THE LONGITUDINAL STIFFNESS OF A JOINTLESS BRIDGE FOUNDED ON LARGE DIAMETER BORED PILES

RIGIDITE LONGITUDINALE D'UN PONT SANS JOINTS FONDE SUR PIEUX A GROS DIAMETRE MOULES DANS LE SOL

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SYNOPSIS: As a part of a special research project concerning the geotechnical design of jointless bridges founded on large diameter piles, the results of horizontal load tests on actual jointless 3-span railway underpass (L = 49 m) are presented in this paper. The horizontal response due to the longitudinal load F = 500 kN was observed and compared with FEM analysis consisting of a 3D soil spring model (2780 DOF) and a 3D material model (38200 DOF). The goetechnical properties of the soil were determined through the standard laboratory and in-situ tests. The structure was modelled with ordinary shell and beam elements including actual reinforced concrete material behaviour.

1. INTRODUCTION

General

During the last decade a growing interest to construct the bridge structure without expansion joints between the abutments and the superstructure has raised. This is due to the numerous advantages in relation to the conventional type of bridge structure. Such advantages could be mentioned the material, constructing and maintenaince costs, which vanish when choosing the jointless type of bridge structure, i.e. the rigid connection between substructure piles and the bridge deck. In this context should also be mentioned the up moved time schedule factor. In addition to the issues mentioned above, an increased need to develop and optimize the jointless bridge structure has came up. This need is due to the increase of speeds especially in railway bridges as well as the demands for more efficient and economical constructing. The increased knowledge of the soil behaviour as well as the continuously growing computer capacity provide excellent facilities to response to this need. In the current design procedure especially the time and strain dependencies of soil parameters are not taken into account well enough. This may in turn cause overestimation of stresses and strains in the bridge structure thus leading to uneconomical design.

The use of the bearingless type of structure provides not only economical advantages, but also structural problems. In this category must be included i.e. the structural and economical length limit of the bridge deck. This limitation is mainly due to the relatively high bending stresses induced especially into the piling at the ends of the deck. This is caused by the thermal expansion and contraction combined with possible deck shrinkage effects. This problem can however be - to a certain extent - taken care of by flexible piling. This in turn creates an additional reguirement into the design procedure; the thermal effects should not be taken into account in ultimate limit state, because it leads to significant overestimation of pile structure. The thermal effects will take place anyway, so there is no point of resisting this phenomenon. The workability of connection elements must however be checked.

The use of IAB-structure when designing bridges is thus a straightforward choise. This type of bridge will introduce disadvantages and advantages, the latter of which are however preferable. It causes additional reguirements for the designer but gives also an opportunity to optimize the structure to a remarkable extent. As far as structural engineering economics is concerned, this choise offers an obvious improvement in better direction.

Linking this paper to the research project

To provide response to the needs mentioned above reguires not only elaborate and updated knowledge of the behaviour of soil but also full scale load tests. Carrying out load tests with actual structures will not only suppress the scale effect but can also be relatively cheep if the design and timing of instrumentations are properly connected with constructing time schedule. Utilizing the facilities provided by numerical methods, such as FEM, and high computer capacity currently available give the designer an excellent opportunity to check whether the theory and practice match with each other or not. In order to check the accuracy of current design procedure by combining the latest information of soil behaviour, FEM-technigue, high computer efficiency, proper parameter estimation, elaborate constitutive soil model and empirical verifications, this research project was thus started. **The results of this paper thus belong to the empirical part of this project.**

The aim of this research project is to examine, how the break loads and temperature effects should be taken into account in the design of joinless bridges from geotechnical point of view. Factors affecting the horizontal response, such as end plates, the stiffness and number of piers, railway tracks, transfer slabs etc., will be examined too. A further goal in the project is also to find out the technical and economical length of jointless bridge structure in the frame of updated knowledge concerning the behaviour of soil and computational methods. In this context the type of pier - drilled pier, steel pipe pile, composite pile - will be examined as well. The analyses will be verified by carrying out not only lateral loading tests but also brake load tests with trains and vehicles. In addition to this seasonal long term measurements will be done.

In the recent design practice several aspects providing more economical substructure are not taken into account. For example the effect of continuously welded tracks or transfer slabs are considered negligible, due to which the estimated horizontal deflections are considerably higher than the real values. This in turn causes overestimation of the stresses in piers. In addition to this should be mentioned the dynamic soil properties under break loads in bridges; the dynamic response of soil can be considerably higher than the corresponding static values thus leading to - if taken into account - smaller lateral deflections and smaller stresses in piers.

The current finnish geotechnical code of practice for design of bridges will be updated by these new aspects verified by load tests. This updated code of practice will consider highway bridges and railway bridges separately and will also propose the use of pile type according to the length of the bridge and the soil conditions.

The contents of this paper

This paper contains results of a series of full scale load tests on actual bearingless railway bridge. The bridge is a 3-span jointless and continuous reinforced concrete underpass, the length of which is 49 m (figure 1). The bored piles (d = 0,9 m - 1,2 m) were reached to the bedrock through clay, silt, sand and base moraine deposits. The longitudinal load F = 500 kN was applied to the other end of the bridge. The loading program consisted of 13 separate tests. The response of structure was then measured and the results were averaged for the comparison of FEM-analysis. The response observed from tests contains first of all the longitudinal, transversal and vertical deflections, the first of which were measured from both ends. The bridge structure is slightly curved, thus a minor transversal deflection was observed. The longitudined stiffness in the early elastic range was then easy to be estimated. In order to determine the shear of tracks of the horizontal capacity the strains - and thus the forces - acting in the tracks as well as the relative slide between the tracks and the deck were also measured. In addition to this the earth pressures in three points on the opposite end plates were also observed. The pressure - deflection relation was then compared with FEM-analysis. To obtain the stresses in the piles the strains of steel rebars in piers were measured. Comparing the strains of rebars with FEM-analysis is very handy because the corresponding values are easily obtained from reinforced concrete beam elements in the FEM-models. The geotechnical parameters of the subsoil as well as the coarse back fill were determined with standard laboratory equipment and in-situ measurements. The first FEM-model (2780 DOF) is based on nonlinear soil springs from the geotechnical point of view. The other model (38200 DOF) in turn uses real soil material behaviour with solid soil elements.

2. THE BRIDGE STRUCTURE ANALYSED AND THE GROUND CONDITIONS

The type of bridge analysed

The bridge structure analysed (figure 1) is a jointless continuous three spanned railway underpass. It is located in the southern part of Finland in about the half way between Helsinki and Turku. The construction material is reinforced concrete with material parameters E = 29,6 GPa, σ_y = 24,5 MPa and ν = 0,2. The reinforcing rebars used belong to the B500P class steel material with σ_y = 500 MPa. The bridge is slightly curved (R = 3280 m) and cantilevered (2 m). The effective length of the structure is 2 + 12 + 14 + 12 + 2 m = 42 m. The total length with wingwalls (2 x 3,5 m) is thus L = 49 m. The cross-section is 6 m wide and has only one pair of continuously welded tracks (UIC 54) with 0,55 m track ballast layer. The bridge is founded on bored piles rigidly connected to the deck and reached to the bedrock. The length of the piers are approximately 25 m. The diameter of the upper parts of the end support piers are D = 0,75 m and middle support piers D = 1,05 m. The diameter of the lower parts of piers are D = 0,9 m and D = 1,2 m, respectively. The structure has end plates at both ends, the dimensions of which are; breadness B = 5,6 m, thickness t = 0,75 m and hight H = 1,9 m together with the deck. The backfill is supported by end bearing piled concrete raft slab. No transfer slabs were constructed.

The ground conditions

The highway underpassing the analysed railway bridge goes through a clay opening, the undrained shear strength of which was observed from the fall cone tests from undistrubed samples and "in-situ" vane shear tests. The slopes were covered with geotextile, gravel and certain cover stone material. The properties of the soil below the clay layer were determined with the combination of dynamic probing tests and the weight sounding tests. The properties of the silt deposit under the clay layer were given as for a friction type of soil. This was because it was mainly coarse and it was not possible to conduct a vane shear test in this deposit. Under this silt deposit there were layers of sand with altering degrees of relative density. Excluding the support 1 there was a layer of moraine of minor thickness on the rock having a relatively high degree of relative density. The piers were reached to the bedrock, the surface of which was situated at hight levels altering from -17,8 m to -12,3 m. There was a slight variation of ground conditions between the neighbouring piers in the end supports. Additional vane shear testing as well as sampling were carried out in the site after finishing the construction and the opening, because the disturbances during the work process most evidently have changed the properties of the soil. Due to the undergoing road opening the ground water table had to be permanently lowered for two meters. The ground conditions with the corresponding parameters for the FEM-analysis (spring model) as well as the bridge structure are presented in figure 1.

3. LOAD TESTS AND EXPERIMENTAL ARRANGEMENTS

The loading program consisted of 13 separate tests, the force-time dependences of which were linear. In the tests the force was created from zero to the total value F = 500 kN in about Δt = 13,6 s. Due to the loading arrangement (figure 2) and the bending moment capacity of piers this was the maximum allowable load. The total force was kept constant for a period of about t = 4 s, after which it was rapidly removed. This arrangement was applied through all the tests, because in preliminary tests no scatter in response was observed in comparison with slower load removal. The total force consisted of two longitudinal forces (2 x 250 kN) both acting on the end support piers (support 1) at the hight of 1,25 m below the deck (figure 2). The forces were created by two hydraulic jacks and controlled by force transducers. The measurements were carried out with sixteen simultaneously active data logger channels. The information was recorded as a function of time, allthough the nature of loading was static. The deflections observed in longitudinal, transversal and vertical directions were measured with special deflection transducers (potentiometers), the accuracy of which are about 10^{-5} m. The deflections were measured in relation to a fixed scaffolding located near the loaded end of the deck (figure 1). The longitudinal deflections were also measured at the opposite end of the bridge. This value is actually the relative movement between the back pile slab and the end plate of the deck. The relative movement between the deck and the tracks were also measured with deflection transducers (potentiometers). This - combined with the recording of strains and thus also forces in the tracks - was carried out in order to find out the share of horizontal capacity received by the track system. The earth pressures on the opposite end plate were observed in three points in about the hight of H/2 of the end plate. The capacities of round shaped (d = 0,2 m) pressure transducers are 200 kPa. No water was absent in the backfill. The strains in the piers were measured 0,25 m below the deck and also in the jacking point. The strain gage recording on the moist concrete surface was not succesfull and thus the gages were installed on the smoothened rebar (d_r = 32 mm) surfaces.

4. THE FEM-ANALYSIS

General

In order to analyse the bridge and the test results theoretically two three dimensional FEM-models were developed. Allthough the geotechnical point of view was emphasized in this context, no liberty could be taken in modelling the structure in a proper way. The models were analysed with ABAQUS-FEM-code. The smaller model was ran in the Kubota 3000 - minisupercomputer owned by the Computer Center of Tampere University of Technology. The larger model had to be ran in Cray-supercomputer owned by the State Computer Center in Helsinki. Despite the 3-D character of the model the total number of DOF of the first model was only 2780 thus being relatively moderate. The elapsed CPU-time per increment with the first model (Kubota 3000-msc) was about 18.9 sec and with the larger model (Cray-sc) about 290 sec. The loads applied in the analyses in addition to the jacking forces were the selfweight of the concrete structure, the track ballast and the track-sleeper-system. In the following texture the spring model

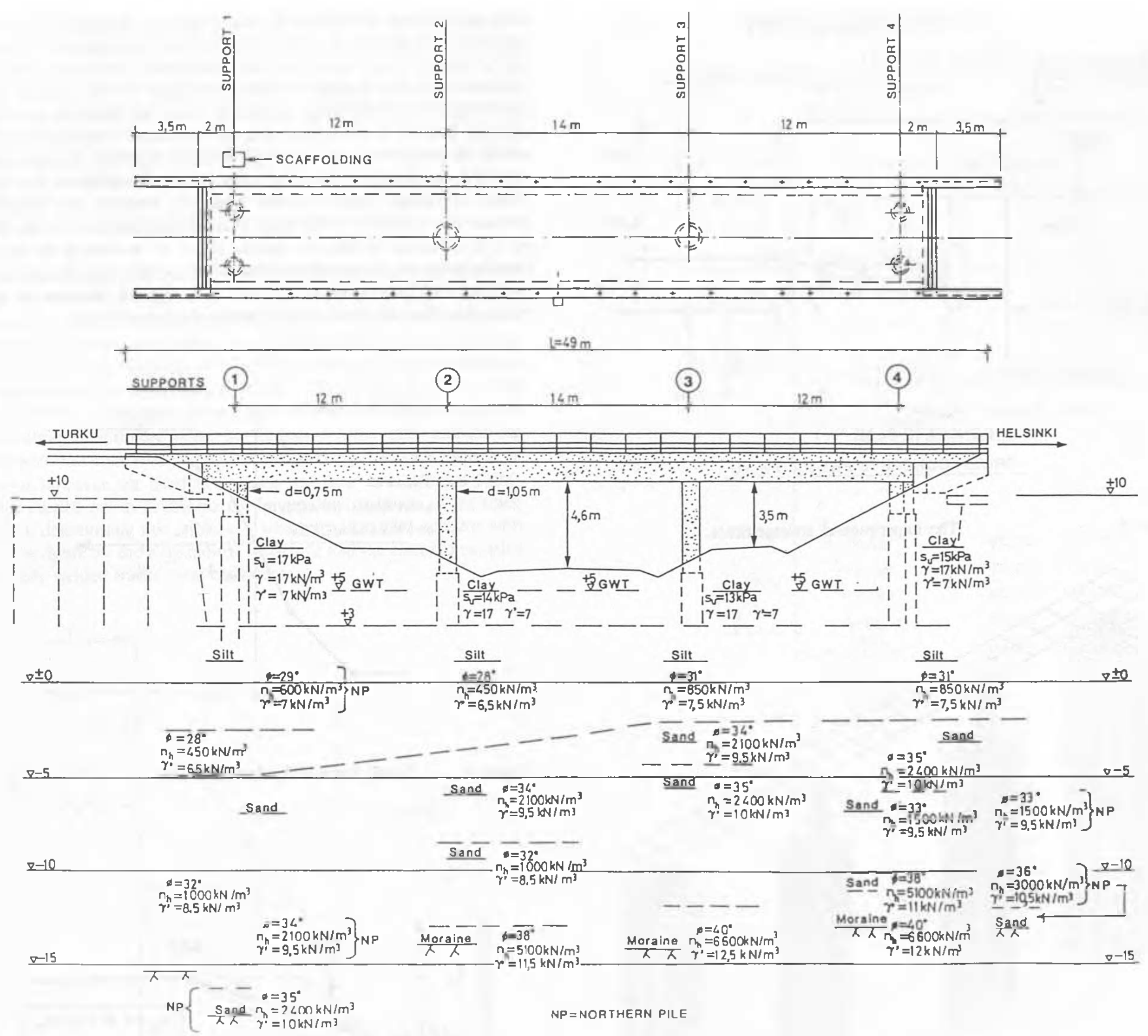


Figure 1. The ground conditions and the bridge structure.

(2780 DOF) and the material model (38200 DOF) are called as FEW3D- and FECM3D-models, respectively.

The structural modelling

The superstructure of the bridge was formed with 92 (FEW3D) four-noded shell elements thus having six degrees of freedom per node. The thickness of these elements forming the deck was t = 1 m. The continuously welded track system was not modelled in particular, but it was considered with altering degree of fixity in relation to the deck - see next part; determining the values of the deck surface friction springs. The end plates were not modelled in particular either, because the effect of the back fill could be taken into account without them in this particular loading case. The steel rebars could also be considered according to the actual reinforcement with special options /1/. The curved shape of the deck (R = 3280 m) was also taken into account. In order to reduce the number of DOF only one half of the real model is analysed in FECM3D-computation (figure 3). The bored pile structure (FEW3D) was modelled with 303 two noded (l = 0,5 m) beam elements with six degrees of freedom per node. In the FECM3D-model the reinforced pile structure was in turn presented by solid elements. The inner (not reinforced) and outer part of the pile was modelled with 6-noded prisms and 8-noded linear bricks, respectively. The reinforcement was considered in similar way as in the shell elements. The elastic and plastic behaviour of concrete as well as the steel rebars were given for the input, allthough the behaviour of the structure was estimated to be elastic. The piers were reached to the bedrock, thus their ends were fixed in vertical direction. To avoid numerical problems additional springs with nominal stiffness were also applied to the ends of the piers.

The geotechnical modelling

In the FEW3D-model the geotechnical part of the analysis was based on modelling the soil with horizontal nonlinear springs and the subgrade reaction method. This is a well-known procedure to represent the behaviour of soil. The popularity of the method is due to the ease and handiness of use not to mention the relatively good accuracy also ascertained by the author /5/. As additional advantages of this method should also be mentioned the

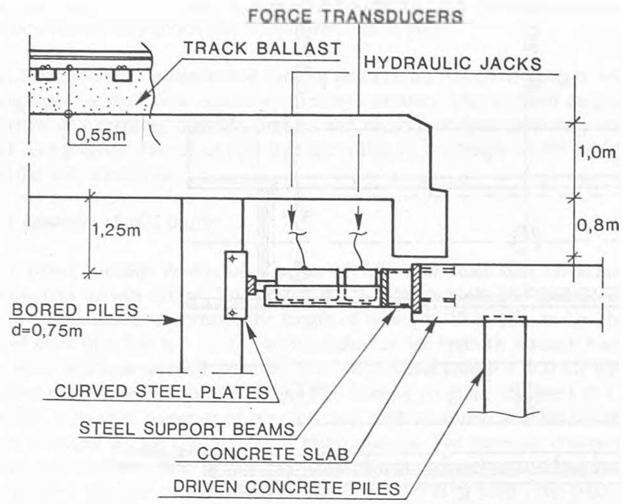


Figure 2. The experimental arrangements.

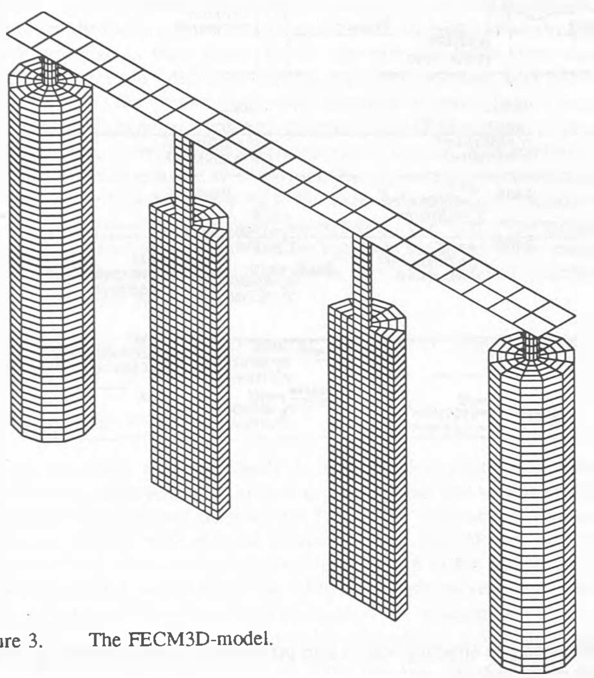


Figure 3. The FECM3D-model.

insensitivity of problems with plasticity algorithm and generally very rapid convergence. The mathematical problems with soil falling into the gap behind the pile when deflected horizontally is also avoided. The initial geostatic analysis step for checking the stress field compatible with applied loads and boundary conditions is not needed either. In the model applied here the initial stress state was not considered, because the effect of it will be examined in the forecoming versions. The values of the elastoplastic soil springs connected to the pier element nodes are given as spring force - deformation couples, the example of which in cohesionless soil is presented in figure 4. Figure 5 shows the determination of the subgrade coefficient, from which the corresponding deflection y is calculated. The difference between the modified value (MM) and the traditional value (TM) of subgrade coefficient for friction soil is also presented. Figure 6 in turn presents the distribution of subgrade coefficient k_h with depth in friction type of soil. In cohesionless soil the subgrade coefficient is assumed to depend on undrained shear strength s_u and pile diameter d but not on depth. The subgrade reaction coefficient n_h was estimated according to the present finnish code of practice for driven piles /4/ and is presented in figure 7. The lateral springs were active in both horizontal directions. An example calculation of soil springs is shown in figure 8. No vertical springs - simulating the possible shape resistance - were set along the piers, because the main interest of this project was the horizontal response in addition to which the piers were reached to the bedrock. Interface elements were not applied in the structure due to the nature of the method either. The weakened values of lateral springs in the boundary between the cohesive and cohesionless soil layers were taken into account according to the reference /4/. The upper part of the clay deposit (z = 1 m) as well as the slope cover material were not considered because of the obvious gap formation and the very small deflection due to the load level applied. Because of the latter reason the effect of slope was not taken into account either.

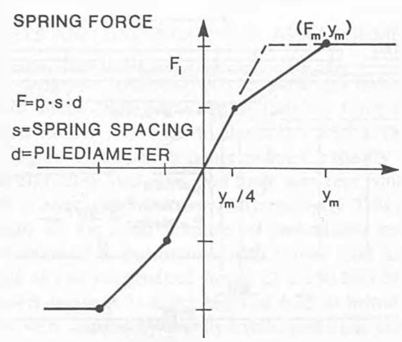


Figure 4. Spring force-deflection relations.

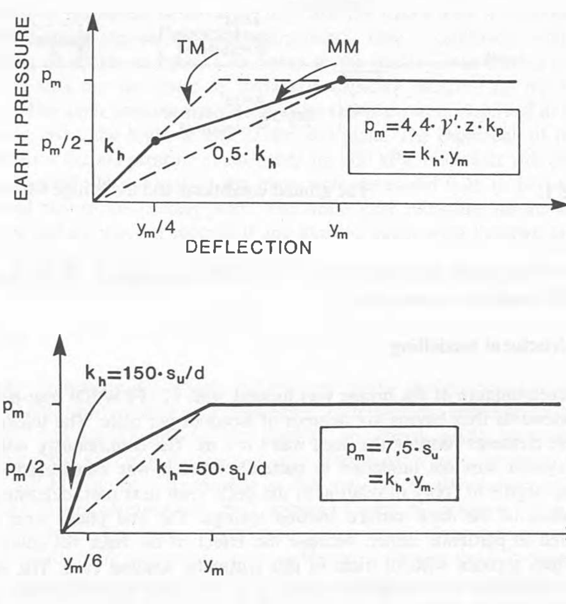


Figure 5. Determination of subgrade coefficient k_h.
a) cohesionless soil
b) cohesive soil.

In the FECM3D-model the cubic soil elements were given the actual elastic properties determined from standard laboratory and "in-situ"-tests. Neither in this model was the initial stress state considered because the effect of it will be examined in the forecoming versions. The "effective diameter" of the soil column around the pile was taken egual to $d_{ef} = 2$ m, which can be assumed as reasonable because of the low load level. Inferface elements were set between the pile and soil elements, allthough there is not much use of them in such a low load level. The upper part of the clay deposit (z = 1 m) as well as the slope cover material were not taken into account due to the assumed gap phenomenon and the very low deflection level.

For both models the effect of tracks as a part of the horizontal capacity was determined by conducting a special load test. The purpose of this was mainly to determine the friction between the ballast and the concrete deck surface. This resistance factor was then applied as a corresponding series of nonlinear lateral springs acting on the deck surface. The stiffness of the deck surface friction springs due to the track-ballast-system is determined as follows. The spring system between the deck surface and the track system was modelled with two individual springs (figure 9). Spring 1 takes into account the relation between the deck surface and ballast assuming the track-system to be fixed. It is however not fixed in practice and thus an additional FEM-analysis was carried out to determine the longitudinal stiffness of the track system. When determining this stiffness - which is taken into account with the spring 2 in figure 9 - the longitudinal resistance and the elastic properties of continuously welded tracks were considered.

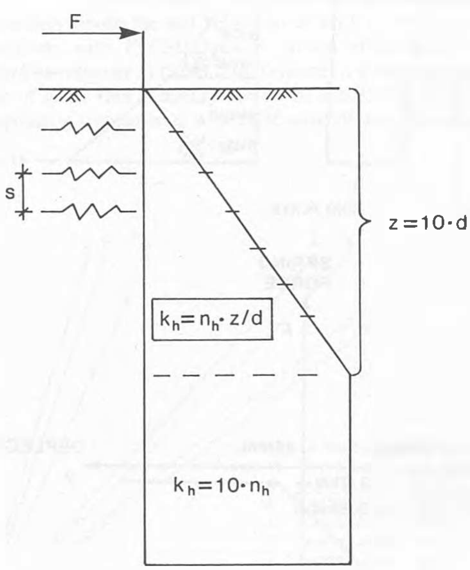


Figure 6. Distribution of subgrade coefficient k_h with depth in friction soil.

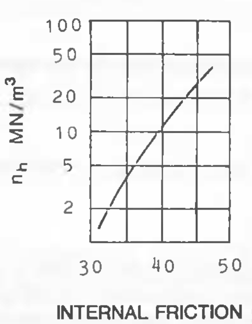
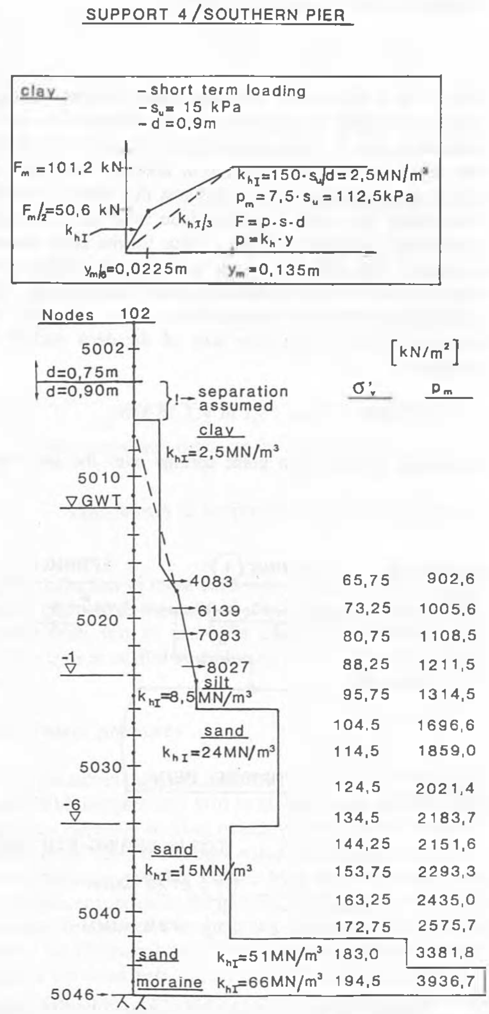


Figure 7. Determination of subgrade reaction coefficient n_h /4/. Below GWT n_h is multiplied by 0,6.



LATERAL SPRING FORCE-DEFLECTION RELATIONS				
NODE	LOWER POINT		UPPER POINT	
	F [kN]	y [m]	F [kN]	y [m]
5017	406,2	0,110	812,3	0,442
5019	452,5	0,082	905,0	0,327
5021	498,8	0,078	997,6	0,313
5023	545,2	0,075	1090,3	0,301
5025	591,5	0,077	1183,0	0,309
5027	763,5	0,035	1526,9	0,141
5029	836,5	0,039	1673,1	0,155
5031	909,6	0,042	1819,3	0,168
5033	982,6	0,045	1965,3	0,182
5035	968,2	0,071	1936,4	0,287
5037	1031,9	0,076	2063,9	0,306
5039	1095,7	0,081	2191,5	0,324
5041	1159,5	0,085	2319,0	0,343
5043	1521,8	0,033	3043,6	0,132
5045	1771,5	0,030	3543,0	0,119

Figure 8. Example calculation of lateral soil springs.

The friction spring force is estimated equal to;

$F_{\mu} = \mu \cdot N,$

where $N = \sigma_v \cdot A_i$ is the weight of ballast layer element. According to a particular load test carried out specially for this analysis the coefficient of friction is taken as $\mu = 1$. The corresponding deflection in order the full friction to be mobilized was discovered to be about $\Delta = 0{,}5$ mm. Due to the arrangement of this particular ballast load test this value is however most evidently somewhat too small in comparison with the real track ballast system thus leading to slightly too stiff a value for the deck surface friction spring 1 in figure 9. The effective breadness of the track ballast on the deck surface was taken as 3,1 m. The vertical pressure on the deck surface due to the track-ballast-system was estimated as $\sigma_v = 10{,}5$ kPa. The fully mobilized friction force on the hole area of the deck surface was thus estimated equal to;

$F_{\mu TOT} = 1{,}0 \cdot 10{,}5\ kN/m^2 \cdot 42\ m \cdot 3{,}1\ m = 1{,}36\ MN.$

This was naturally devided into point springs over the area of the deck surface.

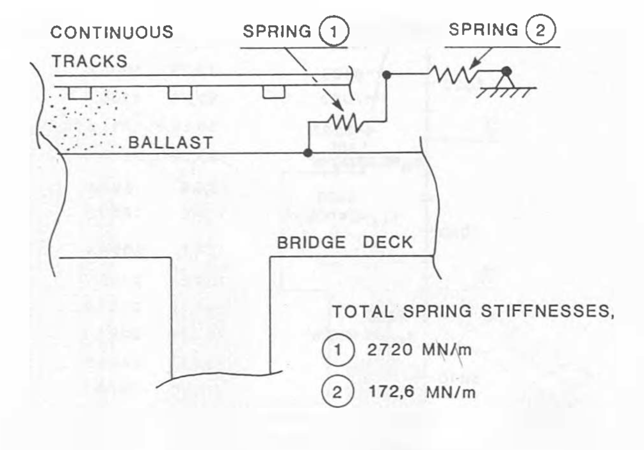


Figure 9. Considering the longitudinal resistance of track-sleeper-ballast-system.

The effect of the back fill of the end plates was considered by applying a number of - to avoid local overstrain effects due to the use of the corresponding total spring - nonlinear springs acting longitudinally to the end of the bridge deck. The values of these springs take into consideration the active, initial and passive earth pressure stage of the backfill and were determined according to the Finnish Code of Practice for Highway Bridges /3/. The stiffness of the back fill total spring is presented in Figure 10b and is determined as follows.

The backfill is constructed with two slightly different cohesionless materials;

- upper layer
  $\gamma_{dmax} = 20{,}01\ kN/m^3$
  $\gamma_{dmin} = 16{,}48\ kN/m^3$
  $D_{av} = 87{,}4\ \%$
  $w_{av} = 6{,}5\ \%$
  $=> \gamma = \underline{18{,}6\ kN/m^3}$

- lower layer
  $\gamma_{dmax} = 20{,}70\ kN/m^3$
  $\gamma_{dmin} = 15{,}89\ kN/m^3$
  $D_{av} = 88{,}3\ \%$
  $w_{av} = 8{,}5\ \%$
  $=> \underline{19{,}8\ kN/m^3}$

The angles of internal friction of the back fill layers determined from triaxial tests are $\phi_p = 38°$ and $\phi_p = 39°$, respectively.

The earth pressure at rest;
$p_o = 0{,}5 \cdot [(9{,}8 + 27{,}9)/2 \cdot (1 - \sin 38°) + (27{,}9 + 47{,}2)/2 \cdot (1 - \sin 39°)] = 10{,}6$ kPa
$-> F_o = 10{,}6\ kN/m^2 \cdot 5{,}6\ m \cdot 1{,}9\ m = \underline{112{,}8\ kN}$

The passive earth pressure;
$p_P = 0{,}5 \cdot [(9{,}8 + 27{,}9)/2 \cdot (1 + \sin 38°)/(1 - \sin 38°) + (27{,}9 + 47{,}2)/2 \cdot (1 + \sin 39°)/(1 - \sin 39°)] = 122{,}1$ kPa
$F_P = 122{,}1\ kN/m^2 \cdot 5{,}6\ m \cdot 1{,}9\ m = \underline{1299{,}1\ kN}$
The corresponding deflection required;
$y_P = 0{,}002 \cdot 1{,}9\ m = 3{,}8\ mm$

The active earth pressure;
$p_A = 0{,}5 \cdot [(9{,}8 + 27{,}9)/2 \cdot (1 - \sin 38°)/(1 + \sin 38°) + (27{,}9 + 47{,}2)/2 \cdot (1 - \sin 39°)/(1 + \sin 39°)] = 6{,}5$ kPa
$-> F_A = 6{,}5\ kN/m^2 \cdot 5{,}6\ m \cdot 1{,}9\ m = \underline{69{,}2\ kN}$
The corresponding deflection required;
$y_A = 0{,}0005 \cdot 1{,}9\ m = 1\ mm$

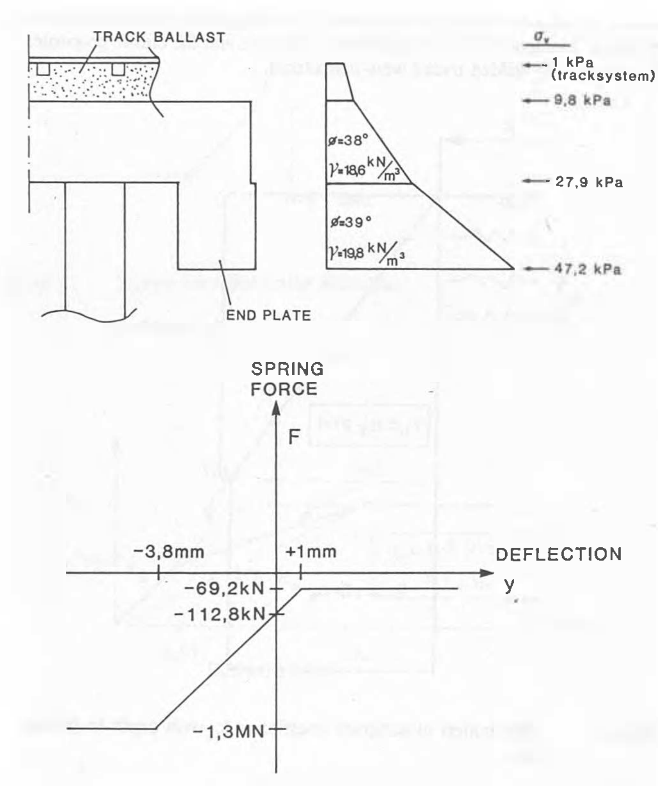


Figure 10. Determination of back fill total spring.

5. RESULTS OF TESTS AND COMPARISON WITH FEM-ANALYSIS

General

In this chapter results of loading tests are analysed in comparison with the corresponding FEM-analysis. In this context no attempt has been made to determine the horizontal capacity experimentally, but both the tests and the analysis show that the load level (F = 500 kN) in comparison with the lateral capacity is relatively low.

Comparison of deflections and longitudinal stiffness

First of all, it is very easy to find out from the test results that the deflections overall due to the applied load appear to be very small. The average value of longitudinal deflection at the scaffolding was 0,34 mm. Using this value one gets the longitudinal stiffness of this particular railway bridge egual to $K = F/y_{sc} = 0{,}5$ MN/3,4 $\cdot$ 10^{-4} m = 1470 MN/m. This value includes the effects of subsoil, back fill, stiffnesses of piers and the track-sleeper-ballast-system. The comparison of deflections shows very clearly that the response is highly influenced by the stiffness of the track-sleeper-system. The stiffnesses given in figures 11 - 14 are obtained from the additional FEM-analysis. In this special FEM-calculation resistances of track-sleeper-system equal to 7 kN/m and 12 kN/m and corresponding mobilization deflections equal to 0,5 mm and 2,8 mm /2/ were used thus giving stiffnesses as follows;

7 kN/m	Δ = 2,8 mm	170,5 MN/m
7 kN/m	Δ = 0,5 mm	403,7 MN/m
12 kN/m	Δ = 2,8 mm	223,3 MN/m
12 kN/m	Δ = 0,5 mm	528,8 MN/m

In the FECM3D-model only the track-sleeper stiffness equal to 170,5 MN/m was used. The stiffness obtained from load tests was calculated using the forces acting on tracks devided by the corresponding deflection. Assuming the spring 1 presented in figure 9 to be real and comparing the test results with analysis of equal stiffness of track-sleeper-system (FEW3D) one can see that the scatter is relatively high. This indicates that the subsoil spring stiffnesses underestimate the soil behaviour in such a low level of strains. The comparison with FECM3D-model shows much more reasonable consistency. The response of the FECM3D-model is even more conservative than results of tests. This is mainly due to the elastic nature of the model. The corresponding comparisons with FEM-analysis are presented in figure 11.

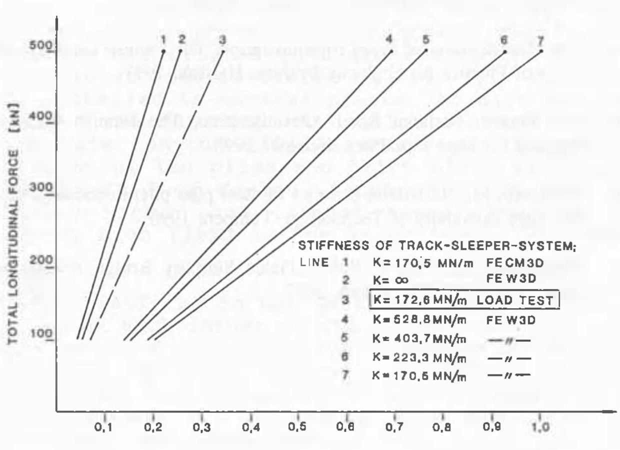


Figure 11. Comparison of longitudinal deflections at scaffolding.

The average observed longitudinal deflection of the end plate was $y_{EP} = 0{,}11$ mm. This indicates a slight elastic shortening of the deck equal to $\varepsilon_d = (3{,}40 - 1{,}1) \cdot 10^{-4}$ m/40 m = $5{,}7 \cdot 10^{-6}$. Figure 12 shows the comparison of longitudinal deflections at the end plate. The results of this are quite similar to deflections at scaffolding in figure 11.

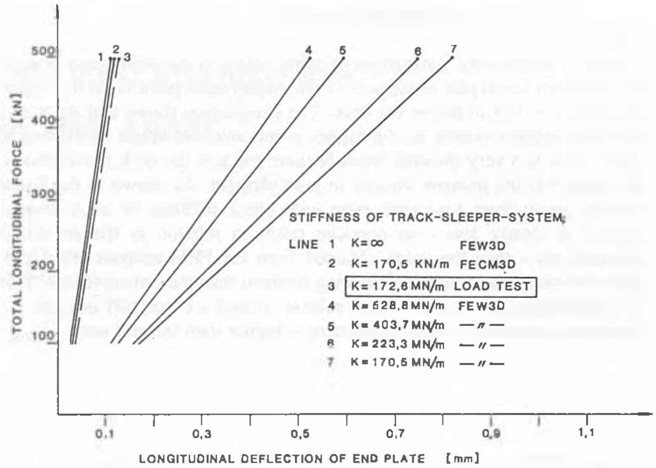


Figure 12. Comparison of longitudinal deflections of end plate.

The permanent deflection in these tests was observed to be negligible (~ 4 %). No logical cumulative increase of maximum or permanent deflection was discovered from test to test. The average transversal and vertical deflections from tests at scaffolding were $y_T = 0{,}07$ mm and $y_V = 0{,}05$ mm, respectively.

Comparison of earth pressures

In figure 13 are presented the earth pressure changes due to the applied load obtained from FEM-analyses and load tests. The earth pressure changes on the end plate were observed in three points. As the figure 13 shows, one of the transducers gives quite a different value in comparison with the other two. This disturbance is probably caused by a piece of stone or a cobble close to the transducer surface in the backfill. Generally it can be seen that - when comparing the lines of corresponding longitudinal stiffness of track-sleeper-system - the FEM-analyses (FEW3D) gives clearly greater values of pressure changes than load tests. This is naturally due to the higher values of movements in the analyses. The scatter between the analysis and tests is thus similar to the comparison of deflections. Again it can easily be discovered the sensitivity of the stiffness of track-sleeper-system to the response.

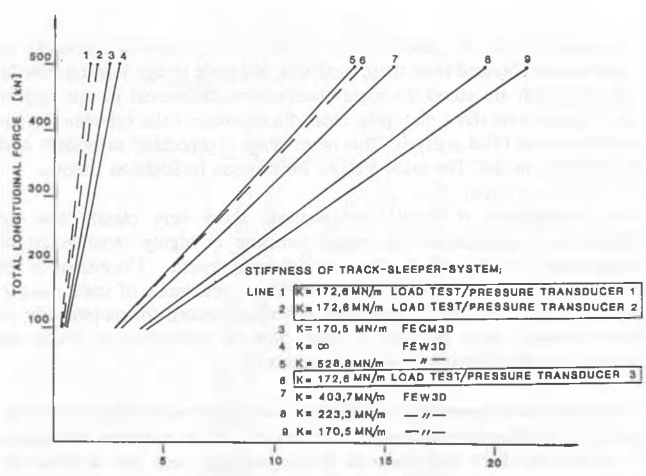


Figure 13. Comparison of earth pressures.

Comparison of reinforcing rebar strains

Figure 14 presents the comparison of strain values in the reinforcing rebar of the southern bored pile of support 1. The observation point is on the tensile side and z = 0,25 m below the deck. The comparison shows that the stiffer the track-sleeper-system is, the higher is the value of strain in the tensile rebar. This is a very obvious result because the less the deck movement is, the greater is the moment created in joint element. As shown in the figure 14, the strain from load tests even with equal stiffness of track-sleeper-system is clearly less - an opposite result in relation to the deflection comparisons - than the value obtained from the FEM-analysis (FEW3D). This indicates a smaller actual bending moment than the estimated one. This is understandeable, because if the subsoil springs are not stiff enough, the calculated moment in the joint element is higher than the real one.

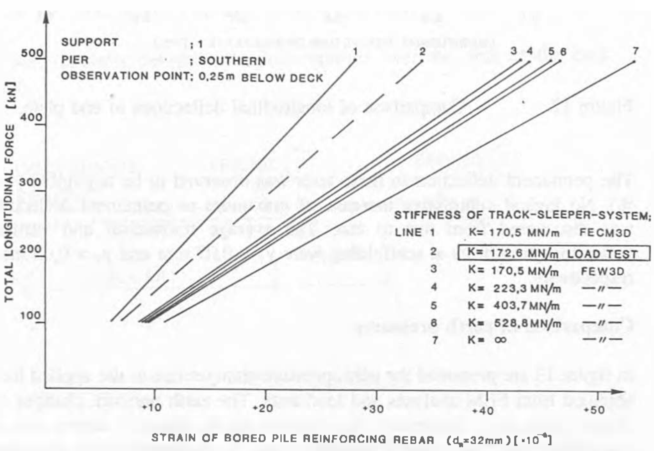


Figure 14. Comparison of strains in rebar of bored pile.

6. CONCLUSIONS

This paper tends to present only results and preliminary analysis of observations obtained from these particular full scale bridge loading tests. In this paragraph are stated the main observations discovered in this project. The comparisons show that quite clear discrepancies exist between the test results and the FEM-analysis. This observation is especially associated with the FEW3D-model. The main reasons for this can be listed as follows.

The comparisons of results and analysis show very clearly that the longitudinal response of the bridge structure is highly sensible to the longitudinal stiffness of the track-sleeper-system. Uncertainties in determining this stiffness, i.e. the longitudinal resistance of track-sleeper-ballast-system and the corresponding deflection needed for mobilization of this resistance, thus produces a great deal of difficulties to check the accuracy of geotechnical modelling the subsoil.

The scatter in the preceding comparisons is also somewhat influenced by the accuracy of measuring equipment. This matter is most evidenty emphasized by of the low strain level occurred in this particular case, due to which the signals must be remarkably amplified. Therefore all measurings should have double certainty in order to ensure the reliability of measurings. This is not however always possible because of the large amount of information to be recorded and the limited number of simultaneously active channels.

The type of principals used in the analysis (FEW3D-model) are basicly created for greater strain levels than discovered in these tests. In addition to this the initial stress state was not taken into account in either of the FEM-models. For these reasons the longitudinal deflections and thus also the response of end plate earth pressure appear to be clearly greater than observed values. The constitutive idealization of soil behaviour used in the FEW3D-model thus assumes the soil not stiff enough in the strain level taken place in these tests. The results of the FECM3D-model show in turn much better agreement with tests. The analysis however serves as a starting point for deeper theoretical basis.

7. ACKNOWLEDGEMENTS

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