

# The case of piles wholly embedded in a settling soil mass – What’s with all the negativity?

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## ABSTRACT

Negative skin friction forces (NSF), or drag load, would need to be considered in the geotechnical design of piles where the soil surrounding a pile undergoes settlement or consolidation after the pile has been installed. Typically, in the Melbourne region, piles that are prone to NSF forces generally terminate in the ‘stable zone’ below the consolidating layer (e.g. bedrock) such that there’s sufficient geotechnical resistance to sustain the combined serviceability load and NSF forces, and to satisfy the serviceability requirement. For the new road bridge over the Murray River at Echuca-Moama (the ‘Dhungala Bridge’), the geotechnical design considered the development of NSF forces in the abutment piles resulting from potential long-term consolidation of the ground due to the construction of fill embankments at the abutments. With the piles wholly embedded in the settling ground, the traditional method of determining a sufficient ‘stable zone’ beneath the consolidating layer to resist the drag load could not be applied and an alternative approach, the ‘Unified Design Method’, was adopted to allow for the drag load in the design.

## 1 INTRODUCTION

The new road bridge over the Murray River at Echuca-Moama, completed in 2022, is a significant infrastructure project that enhances connectivity between the towns of Echuca in Victoria and Moama in New South Wales. Prior to this project, the only road connection between the two towns was a 150-year-old iron girder bridge with a single lane each way, with the nearest alternative crossing needing a 100km round trip. This project addresses the growing traffic demands and provides a critical alternative route to the historic iron girder bridge, alleviating congestion and improving safety.

The new road bridge over the Murray River, called the Dhungala Bridge, is part of the larger Echuca-Moama Bridge Project, which also comprises a crossing over Campaspe River, called Yakoa Bridge. The Dhungala Bridge spans approximately 620m and includes a single lane in each direction, and shared pedestrian and cyclist paths.

The Dhungala Bridge comprises approximately 600m of elevated structures including a 115m long main span over the Murray River. The bridge abutment and the 13 bridge piers are supported on 400mm square driven precast pile groups, with approach embankments at either end up to about 11m high. The fill embankment is a vital element of the project, providing a stable foundation for the approach roads leading to the bridge and allowing the bridge to maintain sufficient clearance for the historic paddle steamers to pass beneath. At the abutments, the foundation piles were installed prior to the construction of the approach embankments.

## 2 APPROACH EMBANKMENTS

The ground conditions along the project alignment typically comprise a surface layer of Quaternary age fluvial, alluvial and flood plain deposits composed of

gravel, sand, silt and clay, over Quaternary age deposits of the Shepparton Formation. The Shepparton Formation is around 80m thick in this area and is underlain by layers of Tertiary sediments with Permian age basement rock at depth in excess of 1,00m.

Based on site investigation findings, the Shepparton Formation typically comprised interlayered loose to medium dense sand and stiff clay that graded to very stiff clay with depth. Groundwater was typically encountered at between 5m and 10m below ground level. At the approach embankments, the clay extends to more than 50m depth with interbedded sand / silty sand bands typically 3m to 8m thick. The typical ground profile is shown in Figure 1.

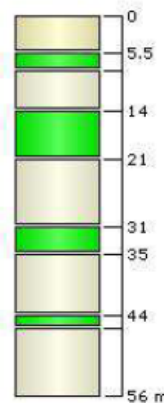


Figure 1. Typical Ground Profile – Depth Below Ground Surface (light brown is clay, green is sand)

On the basis of laboratory tests and CPTs, the following strength parameters were adopted in the geotechnical assessment:

Clay:  $c' = 8 - 12 \text{ kPa}$ ,  $\phi' = 24 - 29^\circ$ ,  $\gamma = 20.5 \text{ kN/m}^3$   
Sand:  $c' = 0 \text{ kPa}$ ,  $\phi' = 37^\circ$ ,  $\gamma = 20.5 \text{ kN/m}^3$

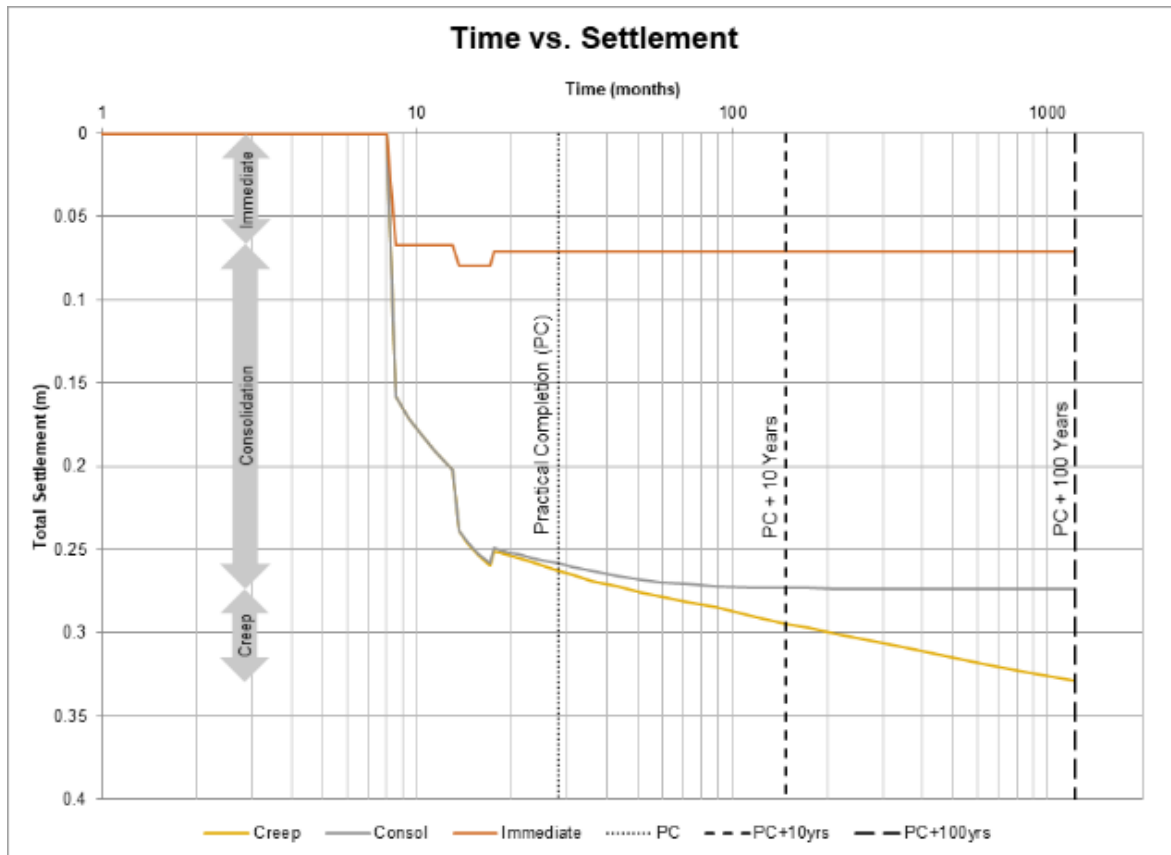


Figure 2. Estimate of Embankment Settlement Over Time

The cohesive (clayey) layers in the alluvial sequence of the Shepparton Formation are expected to undergo time dependent consolidation settlement, where the loading exceeds the pre-consolidation pressure. Where these layers are thick, the settlement can take place over a period of years after construction. For the proposed 11m high embankments, the vertical stress increase will extend to depths of up to 50m or more. The magnitude and rate of consolidation settlement is a function of the compressibility of the soil, the magnitude of stress increase, the thickness of the susceptible layers and their bulk permeability.

Based on laboratory oedometer testing (1-D consolidation) and dissipation tests undertaken during cone penetration testing, the embankment settlement behaviour was analysed using the computer program Settle3 (V5.001) by RocScience. The analyses considered varying ground profiles, height and width of embankment, and the proposed construction program. The estimated embankment settlement over time is shown in Figure 2. The ground settlement profile at 10 years after practical completion is shown in Figure 3. In this figure, the near vertical lines would indicate the sand layers where settlement is expected to occur immediately upon loading, hence no time dependent settlement.

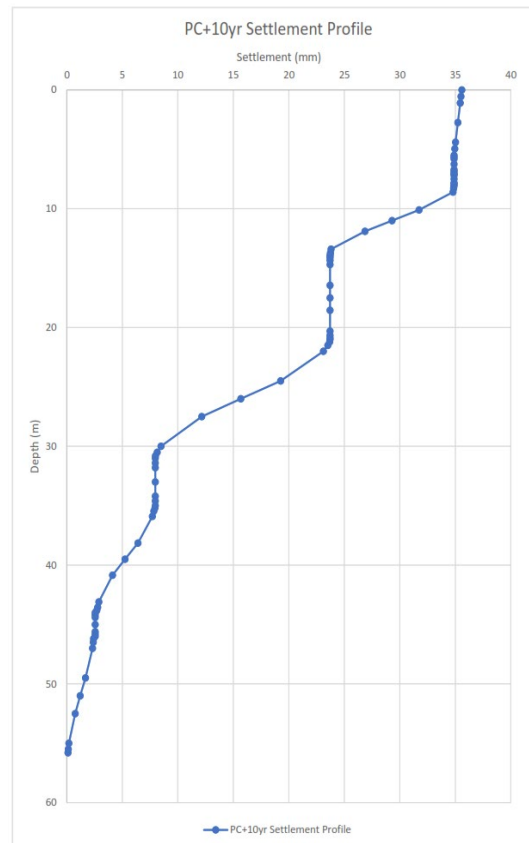


Figure 3. Ground Settlement Profile

A range of techniques exist to help control consolidation, which aims to either improve soil stiffness or reduce drainage path distance. These include deep soil mixing and controlled modulus columns to improve soil stiffness, and vibrostone columns and wick drains to reduce drainage path lengths. However given the depths under consideration, such treatments would generally be impractical or excessively expensive. Other methods include surcharging the embankments for a period during construction to accelerate the consolidation or reducing the embankment weight by the use of lightweight fill, although this method is limited by buoyancy consideration during flood events. Based on site conditions and various constraints, surcharging of the embankments during construction was considered the most appropriate and economical method to ensure settlement requirements are met.

### 3 NEGATIVE SKIN FRICTION

Negative skin friction arises when the soil surrounding a pile undergoes settlement or consolidation after the pile has been installed. Here the ground settlement is induced by the construction of the 11m high fill embankment. As the soil settles, it can adhere to the surface of the pile and drag it downward. The downward force induced by this interaction is referred to as "negative skin friction force" or "drag load". As a consequence, this imposes an additional load that will need to be resisted by the pile foundation, and the pile will experience additional settlement.

NSF load should be considered in terms of pile structural strength and serviceability. The effect of NSF on ultimate structural capacity is different to that for geotechnical ultimate capacity because for the pile to fail geotechnically, it must move down relative to the soil. Once this occurs, negative skin friction loads are relieved, as such NSF is considered to be a serviceability issue in terms of geotechnical performance of the piles and must be assessed separately to the ultimate geotechnical capacity.

### 4 FOUNDATION ASSESSMENT

At the design stage, each bridge abutment was proposed to be supported by 15 No. driven precast piles (two rows of 7 and 8) installed to about 30m below ground level. The ground settlement affects the abutment piles in two ways:

1. NSF load acting on the pile shaft; and
2. Settlement of piles due to consolidation of soil below the pile toe (note ground settlement zone extends to 55m below ground level, as per Figure 3).

In both instances, the ground settlement would increase the settlement of the pile group. In most

projects, there would be a 'stable zone' or non-settling layer where the piles can be founded to provide positive shaft resistance to counteract NSF. For example, in the Melbourne area where we have compressible Coode Island Silt, the piles would be founded in the stable Moray Street Gravels or Melbourne Formation bedrock, which are located within practical depths. One obvious solution to deal with NSF here would be to extend the piles deeper until it is well into the 'stable' zone. However, with the settlement zone extending to 55m below ground level, the piles would need to penetrate well beyond 55m depth to counter the large NSF load, which is not considered practical nor economical.

Effectively, with 30m deep piles in a 55m deep settlement zone, the piles are wholly embedded in a settling soil mass. In order to assess NSF induced settlement of the proposed 30m deep piles, the Unified Design Method (Fellenius, 2019) was adopted. In this method, the aim is to determine the neutral plane of the pile, which is the point where the shaft resistance balances out the negative skin friction force, and is also the point of maximum negative skin friction load. And from the depth of the neutral plane, the corresponding pile settlement can be determined from the ground settlement profile.

To determine the neutral plane for a single pile, the load and resistance curves were generated (see image on the left in Figure 4). The load curve begins at the applied dead load and increases with depth due to NSF load. The dead load of an individual pile was determined from a pile group analysis. For the resistance curve, the curve begins at the ultimate geotechnical capacity of the pile and decreases with depth due to positive shaft resistance. In Figure 4, the negative skin friction force is adopted as the same as the positive shaft resistance, so the curves are a mirror image of each other. Here it is important that the pile toe resistance adopted is reasonable for the anticipated pile movements. For this project, the toe resistance was compared against PDA test undertaken on driven piles that were installed during the early works of the project. Where the two curves intersect is the location of the neutral plane. And from the depth of the neutral plane, the corresponding pile settlement can be determined from the ground settlement curve produced from the Settle3 analysis (Figure 3), and in this instance the estimated pile settlement is about 25mm.

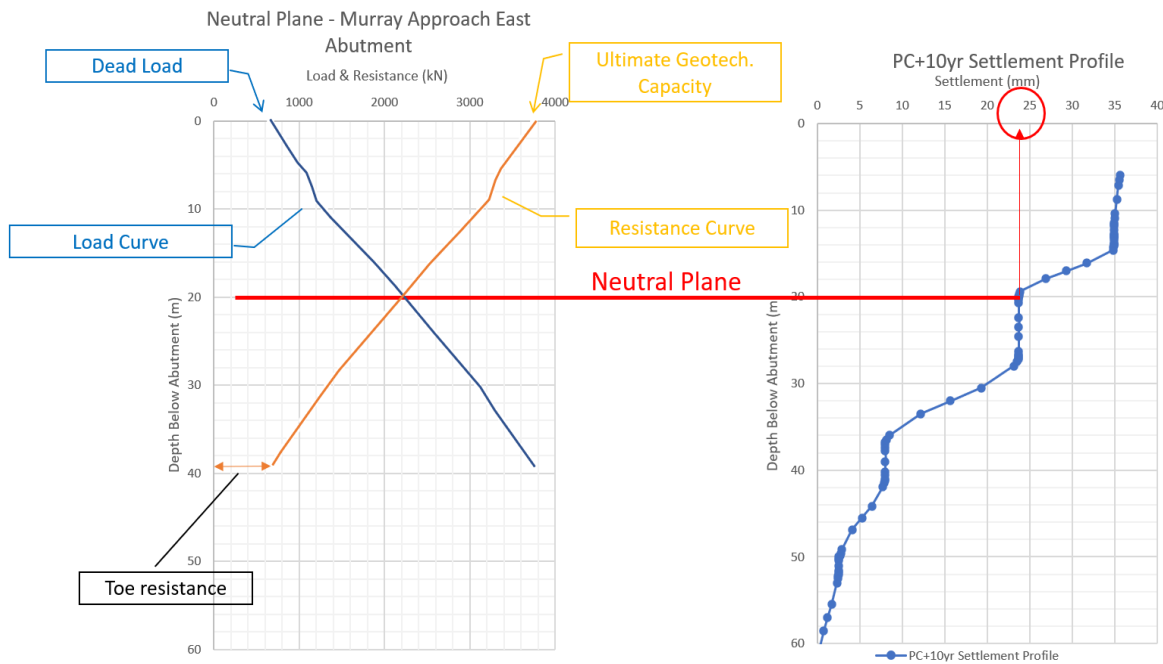


Figure 4. Determining Pile Settlement Using the Unified Design Method

From the load and resistance plot, the negative skin friction load for the pile can be determined, which is the difference between the load at the intersection of the curves and the dead load.

Based on this method of analysis, the pile length can be adjusted to target different pile settlement. Lengthening the pile would lower the neutral plane and hence can target lower down the ground settlement profile.

As a separate assessment, single pile analysis was also carried out using the program Ratz (V4.2) by M.F.Randolph where the down drag function was used to consider the ground settlement, and the resulting pile settlement matched well with the estimated 25mm obtained from the Unified Design Method.

Numerical modelling using PLAXIS 2D was also undertaken to assess the effect of ground consolidation on the abutment piles and found resultant pile axial forces to be of a similar order to those calculated from the Unified Design Method.

## 5 CONCLUSIONS

Negative skin friction acting on piles is an important consideration in pile design, where the ground is prone to settlement and consolidation. Whilst piles can often be extended below the settling soil layer into a ‘stable zone’ to counteract NSF forces, this may not always be practical where the ‘stable zone’ is at significant depth, such as at this project. Here

the Unified Design Method can provide a practical approach in assessing NSF induced settlement for piles embedded in a settling soil mass.

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