Integral Bridges and the Modelling of Soil-Structure Interaction

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No standard approach for the analysis of integral bridges appears in the Eurocodes, AASHTO, or other international codes. This paper considers the approaches most suitable for modelling common integral bridge forms, expanding upon recent UK guidance regarding soil-structure interaction approaches. Issues including material properties, initial stress state and the incorporation of the effects of soil ratcheting are discussed and both continuum and spring-type ('subgrade modulus') finite element models are explored.

1.0 INTRODUCTION

Owing to durability problems associated with movement joints, it is widely accepted that short and medium length bridges are best designed without such joints. This has led to a rise in popularity of integral bridges (no movement joints, no bearings) and semi-integral bridges (no movement joints) for new construction internationally.

Both integral and semi-integral bridges accommodate the thermal expansion and contraction of the superstructure by movement of the abutments or end-screens, which are retaining structures.

Often retaining structures are analysed representing the soil as merely a load – the stiffness of the soil is not modelled. The design proceeds considering only limiting active and passive lateral earth pressures[a]. However, if movements/ deflections of the structure are insufficient to mobilise the limiting values, intermediate values of earth pressure occur, as illustrated in Figure 1 below.

The lateral earth pressure depends on the strain in the soil, which in turn depends on movements in the structure. Structural movements depend on the stiffness of both structure and soil, and on lateral earth pressures. Unless the assumption of limiting earth pressures can be deemed conservative and acceptable, an analysis which somehow reflects this loop is required.

In some retaining structures, use of limiting earth pressures can be demonstrably either over-conservative or unconservative. During the summer expansion of the superstructure in an integral bridge, lateral earth pressures on the abutments can approach the theoretical passive state, especially in the upper portion where horizontal displacements are largest – pressures perhaps an order of magnitude greater than those experienced by the abutments of a non-integral bridge. For some integral bridge arrangements, it is sufficient to carry out the design on the basis of some assumed lateral earth pressure distribution (i.e. a limiting equilibrium approach, as described in Section 2.0 below). For others, the soil stiffness plays a more significant part in the behaviour of the system and an analysis which models the behaviour of both soil and structure – a soil-structure interaction (SSI) analysis – is required (as described in Section 3.0)

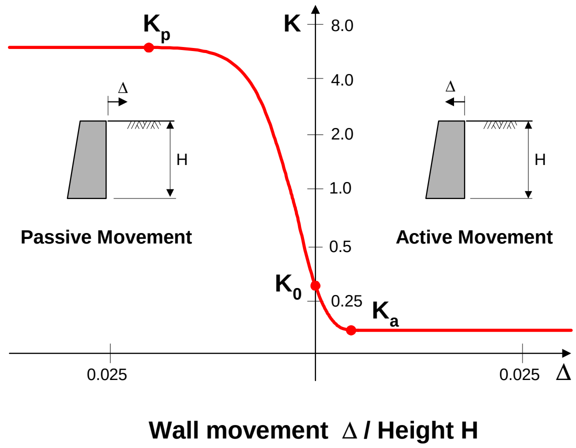


Figure 1. Pressure/ deflection curve[b]

The repeated thermal movements of integral bridge abutments cause particle realignment in granular backfill materials. Year-on-year, the lateral earth pressures each summer increase, as the backfill becomes stiffer, a phenomenon known as soil 'ratcheting' [5]. The effect of ratcheting is that soil stiffness and therefore the maximum lateral pressure can be significantly greater than the 'intermediate' value (K) that would be obtained from a pressure/ deflection curve such as Figure 1 for the expected

[a] Limiting pressures are described fully in Soil Mechanics textbooks – see, for example, Craig [1] Chapter 6.

[b] After NCHRP Report 343 [2]. The lateral earth pressure is illustrated in Figure 1 by reference to an earth pressure coefficient (K) which links lateral pressures to vertical stress in the soil without specific reference to cohesion, for simplicity. The principle stands for various soil types. Also see LRFD [3] C3.11.1 or EN1997-1 [4] Figure C.3.

movement (Δ)[c]. After 100-200 cycles, the increase in stiffness tails off, with maximum (summer) pressures tending to a value[d] which has been empirically linked to backfill properties, geometry and the movement range.

2.0 LIMITING EQUILIBRIUM APPROACHES FOR INTEGRAL BRIDGES

'Limiting equilibrium' approaches for the design of integral bridges generally use an assumed lateral earth pressure distribution and earth pressure coefficient, commonly denoted K*. Where the abutment retains granular material, the pressure distribution and value of K* used should be based on a theory which takes ratcheting into account.

No standard approach for earth pressure distribution behind integral abutments, or for determination of K* appears in AASHTO [3] or the Eurocodes [4] and practices vary [7, 8], with some methods making no allowance for ratcheting.

Figure 2 below shows the assumed pressure distribution given by PD6694-1 [9] for a full height abutment on flexible foundations.

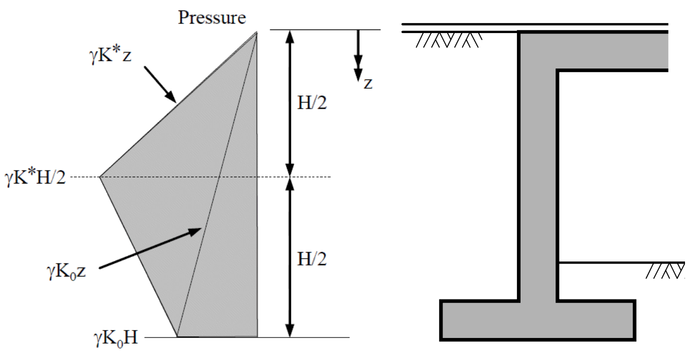


Figure 2. Assumed earth pressure distribution for full height abutment on flexible foundations after PD6694-1 [9] Fig 5

PD6694-1 [9] clause 9.4.3 indicates that for this case, K* can be conservatively calculated using:

$$K^* = K_o + \left(\frac{Cd'_d}{H}\right)^{0.6} K_{p;t}$$

where

H height of the wall

d'_d wall movement range at H/2 below ground level, taken as 0.5 to 0.7 times the design value of the movement range at the top of the wall, based on an assessment of the rotation and flexure in the system. See Figure 3 below.

K_o coefficient of at-rest earth pressure

$K_{p;t}$ coefficient of passive earth pressure determined using the design value of the triaxial ϕ'[e].

C coefficient dependent upon the elastic modulus of the subgrade (E_s):

$C = 0.051E_s + 14.9$ for E_s in MPa

$C = 0.35E_s + 14.9$ for E_s in ksi

and $20 \leq C \leq 66$ in either case.

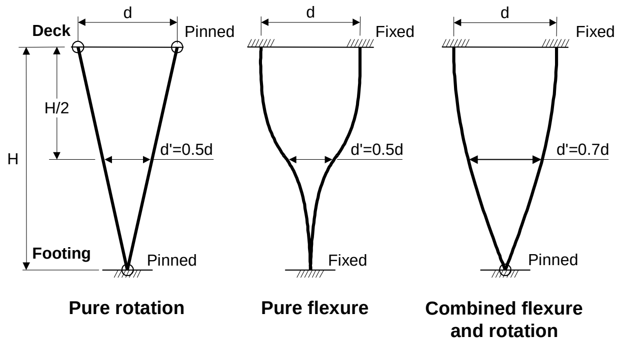


Figure 3. Comparison of different types of rotational and flexural abutment movements after [9] Stage 1 Report, Fig 9

For the shorter height bank pad abutments that accommodate thermal movements by translation without rotation, a simple triangular pressure distribution may be assumed and PD6694-1 [9] clause 9.4.4 gives the following expression for K*:

$$K^* = K_o + \left(\frac{40d'_d}{H}\right)^{0.4} K_{p;t}$$

These expressions and the recommendations of PD6694-1 [9] in general reflect a thorough review of research in the field [9], much improved by comparison to the previous UK guidance in BA42/96 [6][f], as described by Denton et al [12].

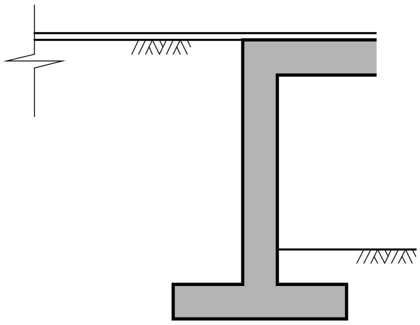
In the review [9] which preceded the publication of PD6694-1 [9], limiting equilibrium approaches based on K* were found to be appropriate for some common integral bridge abutment types such as those illustrated in Figure 4 below. It is usual for these abutment types to be constructed with a free-draining granular material used for backfill – it should be noted that the effects of soil ratcheting may be ignored when the material behind the abutment is a cohesive soil.

[c] By the same token, the relative movements required to reach active or passive conditions suggested by LRFD [3] Table C3.11.1-1 are not applicable.

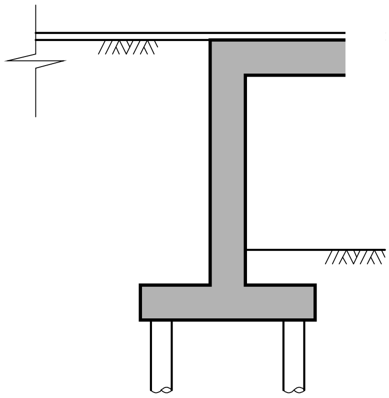
[d] Similar peak values are reached even if the backfill was not very well compacted at placement (BA42/96 [6] clause 3.2)

[e] Reference to PD6694-1 [9] clause 9.4.1 is recommended

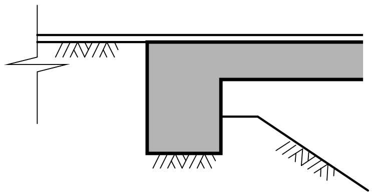
[f] BA42/96 including Amendment 1 [6] was itself a significant improvement upon the first edition [11]. Removal of the limit K*≥ K_p/3 of [11] clause 3.5.4, which had caused some engineers to regard the method as crude and over-conservative, is of particular note.



(i) Full height integral abutment on pad footing



(ii) Full height integral abutment on piles



(iii) Bank pad

Figure 4. Integral abutment types which can typically be designed with K* approaches

In the limiting equilibrium approach, the assumed earth pressures are applied to an analysis model of the structure in question. A 2D frame (beam element) model or 3D model may be used. The grillage (grid) analogy is routinely used for bridge superstructure analysis and some texts advocate extending such models into three dimensions to include abutment walls [13]. However, in-plane effects would be expected to be significant and these can create misleading local in-plane distortions of grid members in a 3D analysis [14]. Therefore 3D shell element models or mixed element models would probably be more appropriate (as in Figure 5 below). But, whatever type of model is used, the soil is represented only as a load – this is not a soil-structure interaction analysis; it is a way of *avoiding* a soil-structure analysis.

From such a structure-only model, load effects (bending moments, shear forces etc) may be obtained for design purposes. Care must be taken to ensure that all components – superstructure, abutment walls or end screens, foundation members/ piles – are designed considering suitable maximum or minimum earth pressures in combination with all other applicable loads, including the corresponding bridge temperature.

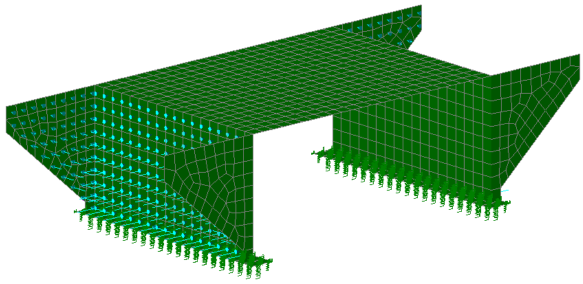


Figure 5. Typical 3D (shell) model of a full-height abutment integral bridge

Where the deck and walls are considered to be at different temperatures (as per EN1991-1-5 [14] clause 6.1.6), large transverse stresses can arise in models similar to the one illustrated in Figure 5 above. If these stresses are used in design calculations, large amounts of transverse reinforcement may be deemed necessary. It must be considered that cracking is, in this instance, due to internal restraint rather than external loading, rendering the crack width calculations of EN1992-1-1 [16] clause 7.3.4 inappropriate. Reference to EN1992-3 [17] section M.3 instead is recommended by Kamali et al [18].

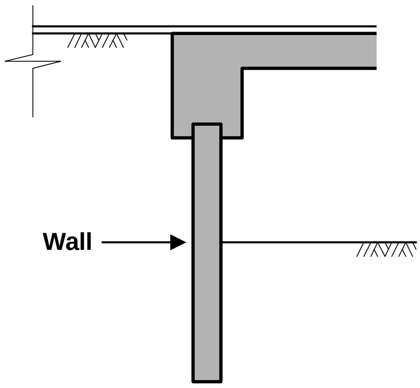
The pressures which should be considered in conjunction with bridge expansion and contraction are illustrated in PD6694-1 [9] Figure 6 and the Finnish Transport Agency Guideline [19] Figure 12. Notably the latter identifies the minimum pressure as being close to zero (rather than K_a) with not only contraction of the bridge but also freezing of the soil at the lowest temperatures. As the bridge expands but temperatures remain below zero, soil pressures increase rapidly as the ground is still frozen, reducing somewhat on thawing.

PD6694-1 [9] suggests that the lateral earth pressure on wingwalls of abutments which support K* pressures should themselves be subject to a pressure distribution similar to that illustrated in Figure 2, but calculated using the greater of K_o or $K_a \times K^*$ (clause 9.9).

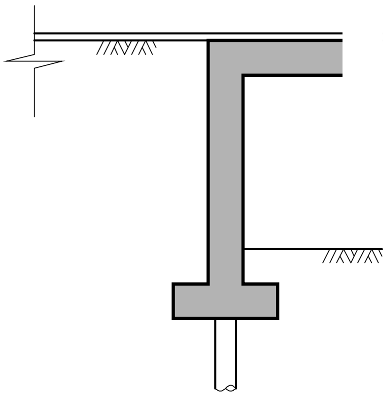
In skew integral abutment bridges, the earth pressures (normal to the walls) create a couple which, unresisted, would cause the bridge to rotate on plan. Furthermore, for bridges such as types (i) and (ii) of both Figures 4 and 6, thermal expansion results in twisting of the top of the abutment walls relative to their bases, along with any intermediate piers. PD6694-1 [9] discusses the rotation and twisting (clause 9.8) but a simple calculation approach is not given. Where such effects are deemed to be significant, a 3D analysis and perhaps SSI analysis may be appropriate.

3.0 SOIL STRUCTURE INTERACTION APPROACHES

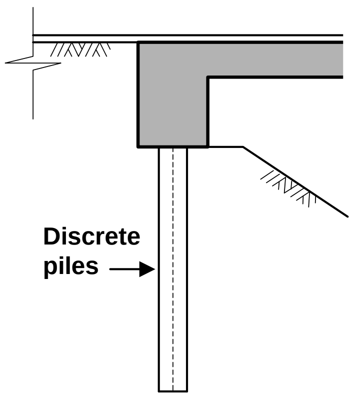
The limiting equilibrium approach is not appropriate where soil stiffness plays a more significant part in the behaviour of the system. This is the situation for a number of common integral bridge types such as those illustrated in Figure 6 below.



(i) Embedded wall integral abutment



(ii) Full height integral abutment on single row of piles



(iii) Bank pad on single row of piles

Figure 6. Integral abutment types which typically require SSI analysis

SSI can be handled by a number of calculation methods including closed form solutions, although these typically consider only simple cases, not extending to integral bridge arrangements. Most real project cases require numerical integration for which the finite difference, boundary element or finite element (FE) methods may be employed. FE approaches to SSI tend to fall into those which represent the soil using continuum elements and those which represent the soil using springs.

Bank pad abutments, often supported on steel H-piles – type (iii) in Figure 6 – are perhaps the most popular integral bridge type where space allows. For these, a 'subgrade modulus' model where the soil stiffness is represented using springs is probably most appropriate. Some authorities require the upper portion of piles to be sleeved in an effort to reduce the effects of SSI. As a result, the movement of the bank pads in and out of the backfill will be relatively greater, due to the lower resistance from the foundations. Soil ratcheting will occur, resulting in increased earth pressures on the bank pads – but the total lateral load arising will be quite limited due to the low height of the pads. Structural models with springs and K* pressures may be adequate, as discussed in section 3.2 below. The ratcheting effect may be further mitigated by the use of pea gravel or other specialised backfill.

Embedded walls (contiguous piled, secant or diaphragm) – type (i) in Figure 6 – are perhaps most popular for short-span underpasses in congested urban areas [7]. For such structures, a full continuum model may be more suitable.

3.1 FE continuum models

3.1.1 Element Selection and Considerations

For the representation of soil masses, beam and shell elements (most familiar to bridge engineers) are inappropriate and continuum elements must be used. In some bridges, 3D effects are of concern, however, often 2D models are sufficient and are certainly recommended for preliminary studies of SSI issues – for example, to assess the sensitivity of results to possible variation in certain parameters. Figure 7 shows the analysis model for an embedded wall underpass (type (i) in Figure 6), where 2D plane strain elements[g] are used to represent the soil and 2D beam elements used to represent the structural members.

As in all FE analyses it is important to ensure that the

[g] Plane strain elements are suitable for thick body problems as it is assumed that the out-of-plane strain is zero and so the out-of-plane stress is non-zero. By contrast, plane stress elements are for thin body problems as they are based on the assumption that the out-of-plane stress is zero and so the out-of-plane strain is non-zero.

number of elements used is sufficiently large that any inaccuracy arising from the division strategy may be deemed negligible by comparison to other assumptions inherent in the analysis. For FE continuum analyses we must also consider that elements give best accuracy at an aspect ratio of 1:1 and equal internal angles, although pragmatically ratios up to 1:3 are usually acceptable in areas of interest, and ratios up to 1:10 may be acceptable in remote regions of the model.

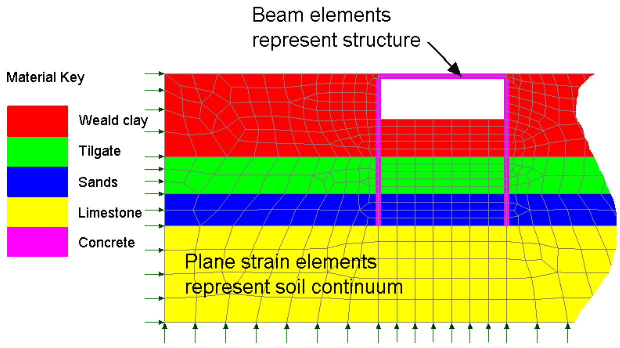


Figure 7. Typical 2D model of a single-span underpass (part model shown)

When modelling a structure interacting with a soil mass, the extent of the model is not straightforward to define: vertical and horizontal boundaries must be imposed on the soil mass at some distance from the structure[h]. Where such boundaries cannot be reasonably defined to match physical boundaries (e.g. free soil face, bedrock) they need to be determined by comparing key results from several models which are identical except for the assumed width or depth. Where the stiffness of the soil (E') has been assumed constant with depth, the predicted deflection under vertical load at the surface will increase as the depth of soil below the structure is increased so other key results should be used for comparison. If E' increases with depth, this effect is less pronounced.

3.1.2 Soil Material Properties and Initial Stresses

The simplest SSI models can assume isotropic linear elastic material properties for the soil. These would require only two parameters in their definition: elastic modulus, (E', perhaps varying with depth) and Poisson's ratio (ν), or shear modulus and bulk modulus. However, clearly most situations which demand SSI analysis also demand a more realistic mathematical representation of the soil to be employed.

There are many nonlinear material models designed to represent soil behaviour. Amongst these, the Mohr-Coulomb model is probably the most widely used ([20] section 3.4.1). Yield is based on a critical shear stress which is dependent on the normal pressure, making it applicable for soils where strength increases with balanced confining stresses. When yield is exceeded, volumetric plastic straining (dilatancy[i]) occurs, and isotropic hardening may be assumed.

To describe a granular material using a Mohr-Coulomb material model, elastic properties must be given (E', ν') along with the initial and final internal angles of friction[j] (ϕ'_1, ϕ'_2), cohesion (c') and dilation angle[k] (ψ) [22]. Effective stress parameters are referenced and the concept of effective stress is important because the *stiffness* of a saturated soil is dependent on whether an increase in load may be carried by fluids, fluids and soil skeleton or skeleton only and the *shear strength* of a soil is dependent on the effective normal stress.

Simple test models can be used to show that the internal stresses in a block of Mohr-Coulomb material correspond to active and passive pressures as predicted by Rankine-Bell equations. Figure 8 illustrates this, showing a test case (plane strain soil block of 10m×10m) with active pressures achieved by movement of the right boundary; movement in the opposite direction will similarly achieve the expected passive pressure.

The results of Figure 8 may be corroborated simply:

Vertical stress at 10m depth

$S_Y = \rho gH = 1.8 \times -9.81 \times 10 = -176.58$

Maximum (initial) lateral stress at 10m depth

$S_X = S_Z = K_o \times S_Y$ where K_o was taken[l] to be 0.5

Thus S_X (initial) = 0.5×-176.58=-88.29

Minimum (final) lateral stress at 10m depth

$S_X = S_Z = K_a \times S_Y$ where

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[h] Where dynamic effects are required to be considered, non-reflective boundaries are typically required. Such considerations are outside the scope of this paper.

[i] Soils comprise a high proportion of void (filled with single or multi-phase fluid): a typical medium dense sand might be 1/3 void by volume; a normally consolidated clay might edge towards 1/2. When the soil is subject to straining, some rearrangement of particles occurs, accompanied by change in volume: this is dilatancy. Either a reduction in volume or an increase in volume can occur, depending on the particle shape and packing arrangements. See [20] section 2.6

[j] For detailed information on friction hardening in Mohr-Coulomb models, see [21].

[k] The dilation angle ψ describes the amount of volumetric change that occurs during plastic straining or shearing. For plastic deformation at constant volume, ψ = 0; for soils that contract when they are sheared plastically, ψ < 0; for soils that expand when they are sheared plastically, ψ > 0; for most real soils, ψ < ϕ'. See [20] section 3.3.4

[l] Various expressions for K_o exist, see Craig [1] Chapter 6, EN1997-1 [4] clause 9.5.2 etc

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} = 0.333$$

Thus S_X (final) = 0.333×-176.58=-58.86

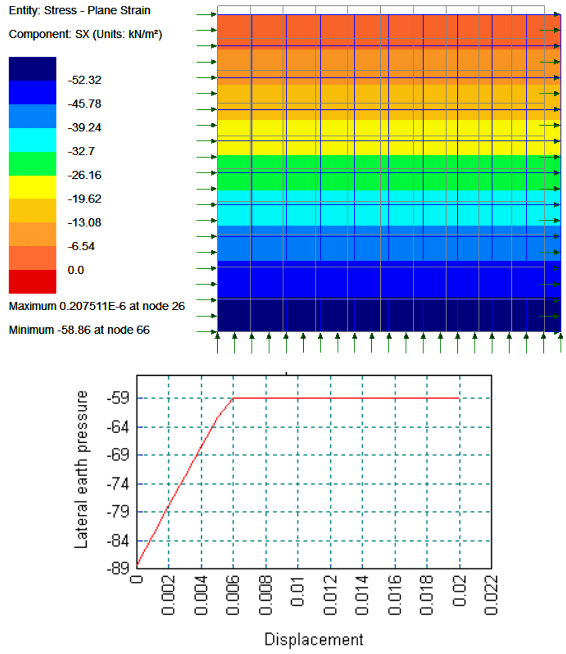

Figure 8. Results from Mohr-Coulomb test model where ϕ'=30

The displacement of the wall (or in this case, boundary) which is required to mobilise the limiting active or passive pressure must be understood to be dependent upon not only the elastic modulus of the material (as might be immediately anticipated) but also upon the *initial stress* in the soil. By varying the assumed value of K_o, the displacements required for full active or passive pressure to be reached is altered, as shown in Table 1.

Table 1: Effect of assumed value for K_o on movement required to mobilise limiting earth pressures

K_o	Limiting pressure at 10m depth mobilised at (mm)	
	Active	Passive
0.1	0	90
0.5	6	90
1.0	22	70
2.0	85	35

The values of Table 1 illustrate the importance of initial stresses in SSI analyses. The applied loads for the initial (equilibrium) state in an FE analysis must include an initial stress which varies with depth, usually based on an assumed K_o.

3.1.3 Representing Interfaces in Continuum-Based Models

The interface between soil and structure typically needs some special consideration in any SSI analysis. Comprehensive FE systems such as LUSAS offer a range of options in this area such as [22]:

a) Joint elements & materials. Known as 'link', 'hook' or 'fuse' elements in some software, joint elements notionally have no length but instead provide a means of connecting two adjacent elements without full fixity, introducing options such as frictional or yielding behaviour.

b) Contact slidelines. Contact algorithms in software such as LUSAS enable the proximity of elements to each other to be detected, allowing transfer of load between one 'component' and another without adjacent elements actually sharing nodes. In the context of SSI, the components would be the soil and the structure, and frictional slidelines would typically be of interest.

c) Elasto-plastic interface materials. A layer of elasto-plastic material (assigned to plane strain or 3D continuum elements) can represent the friction-contact relationship between the soil and the structure. The material reproduces the nonlinear response of a system containing planes of weakness governed by Mohr-Coulomb type laws.

Whichever of these options is utilised, for retaining structures and integral bridges, the crucial consideration is usually back of wall friction, δ. The value of δ cannot be less than zero (a notionally smooth wall) nor exceed that of ϕ' for the material being retained. For many retaining structures, lower bound ϕ' and δ are deemed critical for design and δ=0 is used, as suggested in PD6694-1 [9] clause 7.2.2 and CIRIA C580 [23] section 4.1.4, whereas for integral bridges upper bound values may also be critical. For the design of integral bridge abutments, BA42/96 [6] clause 3.3 states that wall friction should be taken as $\delta=\phi'/2$.

3.1.4 Representing the Structure

It is important to remind ourselves that relative stiffness is crucial to the distribution of loads in an FE model. In the case of a wholly concrete structure, for example, the accuracy of the value used for elastic modulus affects deflections but generally has little effect on load distribution, since the *relative* stiffness is accurate. However, in the case of a concrete structure in contact with the ground, a reasonably accurate relative stiffness may demand more consideration of issues such as concrete cracking and creep deformation. While these are considerable topics in their own right, it should be underlined that reinforced concrete (RC) is generally cracked and therefore has a stiffness significantly less[m] than that which would be assessed using the gross section and the elastic modulus from a code of practice (see [25] section 2.4.2).

3.1.5 When and How to Incorporate the Effects of Soil Ratcheting

Often, the popular embedded wall integral abutment (Figure 7 and type (i) in Figure 6), is constructed as part of a top-down scheme in cohesive soil. As noted earlier, for such soils, the effects of strain ratcheting may be ignored (see PD6694-1 [9] clauses 9.4.5.2 and A3.2) and so a suitable SSI analysis (perhaps utilising a Cam Clay material model) may be used with no further special considerations. The software used must be capable of modelling the staged construction process in conjunction with the use of the preferred nonlinear soil material.

For integral bridges with embedded walls (type (i) in Figure 6) or full height integral abutments on a single row of piles (type (ii) in Figure 6), retaining granular materials, guidance for a suitable SSI analysis incorporating ratcheting is given in PD6694-1 [9] Annex A with further background given in [9] Stage 2 report, Section 5. The recommended approach essentially entails the soil being modelled as a continuum, with an elastic modulus (E') which varies with depth according to an assumed lateral earth pressure profile which can be regarded as a 'quasi-passive' limit, similar in nature to the K* profile described for the limiting equilibrium method above. Along with this, lateral earth pressures are restricted to lie between the active limit and the quasi-passive limit. Therefore a continuum model with Mohr-Coulomb material can be used, together with joint elements – option (a) in section 3.1.3 above – which yield at the quasi-passive limit. The software used must be capable of handling the variation of the material properties (E', K*) with depth in the Mohr-Coulomb and joint materials.

3.2 Use of Springs to represent Soil

For a bank pad abutment on piles (type (iii) in Figure 6), lateral movement at the pile head does not infer plane lateral movement of the soil in the way that lateral movement of a retaining wall does. There may be some arching of soil between piles but judgements are needed. This makes 2D plane strain models less appropriate for such bridges and the use of spring models more attractive.

[m] The cracked stiffness depends upon reinforcement provided but as an indication, ACI 318-08 [24] clause 10.10.4.1 suggests that cracked RC columns and walls may be treated as having a stiffness ~70% of the calculated gross stiffness; cracked slabs having only ~25%.

SSI analyses where soil is represented using springs are widely referred to as 'Winkler spring' or 'subgrade reaction' models. The springs may be used to represent the vertical or horizontal resistance of the soil; in the context of retaining structures and integral bridges it is the horizontal stiffness, characterised by a spring stiffness, k_h (force/length^3), which is of interest. Spring models are suggested in EN1997-1 [4] clause 9.5.4.

Nonlinear springs or joints may be used within an FE model to generate lateral earth pressures for a retaining wall design based on a pressure/ deflection relationship such as that in Figure 9 below:

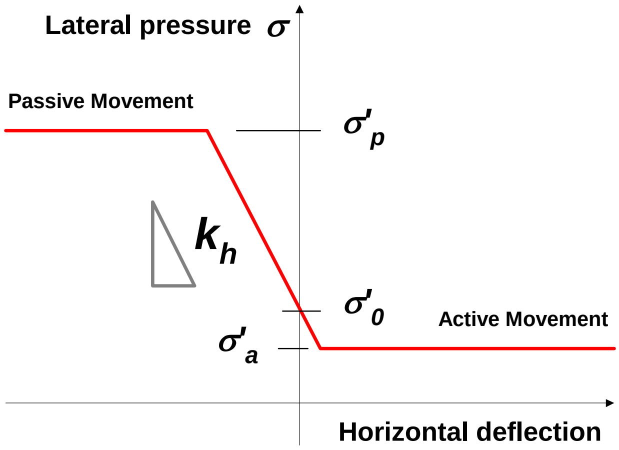

Figure 9. Nonlinear 'soil joint' pressure/ deflection graph [22]

Critically Figure 9 incorporates not only the modulus of subgrade reaction, k_h, but also at-rest earth pressures (σ_o, based on K_o). Neither quantities are considered when retaining walls are designed using limiting earth pressure methods, however they are essential components of SSI analyses (see CIRIA C580 [23], section 5.1). Typically all the quantities represented in Figure 9 – active and passive 'yield points', the spring stiffness, k_h, and the at-rest pressure – vary with depth.

The yielding spring approach illustrated is advocated by Frank et al [26]. It is also applied to integral bridge analysis [27] and, with suitable values of K_a and K* (varying with depth) as the yield points, suitable stiffness, k_h (varying with depth) and initial stresses, might provide an alternative to continuum analyses for embedded wall integral abutments (type (i) in Figure 6). However it is in the analysis of abutments on piles (types (ii) and (iii) of Figure 6) that the use of springs seems most appropriate.

Where piles are installed in level ground, the at-rest pressures are in equilibrium and so are not of interest for most analysis/design purposes. When lateral strains are expected to be small, it may be reasonable to model piles using beam elements, supported by linear elastic lateral springs. For cohesive soils, it is generally considered that spring stiffness may be assumed constant with depth (Finnish Guidelines [19] section 4.3.5.1, Rombach [25]). For granular soils, a linear variation with depth may be used (the Finnish guideline [19] suggests that a linear variation up to a depth of 10*d* and thereafter a constant value for k_h). It may be helpful to consider the variation to be a polynomial of the form:

$$k_h = A + Bz^{\kappa}$$

In this, A, B and κ are empirical constants. Typical variations of k_h with depth for different soil types are suggested by Rombach [25] and illustrated in Figure 10.

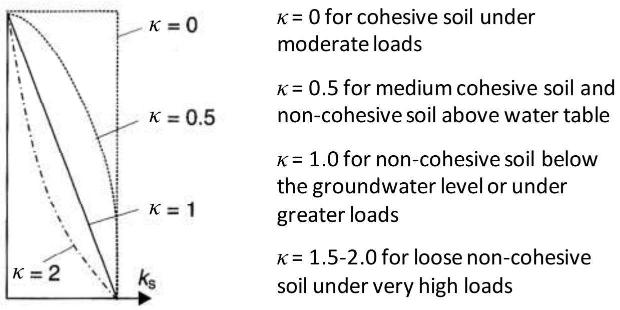

Figure 10. Typical variations of modulus of subgrade reaction with depth for piles (after Rombach [25] Figure 2.39)

Values for k_h are notoriously difficult to obtain, since the spring stiffness is not a fundamental soil property. However some guidance may be found in the Finnish Guidelines [19] section 4.3.5.1 (including a correlation between ϕ' and k_h), while RP2A [28] section 6.8 describes methods for defining pressure-deflection (p-y) curves for laterally loaded piles appropriate to various soil materials. Comprehensive FE software is capable of handling such curves within a nonlinear joint material as in Figure 11 below and the matter is covered in more detail by Reese & Van Impe [29].

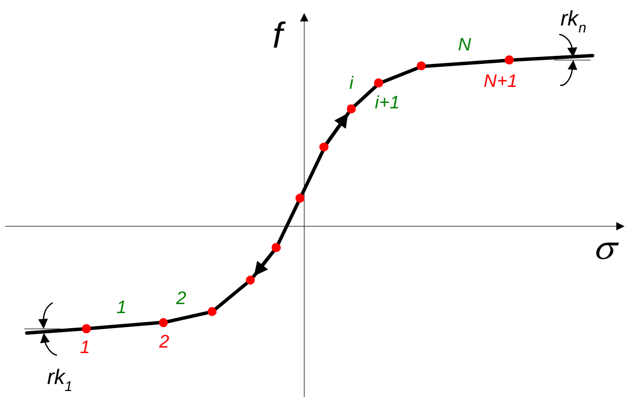

Figure 11. Piecewise joint definition for nonlinear joint material, from [22]

PD6694-1 [9] clause 6.4.9 indicates that SSI analysis is required for bank pad abutments on piles (type (iii) in Figure 6), and draws attention to a particular problem: forward of the piles, the ground slopes away and so the ground stiffness and limiting pressures would be different on each side of the line of piles (see CIRIA C580 [23], clause 7.2).

The remolding of soil over many thermal cycles should also be taken into account as described by Wasserman [30]. This is reflected in RP2A [28], which describes cyclic loads as causing a deterioration of lateral resistance as compared to that observed for static loads, and gives p-y curves for both static and cyclical loading.

For bank pad abutments on piles, then, suitable lateral earth pressures for the end screen may be determined by adopting a K* approach (PD6694-1 [9] clause 9.4.6), while the pile-soil interaction can be handled using linear or nonlinear springs (or 'joints') to represent the soil as illustrated in Figure 12 below. Such approaches to pile-soil modelling have been used for integral bridges – and validated using field measurements and sub-models in the past – see Jayaraman [31], Krizek and Studnička [32], and Albhaisi [33].

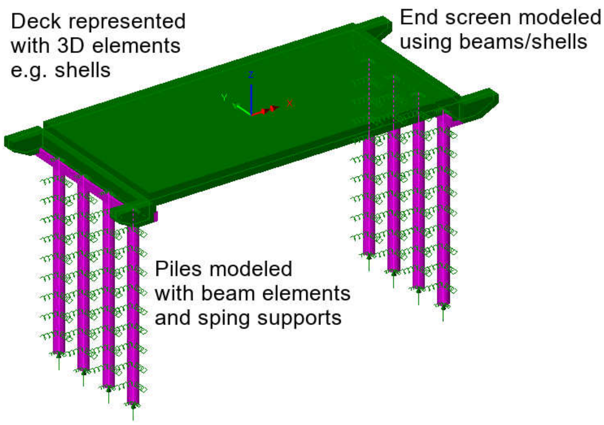

Figure 12. 3D FE model of bridge deck integral with bank pads on piles

4.0 CONCLUSION

Typically the abutment walls or endscreens of integral bridges are backfilled with granular material, where the effects of soil ratcheting should be taken into account. Where cohesive soil lies behind an embedded wall, ratcheting can be neglected. Limiting earth pressure and SSI analysis options have been explored and suggested approaches are shown in Table 2 below.

Table 2. Suggested Analysis Approach by Integral Abutment Type

Integral Abutment Type	Limiting equilibrium	SSI	Notes and reference within PD6694-1 [9]
Full height wall on pad footing	Yes		Granular backfill. Assumed earth pressure distribution from Fig 5 and K* from clause 9.4.3 incorporates ratcheting.
Full height wall on piled footing	Yes		
Bank pad	Yes		Granular backfill. Triangular earth pressure distribution and K* from clause 9.4.4 incorporates ratcheting.
Embedded wall		Yes	Soil modelled using continuum. For granular soils or backfill, modify E' with depth to suit Annex A and restrict pressures to K* from clause 9.4.3. Alternatively nonlinear spring model with similar considerations.
Full height wall on single row of piles		Yes	Soil modelled using nonlinear springs. For granular backfill, modify stiffnesses and limiting pressures to suit Annex A.
Bank pad on single row of piles		Yes	Soil modelled using nonlinear springs. Reduced stiffness and limiting pressures for front face of piles. Granular backfill to end screen. Triangular earth pressure distribution and K* from clause 9.4.4 incorporates ratcheting.

5.0 REFERENCES

1. Craig R.F. 1992. *Soil Mechanics 5^{th} Edition*, Chapman & Hall, London, UK

2. Barker, R.M., Duncan, J.M., Rojiani, K.B., Ooi, P.S.K., Tan, C.K. and Kim, S.G. 1991. *NCHRP Report 343: Manuals for the Design of Bridge Foundations,* Transport Research Board/ National Research Council, Washington D.C., USA

3. *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington D.C., 2012

4. CEN (Comité Européen de Normalisation). 2004. *BS EN1997-1:2004 Eurocode 7: Geotechnical design — Part 1: General rules*, British Standards Institute, London, UK.

5. England, G.L. and Tsang, N.C.M. 2001. *Technical Paper No 2: Towards the Design of Soil Loading for Integral Bridges*, Concrete Bridge Development Group, Camberley, Surrey UK

6. Highways Agency. 2003. *BA42/96 Amendment No 1, The Design of Integral Bridges*, The Stationery Office, London, UK

7. Kunin, J. and Alampalli. 1999. *Special Report 132: Integral Abutment Bridges: Current Practice in the United States and Canada*, Transportation Research and Development Bureau, New York State Department of Transportation, New York, NY, USA

8. Connal, J. 2004. Integral Abutment Bridges - Australian and US Practice. *Austroads 5th Bridge Conference*, Austroads, Sydney, Australia.

9. BSI. 2011. PD 6694-1:2011 Recommendations for the Design of Structures Subject to Traffic Loading to BS EN 1997-1:2004, British Standards Institute, London, UK

10. Highways Agency/ Arup, 2009. Integral Bridges – Best Practice Review and New Research, Phase 2b - Review of Existing Data, Back-Analysis of Measured Performance and Recommendations (Stages 1, 2 and 3)

11. Highways Agency. 1996. *BA42/96, The Design of Integral Bridges*, HMSO, London, UK

12. Denton, S., Riches, O., Christie, T. and Kidd, A. 2010. Developments in Integral Bridge Design, *Bridge Design to Eurocodes: UK Implementation*, ICE Publishing, London, UK

13. Nicholson, B.A. 1998. *Integral Abutments for Prestressed Beam Bridges,* Prestressed Concrete Association, Leicester, UK

14. O'Brien, E.J. and Keogh, D.L. 1999. *Bridge Deck Analysis,* E&FN Spon, London, UK

15. CEN (Comité Européen de Normalisation). 2003. *BS EN1991-1-5:2003 Eurocode 1: Actions on structures — Part 1-5: General actions — Thermal actions,* British Standards Institute, London, UK.

16. CEN (Comité Européen de Normalisation). 2004. *BS EN1992-1-1:2004 Eurocode 2: Design of concrete structures — Part 1-1: General rules and rules for buildings,* British Standards Institute, London, UK.

17. CEN (Comité Européen de Normalisation). 2006. *BS EN1992-3:2006 Eurocode 2: Design of concrete structures — Part 3: Liquid retaining and containment structures,* British Standards Institute, London, UK.

18. Kamali, A.Z, Svedholm, C. and Johansson, M. 2013. *Effects of restrained thermal strains in transversal direction of concrete slab frame bridges.* KTH Stockholm, Sweden 2013

19. Liikenneviraston ohjeita (Transport Agency Guidelines) 11/2012. *Sillan geotekninen suunnittelu - Sillat ja muut taitorakenteet*, Liikennevirasto, Helsinki 2012

20. Muir Wood, D. 2004. *Geotechnical Modelling*, Spon Press, New York, NY, USA

21. Vermeer P.A. and De Borst R. 1984. Non-Associated Plasticity for Soils, Concrete and Rock, *Heron*, 29 (3): 3-64

22. LUSAS. 2013. *Theory Manual Volume 1*, LUSAS, Kingston Upon Thames, Surrey, UK

23. Gaba, A.R., Simpson, B., Powrie, W. and Beadman, D. R. 2003. *CIRIA C580 Embedded Retaining Walls – Guidance for Economic Design*, Construction Industry Research and Information Association, London, UK

24. ACI. 2008. *ACI 318-08, Building Code Requirements for Structural Concrete*, American Concrete Institute, Farmington Hills, MI, USA

25. Rombach, G.A. 2011. Finite-element Design of Concrete Structures: Practical Problems and their Solutions (Second Edition), ICE Publishing, London, 2011

26. Frank, R., Bauduin, C., Driscoll, R., Kavvadas, M., Krebs Ovesen, N., Orr, T. and Schuppener, B. 2004. *Designers' Guide to EN 1997-1, Eurocode 7: Geotechnical design - General rules,* Thomas Telford Ltd, London, UK

27. Faraji, S., Ting, J.M., Crovo, D.S., and Ernst., H. 2001. Nonlinear Analysis of Integral Bridges: Finite Element Model, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 127 (5): 454-461

28. API Recommended Practice RP2A-WSD. Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design, 21st Edition, Dec 2000. American Petroleum Institute, Washington D.C.

29. Reese, L.C. and Van Impe, W. 2000. Single Piles and Pile Groups Under Lateral Loading, Taylor & Francis, London, UK.

30. Wasserman, E.P. 2007. Integral Abutment Design (Practices in the United States), *First U.S.-Italy Seismic Bridge Workshop*, Pavia, Italy.

31. Jayaraman, R., Merz, P.B. and McLellan Pte Ltd. 2001. Integral Bridge Concept Applied to Rehabilitate an Existing Bridge and Construct a Dual-use Bridge, *26th Conference on Our World in Concrete & Structures*, Singapore.

32. Krizek, J. and Studnička, J. 2007. Integral Bridges and Soil–Structure Interaction, *Modern Building Materials, Structures and Techniques 9th International Conference*, Vilnius, Lithuania.

33. Albhaisi, S., Nassif, H., and Hwang, E. 2012. Effect of Substructure Stiffness on Performance of Steel Integral Abutment Bridges Under Thermal Loads, *Journal of the Transportation Research Board,* Transportation Research Board of the National Academies, 2313: 22-32