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Time-Dependent Load Changes in Integral Bridges

Variation des charges dans le temps de ponts monolithiques

Zeitabhängige Lastveränderungen in monolithischen Brücken

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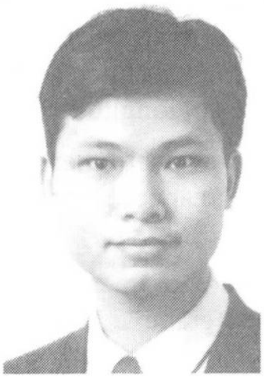

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SUMMARY

The soil/structure interaction of bridge abutments is described in relation to experimentally observed behaviour of drained granular material subjected to cyclical strain loading. For integral bridges, repeated thermal displacements of the deck cause frictional flow in the granular material, and increases in the lateral soil stress behind the abutments. The composite behaviour is analysed in a novel manner, as an elastic-plastic shakedown problem. This approach allows the most influential parameters to be identified, and the limiting load cases for the structure to be defined.

RÉSUMÉ

L'interaction sol-structure des culées de pont est décrite en relation avec le comportement observé des matériaux granuleux subissant des allongements de manière cyclique. Dans les ponts monolithiques, la répétition des déplacements du tablier dûs aux variations de température provoquent des frottements dans le matériau granuleux et augmente les contraintes latérales du sol derrière les culées. Le comportement sol-structure est analysé pour la première fois en temps que problème élasto-plastique. Cette approche permet d'identifier les paramètres principaux du phénomène et de définir les cas de charges limites pour la structure.

ZUSAMMENFASSUNG

Es wird die Boden-Bauwerk-Wechselwirkung an Brückenwiderlagern angesprochen und zu experimentellen Beobachtungen in drainiertem körnigen Material unter zyklischer Dehnungsgeschichte in Beziehung gesetzt. In zusammenhängend gebauten Brücken bewirken die wiederholt auftretenden Temperaturdehnungen in der Fahrbahnplatte Reibungsdeformation im granularen Material und einen Anstieg des horizontalen Erddrucks hinter den Widerlagerwänden. Das Zusammenwirken wird erstmals als elastisches Shake-down-Problem analysiert. Bei dieser Vorgehensweise lassen sich Hauptparameter identifizieren und Grenzbeanspruchungsfälle des Bauwerks definieren.

1. INTRODUCTION

One major source of damage to highway bridges results from deicing salts leaking through deck joints onto sub-surface components. This process causes corrosion and immobilisation of movement joints and bearings, and represents a major element of conventional road bridge repair and maintenance costs. The Department of Transport in the UK has recently adopted the concept of continuous and integral (jointless) bridges as one of the options to avoid the corrosion problems. Integrating bridge deck to the abutment is also an option for retrofitting existing concrete bridges. In the UK, bridge abutments usually act simultaneously as soil retaining structures for the roadway embankment and are typically more than 6m in height. For medium span bridges of this type (length approx. 100m), environmental daily and seasonal temperature fluctuations of the bridge deck (up to 48^{o}C) cause relative movements of the abutments. These in turn cause repeating displacements to be imposed on the granular backfill behind the abutments, where the nature of the soil/structure interaction is to create significant lateral stress escalation in the soil and atttendant increases in longitudinal force in the bridge deck.

Engineers require guidance to evaluate these thermally induced and cyclically-dependent lateral earth pressures acting over the rear face of the abutments and hence the axial force in the bridge deck for design purposes.

This paper describes the nature of the soil/structure interaction in relation to experimentally observed behaviour of drained granular material subjected to cyclical strain loading. An elastic-plastic shakedown model for the interaction between the bridge abutment and the granular backfill is presented. Finally, the most influential parameters are identified and the limiting load cases for the structure are defined.

2. INITIAL STRESS/STRAIN RESPONSE OF GRANULAR MATERIAL

The horizontal earth stress, σ_H, at a depth **z** is defined by γ**zK** where γ is the unit weight of the granular material and **K** is the horizontal soil pressure coefficient (See Figure 1a.). During the initial heating (abutment moves towards the embankment) and then cooling (abutment moves away from the embankment), the horizontal soil stress path follows C-b-A in Figure 1b. In the case of cooling first *before* heating the path is C-a-B. The initial **K** value (point C) and profile of the initial responses curves (C-A or C-B) are dependent on density of the soil which is controlled by the degree of compaction. Point **a** and **b** are dependent on the temperature applied to the structure. During the second and subsequent thermal cycles, loading paths have origins which are no longer on the virgin loading curves, CB and CA.

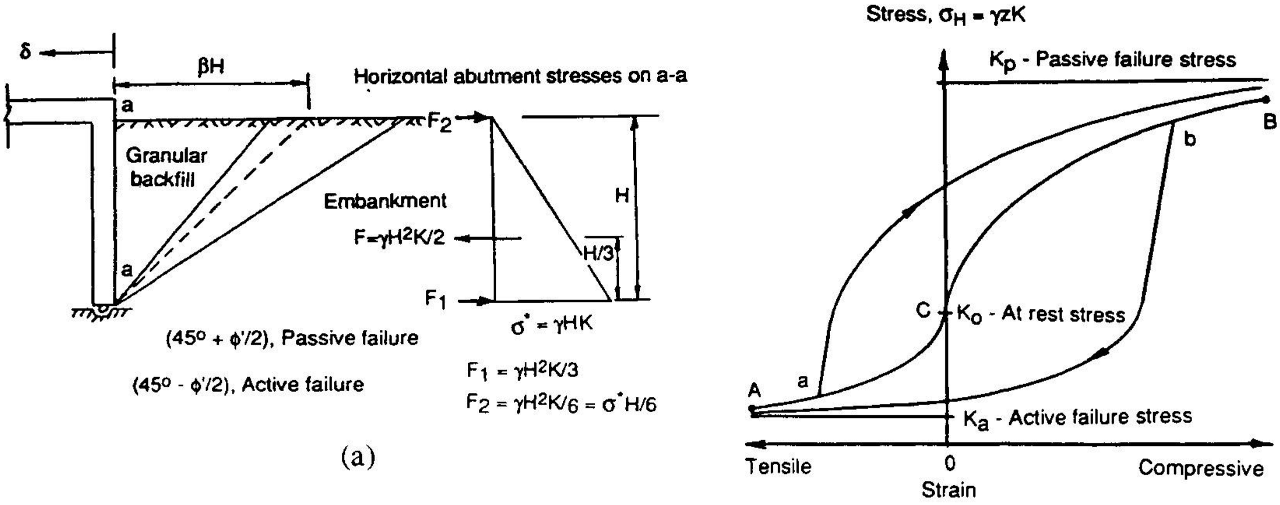
3. SOIL/STRUCTURE INTERACTION

For a small number of cycles the volume of backfill which is strained plastically is difficult to identify. However, after many cycles, experimental observations of deformation patterns have revealed that β (see Figure 1) approaches unity. Initially, the deck is in compression due to the horizontal soil pressures at the abutments. Then for a temperature change Δ**T** (+ve for temperature rise), the *free thermal expansion* of the bridge deck is $L\alpha\Delta T$, where α and **L** are the coefficient of thermal expansion and length of the deck respectively. For a unit width of the abutment, this will cause a change in horizontal earth force, ΔF_2 (see Figure 1a.) and a complimentary equal change in the deck force, ΔP (+ve for compression). The corresponding deck displacement caused by the change of soil pressure, from the *free thermal length* $L(1+\alpha\Delta T)$ at one end of the deck, (symmetry assumed) is given by

$\delta_1 = \Delta P/k_b$ +ve for *contraction*, where,

k_b = 2AE/L axial stiffness of the deck relative to P
$\Delta P = \Delta F_2$ thermally induced *increase* in axial compressive force

The corresponding displacement of the wall/soil interface is defined as, in Figure 1a, **δ = (L$\alpha\Delta$ T/2-δ_1)**. This is absorbed within the soil wedge defined by **β** (=1).



(a) (b)

Figure 1. (a) Lateral soil pressure on a retaining wall type bridge abutment
(b) Initial Stress/Strain Response of Drained Granular Material
(i) unload **C-a** then reload **a-B** or (ii) load **C-b** then unload **b-A**

The analysis shown above represents an half-cycle temperature change. The daily and seasonal cyclic thermal movements of the bridge deck lead to a more complicated problem of continuous fluctuating strain changes in the granular backfill. A soil/structure interaction diagram is developed to enable a graphical presentation of the relationships between the stress paths of the soil, the stiffness of the bridge deck and the displacements of the deck/soil interface. Figure 2 represents a typical interaction diagram of this type for a "soft deck" structure, this will be further explained in next section.

Figure 2 has been derived from Figure 1(b) by suitable scaling of the axes; **$\delta = \beta H \varepsilon_H$** and **$F_2 = \sigma_H{}^*HK/6$**. Lines **x-x** and **y-y** represent the bridge deck force/displacement responses at the two temperature limits. The effect of the cyclic temperature changes in the bridge deck creates an overall incrementally ratcheting inward displacement (towards the deck) of the wall/soil interface and an escalation of the horizontal force between the full-height retaining-wall abutment and the granular backfill embankment following the unclosed loops c-d-e-f-...of Figure 2.

4. SHAKE-DOWN STATE

During the cyclic straining process, the bulk volume of the soil particles may reduce and their packing arrangement changes. The horizontal earth pressure coefficient, **K**, will build up. Eventually the material tends either to (i) a *shakedown state*, or (ii) *failure* of the granular material (ie **$K = K_p$** or **K_a**) or *failure* of the structure. The shakedown state is characterised by stresses which repeat on a cycle by cycle basis with no overall volume change and an upper stress ratio, **K_u**, which is always the reciprocal of the lower stress ratio, **K_l**, i.e. **K_uK_l** = 1. The existence of a shakedown state (see Figure 2) is dependent upon the magnitude of the imposed fluctuating soil strain (**$\Delta\delta/\beta H$**) from the bridge deck, the stiffness (**2AE/L**) of the deck and the horizontal residual soil modulus (E_g). It is defined by repeating elastic-plastic (friction) behaviour in the granular material and elastic behaviour in the structure. The existence of such a state has been verified experimentally [1] and values for the residual soil modulus for *sand* under cyclic strain loading have been observed to be proportional to the overburden pressure. $\mathbf{E_g}$ can be defined as $\mathbf{N\sigma_v}$ where σ_V is the vertical stress on the soil element considered and **N** is a constant coefficient. $\mathbf{E_g}$ is, therefore, dependent on the abutment wall height **H** and varies linearly with depth.

Because both the temperature change $\mathbf{\Delta T}$ and abutment wall height **H** are governed practically by the environment and topographical conditions, the interaction diagrams, as shown in Figure 2 and 3, identify two types of integral bridge, depending upon deck stiffness :

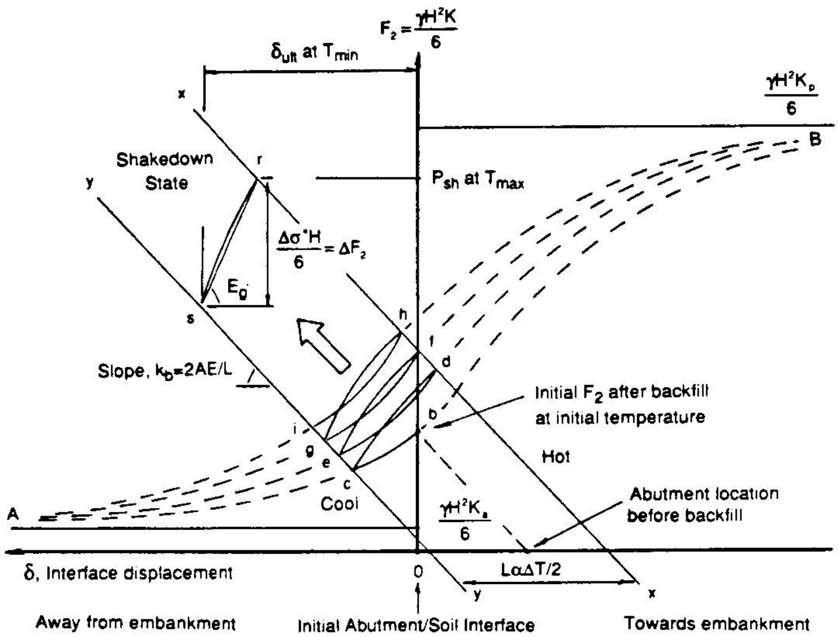


Figure 2. A Soil/Structure Interaction Diagram for "Soft Structure"

(i) **Soft deck**; small $\mathbf{k_b}$. Cyclically imposed displacements of the soft deck on the granular backfill lead to a *shakedown state* for any value of the residual soil modulus, $\mathbf{E_g}$. When this value is not known, a shakedown solution may still be obtained by using an upper bound value of $\mathbf{E_g} = \infty$. Then, the limiting upper bound value of the fluctuating stress, $\mathbf{\Delta\sigma_u^*}$, at the toe of the abutment wall (see Figure 1) may be evaluated from Figure 2, as $\mathbf{\Delta\sigma_u^* = 6AE\alpha\Delta T/H}$. The corresponding maximum fluctuating force in the deck is $\mathbf{\Delta P = AE\alpha\Delta T}$. By applying one the shakedown requirements, $\mathbf{K_u K_l} = 1$, the maximum shakedown force in the deck, $\mathbf{P_{sh}}$, and maximum abutment displacement, $\mathbf{\delta_{ult}}$, can then be scaled from the interaction diagram of Figure 2.

(ii) **Stiff deck**; large $\mathbf{k_b}$. For a stiff bridge deck, the upper bound $\mathbf{\Delta\sigma_u^*}$ evaluated by the formula in (i) above is larger than the maximum permissible stress defined by $(\mathbf{K_p - K_a})$. This implies that the actual residual modulus, $\mathbf{E_g} \neq \infty$, of the granular backfill *at shakedown* should be determined and the corresponding fluctuating stress $\mathbf{\Delta\sigma^*}$ (see Figure 3) should remain within the stress limits defined by $\mathbf{K_p}$ and $\mathbf{K_a}$ for a safe solution to exist. Otherwise, failure of the embankment will occur.

Figure 3 represents an interaction diagram for a *typical* highway bridge deck (stiff deck) and the granular backfill. The dimensions for the bridge are: length 100m, sectional area of the concrete deck per unit width 0.5m^2 and abutment height 6m.

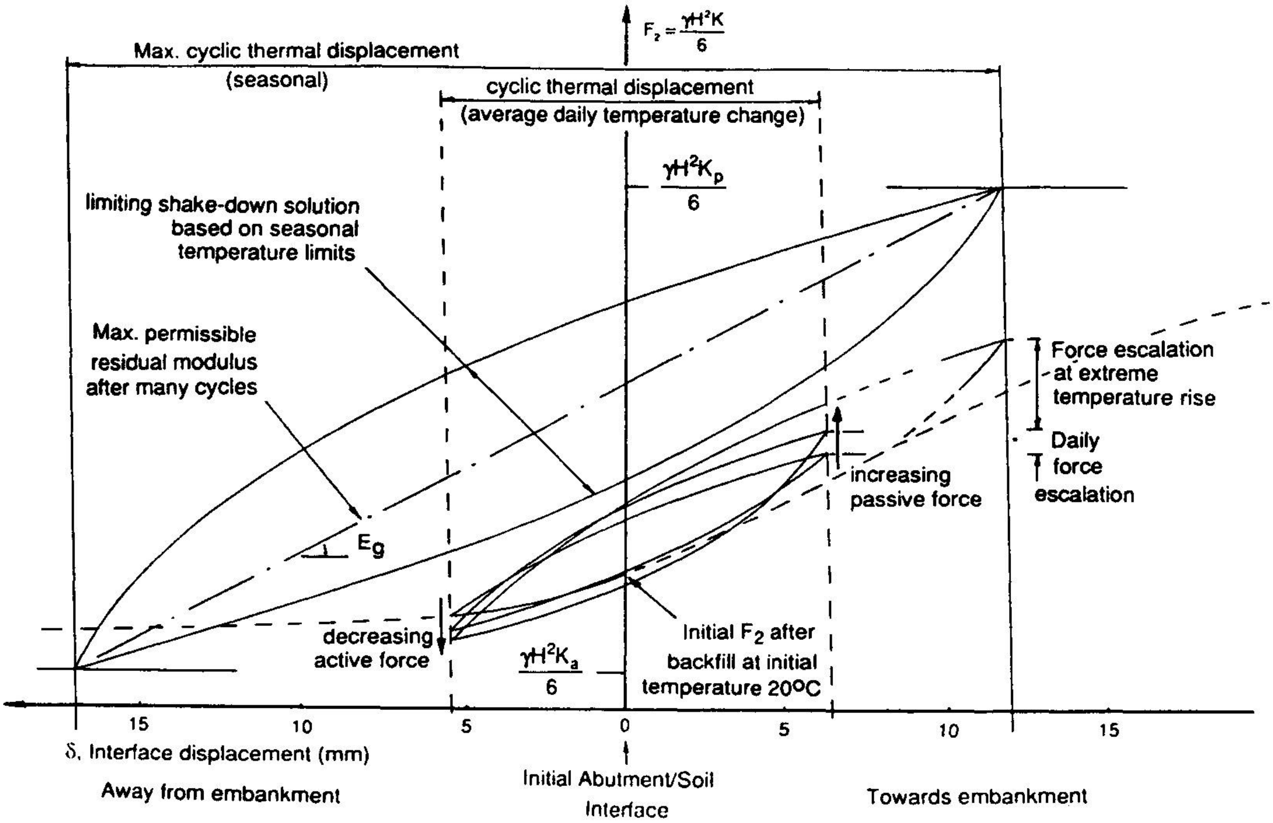


Figure 3. A Soil/Structure Interaction Diagram representing a typical highway concrete bridge (Stiff deck)

5. SHAKE-DOWN STATE FOR STIFF DECK

As shown in Figure 3, for a *typical* stiff bridge deck, the soil/structure interaction mechanism is strain-driven. Concerning the stability of the embankment, the shortening of the bridge deck due to the soil pressure is negligible. For a *Shake-down* solution to exist, the fluctuating horizontal soil stress at the toe of the wall, $\Delta\sigma_u^*$, should be less than the maximum permissible stress defined by ($\mathbf{K_p}$ - $\mathbf{K_a}$). It is then dependent on the residual soil modulus (variable with abutment height, **H**)and the fluatating soil/structure interface movement $\Delta\delta$ (variable with length of deck, **L**). Figure 4 represents the limiting relationship between the residual soil modulus coefficient **N** and the deck length **L**, with the wall height **H** for a temperature variation of 48°C. The residual modulus coefficient **N** increases with number of strain cycles. For sand, $\mathbf{E_g}$ has been found experimentally [1] in the range of 550 (15 cycles of 0.1% fluctuating strain) to 900 (100 cycles of 0.08% fluctuating strain). The actual movements that take place daily in most times of the year are always less than theoretically predicted. As shown in Figure 3, the number of cycles required to escalate the soil pressure by small daily movements is expected to be substantial. Althought the seasonal or extreme variations may give a more significant rise of soil pressure, the number of extreme cycles in the life time of the bridge will be limited.

6. CONCLUSION

6.1 A soil/structure interaction diagram relating the stiffness dimensions of an integral bridge to the soil preperties of embankment is presented. The composite behaviour is analysed as an elastic-plastic shakedown problem. Axial deck stiffness is the dominant feature in dividing the problem in two classes : *stiff* deck and *soft* deck.

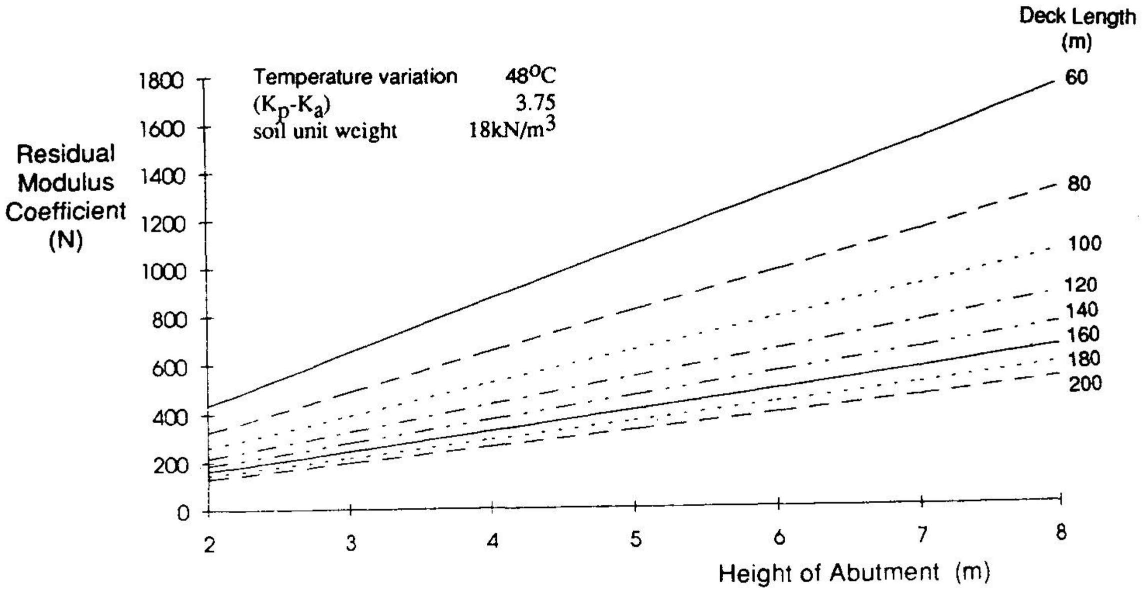


Figure 4. Maximum allowed residual soil modulus coefficient **N** for the deck length **L** and abutment height **H**.

6.2 Figure 2 shows that, an increase in deck length, **L**, increases the free thermal expansion $L\alpha\ \Delta T$ but reduces the axial stiffness, $\mathbf{k_b} = \mathbf{2AE/L}$. Therefore, for the soft deck case, the maximum fluctuating stress in the granular backfill is independent of **L**, although the rate of approaching the shake-down state will be slower for larger **L**. However, due to the possible relatively higher interface displacement away from the soil, significant settlement in the region approaching the bridge deck may cause cracks at road surface.

6.3 Most bridge decks are classified as *stiff*. When the residual modulus of the granular material at shakedown predicts a failure, the bridge designer should consider either modifying the soil properties ($\mathbf{E_g}$) by ground improvements or reducing the axial stiffness of the deck. Another alternative is to span the carriageway over the failure region by a run-on slab.

6.4 Further investigation is required to understand the combined effect of the number of strain cycles and their magnitudes to the ultimate residual soil modulus coefficient **N** and the rate of approaching this value.

6.5 Although the initial temperature and soil pressure during the completion of the bridge will not affect the ultimate shake-down stress imposed on the bridge deck, the ultimate soil/structure interface displacement and the volumetic change of the granular backfill are dependent on them and may cause unacceptable settlement of the embankment. Excessive initial compaction on granular backfill will either reduce the number of strain cycles required to reach the shake-down state or cause dilating failure to the embankment.

6.6 Although the problem of expansion joints has transferred from the bridge to the approaches, it is considered both (a) cheaper to repair damage to the junction between run-on slab and the carriageway and (b) easier to restore highway construction at grade.

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