

TRANSITION TREATMENT DESIGN FOR BRIDGE APPROACH EMBANKMENTS – FROM A PRACTITIONER’S PERSPECTIVE

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ABSTRACT

Transition treatments in soft soil are implemented for the purpose of achieving differential settlement criteria between a rigid piled abutment and an approach embankment founded in soft soil. With carefully designed transition treatments, bridge abutments and their supporting piles can be constructed early in the construction program without the need to wait for embankment settlement to conclude – thus, facilitating an efficient construction process.

Methods to reduce or control settlement, include the use of piles/columns in the embankment foundation, and other ground improvement techniques such as preloading/surcharging. Piles/columns are used in varying frequency and quantity within the approach embankment to achieve the desired transitional effect and “protection” for the abutment piles from the lateral movement of the foundation soils. Overburden loads are often transferred to the piles/columns through a gravel mattress embedded with geotextiles, commonly referred to as a load transfer platform (LTP). The design of LTP’s depends heavily on the degree of support provided by the soil matrix surrounding the pile/column treatment.

The success of transition treatments can be measured by the magnitudes of settlement within the transition zone and achievement of differential settlement criteria. Methods to reduce long term creep settlement include the use of surcharging pressures to over-consolidate clay soils. In this paper, the measurement of differential settlement in the form of change-in-grade (CIG), i.e. change of slope between two straight lines, are discussed and clarified – removing some ambiguities and confusion which currently exist in practice.

1 INTRODUCTION

Bridge superstructures are typically composed of large loads, requiring substantial foundations to support them. The allowable settlements and deformations for a bridge are usually very small, in the order of tens of millimetres. Accordingly, bridge abutments are typically supported on piles founded within competent ground stratum, such as rock.

Approach embankments supporting roadways preceding these rigid abutments are likely to experience settlement, which can be particularly large if soft soils are present. In some cases, these settlements can continue for up to tens of years post construction. While the embankment integrity and road pavement structure may be able to absorb large total and differential settlements, such large settlements will contrast suddenly with the rigid, almost immovable bridge abutments. If not treated properly, such zones will produce a “bump at the end of the bridge” (Briaud et al., 1997), which can be unpleasant and hazardous to drivers (Briaud et al., 1997). If left untreated, the continued impact of vehicles at the bump may cause the approach slabs and bridge deck to deteriorate (Wahls, 1990), adding costs to the operation and maintenance of the road network. As such, transitional zones are often required to “soften” the settlement progression between earthen embankments and bridge abutments.

Engineering of the soft soil alone beneath the approach embankment has its own set of unique challenges, bringing a high level of risk and uncertainty to the project (Hsi, 2016). When coupled with the task of producing a gradual transitional feature against a rigid piled abutment, the complexity increases, and the task of producing an optimal engineering solution becomes particularly challenging.

In recognition of the challenges associated with transition treatment design, this paper presents a brief accumulation of the findings from the authors’ experience in the design and construction of bridge transition treatments in soft soils. The introductory sections present the basic framework behind the various zones adopted for typical transition treatments, including the functionality of each zone, and the typical treatment arrangement. The latter sections provide recommendations towards addressing more complex issues of transition treatment, such as the protection of abutment piles from the impact of embankment settlement, clarifying ambiguities in the design of load transfer platforms, and the available techniques currently used in industry to overcome these. The paper concludes with a discussion on the differential settlement criteria, commonly referred to as change-in-grade, to which all transition treatments strive to achieve, and the practical application of predicting differential settlement profiles incorporating creep effects. Relevant

case studies will be offered to showcase various arrangements of transition treatments and successful applications in practice.

While this paper provides a collective summary of personal experiences encountered by the authors, and have generally resulted in successful outcomes, the authors would advocate that practitioners to form their own opinions and conclusions based on sound research and practical experience. Any critical discussion about these issues to further our knowledge in tackling these problems is welcome. As this paper has been written from a practitioner's perspective, sophisticated theories and approaches to address complex issues, which are generally difficult to apply in practice, are avoided.

2 PURPOSE OF TRANSITION TREATMENTS

In light of the marked level difference that can occur between a rigid piled abutment and an approach embankment founded on soft soils, transition treatments are designed with the following objectives in mind:

- Achieving differential settlement criteria – by implementing foundation treatments beneath approach embankments. Designers can limit settlement magnitudes to occur at a gradual rate away from piled abutments towards untreated embankment zones – thereby achieving the required differential settlement criteria.
- Providing a buffer zone to allow early construction of abutment piles – construction of embankments on compressible ground will induce vertical and lateral deformations in the foundation, which can be particularly prominent in soft soils. Accordingly, the installation of abutment piles adjacent to settling approach embankments will subject the piles to ground deformations, creating undesirable bending moments and shear forces. Transition treatments can also be used to eliminate deformations in the soft soil generated by the settling embankment, effectively shielding the abutment piles from deforming foundation soils, and allow the piles to be constructed at an early stage of the construction program.

3 TREATMENT ARRANGEMENT

In order to satisfy the objectives of transition treatments, ground treatment elements must be placed into the foundation beneath approach embankments to reduce ground deformation. The specific arrangements of these elements are then manipulated in frequency, quantity and type to achieve the desired transitional effect. In the authors' experience, the various treatment elements can be broadly categorised into three zones, i.e. Structural Zone, Surcharge Zone and Embankment Zone, each performing a unique settlement mitigation function, yet collectively forming a gradual transitional function away from the piled abutment. An illustration of a typical transition treatment arrangement suggested by the authors is shown in Figure 1.

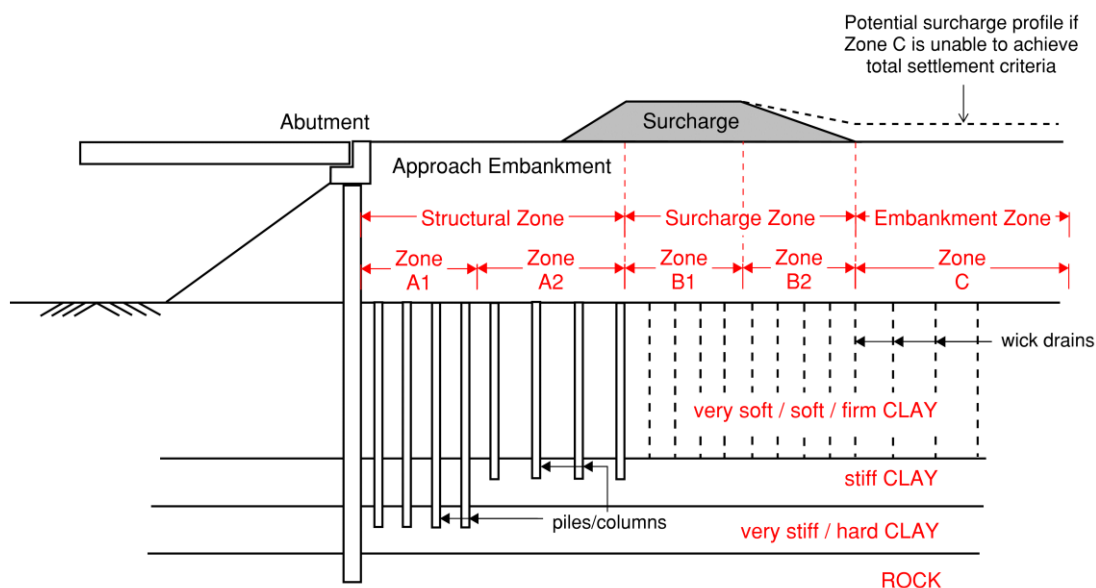


Figure 1: Typical arrangement of transition treatment

The three zones of transition treatment include the following foundation treatments:

- Zone A (Structural Zone): Foundation treated with piles/columns which provide full support to the overlying embankment. Zone A is divided into Zone A1 and Zone A2, as follows:
 - A1: Piles/columns extend to a competent soil layer, e.g. very stiff to hard clay, dense to very dense sand or rock. Negligible settlement is expected within this zone.
 - A2: Piles/columns extend to a less competent soil layer, e.g. stiff clay or medium dense sand, and not weaker. Minimum settlement is expected within this zone.
- Zone B (Surcharge Zone): Preloading and surcharging (with/without wick drains). Zone B is divided into Zone B1 and Zone B2, as follows:
 - B1: Maximum preload/surcharge height is constructed to a uniform level. Maximum surcharge is used to achieve a high over-consolidation ratio (OCR), in order to reduce long term creep settlement.
 - B2: Preload/surcharge heights are constructed at progressively reduced levels away from the bridge abutment. The purpose of this treatment is to achieve a varying OCR value in soft soil to achieve a gradual transition of creep settlement.
- Zone C (Embankment Zone): Preloading (with/without wick drains) at final embankment level. No reduction in creep settlement is attempted with surcharging if the long term total settlement criterion is met.

Zone A treatments are also often used as a “buffer zone” to reduce the impact of ground movement (settlement and lateral movement) on abutments, such that the abutment piles can be installed early during construction.

Consolidation settlement in Zones B and C is expected to be completed during the preload/surcharging period, i.e. the long term settlement comprises only creep settlement.

Variations to the typical transition treatment arrangement should be noted, including:

- The above arrangement can be modified and adjusted, provided the differential settlement criterion is met.
- In the event where a Structural Zone is not provided to form a buffer zone for the early construction of bridge abutment piles, preload/surcharge transition treatment can be utilised which requires abutment construction to be delayed. This process involves filling the approach embankments to the final surface or surcharge level, inclusive of the bridge abutment zone. Construction of the piled abutment is placed on hold until consolidation settlement is complete, or when the residual settlement is reduced to acceptable levels (Hsi et al., 2013). The fill placed in the abutment zone is removed prior to installation of the bridge piles and headstock. This solution is best suited for situations where the construction schedule allows piling works to be undertaken later in the construction program, and independent of the project critical path – as there are inherent difficulties in predicting consolidation settlement durations.
- Surcharging for the general embankment zone (Zone C) may still be required if the total long term settlement criterion cannot be met.

4 TREATMENT SOLUTIONS

Within each of the treatment zones, different structural elements are used to achieve the required support and settlement function. Refer to Hsi (2016) for available ground treatment options. The following sections summarise the commonly used elements within each treatment zone, and the means by which these elements achieve the desired treatment outcome.

Zone A – Structural Zone

Examples of elements used within Zone A include “rigid” inclusions, such as, pre-cast concrete piles, timber piles, CFA columns, concrete injected columns (CIC’s), and deep soil mixing (DSM) columns. These elements possess the structural rigidity to fully support and transfer all embankment loads to the competent soil/rock layer at depth, whilst ensuring minimal load transfer to the softer, highly deformable stratum. The authors recommend that piles/columns utilised in Zone A are socketed in stiff clays or medium dense sands, as a minimum. The successful performance of these rigid elements ensures that negligible to minimal settlement is experienced within Zone A.

Installations are formed in a regular grid pattern, which effectively improves the strength and reduce the compressibility of the treated ground mass. With these elements, settlement magnitudes are significantly reduced and the embankment is allowed to be constructed rapidly, without concern for undrained failure of the soft soil (Hewlett and Randolph, 1988). Inclusions have the added benefit of protecting the abutment piles from lateral loads imparted by lateral movement of soft soils as they settle beneath the neighbouring embankment founded upon untreated ground; and must also be arranged

strategically to achieve an acceptable degree of transitioning between the embankment and piled abutment. These treatments are normally attempted to be minimised due to their high cost, in areas where settlement criteria are particularly stringent, or when scheduling pressures do not allow adequate time for consolidation to take place.

Zone B – Surcharge Zone

Typical examples of elements used within Zone B include “semi-rigid” inclusions such as stone columns, sand columns, and “soft” inclusions such as wick drains, and vacuum consolidation, in conjunction with preloading and/or surcharging. While semi-rigid and soft inclusions can be used as standalone transition treatment solutions, they are often used as a complimentary measure for other more intensive forms of transition treatment. These treatments used at the interface between the zone of rigid inclusions (Zone A) and the unsupported embankment (Zone C).

With the exception of stone columns and sand columns, which provide partial support to the embankment, the primary function of elements in Zone B is to accelerate settlement of the soft soil, by applying overburden loads in excess of the final embankment profile during construction. This will force consolidation settlement to occur quickly. Prior to installation of the pavement and structures, the excess overburden can be removed, thus ensuring only minimal residual settlement remains between the embankment and piled abutment (Hsi and Martin, 2005). Surcharging of the soft soil can be used to over-consolidate the ground, and minimise creep settlement magnitudes (see Section 10.3 for details).

Treatments in Zone B are arranged such that the preload/surcharge overlaps with the rigid inclusions to provide a smoother transition. Details of how preload/surcharge with wick drains was used successfully in conjunction with rigid inclusions from the Yelgun to Chinderah (Y2C) Freeway project is shown in Figure 2, (Hsi and Martin, 2005).

Zone C – Embankment Zone

Zone C essentially forms the unsupported embankment zone whereby the soft soil is subject to the full impact of the overburden load. Treatments used in this zone are kept to a minimum, whilst ensuring the total settlement criteria are met. If long term total settlement criteria are expected to be exceeded, preloading with wick drains is commonly used to accelerate settlement prior to construction of the road pavement. Surcharging can also be used to minimise creep settlements, in the event total settlement magnitudes in the long term are excessive.

Surface Treatment Options

To further prevent sharp changes in settlement between rigidly supported and unsupported embankments, geotextile reinforced mattresses can be installed below the pavement across Zones A and B. This treatment can be implemented post preloading/surcharging and prior to pavement construction. This is used as a complimentary measure to more intensive forms of transitional treatment, and cannot be used as a standalone solution. On the Yelgun to Chinderah Freeway (Y2C) project in NSW, this mattress was formed using geotextile reinforcement sandwiched within a rock mattress of 900mm thickness (Hsi and Martin, 2005). A schematic arrangement of the Y2C bridge approach treatment is shown in Figure 2.

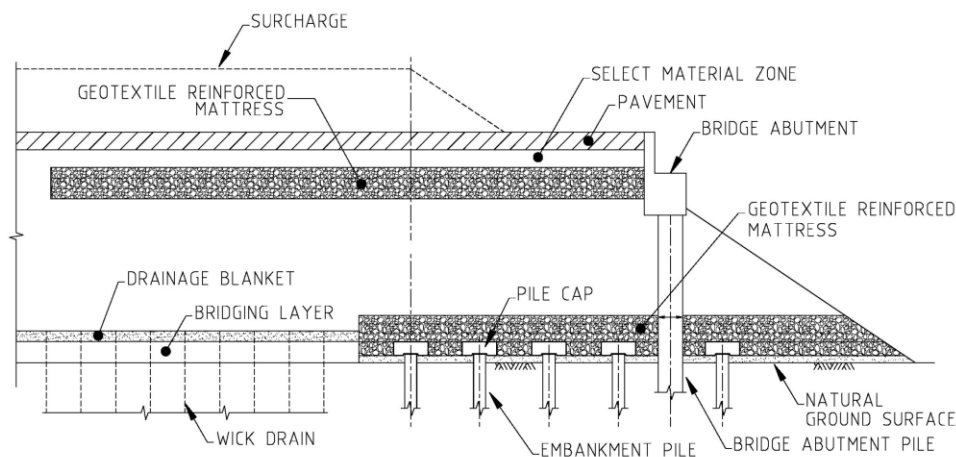


Figure 2: Bridge approach transition zone in soft soils for Y2C (Hsi and Martin, 2005)

5 ISSUES RELATING TO TRANSITION TREATMENTS

The sections below discuss some specific considerations that are often incorporated into transition treatment design and construction.

- **Pile/column arrangement in the Structural Zone.** There are many documented instances of transition treatments utilising “floating” piles/columns in the Structural Zone, which adopt gradually reducing pile/column lengths away from the piled abutment, to achieve a gradual increase in settlement. The piles/columns are often designed to terminate within soft to firm clay. This solution contrasts with the authors’ suggestion to terminate all piles/columns in Zone A within stiff clay or better (as detailed in Section 3). Section 6 provides discussion on the potential pitfalls of the “floating” pile/column system versus the authors’ recommended solution.
- **Impact of lateral soil movement on abutment piles.** Embankments founded on soft soil will induce both vertical and lateral deformation in the foundation. Abutment piles located adjacent to settling embankments will be subjected to these foundation movements, resulting in induced bending moments and shear forces. Section 7 will discuss the methodologies used to determine the magnitude and distribution of forces for piles in this situation; and also how piles/columns utilised for transition treatment can be used to create the “buffer zone” mentioned in Section 3, to eliminate the impact of the ground deformation on the abutment piles.
- **Design methodologies for load transfer platforms (LTP).** LTP’s provide a load distribution function, by effectively bridging between rigid inclusions installed within the Structural Zone, and distributing overburden embankment loads onto the rigid inclusions beneath.

Among the many competing design methodologies for LTP’s, the main point of difference lies in the quantum of resistance provided by the foundation soil between the rigid inclusions to support the overburden loads. Based on the varying design philosophies, the tensile loads to be sustained by the LTP reinforcement will differ significantly, and axial loads on the rigid inclusions will differ as a result. Section 8 discusses the interaction theories within piled embankments proposed by numerous researchers and industry codes, and the corresponding methods used to ascertain the forces induced within LTP reinforcement.

- **Measurement of differential settlement.** The performance of a transition treatment is measured by the achievement of the design differential settlement criteria. This is often relayed by stipulating a maximum change-in-grade (CIG) criteria for the finished pavement surface. The means, by which CIG is measured, is often not well defined. Section 9 provides commentary on the potential source of the CIG criteria adopted in industry today, and how this is measured for current transition treatment design applications.
- **Creep settlement.** The assessment of creep settlement plays a crucial role in transition treatment design, as creep deformation will form the majority of the settlement experienced by the approach embankment and road pavement during its serviceable life. Section 10 provides discussion on the following creep related topics:
 - Creep settlement assessment methods. There are two competing theories on creep settlement in the industry, which argue that the commencement of creep settlement – either at the commencement of primary consolidation or at the completion of primary consolidation. The point at which creep settlement commences drastically affects the calculated magnitude of creep, and the necessary treatment methods within the transition treatment.
 - Creep reduction methods. The use of surcharging to reduce creep magnitudes is well documented and used frequently in industry. Documented examples of creep magnitude based on achieved OCR from surcharging are provided in Section 10.3.

6 PILING ARRANGEMENT IN THE STRUCTURAL ZONE

To produce a gradual settlement transition, inclusions used for foundation treatment are installed at varying intensities and frequencies beneath approach embankments. By increasing the spacing and/or reducing the inclusion lengths progressively away from the piled abutment, the support provided by the inclusions is gradually reduced, allowing settlements of the embankment to transition from negligible displacements at the piled abutment, to maximum settlements beneath the unsupported embankment.

Typical arrangements in the Structural Zone utilise a gradual reduction in the pile/columns lengths, which progressively shorten away from the pile abutment. In an ideal condition, the thickness of soft soil beneath the inclusions should gradually increase, resulting in increased settlement, as illustrated in Figure 3.

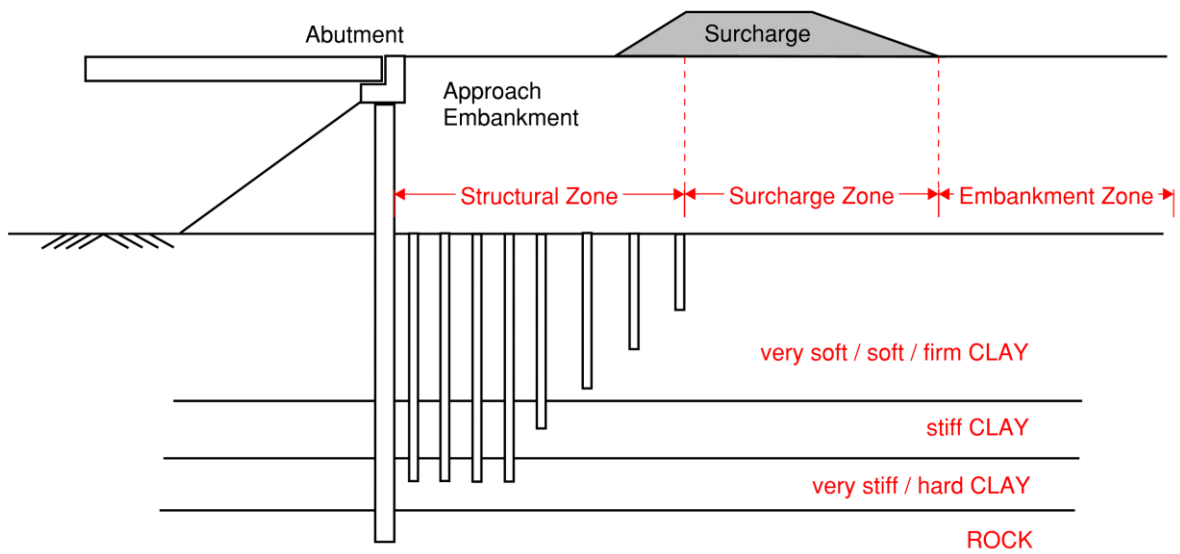


Figure 3: Typical transition treatment illustrating progressively shortened inclusions away from the pile abutment

However, this design may be flawed as it assumes uniformity (or linearity) of the various soil horizons. By assigning a pre-determined inclusion length based on the assumed ground profile, there may be potential for larger or smaller thicknesses of soft soil to exist beneath the inclusions, other than that intended by the designer. The variability of the soft soil thickness may cause the embankment to settle at a different magnitude than predicted.

Rather than reducing the inclusion lengths away from the pile abutment, the authors prefer a different arrangement in the Structural Zone – by placing the inclusions at increasing offsets, i.e. centre-to-centre spacing between the inclusions, which are founded into competent stratum (stiff clay or better) below the soft soils. The increased offsets between the inclusions have the desired effect of reducing the overall support to the foundation, thereby generating a transitional settlement profile along the approach embankment. The advantage of this alternative, is that the inclusions can be installed relatively confidently into competent ground stratum, based on the construction resistance encountered during installation. This solution can overcome the risks associated with shortening the inclusion lengths, particularly in variable ground profiles which have non-uniform thicknesses of soft soil. Use of increasing inclusion spacing was implemented successfully on the Brunswick Heads to Yelgun (BH2Y) Freeway project in NSW (Hsi et al., 2008), as illustrated in Figure 4.

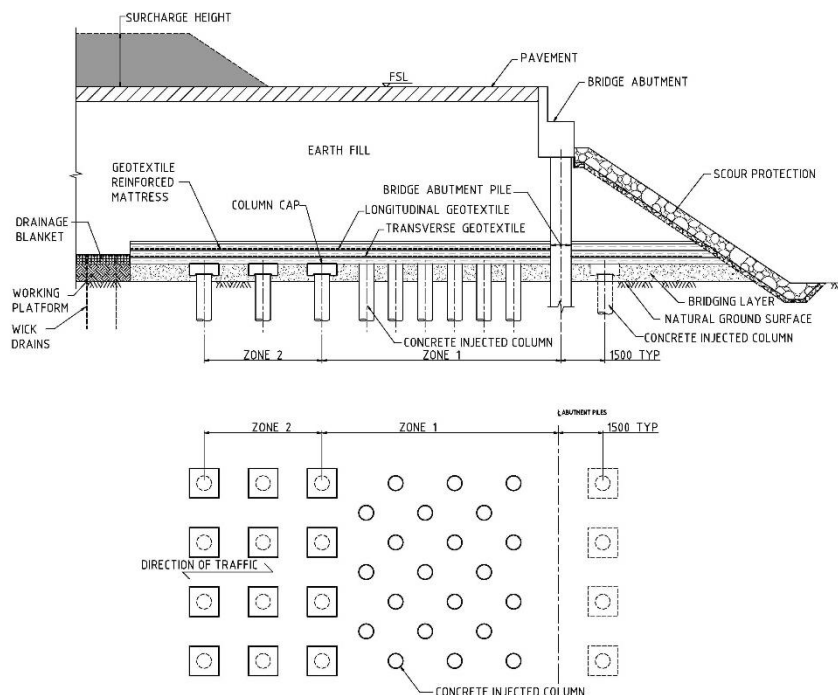


Figure 4: Bridge approach transition zone in soft soils for BH2Y (Hsi et al., 2008)

7 LATERAL SOIL MOVEMENT

7.1 LATERAL SOIL MOVEMENT ASSOCIATED WITH SETTLEMENT

Settlement of soft soil is often accompanied with lateral extrusion. In situations where embankments on soft ground are settling adjacent to bridge abutment piles, the lateral soil movement will induce bending moments and deflections in the piles, as shown in Figure 5. In severe cases, this can cause structural damage to the piles and bridge structure. Because of this, it is not uncommon for designers to incorporate caissons to shield the abutment piles from lateral soil displacements, or delay pile installation until settlement of the embankment is almost complete (Stewart et al., 1994)

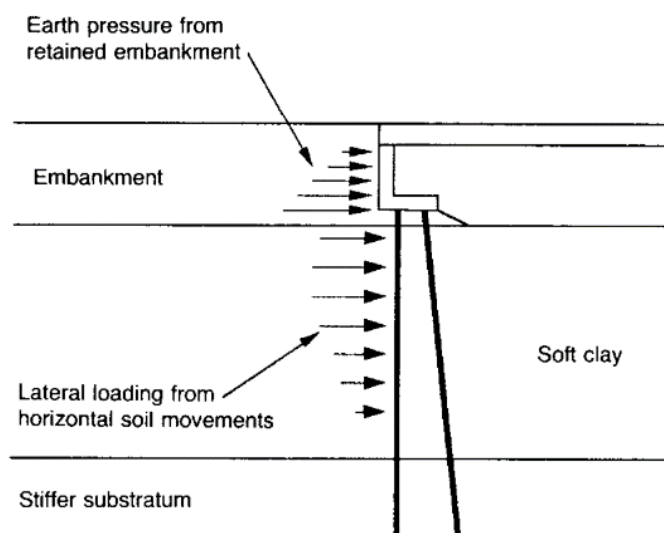


Figure 5: Bridge abutment piles subject to lateral loads due to settling embankment (Stewart et al., 1994)

Alternatively, the piles can be designed to support the lateral loads, by ensuring the piles are designed for the bending moments and deformations induced by the lateral ground movement. For this assessment, the “free-field” lateral deformation at the pile location can firstly be ascertained. “Free-field” (or “greenfield”) lateral deformation pertains to the ground movement assuming the piled structure is not present. This ground deformation can be ascertained using numerical or empirical methods

In order to calculate the induced bending moment in the piles, software incorporating soil-to-structure interaction and structural deformation prediction capabilities due to soil deformation can be used. Such programs include PALLAS (Hull, 1998) and LPILE (Ensoft, 2016), which require the soil movement profile as input. The bending moment profile is calculated based on the stiffness’s of the pile/column and relative movement of the soil. Alternatively, analytical solutions can be used to simulate soil-to-structure interaction for single pile response, such as that presented by Goh et al. (1997).

7.2 USE OF EMBANKMENT PILES/COLUMNS TO REDUCE LATERAL MOVEMENT

In lieu of designing abutment piles to resist lateral ground movements, a “forest-of-piles/columns” installed within the Structural Zone, can provide a “buffer zone” to shield the bridge abutment piles from lateral ground movements, as shown in Figure 4. By improving the overall rigidity of the soft foundation, the rigid piles/columns provide a protective barrier around the abutment piles, preventing them from the impact of ground settlement or lateral soil movement (Hsi et al., 2008).

The successful use of the Structural Zones in the Brunswick Heads to Yelgun Freeway project, shown in Figure 4, eliminates the impact of embankment settlement on the abutment/structure piles and minimises the differential settlement at the interface between the piled structures and non-piled embankments. In addition to the piles/columns incorporated into the Structural Zone, a geotextile reinforced mattress was placed over the CIC’s to transfer the embankment loads onto the CIC’s. These CIC’s were designed to carry the full embankment load and hence no significant ground settlement would occur due to compression of the soft ground. Consequently, lateral loads on the piles were also significantly reduced.

The bridge approach analyses involved modelling of surcharging, geotextiles, CIC's, piles, which were complex systems with interaction between all the working components. Numerical modelling software PLAXIS (PLAXIS, 2002) was adopted to analyse the effectiveness of the system. In order to calculate the induced bending moment into the piles, the soil-structural interaction program PALLAS (Hull, 1998) was used, which required the soil movement profile at the pile location as input. The free field soil displacements at the location of the piles as predicted by PLAXIS were used as input to PALLAS. From the entered displacement profile, the PALLAS program then calculated the bending moment profile based on the stiffness of the pile.

Figure 6 shows the exaggerated deformed mesh from PLAXIS simulating the embankment during construction (Hsi et al., 2008). It can be seen that CIC's in Zone 1 (refer to Figure 4) were modelled as a block (dark shaded blue area in Figure 6). This assumption was achieved by prevention of "soil flow" around the piles. As the CIC spacing was small in Zone 1, soil movement between the rigid inclusions would be unlikely to occur. It can be seen that the Zone 2 (refer to Figure 4) CIC's provided an effective transition zone between the hard response of the Zone 1 CIC's and the soft response of the general embankment. During construction, measured lateral deflection in the soil surrounding the abutment piles was small (up to 12mm), proving the success of this treatment.

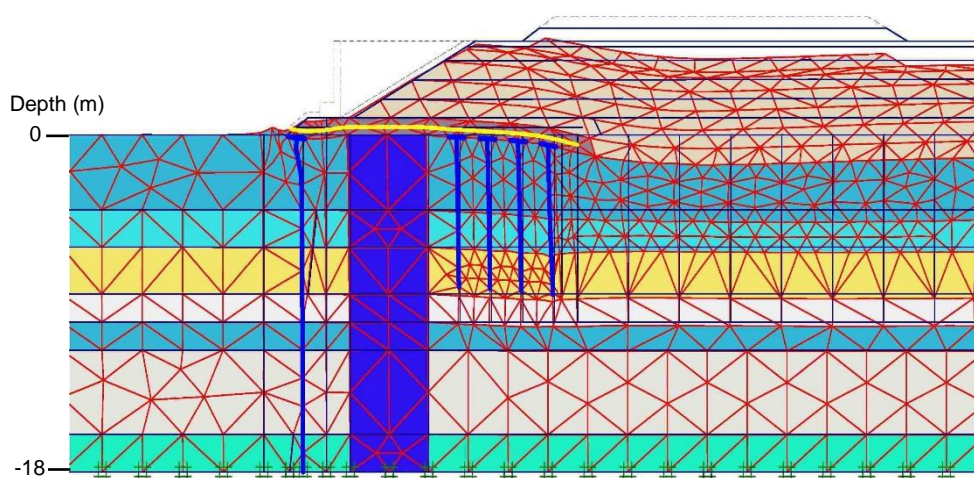


Figure 6: Deformed mesh under embankment loading during construction (Hsi et al., 2008)

7.3 STRUCTURAL INTEGRITY OF UNREINFORCED COLUMNS SUBJECTED TO LATERAL SOIL MOVEMENT

In the event CIC's are utilised within the Structural Zone, careful assessment of their structural integrity is required due to their limited bending capacity in the absence of steel reinforcement in CIC's. Figure 7 illustrates a purely elastic concrete column subject to vertical and lateral loads.

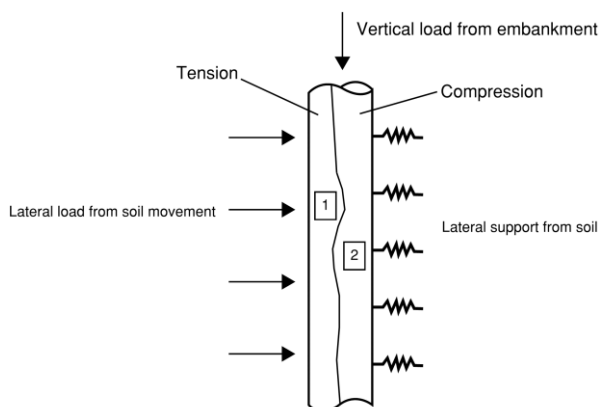


Figure 7: Stresses generated in CIC subjected to vertical and lateral loads

When a concrete column is subject to the above actions, the column will deflect, generating compression on the inner face (embankment side) of CIC; and tension on the outer face, as shown in Figure 7. Considering an infinitesimally small element on the tension face, as shown in Figure 8 – if the major principal stress σ_1 is greater than the factored tensile

capacity of concrete, the concrete will crack (Park & Paulay, 1975). The same philosophy applies for elements in the compression face of the CIC, as shown in Figure 9. If the major principal stress σ_1 is greater than the allowable Unconfined Compressive Strength (UCS) of concrete, the concrete will fail in compression due to shear failure.

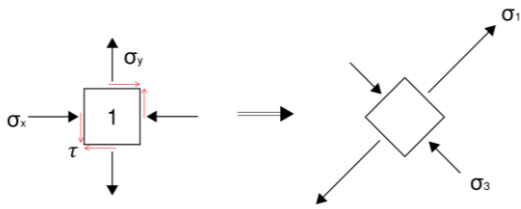


Figure 8: Free body diagram on infinitesimally small element on tension face of the CIC's

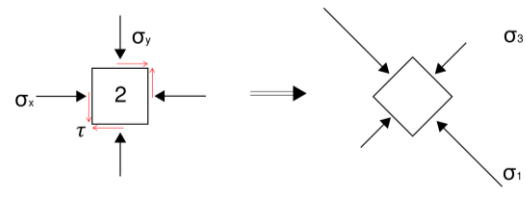


Figure 9: Free body diagram on infinitesimally small element on compression face of the CIC's

To maintain the structural integrity of the CIC's subjected to lateral soil movement, the authors suggest that low or zero tension cut-off is assumed for the CIC's in the numerical and/or structural analysis. This assumption ensures that the CIC's do not carry undue tension loads and hence maximum compression loads can be calculated.

8 LOAD TRANSFER PLATFORMS

As mentioned in Section 4, rigid inclusions are often used in the Structural Zone to produce a pile/column supported zone. This system is typically referred to as a “piled embankment”. Due to the high cost of rigid inclusions, the spacing between these elements is often maximised with spacing ranging generally between 1.3m to 2.5m, and inclusion diameters ranging between 300mm to 600mm. Due to the large spacing, rigid inclusions are commonly complimented with a “load transfer platform”, or LTP, to effectively “bridge” between the rigid inclusions and help evenly distribute the overburden loads onto the inclusions. LTP's comprise a gravel mattress embedded with layers of geosynthetic reinforcement. LTP's are positioned to sit directly above the rigid inclusions, at the interface with the earthen embankment. Typical examples of LTP's are illustrated in Figure 2 and Figure 4, where the LTP's are referred to as “geotextile reinforced mattresses”.

The interaction between the embankment, LTP, foundation soil, and rigid inclusions is a complicated phenomenon, and has been the subject of many studies and research. The ubiquity of piled embankments has spawned numerous guidelines and codes covering the design of the LTP's and the rigid inclusions. However, due to the complex interaction between the various components, the design of piled embankments still has elements of ambiguity, resulting in inconsistency of design outcomes in the industry today. In recognition of such design variability, this paper discusses various approaches used in Australia, and suggests one that is based upon the relative displacement of foundation soils to the LTP's.

Among the numerous design methodologies exist within the industry, the main point of difference lies in the quantum of resistance provided by the foundation soil between the rigid inclusions to support the overburden loads. Based on the varying design philosophies, the tensile loads to be sustained by the LTP reinforcement will differ significantly. The sections below detail the interaction theories within piled embankments proposed by researchers and industry codes, and the corresponding methods used to ascertain the forces induced within the LTP reinforcement. Refer to Li and Hsi (2016) for determination of geosynthetic reinforcement strength requirement.

8.1 TRADITIONAL LTP DESIGN APPROACH AND BEHAVIOUR

Traditional design philosophies for LTP analyses assume that foundation soils between the rigid inclusions do not provide support to the embankment, i.e. all of the overburden load is assumed to be transferred to competent layers at depth via the rigid inclusions. The LTP's function as a bridging layer to facilitate the distribution of the overburden load onto the rigid inclusions. This methodology is adopted in the British Standard, BS8006:2010, Code of Practice for Strengthened/Reinforced Soils and Other Fills. The theory behind this philosophy is that the soft foundation soils may settle over time, and separate from the underside of the embankment resulting in a reduction of support.

- **BS8006:2010 Method 1 – Marston's formula**

Marston's formula was originally developed for the purposes of determining the magnitude of loads acting on buried conduits (Spangler and Handy, 1973). According to Marston, the loads above a buried conduit can be simulated as a soil column. In situations where the ground around the soil column settles a greater amount compared with the buried conduit, the load on the conduit increases due to an “inverted arch action” in the overburden soil. In recognition of the similarities between unyielding rigid inclusions beneath an embankment

with buried conduits, the outcomes of Marston's solutions were adapted for rigid inclusions beneath embankments in BS8006:2010.

Based on Marston's formula, the distribution of overburden pressures does not account for the arrangement and spacing of rigid inclusions, as this formulation was derived for plane strain conditions simulating an infinitely long conduit. In the author's opinion, this aspect of Marston's formula diminishes its validity towards simulating the behaviour of LTP's.

- **BS8006:2010 Method 2 – Hewlett and Randolph, 1988**

The second reinforcement design method in BS8006:2010 is the more commonly used approach, developed by Hewlett and Randolph (1988). Hewlett and Randolph (1988) undertook a series of scaled model tests in the laboratory simulating an embankment over a rectangular grid of rigid inclusions. The models indicated that the inclusions provided support to the embankment via arching of the free-draining, granular embankment fill. The analytical solution which ensued from the laboratory experiment, assumed a region of arched embankment fill above the rigid inclusions. The arches formed a series of domes – the culmination of which create a vault-like structure.

An estimate of the proportion of embankment load taken directly by the inclusions, was ascertained based on the failure conditions of the soil arches. The proportion of the embankment load which was not supported by the inclusions was then considered to act directly on the subsoil, and thus equated to the vertical load taken by the LTP reinforcement.

8.2 ALTERNATIVE LTP DESIGN METHODS

Contrary to conventional methods such as Hewlett and Randolph (1988), there are numerous published design methods which consider partial support offered by foundation soils between the inclusions. By considering the contribution of these soils, a consequential reduction in the load carried by both the reinforcement and rigid inclusions may eventuate, potentially resulting in leaner, more optimised ground treatment solutions. BS8006:2010 cautions against these methods, warning that long term consolidation of the foundation soil may result in loss of contact with the underside of embankment or LTP.

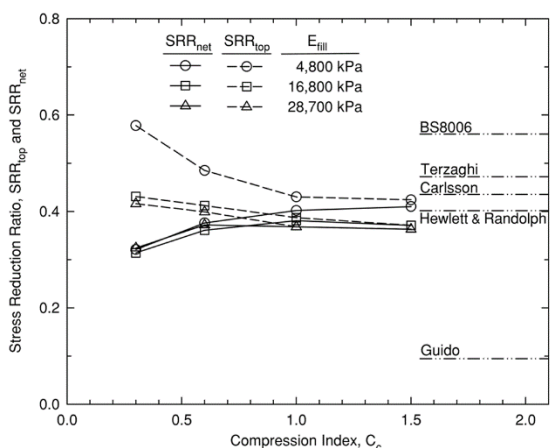
The sections below describe some of the research and philosophes behind published design methods to ascertain the proportion of overburden loads carried by rigid inclusions, while accounting for the support provided by foundation soils.

- **Influence of Clay Compressibility on Geosynthetic Loads in Bridging Layers for Column-Supported Embankments (Stewart and Filz, 2005)**

This paper outlines a parametric study carried out for a piled embankment undertaken using FLAC 3D (Fast Lagrangian Analysis of Continua in 3 Dimensions) by ITASCA 2002. The primary purpose of the study was to evaluate the effects of compressibility in the foundation soil on the proportion of load transferred to the rigid inclusions. The numerical model was verified against measurements taken from an instrumented test embankment in Virginia, USA. The results of the parametric analysis are shown in Figure 10.

Based on the results of Stewart and Filz's study (2005), it appears that foundation soils of higher compressibility, i.e. higher compression index, C_c , result in reduced loads on top of the LTP reinforcement, i.e. decrease in SRR_{top} , and increase in net loads on the reinforcement, i.e. increase in SRR_{net} . The decrease in downward load on the reinforcement can be explained by the rigid inclusions attracting a greater proportion of the load due to its increasing comparative stiffness to the compressible ground. The increase in SRR_{net} is due to the reduction in supportive reaction provided by the compressible soils (see Figure 10 for definitions of SRR_{top} and SRR_{net}).

As the foundation soil becomes more compressible, the numerical solutions converge towards the analytical outcome derived by Hewlett and Randolph (1988), as shown in Figure 10. This suggests that Hewlett and Randolph's method (1988) is numerically accurate when the foundation soils are highly compressible, such that they provide minimum support to the embankment.



SRR_{top} = vertical load on top of reinforcement (in area outside the pile cap), normalised by weight of embankment fill overlying reinforcement

SRR_{net} = net vertical load (downward load on top of the geosynthetic minus the upward load on the bottom of the geosynthetic) in the area outside the pile cap, normalised by the weight of the embankment fill overlying the reinforcement, i.e. net vertical load felt by the reinforcement.

Figure 10: Results of parametric numerical analysis for varying foundation soil compressibilities (Stewart and Filz, 2005)

• **Varaksin et al. (2014)**

Varaksin et al. (2014) summarised the analytical solutions documented by the French nationwide research program ASIRI (Soil Improvement by Rigid Inclusions) to determine vertical pressures on rigid inclusions. According to Varaksin et al. (2014), ASIRI considers two possible failure modes of the LTP. Based on the realisation of either of these two cases, the pressure at the top of the inclusions can be determined.

- Failure mechanism 1: Prandtl's failure mechanism, as shown in Figure 11. This bearing capacity-type mode of failure is postulated to occur in LTP's overlain by rigid structural elements such as a concrete slabs, raft or footings, or when the embankment thickness is sufficient to avoid punching failure.
- Failure mechanism 2: Punching failure through the LTP corresponds to a shear cone forming within the LTP. This is particularly applicable for light overburden pressures.

Based on equations provided by Varaksin et al. (2014), q_p^+ can be determined from q_s^+ , q_0 and the intrinsic parameters of the LTP. The maximum load attracted to the head of the inclusion is a function of inclusion spacing, shear strength of the LTP and the in-situ soil. The validity of these equations has been verified with physical modelling using centrifugal testing by ASIRI which reported good agreement with the nominated equations and recorded test measurements. ASIRI also undertook verification using numerical analyses for which a reasonable degree for compatibility was found.

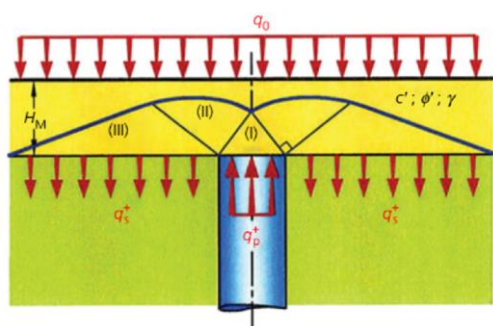


Figure 11: Prandtl's failure mechanism for thick embankments (after Varaksin et al., 2014)

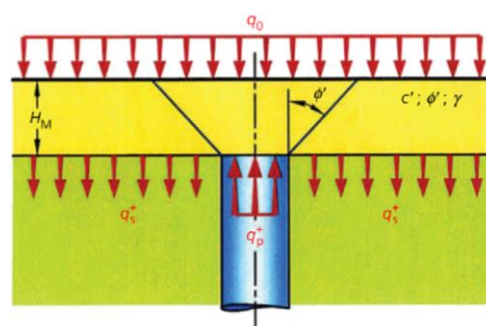


Figure 12: Punching failure for thin embankments (after Varaksin et al., 2014)

Note:

q_p^+ denotes the stress at the inclusion head

q_s^+ is the stress on the in-situ soil.

q_0 is the uniformly distributed load applied to LTP

8.3 RECOMMENDED LTP DESIGN PROCEDURE

Based on the design philosophies discussed above, the design of LTP's is heavily dependent on the predicted settlement behaviour of the soft soil between the rigid inclusions:

- If the foundation soil is expected to settle significantly relative to the LTP, the rigid inclusions need to be designed to transfer the full quantum of the embankment loads to competent ground at depth; and the LTP will be used to bridge between the inclusions. Solutions such as those documented in BS8006:2010 should be used.
- If the foundation soil is expected to maintain contact with the underside of the LTP over the design life of the structure, load sharing between the LTP and the soil can be considered. Numerical modelling can be used to assess the quantum of support provided by the rigid inclusions, LTP and the soil.

Whether the foundation soil will provide support to the LTP needs to be assessed by the designer.

9 DIFFERENTIAL SETTLEMENT

The success of the transition between an approach embankment and a piled abutment is typically governed by the achievement of pre-determined differential settlement criteria. On many of the past projects in Australia, this is typically stipulated as a maximum “change-in-grade” (CIG) of 0.3% for rigid (concrete) pavements and 0.5% for flexible pavements.

The 0.3% and 0.5% CIG criteria bears a close approximation to the grade change criteria stipulated in Austroads (2016), and the RTA Road Design Guide (1989), which contain differential settlement criteria for varying vehicle speeds, as shown in Table 1.

Table 1: Maximum change-in-grade for roads (reproduced from Austroads, 2016, and RTA Road Design Guide, 1989)

Design Speed (km/h)	Maximum Grade Change without Vertical Curve (%)
40	1.0
60	0.8
80	0.6
100	0.4
120	0.2

According to the Wooldridge et al. (1997), this criterion appears to have been developed based on passenger comfort while navigating a vertical curve in sag. By definition, vertical curves are required in the vertical alignment of a road to transition from one straight grade to the next (Austroads, 2016). The nomenclature used to define vertical curves are illustrated in Figure 13.

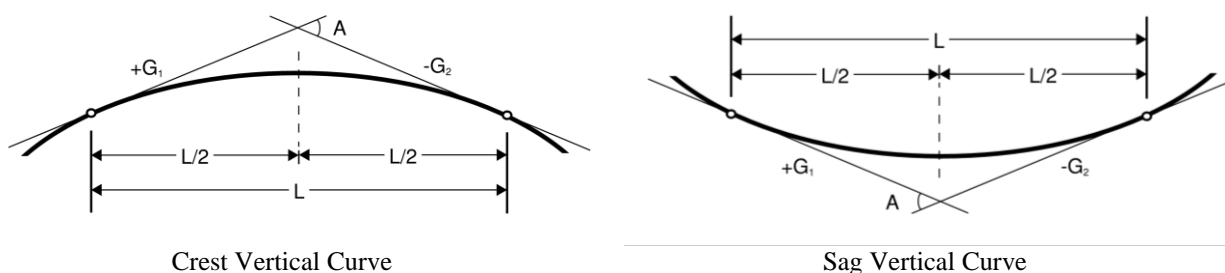


Figure 13: Crest and sag vertical curve nomenclature (adapted from AASHTO, 2001; Austroads, 2016; and RTA, 1989)

Note:

A= algebraic difference in grade (%)

L= length of vertical curve (m)

G1, G2 = tangent grades (%)

Passenger comfort is linked to the maximum degree of radial acceleration experienced by the passengers. While there is no general agreement on the radial acceleration that can be tolerated without producing discomfort, AASHTO (2001)

suggests a value of 0.3 m/s². The equation developed by AASHTO (2001) for vertical curve length corresponding to this control is shown in Equation 1.

$$L \geq \frac{AV^2}{46.5} \tag{1}$$

Note:

- L = length of vertical curve, ft
- A = algebraic difference in grades, %
- V = design speed, miles per hour

Equation 1 has been shown by road authorities in the United States, such as the Ohio Department of Transportation (ODOT, 2010), to correlate to the following maximum CIG's, shown in Table 2. The corresponding design speed in kilometres per hour (km/hr) has been provided for reference.

Table 2: Interpretation of maximum CIG based on passenger comfort criteria (adapted from ODOT, 2010)

Design Speed V (mph)	Approximate Design Speed (km/hr)	Grade Change A (%)
25	40	1.85
40	60	0.75
50	80	0.45
60	100	0.30
70	110	0.25
75	120	0.20

All grade changes, A, have been determined assuming a vertical curve length L of 25 feet (i.e. 7.6m)

For a design speed of 110 km/h, the corresponding grade change (A) is approximately 0.3% – similar to that adopted on numerous Australian road projects designed to 110 km/h (refer to Table 3).

Throughout the transition area, due to the various treatment arrangements, the curvature of the predicted pavement profile may vary substantially within a short distance, making it difficult to assess the grade change using the definition as shown in Figure 13. From the practical point of view, change-in-grade for the transition treatment assessment is determined as the change of two adjoining slopes, as shown in Figure 14 and Equation 2. The quantification of CIG is shown in Equation 3.

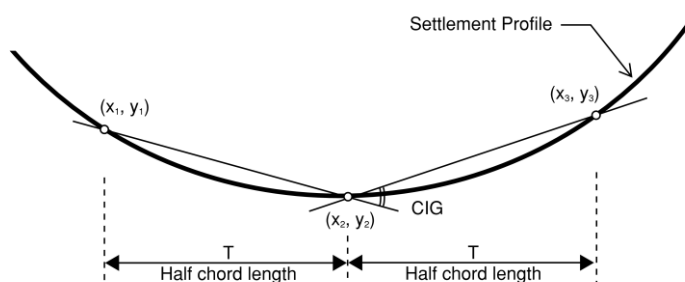


Figure 14: Authors' suggested measurement of change-in-grade, CIG

$$CIG = \left(\frac{y_1 - y_2}{x_1 - x_2} - \frac{y_2 - y_3}{x_2 - x_3} \right) \times 100\% \tag{2}$$

The distance over which CIG is measured, is typically referred to as the half chord length, T, as defined in Equation 3. The value of T on recent projects the authors have partaken in, has generally fallen between 5m to 10m. This coincides with the assumed L of 25 feet (7.6m) used in the derivation of the grade change criteria shown in Table 2.

$$T = |x_1 - x_2| = |x_2 - x_3| \tag{3}$$

The authors note there are differences between the measurement of CIG presented in Figure 14 and that dictated by the vertical curves defined in AASHTO (2001), Austroads (2016) and RTA (1989), from which the passenger comfort criteria are derived, as shown in Figure 13. However, it should be noted that the CIG criteria presented in Table 1 and Equation 1, are road design criteria for which vertical curves, and curve lengths are designed specifically to achieve a change-in-grade – and not to assess change-in-grade resulting from differential settlement over a short length of the roadway. It appears road authorities have *adapted* road design criteria to allow enforcement of geotechnical design limitations. Accordingly, the authors consider the use of secant lines to connect points on the settlement curve to be a practical means of establishing the settlement profile, upon which the CIG can be determined. It is the authors' experience that this definition of CIG has been used in the geotechnical industry for the past 25 years and has attracted little debate.

Examples of differential settlement criteria for existing road projects in Australia are provided in Table 3. Similar values appear to be adopted overseas, as summarised in Table 4.

Table 3: Summary of differential settlement criteria for selected projects in Australia

Source	State	Pavement Type	Prescribed Criteria				
			T (m)	D _{max} (mm)	CIG (%)	R (m)	Design Speed (km/hr)
TMR (2015)	QLD	Concrete	5	50	0.3	(1667)	-
TMR (2015)	QLD	Flexible	-	-	0.5	-	-
RMS (2013)	NSW	Concrete	5	-	(0.3)	1700	-
Yelgun to Chinderah Freeway	NSW	Concrete	(5)	100-160	0.3	(1667)	110
Tugun Bypass	QLD	Concrete	10	30-200	0.3	(3333)	110
Oxley Highway to Kundabung	NSW	Concrete	10	50	0.3	-	110
Ballina Bypass	NSW	Flexible	(5)	50-100	0.5	(1000)	110
Gateway Upgrade	QLD	Flexible	(5)	50-200	0.5	(1000)	110
Ipswich Motorway Upgrade	QLD	Flexible	10	25-100	0.3	(3333)	110
Peninsula Link	VIC	Flexible	5	50	0.4	(1250)	110

T = Half-chord length (refer Figure 14)
 D_{max} = Maximum allowable post-construction settlement
 CIG% = Maximum allowable change-in-grade (refer to Figure 14 and Equation 2)
 R = Minimum radius of curvature for virtual segment fitted
 () = values inferred in the absence of data
 - = Not available

Table 4 provides a summary of the differential settlement criteria for bridge transition zones suggested by international publications. The values range from 0.4% to 1%, which generally supports the 0.3% to 0.5% criteria adopted in Australia.

Table 4: Summary of internationally adopted differential settlement criteria in bridge transition zones

Publication	Design Criteria	
	D _{max} (mm)	CIG (%)
Hoppe (1999)	60	0.5%
Hearn (1997)	25 - 100	0.4% to 0.8%
Long et al. (1985)	50 -75	0.5%
Stark et al. (1995)	75	0.5%
Wahls (1990)	13	0.4% to 0.5%
Wilk (1981)	-	0.3% to 1%

D_{max} = Maximum allowable post-construction settlement
 CIG % = Maximum allowable change-in-grade (refer to Figure 14 and Equation 3)

10 CREEP SETTLEMENT

Creep is conventionally defined as the settlement of soil under constant effective stress (or zero excess pore water pressure), which continues indefinitely, following a straight line on a strain-log-time plot. This phenomenon is particularly prevalent in soft soils. The hypothetical mechanisms by which creep is manifested in earthen materials includes, among others:

- Rearrangement of soil particles, and
- Visco-plastic behaviour, i.e. delayed shear, such as that observed in viscous fluids

The estimation of creep settlement is particularly relevant for approach embankments transitioning to piled bridge abutments, as areas of the approach embankment without treatment of the soft ground are subject to creep, long after construction is complete. Failure to account for creep accurately may result in differential settlements along the alignment that exceed the allowable transitional criteria.

10.1 CREEP SETTLEMENT THEORIES

Currently, there are two schools of thought on the initiation of creep settlement, both proposed by Ladd et al. (1977), namely,

- Hypothesis A: creep commences after the end of primary consolidation.
- Hypothesis B: creep commences in tandem with primary consolidation.

Using Hypothesis A, the calculation for creep settlement is based on Equation 4.

$$\varepsilon_c = \varepsilon_c^s = c_{\alpha\varepsilon NC} \times \log \frac{t}{t_p} \quad (4)$$

Where ε_c = total creep strain, ε_c^s = creep strain during secondary consolidation, $c_{\alpha\varepsilon NC}$ = normally consolidated creep strain index, t_p = time at end of primary consolidation, t = elapsed time since loading commenced.

For Hypothesis B, the estimation of creep is shown in the Equation 5 where it is a combination of creep strain in the primary and secondary consolidation stages.

$$\varepsilon_c = \varepsilon_c^p + \varepsilon_c^s = \varepsilon_c^p + c_{\alpha\varepsilon NC} \times \log \frac{t}{t_p} \quad (5)$$

Where, ε_c , ε_c^s , $c_{\alpha\varepsilon NC}$, t_p , and t = as per Equation 4, and ε_c^p = creep strain during primary consolidation

Although Hypothesis B assumes that creep occurs concurrently with primary consolidation where excess pore water pressure dissipates with time, this does not necessarily suggest that the accompanying creep is caused by change in effective stress, as the material may undergo visco-plastic deformation.

10.2 LABORATORY TESTING

Both Hypotheses A and B require consolidation parameters $c_{c\varepsilon}$, $c_{r\varepsilon}$ and $c_{\alpha\varepsilon NC}$ as inputs to determine primary and creep settlement magnitudes. Conventionally, these values have been determined from incrementally loaded oedometer (ILO) tests, as per Australian Standard AS 1289.6.6.1 (1998).

ILO tests impose an incrementally larger load on the selected sample to establish the primary consolidation parameters, $c_{c\varepsilon}$, $c_{r\varepsilon}$, subjected to various magnitudes of overburden stress. The creep strain index, $c_{\alpha\varepsilon NC}$, is determined by ceasing to increase the load on the sample and allowing it to settle under constant effective stress.

The authors note that any creep mechanisms, such as the rearrangement of particles or visco-plastic behaviour, that may occur during the primary consolidation period will also be captured in the $c_{c\varepsilon}$ and $c_{r\varepsilon}$ values established during the ILO test.

10.3 CREEP REDUCTION

There are two approaches to reduce the creep settlement, i.e. either reducing the thickness of the clay layer (H) or reducing the creep strain during the secondary consolidation stage (ε_c^s). Since reducing H is usually not possible for clay layers at depth, reducing ε_c^s post construction is commonly the approach taken in practice.

Ladd (1971) and subsequent recent authors (e.g., Terzaghi et al., 1996 and Conroy et al., 2015) have pointed out how the secondary consolidation stage follows a different pattern after the removal of a surcharge preload, as shown in Figure 15. Figure 15 shows that by surcharging, essentially increasing the OCR of the consolidating soil, the creep strain index $c_{\alpha\varepsilon}$ can be reduced.

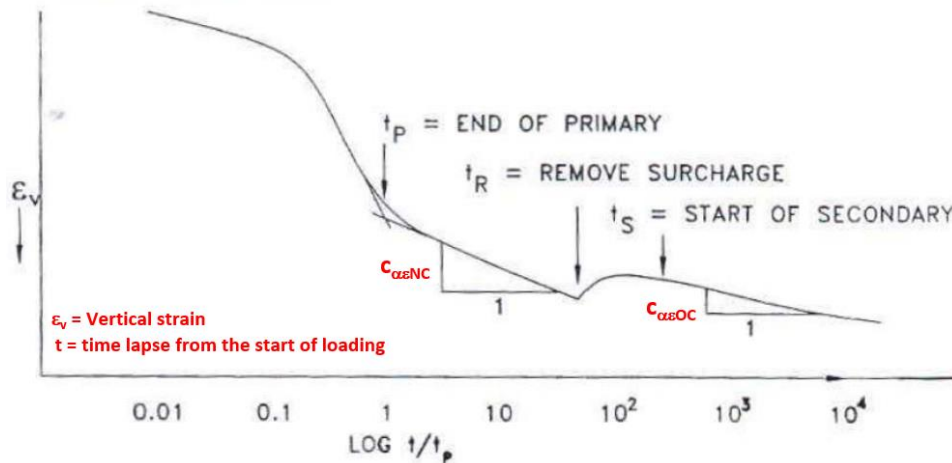


Figure 15: Effects of surcharging on secondary compression (Ladd, 1971)

Mesri et al. (1994) proposed the creep settlement after removal of surcharge ($\delta_{after\ surcharge}$) can be estimated using Equation 6.

$$\delta_{after\ surcharge} = \varepsilon_c \times H = c_{\alpha\varepsilon OC} \times \log \frac{t}{t_s} \times H \quad (6)$$

Where, ε_c = creep strain, H = thickness of soil layer, $c_{\alpha\varepsilon OC}$ = over consolidated creep strain index, t_s = time at start of creep post removal of surcharge, t = elapsed time since loading commenced

The value of $c_{\alpha\varepsilon OC}$ can be estimated using Figure 16. This figure shows the reduced $c_{\alpha\varepsilon OC}$ as a function of adjusted amount of OCR (AAOS), where AAOS = OCR - 1.

10.4 CREEP SETTLEMENT IN PRACTICE

Creep prediction methods following both Hypothesis A and B are commonly used in industry. The sections below summarise common ways in which these theories are applied using parameters established from the commonly available ILO tests, and the limitations of these methods in practice.

10.4.1 Calculations Based on Hypothesis A

Use of the consolidation parameters, $c_{c\varepsilon}$, $c_{r\varepsilon}$ and $c_{\alpha\varepsilon NC}$ parameters established from ILO tests are suitable for the application of conventional settlement and creep assessments, following the formula shown in Equation 7.

$$\delta = H \cdot \left[c_{r\varepsilon} \log \left(\frac{\sigma'_p}{\sigma'_i} \right) + c_{c\varepsilon} \log \left(\frac{\sigma'_f}{\sigma'_p} \right) + C_{\alpha\varepsilon} \log \left(\frac{t}{t_p} \right) \right] \quad (7)$$

Where, σ'_i = initial pressure, σ'_p = preconsolidation pressure, σ'_f = final pressure

Given that primary settlement and creep settlement occur at discrete times, and are calculated separately, conventional calculations follow the Hypothesis A approach to assess creep settlement. Reduction of creep due to over-consolidation can be accounted for using the graphs shown in Figure 15 and Figure 16.

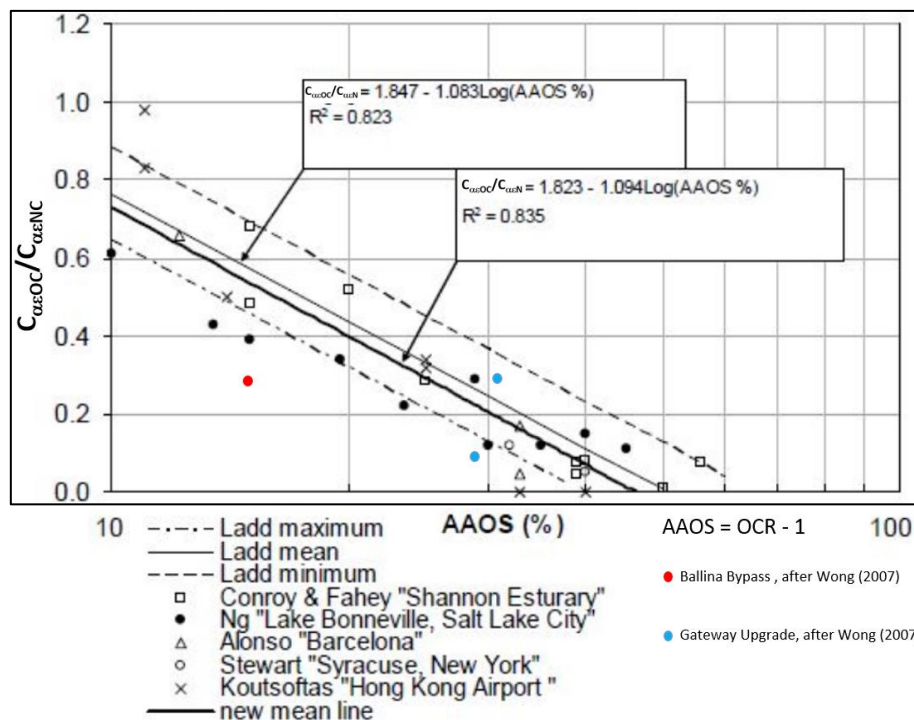


Figure 16: Creep improvement ratio after surcharge (Conroy et al., 2015)

Ladd and DeGroot (2004) suggest the use of Hypothesis A is best suited to over-consolidated, isotropic and rate-insensitive clays, from which the closest settlement predictions have been found using this approach.

10.4.2 Calculations Based on Hypothesis B

Creep calculations based on Hypothesis B can be done using the Soft Soil Creep (SSC) model of PLAXIS. However, use of the parameters, $c_{c\varepsilon}$, $c_{r\varepsilon}$ and $c_{\alpha\varepsilon NC}$, derived from ILO tests, in the SSC model may lead to erroneous results. This is because the measured consolidation rate from the ILO test already includes the creep rate. The summation of $c_{c\varepsilon}$, $c_{r\varepsilon}$ and $c_{\alpha\varepsilon NC}$ during primary consolidation effectively over predict the quantum of consolidation settlement.

More advanced soil testing techniques, such as the Constant Rate of Strain (CRS), published in Wissa et al. (1971) can be used to establish $c_{c\varepsilon}$ and $c_{r\varepsilon}$ values (without creep) which are more compatible with Hypothesis B. The CRS test controls the strain rate applied to the soil sample to prevent excess pore pressures dropping to zero. Accordingly, the $c_{c\varepsilon}$ and $c_{r\varepsilon}$ values established from the CRS test are not expected to include creep strain rate.

According to Ladd and DeGroot (2004) and Vermeer (2005), use of Hypothesis B is best suited to lightly over-consolidated (i.e. OCR less than 2), rate-sensitive, structured and highly anisotropic clays, from which the closest settlement predictions have been found using this approach.

It is noted that although the SSC model of PLAXIS captures both the consolidation and creep settlements concurrently, it is unable to predict reduced creep settlement due to over-consolidation of soil. Consequently, this model will over-predict the post surcharge strain. This is highlighted in recent studies such as Yuan et al. (2015) and Fatahi et al. (2013). The authors are not aware of commercially available software, which can capture creep reduction due to surcharging effects.

11 CONCLUSIONS

Within the Structural Zone, the authors recommend terminating the columns/piles within stiff clay or medium dense sand, as a minimum. Installation of “floating” piles/columns in soft to firm clay or very loose to loose sand have the potential risk of terminating in layers of soft clays of unknown thicknesses – resulting in unexpected settlement magnitudes.

To quantify the impact of foundation deformations on bridge piles, “free-field” lateral deformation at the pile location must firstly be ascertained. Soil-structure interaction programs such as PALLAS (Hull, 1998) and LPILE (Ensoft, 2016) can then be used to determine the bending moment profiles based on the entered soil movement profile. In lieu of

designing abutment piles to resist lateral ground movements, piles/columns installed within the Structural Zone, can provide a “buffer zone” to shield the bridge abutment piles by improving the overall rigidity of the soft foundation.

The conventional design approaches for LTP's assume that foundation soils do not provide support to the embankment – based on the philosophy that soft foundation soils will settle over time, separating from the underside of the embankment. Accordingly, all of the overburden loads are assumed to be transferred to the rigid inclusions. Alternative design methods consider partial support offered by foundation soils between the inclusions. Consequentially, reduction in the load carried by both the reinforcement and rigid inclusions may eventuate, resulting in leaner, more optimised ground treatment solutions.

Differential settlement criteria for pavements are typically quantified in the form of CIG limits of 0.3% to 0.5%, both within Australia, and internationally. Values of limiting CIG are found to correlate to passenger comfort criteria. Measurement of CIG in transition treatment zones can be executed based on the authors' methodology shown in Figure 14.

The assessment of creep settlement plays a crucial role in transition treatment design, as creep deformation forms the majority of the settlement experienced by the approach embankment and road pavement during their serviceable lives. The two current competing theories on creep settlement, Hypothesis A and B, dispute the point of commencement of creep. Conventional settlement and creep calculations follow the philosophies of Hypothesis A – which can also incorporate creep settlement reductions due to over-consolidation of soft soil. Finite element software, PLAXIS, utilises a constitutive model, Soft Soil Creep (SSC), to assess creep settlement, which follows the Hypothesis B approach. Use of ILO test results in the SSC model may lead to over-prediction of consolidation settlement. Reduction in creep using surcharge cannot be properly captured using currently available software models.

This paper documents the authors' past experience with transition treatment. However, this is by no means a comprehensive list of the relevant issues and challenges. The authors welcome an open discussion of experiences and methods to the contrary.

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