# Soil-structure-interaction of jointless bridges on piles

## Interaction sol-structure de ponts sans jambages sur pieux

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ABSTRACT: New aspects of modeling the soil-structure-interaction (SSI) concerning jointless non-abutment bridges on piles are given in this paper. Factors affecting the longitudinal stiffness of such a bridge with corresponding discrete finite element (FEM) modelings are presented. A FEM-analysis of a longitudinal load test of an actual highway bridge is also given. When creating modeling principles latest information of soil and bridge behaviour including stress and strain dependencies are utilized.

RESUME: L'àrticle presente de nouveaux aspects de modelation de l'interaction terrain-suructure (SSI) concemant les ponts sur piliers sans jambages et sans joints. Sont egalement presents les facteurs affectant la rigidite longitudinale d'un tel pont avec les modelations d'elements finis discrets correspondants (FEM). Une analyse FEM d'un test de charge longitudinale d'un reel pont d'autoroure est aussi proposee. Lors de la creation de principles de modelation, on utilise les informations les plus recentes concemant le terrain et le comportement du pont, y compris les facteurs dependant de l'effort et de la fatigue.

## 1. INTRODUCTION

### 1.1 *General*

Soil-structure-interaction problems have attracted a lot of attention during the last decades. The growing interest in such problems has resulted in a variety of analysis methods. Several analytical and numerical methods have been introduced lately to solve various SSI-problems. The structural behaviour as well as certain geotechnical aspects, like nonlinearity, are well known in these days. However, only the simultaneous consideration of stress-strain, temperature and time dependencies of soil as well as a numerical method with proper constitutive soil model provides an accurate analysis. Any shortage of these, e.g. linear soil model, analytical solution, coarse system model etc., may lead to inproper analysis.

Among other cases a jointless bridge with rigidly connected piles is a very important application of soil-structure-interaction. The growing interest to construct bridges without expansion bearings between abutments and superstructure is due to numerous advantages, e.g. material, contructing and maintenaince costs as well as better constructing time schedule in relation to conventional bridges. More efficient and economical constructing methods are reguired on these days. The increased knowledge of soil and bridge behaviour as well as the continuously growing computer capacity needed with numerical methods provide good facilities to response to this demand.

### 1.2 *Contents of this paper*

This paper gives some new aspects of SSI-modeling principles concerning factors affecting the longitudinal stiffness of a non-abutment bridge on piles. These factors are the end plate-backfill-, transition slab-backfill- and pile-soil- interaction. These principles are based on the discrete finite element method. The so called **spring-model**, in which the soil is modeled with separate independent linear or nonlinear springs, is applied.

An illustrative example is also given. This presents a FEM-analysis of a longitudinal load test on actual single span jointless highway bridge founded on large diameter bored piles. The results are compared with the corresponding FEM-analysis, in which the longitudinal stiffness factors are simultaneously taken into account.

## 2. PRINCIPLES OF SSI-FEM-MODELING

### 2.1 *General*

The well known **spring-model** provides a handy way to simulate the behaviour of various SSI-problems. Similarities to the subgrade reaction theory exist, although certain modifications (Koskinen, 1990, 1991, 1994 and unpublished) have been accomplished. The model excluding the definition of initial stiffness is basicly similar to that presented in references e.g. Greiman et all (1986 and 1988). Although the continuity is neglected, the accuracy in principle is good. This is also verified by the author (Koskinen, 1990) in the case of a laterally loaded pile. Significant advantages of the model are the absense of problems with plasticity algorithm as well as the high convergence rate.

### 2.2 *Determination of springs for end plate-backfill-interaction*

The force-displacement-relationship (Figure 1) till half the ultimate value is connected with the elastic modulus $E_{bf}$ defined with triaxial test. The initial stiffness $k_{bf}$ is defined according to equation

$$k_{bf} = E_{bf} / H_{ep}, \tag{1}$$

in which $H_{ep}$ denotes the hight of the end plate. The displacement corresponding half of the ultimate spring force is determined according to equation

$$y_{ie} = 0.5 \bullet p_p / k_{bf}. \tag{2}$$

In this equation $p_p$ denotes the ultimate (passive) stress state of soil.

The ultimate total spring force is defined with the stress state and friction angle according to equation

$$F_p = 0.5 \bullet \gamma' \bullet H^2_{ep} \bullet B_{ep} \bullet K_p. \tag{3}$$

The symbol $K_p$ denotes the passive earth pressure coefficient and $B_{ep}$ the width of end plate. The corresponding displacement is determined according to equation

$$y_p = 4 \bullet y_{ie}. \tag{4}$$

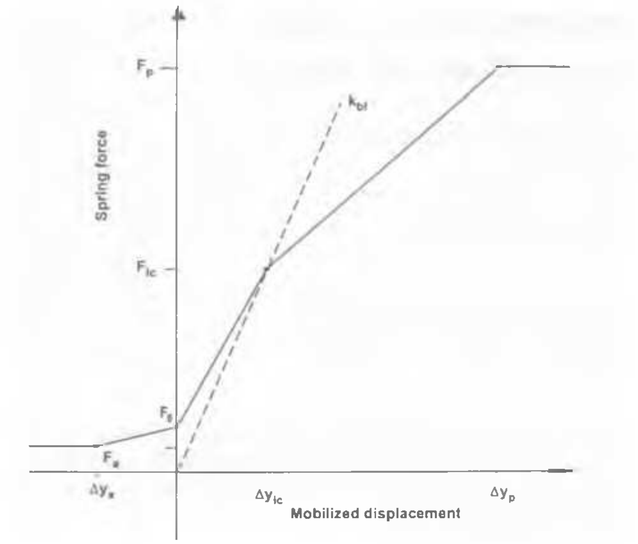


Figure 1. Determination of spring for end plate-backfill-interaction.

The force corresponding the initial stress state is obtained from equation

$$F_o = 0.5 \bullet \gamma' \bullet H^2_{ep} \bullet B_{ep} \bullet K_o . \tag{5}$$

and the force corresponding the active state with equation

$$F_a = 0.5 \bullet \gamma' \bullet H^2_{ep} \bullet B_{ep} \bullet K_a . \tag{6}$$

The displacement corresponding the active state is obtained from equation

$$y_a = -0.25 \bullet y_p . \tag{7}$$

The total spring force is devided into partial springs affecting the nodes of the end plate elements.

2.3 *Determination of springs for transition slab-backfill-interaction*

The envelope path of the total spring may be modeled according to the principles presented in Figure 2 (Koskinen, unpublished). The casting technique as well as the effect of plastic sheet under the slab may be taken into account in the determination of the total friction spring, which can be devided into partial springs to the nodes of end plate.

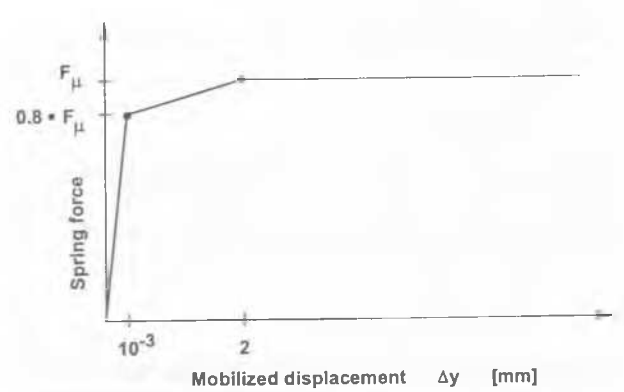


Figure 2. Determination of friction spring for transition slab-backfill-interaction.

The total friction spring is defined according to equation

$$F_\mu = a \bullet b \bullet \sigma_v \bullet \mu \bullet A_{ef} . \tag{8}$$

in which the reduction factor a denotes the effect of possible plastic sheet under the slab as follows:

a = 1.0 (no plastic sheet)

a = 0.9 (with plastic sheet)

The reduction factor b denotes the effect of casting technique as follows:

b = 1.0 (cast-in-place slab)

b = 0.9 (element slab)

The friction coefficient between concrete surface and gravel is

$\mu$ = 0.5 (concrete-gravel)

and $A_{ef}$ denotes effective area of the slab.

The mobilized displacements of ultimate friction spring force may be considered as

$y_{mob} = 2.0$ mm (gravel).

The transition slab is generally constructed on the compacted fill. During a relative short period of time the interface between the slab and fill will however be gapped. This phenomenon is valid in the connection areas of the transition slab and end plate. In case of precast slab this phenomenon is emphasized.

The effective frictional area $A_{ef}$ of the slab may also be significantly smaller than the nominal area. This is due to the abovementioned gapping effect as well as the flow of fill underneath the wing walls. The effective area may be considered according to Figure 3.

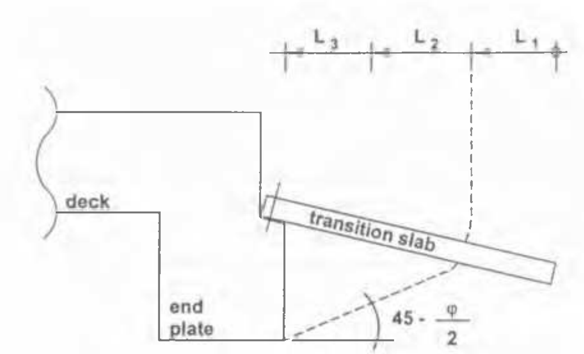


Figure 3. Determination of effective area of transition slab

In the determination of the effective area following phases must be considered (Koskinen, unpublished);

1) situation immediately after construction;

- small displacement level

$$A_{ef} = B \bullet L_3 \tag{9}$$

- large displacement level

$$A_{ef} = B \bullet L_1 \quad (10)$$

2) situation in long period of time

- small displacement level

$$A_{ef} = B \bullet L_2 , \quad (11)$$

in which

$$L_2 = 0.75 \bullet L_3 \quad (12)$$

- large displacement level

$$A_{ef} = B \bullet L_1 \quad (13)$$

In abovementioned equations (9) - (13) the effective width B may be taken as 90 % of the nominal value. The intermediate values of those presented may be interpolated linearly.

### 2.4 *Determination of springs for pile-soil-interaction*

The early part of the force-displacement-relationship is determined with the elastic modulus $E_s$, which gives the initial stiffness (Figure 4) estimated as an average value of Broms (1972), Pyke et al (1984) and Swane et al (1982) according to equation

$$k_h = 0.83 \bullet E_s / D , \quad (14)$$

in which D is the diameter of the pile. The corresponding displacement is obtained from equation

$$y_{ic} = 0.5 \bullet p_p / k_h . \quad (15)$$

The latter part of the path is defined with the ultimate strength of soil. The ultimate spring force with cohesionless soil is defined with equation

$$F_p = 4.4 \bullet \gamma' \bullet z \bullet K_p \bullet s \bullet D. \quad (16)$$

The ultimate spring force with cohesive soil is defined with equation

$$F_p = 7.5 \bullet s_u \bullet s \bullet D. \quad (17)$$

In these equations

$s_u$ = undrained shear strength of soil
s = spacing of soil springs

The corresponding displacement is obtained from equation

$$y_p = 4 \bullet y_{ic} . \quad (18)$$

The spring force in the initial stress state is given by equation

$$F_o = \sigma'_v \bullet K_o \bullet s \bullet D. \quad (19)$$

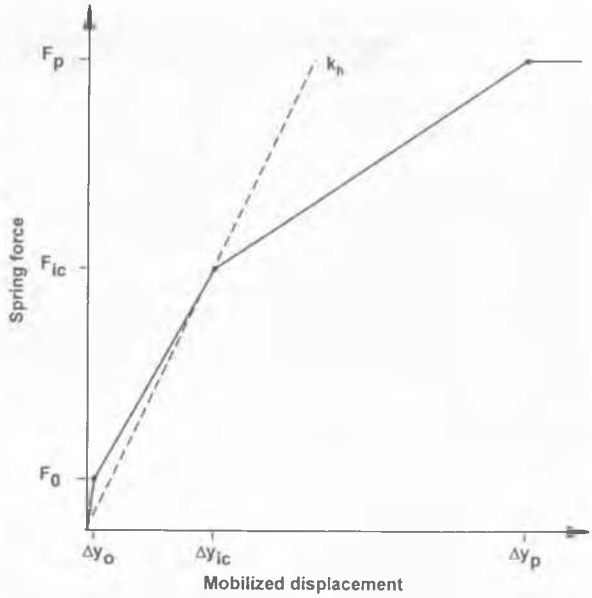


Figure 4. Determination of springs for pile-soil-interaction.

## 3. EXAMPLE CALCULATION, TEURO HIGHWAY BRIDGE, LOAD TESTS AND FEM-ANALYSIS

### 3.1 *The bridge*

The Teuro bridge is a continuous and cantilevered non-abutment single span highway bridge (Figure 5) founded with rigidly connected bored piles. It is located in the southern Finland between Hausjärvi and Lammi. The bridge is straight and the total length is L=33m. The effective width of the deck is B=7.4m. The average length of piles reached to the bedrock is L=9m. The diameters of support columns are D=0.9m. The bridge has end plates at both ends with width $B_{ep}$=7.4m, hight $H_{ep}$=1.9m and thickness $t_{ep}$=0.6m. Transition slabs with size B • L = 3m • 7.2m are constructed.

### 3.2 *The ground conditions*

The base soil of this highway bridge site consists of silt and sand deposits with altering degrees of relative density. In the support 1 there is a layer of moraine on the rock having a relatively high degree of relative density. The surface of rock is at the hight ∇ +76.7 in both supports. Weigh soundings, dynamic probings, vane shear tests as well as samplings were carried out on the site. The laboratory tests consisted of triaxial and oedometer tests. The ground conditions are presented in Figure 5.

### 3.3 *Load tests and experimental arrangements*

The bridge was longitudinally loaded by two forces (2x330kN) acting at the end support columns at the hight of 1.25m below the deck according to Figure 6. Longitudinal displacements, earth pressures on end plate and strains of reinforcement bars in columns at 0.3m below the deck were measured during the test.

### 3.4 *The FEM-analysis*

#### 3.4.1 General

In order to analyse the bridge load test theoretically a special spring-model (Koskinen, unpublished) was developed. Although

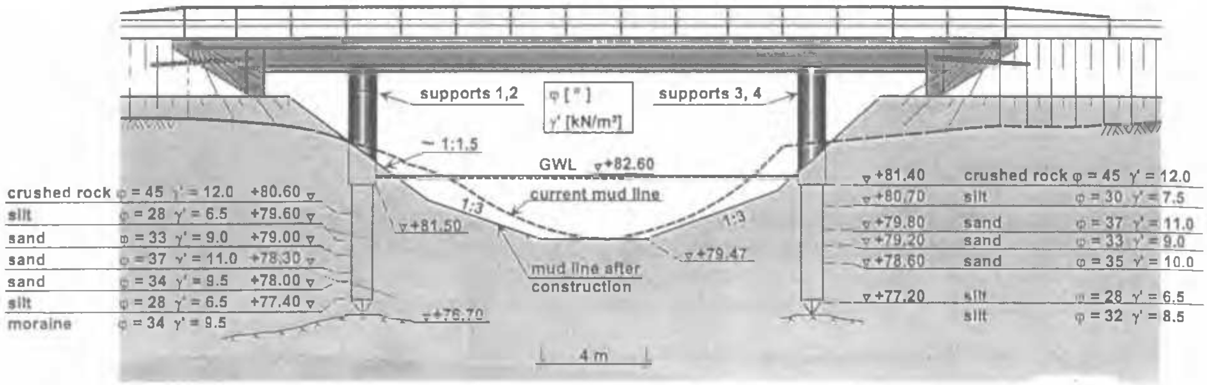


Figure 5. Bridge and ground conditions

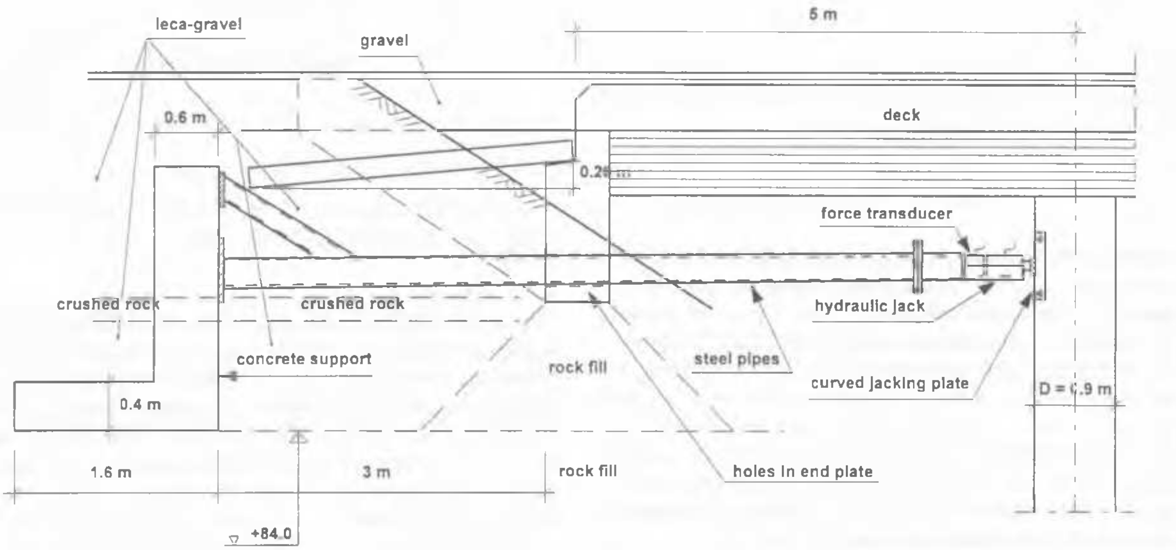


Figure 6. The experimental arrangements.

the geotechnical point of view was emphasized, no liberty could be taken when modeling the structure in a proper way. The model was analysed using the ABAQUS-FEM-code (Hibbit et al, 1992). The model was run in the Alpha-Cluster-supercomputer owned by the Computer Center at the Tampere University of Tehnology. The DOF-value of the model was 1350 thus being relatively moderate.

### 3.4.2 The structural modeling

The deck of the bridge as well as the end plates were modeled by four noded shell elements having six degrees of freedom per node. The thickness of these elements were t=1m. The reinforcement bars in the columns and piles were modeled according to special options (Hibbit et al, 1992). The concrete columns and piles as well as the steel cover were modeled with two node beam elements. The material properties of concrete and structural steel as well as reinforcement bars were considered according to actual behaviour.

### 3.4.3 The geotechnical modeling

In this analysis model (Figure 7) all the interactions affecting the longitudinal stiffness were modeled with springs according to the principles presented in section 2. The springs provided nonlinear behaviour, although the actual behaviour appeared to be linear No vertical springs were applied being unnecessary in this case.

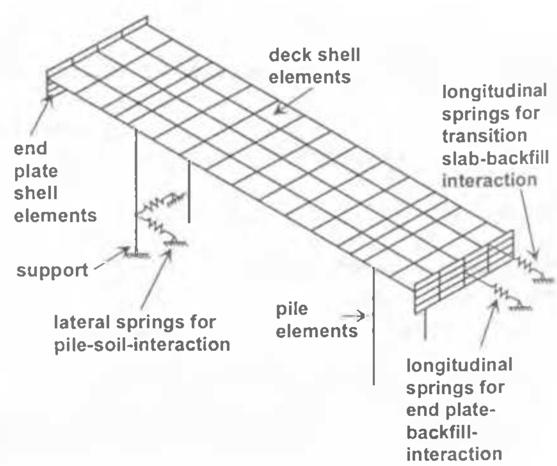


Figure 7. The total spring-model.

At the time of load tests the backfill was constructed only with the leca-gravel fill and the transition slab on it. The properties of leca were;

$E = 5.0$ Mpa
$\nu = 0.15$
$\gamma = 6.0$ kN/m$^3$
$\varphi = 35^\circ$

These properties with equations (1) - (7) resulted in the backfill spring, which was devided into partial springs presented in Figure 8.

| node | spring type | spring forces [kN] $F_a$ | $F_o$ | $F_{lc}$ | $F_p$ |
|---|---|---|---|---|---|
| 2 | *) | 0.27 | 0.43 | 3.73 | 7.46 |
| | **) | 0.13 | 0.21 | 1.86 | 3.73 |
| 3 | *) | 0.82 | 1.30 | 11.19 | 22.38 |
| | **) | 0.41 | 0.15 | 5.59 | 11.19 |
| 4 | *) | 0.99 | 1.57 | 13.51 | 27.03 |
| | **) | 0.49 | 0.79 | 6.75 | 13.51 |
| 5 | *) | 0.49 | 0.79 | 6.75 | 13.51 |
| | **) | 0.25 | 0.39 | 3.39 | 6.75 |

*) middle springs
**) edge springs

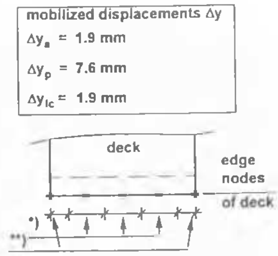


Figure 8. Partial backfill springs.

The subscript ic denotes the intersection point of the path (Figure 1), where the behaviour changes from elastic to plastic.

An example of stiffness distribution in the soil around piles of support 1 is presented in Figure 9.

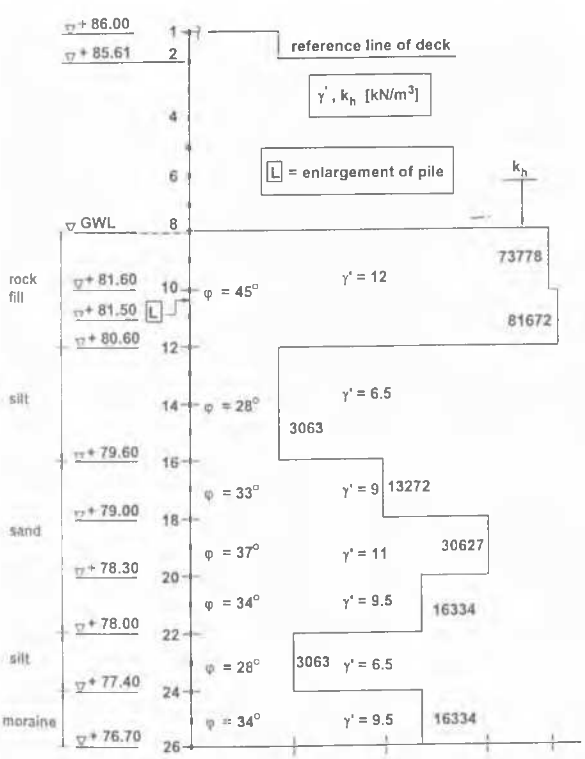


Figure 9. Horizontal stiffness distribution in the soil around pile of support 1.

## 4. COMPARISON OF TEST AND FEM-RESULTS

### 4.1 *Longitudinal stiffness and displacements*

The longitudinal displacements due to the applied load $\Delta F$=660 kN were small. The average observed value was $\Delta y$=2.05 mm. Using this value one gets the longitudinal stiffness of this particular highway bridge $K = \Delta F / \Delta y = 0.66$ MN / 2.05 mm = 322 MN/m. This value includes the effects of transition slab, backfill, stiffness of columns and piles as well as soil around piles.

The theoretical displacement was $\Delta y$=3.7 mm. The discrepancy of results is caused by the fact, that the stress history of backfill and the soil around piles as well as the soil properties in the immediate vicinity of the soil-structure-interface cannot be estimated accurately.

### 4.2 *Earth pressures*

The earth pressures on the end plate were measured in three points at half the hight of the end plate. The pressure transducers gave results approximately $\Delta p$=2kPa. The empirical result thus gives a stiffness of backfill with such a displacement equal to $K = \Delta p / \Delta y$ = 2kPa / 2 mm = 1.000 kN/m$^3$. The theoretical earth pressures were clearly greater than empirical ones. This is however due to possible flow of the loose leca-gravel underneath the wing walls. Such flow takes place due to the lack of external embankments outside of wingwalls. It must also be noted, that the stress history of backfill cannot be estimated accurately.

### 4.3 *Strains of reinforcement bars*

The observed strain of the main reinforcement bar in the support 1 was $\Delta\varepsilon$ = 75 micro strain units, while the theoretical one was $\Delta\varepsilon$ = 80 micro strain units. The discrepancy is thus guite nominal. The theoretical value is larger than the empirical one, which indicates a greater theoretical bending moment. The result is analogic to displacements.

## 5. CONCLUSIONS

This paper contains the SSI-modeling principles of factors affecting the longitudinal stiffness of a jointless bridge on piles. The principles presented here provide reasonable accuracy (Koskinen, unpublished) and are thus recommended for use. The approach for the determination of soil behaviour with springs is given, which is applied in the example calculation.

The comparisons show, that clear although not extremely large discrepancies between the test results and FEM-analysis take place. Such differencies are likely to appear due to the numerous factors affecting the longitudinal stiffness of such a system.

An additional inaccuracy with the analysis is introduced due to the shortcomings in the parameter estimation procedure. The properties of lower layers of soil around piles were estimated only on the basis of soundings. Additional uncertainties appear, because soil properties cannot be accurately predicted in the immediate vicinity of structure. The significance of this is emphasized, because the strain level in these tests was low.

The results show, that the behaviour of such a bridge is linear at the load level applied. The utilization of ultimate longitudinal capacity is thus low. The same tendency is observed with break load tests (Koskinen, unpublished), although not presented here. The system is thus very stiff in relation to the applied load.

The study gives an estimation of the longitudinal stiffness of a typical highway bridge. The stiffness given is naturally valid only at the load level applied. A teoretical study providing all those factors affecting the longitudinal stiffness shows, that with the load F=660kN the relative portions of these factors are roughly as follows;

- end plate-backfill-interaction 2/3
- track-sleeper-ballast-deck-interaction 1/5
- pile-soil-interaction, rigid connection 1/5

These portions are somewhat influenced by the load level, but a good estimation of mutual relations is however given.

## 6. ACKNOWLEDGEMENTS

This paper has been prepared on a basis of a research project (Koskinen, unpublished) concerning the longitudinal behaviour and SSI-problems connected with continuous non-abutment concrete bridges with rigidly connected large diameter piles. It has been carried out with support of The Finnish Academy of Science, The Finnish National Road Administration, The Finnish National State Railways, Rautaruukki Ltd., MVR-Haka Ltd. and Kreuto Ltd. It is supervised by professor Jorma Hartikainen at Tampere University of Technology.

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