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Challenges and opportunities in solar farm construction

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ABSTRACT: Solar farm developments can cover several hundred hectares in land area and comprise tens of thousands of pile installations. Adoption of sub-optimal pile types or installation methods, poor characterisation of ground variability and geotechnical risk, or inappropriate design assumptions can therefore result in significant cost or time overruns. Short piles which support solar trackers are particularly susceptible to the shrink/swell behaviour of soils within the active zone. By way of case study, this paper looks at some approaches to dealing with these challenges, including the benefits of pile testing to optimise pile lengths and help refine construction methodologies. The benefits of considering site-specific climate/weather data and finite element modelling of pile performance are demonstrated to provide a sensible rationalisation of shrink/swell pile performance. Applicability of Australian Standards is also discussed. These approaches have been demonstrated to result in significant cost and program benefits, compared with less rigorous approaches.

1 INTRODUCTION

1.1 *Background*

According to the Australian Photo Voltaic Institute (APVI), solar projects which will contribute up to 1.3 GW of electricity to the national grid are currently under construction across Australia, with a further 35 GW worth of development in various stages of planning (APVI, 2017). From a geotechnical perspective, these projects appear simple enough; foundations typically comprise steel sections installed to relatively short embedments via either direct drive or pre-boring methods. However, these developments may cover hundreds of hectares of land area, and comprise tens of thousands of pile installations. Failure to select optimal pile types or installation methods, inadequate characterisation of geotechnical variability or risk, or sub-optimal assessment of pile embedment lengths can therefore result in significant cost or program impacts to projects. This paper reviews some of the typical geotechnical considerations encountered in these projects, and via a case study, highlights how these can be addressed to ensure an efficient design outcome. In particular, this paper will discuss:

- Typical design load cases and pile types
- Site investigation and pile testing
- Installation methods
- Design in the active zone
- Applicability of Australian Standards.

1.2 *Case study particulars*

Examples discussed in this paper will be illustrated via a particular case study from the authors' recent experience. This project comprised the installation of over 60,000 piles across a 200ha site in Far North Queensland. The original design of these piles was based on approximately 50 boreholes drilled to 6 m in depth, and SPT testing at usual intervals of approximately 1.5 m. Ground conditions comprised very stiff to hard silty clays and very dense clayey sands of alluvial origin, below a thin veneer of topsoil. No bedrock was encountered within the excavation depths across the site, however, occasional weakly-cemented lenses were encountered. Auger drilling did not refuse at any borehole locations, although SPT N values varied from N=13 to refusal. Initial pile length estimates by others called for variable pile lengths of between 2.8 m to 4.4 m embedment based on a conventional design approach and correlation with borehole information, and all piles were assumed to be able to be directly driven with a 1.3KJ accelerated hammer.

Early during the construction phase, significant difficulties in pile installation were observed, with piles refusing at varying depths. Zones of probable refusal were difficult to reliably predict. Installation rates slowed to as low as 15% of targets. As a consequence of subsequent pile testing and redesign considerations as described herein, piles were shortened to 2.3 to 2.6

m embedment, and installed via a combination of direct drive and pre-boring means.

2 DESIGN LOAD CASES AND PILE TYPES

2.1 *Typical load cases*

It is common for design loads to be determined by the solar tracker supplier, based on the unique operational characteristics of the superstructure. Loads are normally assessed in accordance with AS1170, and for Australian conditions, consider the following load cases:

- Uplift due to wind loads;
- Lateral load / overturning moment due to wind loads;
- Compression loads due to self-weight and down force of wind;
- Instrumentation or equipment loads such as cable trays or motors.

Whilst all load cases need to be considered, overturning / lateral loads normally govern, along with vertical movements associated with shrink/swell potential of the soil. Solar trackers are usually aligned east-west so that panels can follow the path of the sun throughout the day. Horizontal loads and axial loads alike may typically be in the range of 5 kN to 20 kN. Associated moments depend on the height of panels above ground level, as well as any additional torque moment applied as a result of the tracker mechanism itself. In areas prone to cyclonic winds, such as the case study site, lateral loading tends to be at the upper end of this range. In non-cyclonic areas, such as Victoria, loads tend to be lower.

The application of these load cases needs to consider the changing state of the soil profile throughout the design life of the solar farm (typically 20 to 30 years). For example, cracking due to drying of the soil can result in reduced capacity in both axial or lateral resistance. Conversely, extreme wetting or flooding of the site may result in softening of the soil near the ground surface. Consideration of these factors at the case study site indicated that the critical design load case was one where the design lateral load applied when the soil softened as a result of very heavy rainfall or flooding. Consideration of shrink/swell effects on the long-term vertical displacement of piles was also a critical factor.

2.2 *Pile types*

Pile types are often dictated by the solar tracker supplier, and include UC, UB, Z, RHS, C and Σ sections. Sections are often specified in overseas (for example, ASTM) designations, making it somewhat difficult to identify appropriate local equivalents. At the case study site, adopted piles were W6x15 (ASTM) designation, which were acceptably similar to an Austral-ian 150UC23 section, however at other sites, the au-thors have observed significant differences between specified pile types and the 'nearest equivalent' locally manufactured section. This is important to note, as equivalencies need to be considered if alternative sections are adopted for pile testing purposes, or when applying engineering judgement based on experience with locally available pile types. It also highlights the need to optimise pile designs as best as practical; pile sections which are non-standard for local manufacturers may be difficult to come-by if additional steel is required. An oversupply may be equally difficult to dispose of.

3 SITE INVESTIGATION AND PILE TESTING

3.1 *Site investigation*

Site investigation methods adopted for solar farm projects in Australia typically comprise augured boreholes with in-situ testing often limited to SPT tests, and laboratory testing comprising conventional index testing, along with I_{ss} testing to assess shrink/swell behaviour. Site investigations are often procured at very early stages, pre-design, by developers or clients who may have a limited understanding of geotechnical engineering. As a consequence, investigations are often not necessarily as comprehensive as a de-signer may hope. For example commencement of SPT testing at 1.0 or 1.5 m depths and then conducting at subsequent 1.5 m intervals, as is normal practice for other projects, invariably fails to capture in-formation on the strength or consistence of the near-surface soils which may prove crucial to the capacity of piles founded at shallow depths. Where ground conditions permit, CPT testing may prove more valuable s it can provide a continuous strength profile at these important shallow depths.

At the case study site, boreholes were performed at approximately 200 to 250 m spacings. Whilst these gave a reasonable overall characterisation of the general material types across the site, this frequency was inadequate to properly understand the variability of cemented lenses which appeared to impede pile driving, and had a scale of variability in the order of tens of metres, or less. This highlights the inherit risks and uncertainty in such limited investigation for a project of this type.

Laboratory testing for shrink-swell properties were conducted on 30 samples, with I_{ss} values ranging from 0.1 % to greater than 5.0%, and with measured swell pressure values of between 25 kPa and 1600 kPa. Such variability is difficult to manage in a design process (and even harder to explain to a non-geotechnical engineer) without sufficient sample quantity to prepare a statistical representation of the data. Fortunately, at the case-study site, the number of samples available allowed for a basic statistical assessment and a '95th percentile' and average value could be estimated with reasonable confidence.

3.2 *Pile testing*

Given the limited ground information available at the case study site, pile testing proved to be a critical means of assessing pile capacity, rather than conventional design approaches which rely on correlations with SPT tests. A total of 31 axial pull out tests and 9 lateral load tests were completed across the site for the purposes of pile design, with a further 50 of each type of test (100 in total) completed post-construction as proof tests.

Test piles were installed to depths of up to 3.0 m (typically 2.6 to 2.8m) below ground level. Test piles were initially driven, but if unable to do so then they were installed instead in a pre-bored hole grouted with stabilized sand.

Axial tests were conducting using a 20 tonne franna crane and appropriate lifting slings and load cells. An example is shown in Figure 1. For piles with typical embedments of 2.8m the lowest load which induced axial pull out of a pile was 10 tonnes (100kN) although the vast majority of test piles show negligible creep movement under the maximum available pullout load of 20 tonnes (200 kN). This suggests ultimate axial shaft resistances of over 120 kPa, assuming plugged section behaviour. Such values are higher than might be predicted from evaluation based on SPT testing alone.

Figure 1. Typical axial test setup (lifting shackles, load cell and dial gauge shown)

Lateral tests were conducted using a hydraulic jack and load cell, and the franna crane as a reaction. Up to 5 tonnes (50 kN) of lateral force was applied at a height of approximately 1.2 m above ground level. Figure 2 shows a typical test setup. Pile deflections under load were measured using dial gauges at both ground level and the top of pile, and the measured load-deflection curve was then used to calibrate a p-y model using the software package *LPile*. Given the predominately clayey nature of the soils at the site, a Reese Stiff Clay model was adopted, with the design s_u modified until good agreement with the site measured load-deflection curve was found. An example plot is shown in Figure 3.

Figure 2. Typical lateral test setup showing jack, load cell and dial gauge. 1 dial gauge was used during proof tests as shown, 2 during design tests.

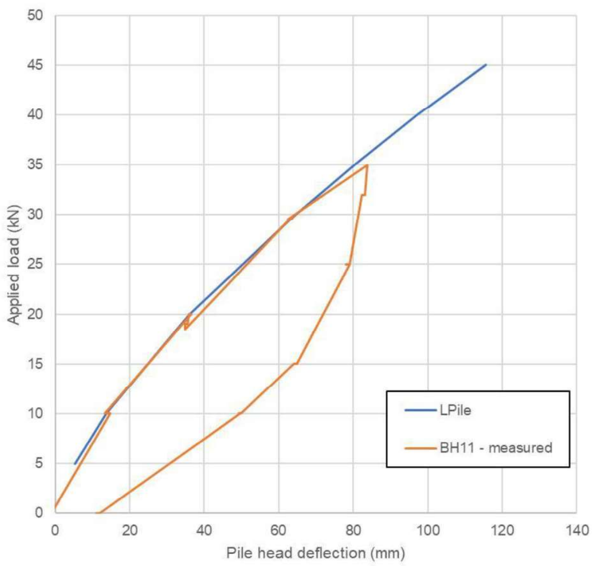

Figure 3. Typical lateral test calibration plot

Lateral test results needed to be adjusted for soil state. As mentioned in Section 2.1, the governing soil condition for the lateral load case at the site was a case where surficial soils had softened due to an extreme wetting event. A general assumption of 50% loss in strength and stiffness was assumed for design purposes. Fortuitously, this was able to be tested when the site experienced a month of heavy rainfall. Over 470 mm of rainfall fell at the site over a one-month period. Review of local weather station information suggested this was a 93rd percentile event, and as a consequence was considered appropriate to validate the design assumption of soil softening. The example calibration plot shown in Figure 3 shows the results of a load deflection test undertaken a few days after rain had ceased and minor flooding of the site had subsided. The plot shows good agreement between the field data and the design model which assumed a 50% reduction in soil strength over the top 0.9 m of the soil profile.

As a consequence of the pile testing program undertaken across the site, the design embedment length of piles was successfully reduced to between 2.3 m to 2.6 m embedment, from the range of 2.8 m to 4.4 m originally allowed for. This resulted in significant time and monetary savings across the project.

4 INSTALLATION METHODS

Solar farm piles are typically installed by driving, either directly into the undisturbed ground or into a pre-bored hole, which has been backfilled, usually with stabilized sand or similar. Both undersized and over-sized pre-bored holes are common.

Contractors' preference is usually to drive the piles wherever possible, as the cost of installation can as much as double once pre-boring is implemented. Similarly, where pre-boring is introduced, there is a tendency to err towards an 'undersized' pre-bore, to minimize the total volume of backfill required. At the case study site, the use of under-sized pre-bored holes was a false economy as many piles still refused when their flanges engaged with the hard in-situ soil. It was also found to be difficult to achieve positional tolerance of the piles with this method, with piles tending to twist.

At the case study site, the contractor elected to place stabilized sand in the pre-bored hole, prior to pile driving. Piles were driven up to approximately 24 hours after stabilized sand placement. The stabilized sand was a 1.0 MPa UCS mix, consistent with Queensland Main Roads specification MRTS04 (QDTMR, 2017). Figure 4 shows a stabilized sand test pile which was extracted during an axial pull out test. It can be seen from this figure that the stabilized sand itself shows no visible sign of cracking or disintegration due to the driving process. Where the stabilized sand has broken away from the pile during extraction, the pile itself appears 'clean' and does not appear to have been in contract with the soil. These observations suggest that driving into the stabilized sand is an acceptable construction technique, however the authors' would also recommend site trials, as results may differ with different equipment, piles or mix designs.

Figure 4. Extracted pile in stabilized sand.

5 DESIGN IN THE ACTIVE ZONE

5.1 *Thickness of active zone*

Solar farm piles can be affected by shrinking and swelling effects if founded in expansive soils due to drying and wetting arising from variable climate patterns. In this environment, piles move vertically together with the surrounding soil. It is possible for cyclic heave of piles to be greater than settlement in response to these climatic effects, which can lead to progressive 'jacking' of piles over multiple shrink-swell cycles.

The likelihood and magnitude of pile behaviour in response to seasonal moisture variations is difficult to reliably predict. AS2870-2011 provides guidance on the estimation of the thickness of the 'active zone' (the depth over which soil moisture is likely to vary due to seasonal climatic cycles), and the characteristic surface movements (y_s) which might be expected for different soil types. Whilst AS2870 is written for the specific purpose for the design of foundations of residential buildings, its guidance is still useful for the assessment of foundations of the type described herein.

AS2870-2011 suggests that the thickness of the active zone of soil is a function of the Thornthwaite Moisture Index (TMI) of the local area, calculated on a long-term average of monthly rainfall and temperature data over at least 25 years. At the case study site, the TMI was calculated as follows:

- Monthly rainfall statistics and average monthly temperatures were obtained from the Australian Bureau of Meteorology website for the nearest weather station to the site, with at least 25 years of continuous data, as recommended by AS2870-2011;
- The TMI was estimated using "Method 2" described by Karunarathne et al (2016), with the active zone thickness based on the average of yearly TMI values over the range of interest.

Based on this methodology, a site specific TMI of -1.8 was estimated. AS2870-2011 recommends a climate zone classification of "2" and active zone thickness of 1.8 m for a TMI between -5 to +10. Figure 5 shows the calculated TMI for the site.

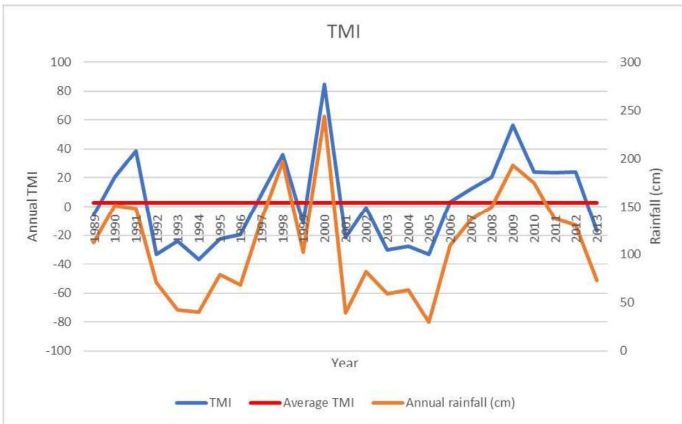

Figure 5. Site specific TMI

The climate zone for the site was also checked using the published map by Fox (2000) which suggested a climate zone of 3, and an active zone thickness of 2.3 m. Thus, a saving of 0.5 m of active zone depth (and and thus potential pile embedment) was able to be realized immediately by completing a site-specific TMI estimate.

5.2 *Estimating pile movements*

Once the thickness of the active zone was established, estimates had to be made of the likely swell/heave movements the piles may experience across the site. This was completed using the software package *Plaxis 3D*.

The pile was modelled as a rectangular block element of 150 mm square, which assumes the pile would behave in a plugged manner. The soil was modelled as a Mohr-Coulomb "Undrained B" material, with undrained shear strength of 75 kPa and E' of 30MPa which was considered reasonable for fully softened conditions (maximum swell) at the site. A shaft resistance of 50kPa was assumed along the pile shaft below the active zone, which was considered conservative compared with the axial tests completed at the site.

Swelling of the soil was simulated using a volume expansion model. Swelling was allowed to occur over the top 1.8 m of the pile to simulate the active zone, and pile movement was assessed at an average surface movement of 50 mm, which was consistent with the average characteristic surface movement estimated for the site, based on measured I_{ss} values and the active zone thickness in accordance with AS2870-2011. Sensitivity analysis was also undertaken for surface movements of 25 mm and 75 mm. Figure 6 shows an example of the output obtained. The results suggested that for a 50 mm characteristic surface

movement, the pile would move vertically by up to 11 mm. On subsequent drying and shrinking of the soil, the pile would then move down by 6 mm, resulting in net movement of 5 mm. Sensitivity analysis for 25 mm and 75 mm surface movements suggested a range of 2 mm to 10 mm of pile heave might be experienced over a shrink swell cycle. The number of shrink-swell cycles expected over the design life of the facility was not estimated, however the results were considered to be reasonable and acceptable by all stakeholders.

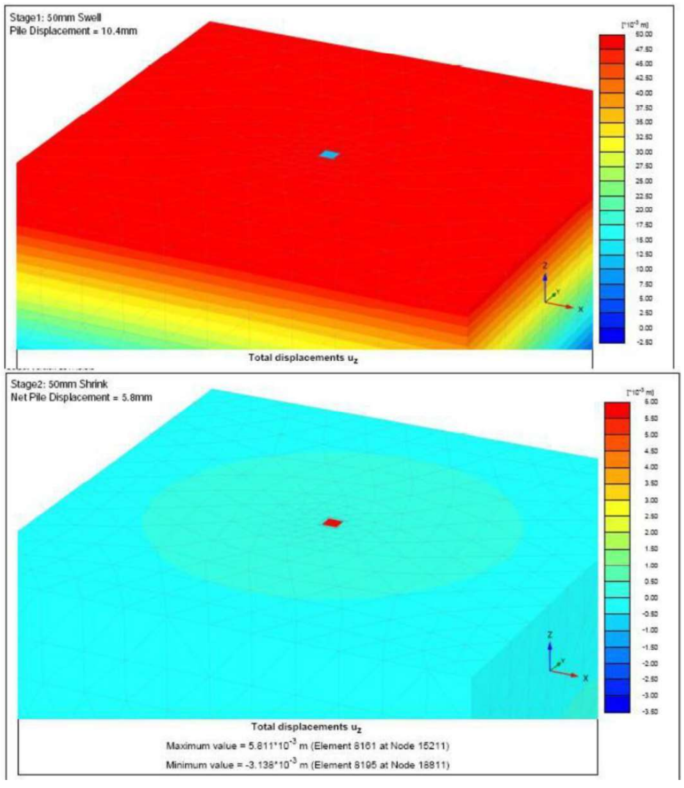

Figure 6. Example Plaxis output – Shrink/swell assessment

5.3 *Validation of pile movements – comparison with published data*

A useful case study by which to compare the Plaxis estimates of pile performance is provided by Fityus and Delaney (2001). These authors studied the swell movements of 125 mm to 200 mm diameter timber poles embedded in soils with an active zone of around 1.7 m thickness. Pile movements reported by Fityus and Delaney were in the order of 10% of free field surface movements for piles embedded around 2.0 m, and around 22% for piles embedded to around the same depth as the active zone. Predicted movements by the Plaxis 3D modelling for the steel piles of similar exterior dimension were consistent with these observations.

6 APPLICABILITY OF AUSTRALIAN STANDARDS

It is useful to note the applicability of relevant Australian standards to the design of solar farm piles, such as those at the case study site. AS2870-2011 has been discussed already, insofar as its applicability in the estimation of surface movements and active zone thicknesses. Whilst this standard provides useful indicative guidance on active zone thicknesses for example, greater economy can sometimes be achieved by estimating this using other published methods. The standard itself is not clear on which method of calculating TMI is preferred. Karunarathne et al (2016), used in this case study, discusses the implications of different methods. It is possible that adopting different approaches would yield different results, and could lead to either more or less favorable outcomes, and therefore consistency of approaches is important, and thus estimating the active zone thickness. The standard also does not explain how the active zone thickness directly relates to TMI.

AS2159-2009 is the other important Australian standard, as it relates specifically to the design and installation of piles. Of particular interest in AS2159 is its discussion of pile testing and in particular the "requirement to test". Section 8.2.4 of the standard, for example, suggests that proof testing is required on at least 1% of piles whenever the average risk rating (ARR) calculated in Section 2 of the standard is greater than 2.5 and the adopted geotechnical reduction factor for ULS design cases is greater than 0.4. For a site like the case study site, this would require over 600 pile tests to be completed. It also means that a very small change in the ARR (which could occur through different interpretations of the individual risk ratings (IRRs) in Section 4.3.2 of the standard) could make a very significant commercial impact to a project, if pile testing requirements jump from 0 to 600+ piles through a very small change in this evaluation.

As has been discussed herein, pile design at the case study site, and at others which the authors' have been involved at have been governed by shrink swell effects of the soil and modelling of lateral performance if the soil were to soften by an estimated amount. Neither of these circumstances can readily be evaluated using pile testing of the type intended by the relevant clauses in AS2159, and consequently, it may be possible to complete significantly fewer tests for the same level of confidence. Ultimately then, deciding the number of tests to be completed relies on a cooperative, transparent conversation about risk, and agreement between designers, reviewers, contractors and end-clients.

7 CONCLUSION

Good practice design and construction of solar farms requires a sound understanding of the geomechanics of near surface soils, as well as an appreciation of the limitations of construction techniques and site investigation data typically available. The scale of developments means relatively small design refinements can magnify significantly, as can conservatisms. As has been demonstrated, targeted pile testing, site specific consideration of shrink-swell properties and consideration of the applicability of appropriate codes and standards can yield significant project benefits when applied appropriately.

8 ACKNOWLEDGEMENTS

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