced initial steep increase in settlement during the construction and progressive raising of the surcharge bund. Whilst data from a limited number of RSGs (Section 5 and 6) suggest an initial 'heave' this is attributed to the selected monitoring instrumentation that was rapidly replaced by a precise level monitoring instrument with a +/- 1mm accuracy; it is not considered that the underlying ground 'heaved'. Some of this observed heave, however, could be linked to RSG disturbance whilst placing fill materials.

Beyond the construction period it is observed that no further discernible settlement occurs (Figure 6). This observation confirmed that settlement of the underlying strata was limited to immediate settlement with little evidence of further consolidation settlement. This observed behaviour gave confidence that the reported weathered Mudstone, recorded as residual Clay, was more likely simply a drilling induced phenomenon opposed to an in-situ condition and likely long-term material behaviour. As such, risk of future consolidation settlement of the permanent structure could be discounted.

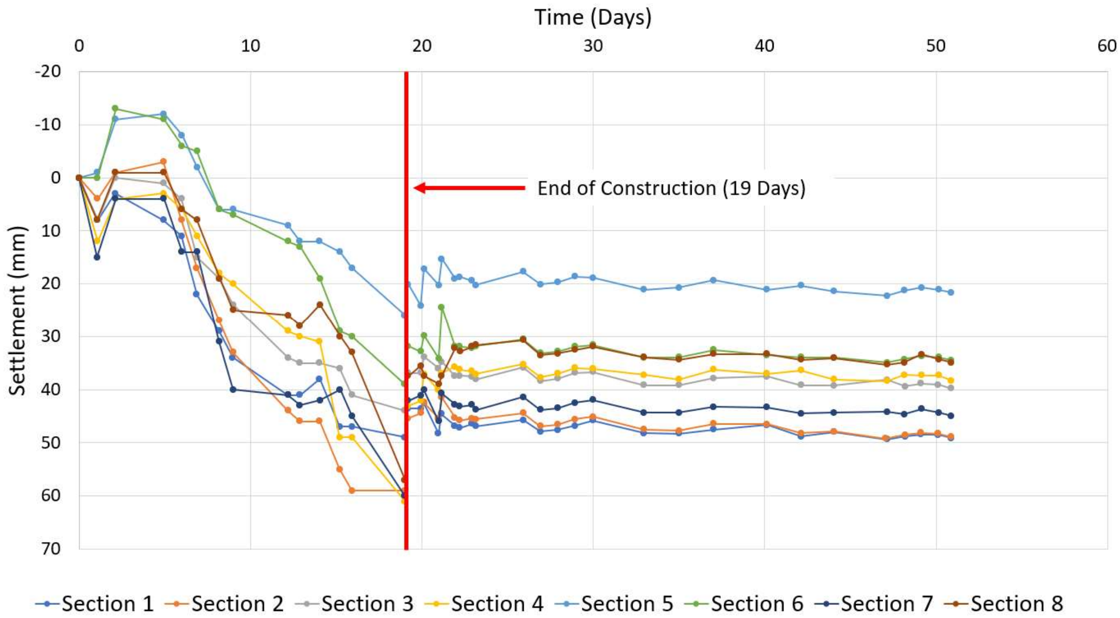

Figure 6: RSG Settlement Monitoring

The settlement of each RSG against that predicted from the initial settlement analyses to determine the size of bund required and surcharge timescales (Figure 3) is summarised in Table 3. The observed settlements for the surcharge bund were in the range of 20mm-47mm; this is comparable with the initially predicted magnitude of settlement. Overall, the agreement between actual and predicted settlements was favourable, with observed variations primarily attributed to inherent differences between interpreted and actual ground conditions. It is however, notable that the predicted total settlement was realised within 51 days, considerably sooner than anticipated. Figure 7 illustrates the predicted and actual settlement profiles for RSG 2, illustrating a very close alignment between predicted and observed.

Table 3: Predicted versus observed settlement

RSG Number	Predicted Settlement from 3 Months of Surcharging (mm)	Actual Settlement at 51 days (<3 months) of Surcharging (mm)	Difference Between Actual and Expected (mm)	Percentage of Predicted Settlement Achieved (%)
RSG 1	34	47	+13	138
RSG 2	46.5	46	-0.5	99
RSG 3	50	38	-12	76
RSG 4	42	38	-4	90
RSG 5	31	20	-11	65
RSG 6	32	33	+1	103
RSG 7	37	43	+6	116
RSG 8	29	34	+5	117

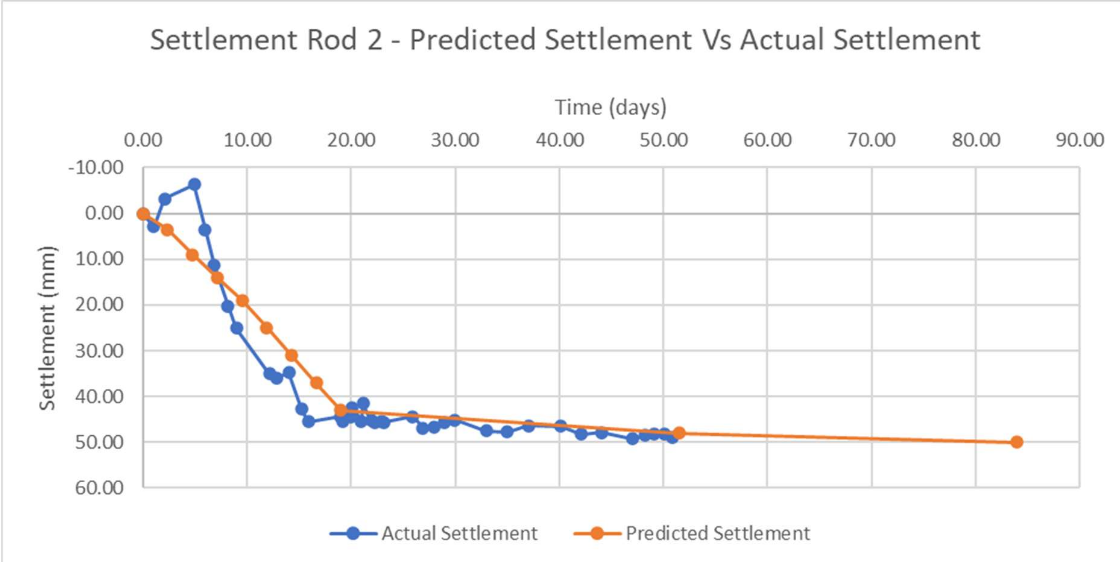

Figure 7: Example actual settlement vs predicted settlement for RSG2

Surcharge Review & Removal

It was observed that actual and predicted settlement magnitudes were comparable and further that settlement had plateaued and was not showing evidence of ongoing consolidation settlement. Furthermore, observed settlements were comparable of greater than that predicted for the permanent structure. It was therefore concluded after a period of 51 days, some 40 days earlier than was predicted, that sufficient settlement had occurred and that the surcharge bund could be removed early; effectively saving a month on the overall construction programme.

There was some concern that on removal of the surcharge bund the underlying strata could partially heave due to elastic rebound. As such, RSGs were carefully monitored during removal of the bund, however no particular rebound was observed during deconstruction.

Overall, the surcharge bund surpassed expectations by achieving the desired results in less time than predicted. Furthermore, with the actual settlements being very close to that predicted this gave additional confidence that the developed ground model was a reasonably accurate reflection of true ground conditions.

ACCURATE IDENTIFICATION OF THE POINT OF TRANSITION BETWEEN COMPETANT AND WEATHERED ROCK

After the removal of the surcharge bund it was important to accurately locate the transition between competent and weathered rock such that a number of bespoke SR wall panel units could be placed to span this transition. Trial trenches were located based on the 3D ground model and were excavated under supervision of a Geotechnical Engineer. Figure 8 illustrates the inspection trenches employed to pinpoint the actual transition location. On identifying the point of transition this was recorded by site engineers to allow wall panels to be accurately located during future construction.

Figure 8: Transition from competent to weathered rock inspection trenches

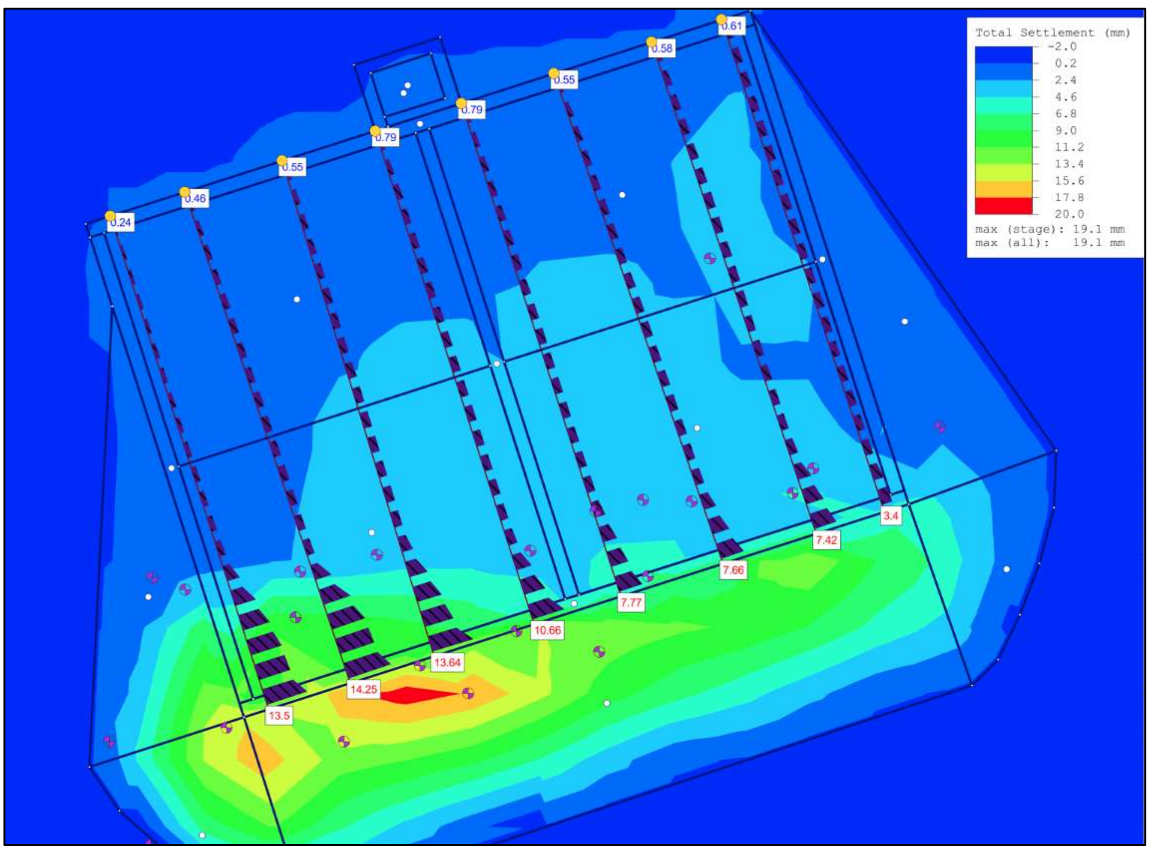

Figure 9: Settle3 Extract Showing Residual Settlement After Surcharging

BACK ANALYSIS OF SURCHARGE MONITORING DATA

The Settle3 settlement model was revised to reflect the accurate position of the transition from weathered to competent rock. Following revision to the ground model the post surcharge model was used to output revised representative modulus of subgrade reaction (ground stiffness) design parameters for the improved consolidated ground. Revised design parameters were adopted in structural finite element design to determine what, if any, long term settlement might be realised by the structure. Figure 9 illustrates the predicted long-term settlement based on worst credible structural loading. The maximum predicted future settlement directly below the structure is in the order of 14mm. Also, the likely maximum angle of distortion (differential settlement) over the weathered rock exceeds 1:1,000 which was deemed acceptable.

CONSTRUCTION SETTLEMENT MONITORING

As previously discussed, three significant loading cases are expected to induce the largest magnitude of settlement: landing wall units; water testing and backfilling. Throughout construction settlements of the structure will be monitored at these key stages. Monitoring positions will be set up on the setting-out strip, the external face of the walls, and the top of the walls at locations around the site. Baseline readings will be taken before any load is applied allowing for the calculation of cumulative settlement.

Construction to date has largely been over the identified competent rock and only marginally encroaching on the identified weathered zone. Fi gure 10 illustrates the extent of progress to date (May 2024). Observed settlements from site monitoring have consistently been below that predicted. This alignment between observed settlement and predicted behaviour instils further confidence in the accuracy of the ground model in Settle 3.

Figure 10: Overview Photo of the Site (taken on 09/05/2024)

PRECAUTIONS DURING WATER TIGHTNESS TESTING

When such structures are subject to water tightness testing (commonly referred to as a drop test) it is normal that one cell is initially filled and tested with water then pumped to other cells to test each cell individually. On this site however, there remains a low residual risk of differential settlements inducing excessive cracking over the transition between weathered and competent rock. It is unusual for such a structure to be constructed over strata with such significantly contrasting relative stiffnesses. To best mitigate the risk of excessive cracking being induced during testing it is proposed to take a different approach to initial filling and the resultant significant first ground loading.

For this structure it is planned that first filling for testing will introduce water into both cells simultaneously to 50% of the capacity of each cell. This will in effect allow even load distribution and significantly reduce the risks of differential settlements between the two cells. At this point the water in the west cell will be transferred to the east cell (lesser expected settlement magnitudes) and the east cell will then be fully tested. The water will then all be transferred to the west cell and this cell fully tested. In doing this it will in essence avoid 'shock' loading either SR cell and significantly reduce the risk of differential settlement between the cells. Close monitoring of settlement will be undertaken throughout this stage of work such that, if required, further measures can be implemented to avoid excessive structural distress.

SUMMARY AND CONCLUSIONS

Prevailing ground conditions beneath the proposed structure represented a significant geotechnical challenge owing to the stark contrast in relative ground stiffness's resulting in a significant risk of excessive structural settlements. To reduce this risk extensive ground truthing, detailed ground modelling and 3D settlement analyses were undertaken. It was ultimately concluded that sufficient ground improvement could be achieved by surcharging the site to induce potential future settlement early in construction.

Construction and monitoring of a significant but simple surcharge bund have removed the risk of initially intolerable predicted structural settlements. This has allowed the SR to be designed and constructed on a shallow reinforced pad foundation instead of a potentially more costly and carbon intense alternative foundation solution.

Borehole data suggested that Mudstone units weathered to a residual Clay may be present that could result in a long-term consolidation settlement risk. Observations from settlement monitoring provided evidence that underlying strata was largely granular in behaviour with no evidence of potential for long term consolidation settlement. It was concluded that the Mudstone was disturbed during drilling with water flush. Interpretation of factual data should not simply be taken on face value; experience, judgement and further proving should be applied such that over-conservatism does not creep into design.

Construction activities and continued monitoring to date has confirmed observed structural settlements less than predicted.

A residual risk of inducing potential differential settlement during first filling of the SR for water tightness testing was identified. To best reduce this risk it is planned to fill the individual SR cells concurrently during first filling to 50% of their individual capacity; this is generally not an industry-followed procedure. This methodology will in effect smooth initial structural differential settlement between the individual cells. It is recommended that this procedure be adopted as industry good practice for future such structures.

REFERENCES

Aujla N, Edmondson M and Dixon M (2021). High Leys service reservoir: a semi-precast construction. *Dams and Reservoirs* **31(4)**: 126-132, http//doi.org/10.1680/jdare.21.00029