SEISMIC ANALYSIS OF BRIDGES INCLUDING SOIL-ABUTMENT INTERACTION

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SUMMARY

This study investigates the effects of the soil-abutment interaction on seismic analysis and design of integral bridges. Past experience and recent research indicates that soil-structure interaction plays a very important role on seismic response of bridge structures. Abutments attract a large portion of seismic forces, particularly in the longitudinal direction. Therefore, participation of backfill soil at the abutments must be considered. A design driven methodology to model the abutment stiffness for either linear or non-linear analysis, considering the backfill and the pier foundation, is presented. An iterative design procedure of successive linear dynamic response analyses that takes into account the non linear behaviour of the abutments caused by backfill soil yielding is developed. Also, a non-linear static analysis of the bridge-soil system is conducted. A three-span bridge with monolithic abutments is selected to demonstrate the proposed procedures. Parametric studies demonstrate that, if the bridge is analysed with the proposed methodology instead of a simple procedure that ignores backfill stiffness reduction, the calculated forces and moments at the piers are greater by 25%-60% and the displacements by 25%-75%, depending on soil properties.

INTRODUCTION

Foundation behaviour plays a major role on the performance of highway bridges during earthquakes. For many highway bridges, abutments attract a large portion of the seismic force, particularly in the longitudinal direction. After the 1971 San Fernando earthquake, it became quite evident that many abutments had been subjected to large seismic forces. On many bridges, abutment damage was the only damage reported indicating that abutments attracted a large portion of the seismic force.

Soil-abutment interaction under seismic loads is a highly non-linear phenomenon. This non-linearity plays important role in the overall structural response [Spyrakos 1990, Spyrakos 1992, Maragakis 1989]. As a result there is a definite need to develop a proper methodology to design bridges including the effects of soil-abutment interaction.

Some guidance is currently provided by Caltrans Bridge Design Aids and the AASHTO [Caltrans 1989, FHWA 1986]. Both documents recognise the highly non-linear behaviour that could be caused by large deformations in

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the backfill at the abutments during seismic excitations. This paper presents seismic design oriented procedures of modelling and analysing highway bridges including soil-structure interaction. Emphasis is placed on modelling of abutment system, and the development of two analysis procedures that account for the non-linear behaviour of abutments. The first is an iterative design procedure utilising successive linear analyses. The second is a non-linear static analysis using non-linear springs to account for backfill soil stiffness.

MODELLING BACKFILL SOIL STIFFNESS

For various abutment configurations and soil conditions, a general form of abutment wall-backfill stiffness equation that considers passive resistance of soil, as recommended by Wilson [Wilson 1988] can be used to estimate the longitudinal stiffness of the end-wall and the transverse stiffness of the wing-wall, that is:

$$K_S = \frac{E_S}{(1-\nu^2)\cdot I} \tag{1}$$

where K_S is soil stiffness per unit deflection per unit wall width; E_S is the Young's modulus of the backfill soil; ν is the Poisson's ration of the backfill soil; and I is a shape factor. Representative values of I are given in Table 1 [Lam].

Table 1. Shape factor for abutment stiffness

[L/B]	*Shape Factor [I]*
1	0.80
5	1.70
10	2.00
20	2.40
L: Dimension, Long side of contact area B: Dimension, Short side of contact area	

Equation 1 is used for the evaluation of vertical displacement of a uniformly loaded area resting on an elastic half-space, which is available in standard geotechnical references [Poulos 1974]. Thus, for a rectangular area with dimensions *a x b* (*b* is the shorter dimension) the vertical displacement is given by:

$$\delta_z = \frac{(1-\nu^2)}{E_S} pbI \tag{2}$$

where, p is the uniform load per unit area of the rectangle.

Evaluating soil stiffness as described above is just one possible approach to account for translational stiffness of end- and wing-walls. Other models [Veletsos 1971], which have received widespread use in estimating foundation stiffness and are equally as convenient to use, could have also been adopted in this problem.

Equation 1 allows for input of site specific soil parameters and abutment wall configurations. As the length to height ratios for wing-walls are somewhat smaller than end-walls, equation 1 suggests a lower shape factor I, or a higher soil stiffness (K_S) for wing walls as compared to end-walls.

MODELLING PILE STIFFNESS

Pile footings are the most commonly used foundation systems to support bridges. A pile foundation can be incorporated in bridge analysis by several models, including: (1) equivalent cantilever, (2) uncoupled base spring and (3) coupled foundation stiffness matrix. The third model is the most elaborate in representing foundation stiffness in a dynamic response analysis of the overall bridge. The main draw back relates to the added effort to develop the coefficients in the stiffness matrix. In this study a simplified procedure that has been developed by Lam, Martin and Imbsen [Lam] is used to evaluate the translational stiffness of the pile-group at the abutments.

Translational pile stiffness can be obtained for a combination of bending stiffness of the pile (*EI)* and the coefficient of variation of soil reaction modulus E_S with depth (*f*). Proper diagrams are given in [Lam]. There are several simplifying assumptions in the presented approach. The embedment effect has not been taken into account in the procedure. Therefore the recommendations are conservative and appropriate for shallow embedment conditions. The pile group interaction is neglected for simplicity, a simplification that at special circumstances should not be made.

MODELLING ABUTMENT STIFFNESS FOR LINEAR ANALYSIS

The abutment that is used for the analysis is a monolithic type with pile foundation as shown in Figure 1. For simplicity only the translational (longitudinal and transverse) stiffness of abutment is incorporated in the bridge model for the analysis. Other methods of modelling the abutment stiffness can be found in the literature [Wilson 1988, Maragakis 1989] Proper values of spring constants in the longitudinal and transverse directions are calculated from the backfill soil and pile foundation stiffness according to the following assumptions:

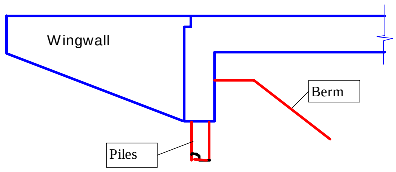


Figure 1: Monolithic abutment

In the longitudinal direction, when the structure is moving toward the soil, the full passive resistance of the soil is mobilised, but when the structure moves away from the soil no soil resistance is mobilised. The total structure stiffness would be unrealistically high if the full passive resistance were used at both abutments. In a dynamic analysis, as an approximation one-half of the total backfill soil stiffness is located to each abutment (Figure 2). In quasistatic analysis the full backfill soil resistance is located to the abutment toward which the superstructure moves (Figure 3). The backfill soil stiffness K_{soil} and the pile stiffness K_{pile} are additive until the soil capacity is exceeded at which point the pile stiffness K_{pile} alone controls the force-deformation behaviour [Priestley 1996]. In any case, it is important that the total stiffness of the system in the longitudinal direction is determined with the most possible accuracy to obtain a realistic evaluation of the system’s response. The reduction of stiffness at the abutments, in a dynamic analysis, requires adjustment of the computed resultant forces. When half springs are used, the resulting forces from the analysis should be doubled at each abutment.

In the transverse direction, the flexible wing-walls are not usually fully effective and some judgement is necessary to estimate stiffness realistically. The effective width is taken as the length of the wing-walls multiplied by a factor of 2/3. Also, the soil between the wing walls is more effective (≈100%) than the exterior soil (≈33%). The assumptions are based on several experimental tests and field inspections on abutment response and lead to conservative results for the design of bridge [Caltrans 1989].

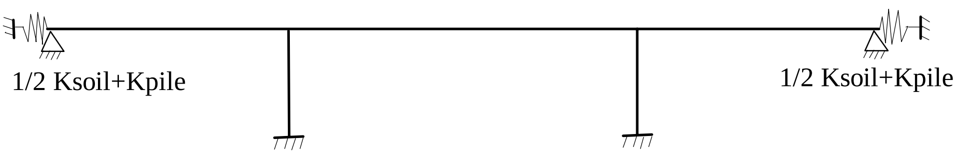


Figure 2: Abutment stiffness for dynamic analysis

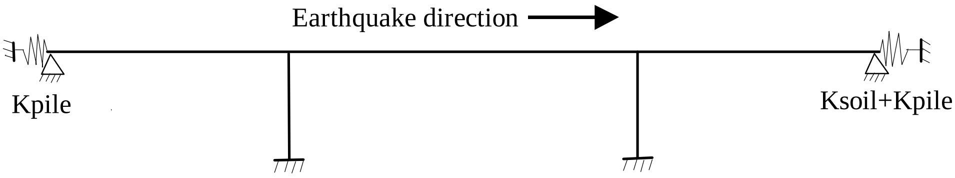


Figure 3: Abutment stiffness for quasistatic analysis

ITERATIVE ANALYSIS PROCEDURE

An iterative analysis and design procedure that consists of successive linear dynamic analyses is described. The iterative procedure accounts for the non-linear behaviour of abutment systems due to backfill soil yielding. The presented procedure has been calibrated to Greek seismic codes and Eurocode 8-Part 2 as well as to current bridge design practice. A schematic presentation of the three-step procedure is given in Figure 4.

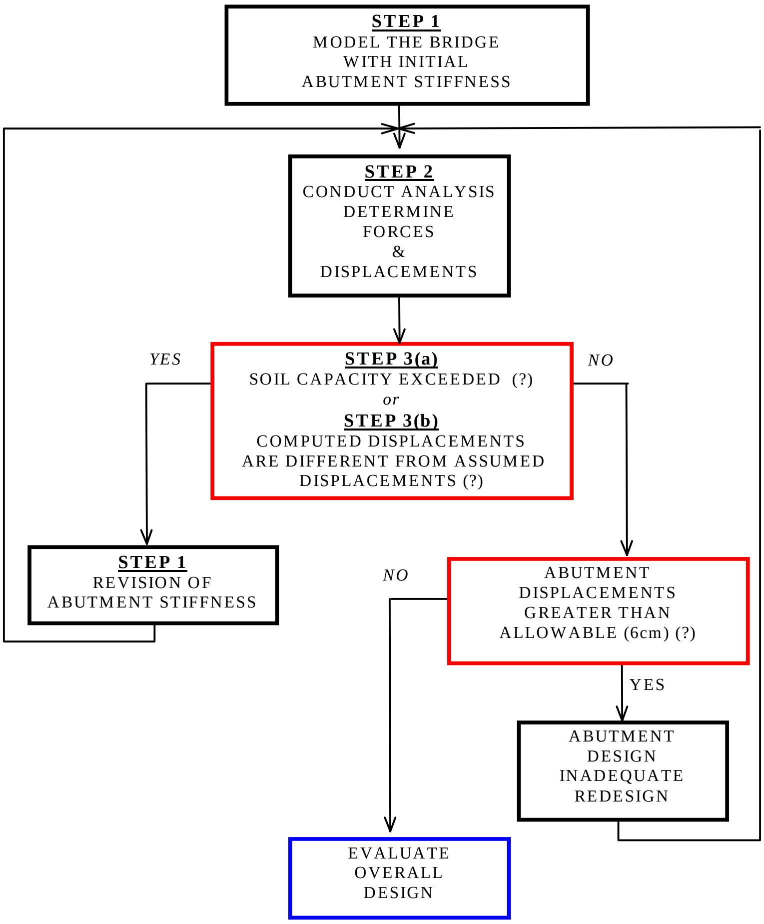


Figure 4: Schematic presentation of Iterative Analysis Procedure

STEP 1: Evaluate the abutment stiffness and the abutment load-displacement characteristics. Assume initial abutment stiffness in longitudinal and transverse direction. The stiffness should be compatible with the backfill soil stiffness and the foundation type at the abutment. The contribution of the approach slab to abutment stiffness is neglected for simplicity. Soil stiffness and pile foundation stiffness are determined. Load-displacement diagrams for both directions are constructed as shown in Figures 5 and 6.

STEP 2: Perform the analysis using the abutment stiffness, conduct linear analyses of the overall bridge to determine forces and displacements. This step is usually repeated as many times as required to arrive at an acceptable solution according to the schematic of Figure 4. Usually three iterations suffice.

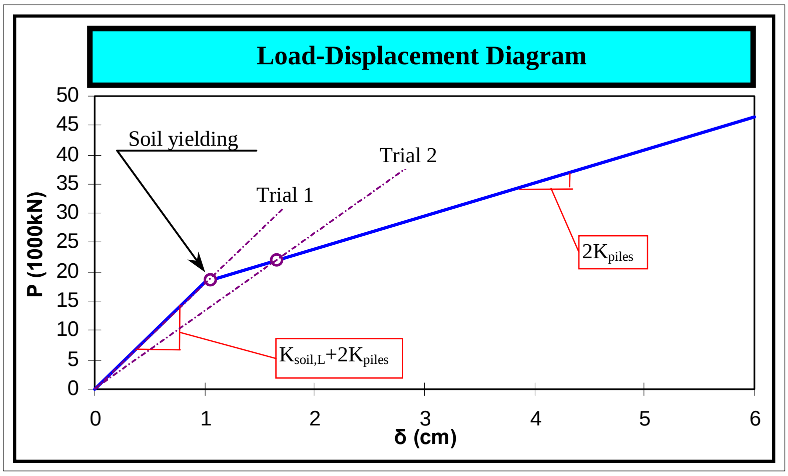


Figure 5: Load-displacement diagram for both abutments in the longitudinal direction

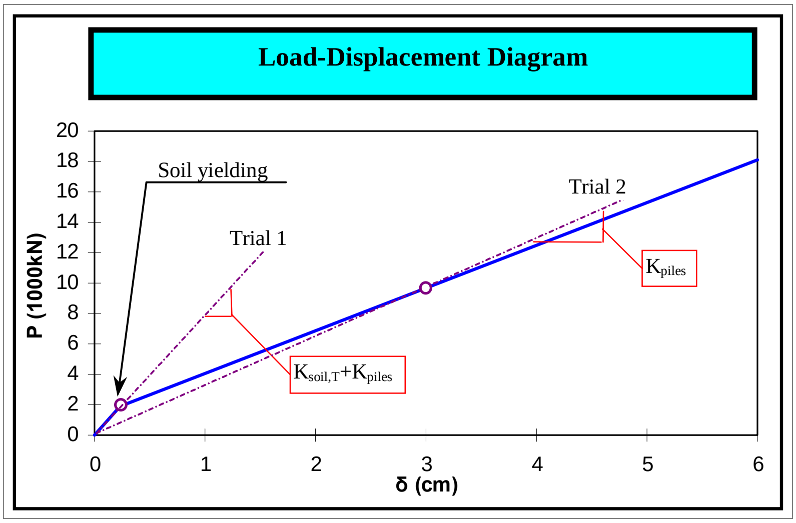


Figure 6: Load-displacement diagram for each abutment in the transverse direction

STEP 3(a): After the first iteration, check that the soil capacity has not been exceeded. If the peak soil pressure exceeds soil capacity the analysis should be repeated with reduced abutment stiffness, using an equivalent linear stiffness (see Trial 2 in Figures 5&6) to reflect plastic yielding of the backfill soil. The equivalent linear stiffness for each direction is evaluated on the basis of load-displacement characteristics and assumed displacements.

STEP 3(b): Continue with subsequent iterations and compare for each iteration the displacements against the value assumed for the equivalent linear abutment stiffness. This check is needed to ensure that the assumed abutment stiffness reflects the load-displacement characteristics properly. If the difference in the assumed stiffness between two successive iterations is excessive, the analysis should be repeated with revised stiffness until convergence is achieved.

CHECK: Examine for excessive deformations. After the 1971 San Fernando earthquake, field inspections revealed that abutments which moved up to 6cm in the longitudinal direction into the backfill survived with little need for repair. Caltrans and Eurocode 8 suggest that this limit should be maintained. Deformation greater than 6cm in the abutment foundation should be avoided for stability and structural integrity.

MODELLING ABUTMENT STIFFNESS FOR NON-LINEAR ANALYSIS

Instead of conducting the iterative procedure to account for the backfill soil yielding at abutments, a non-linear static analysis or a non-linear time-domain dynamic analysis can be implemented. In this work the static non-linear analysis is presented.

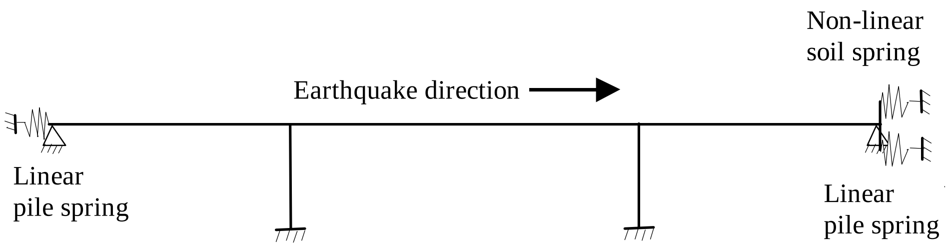


Figure 7: Abutment stiffness for non-linear static analysis

Two springs are used for modelling the stiffness of the abutment toward which the structure moves (Figure 7). The first is a non-linear spring, representing the backfill soil stiffness with constant K_{soil} and yield limit at the point where the pick soil pressure is reached (Figure 8). The second is a linear spring representing the pile foundation stiffness with constant K_{pile} (Figure 9). At the opposite abutment only the second spring is set.

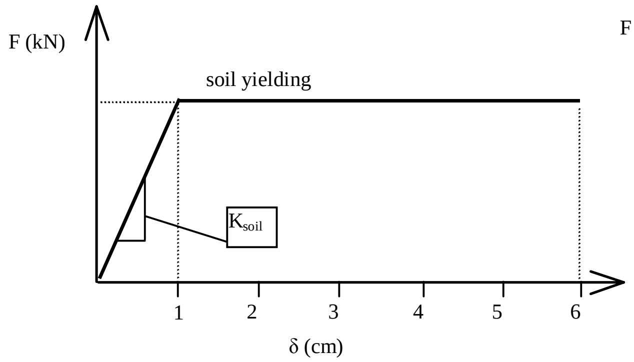


Figure 8: Non-linear soil spring

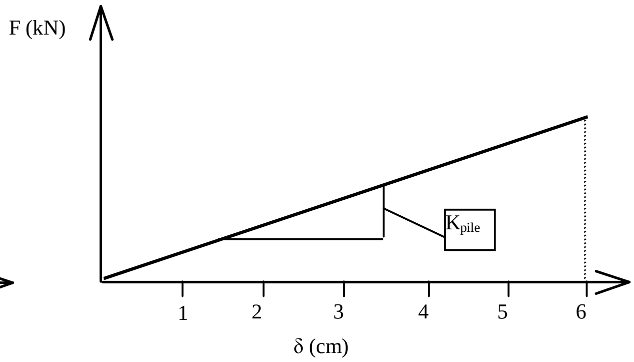
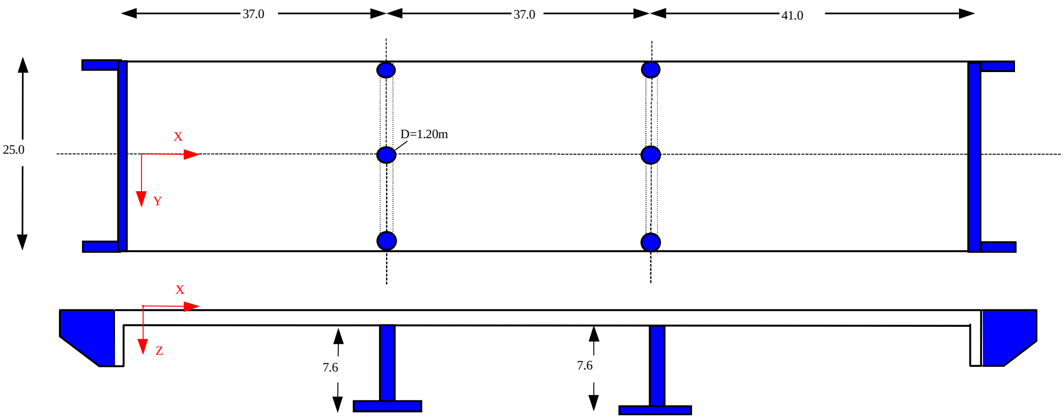


Figure 9: Linear pile foundation spring

BRIDGE EXAMPLE

The two procedures are demonstrated with a representative example. Consider a 115m long three-span bridge with a prestressed concrete box girder deck in monolithic connection with bents and abutments. There are three circular columns at each bent. The width of the bridge is 25m; geometric characteristics and moments of inertia are shown in Figure 10. Spectra of the Greek seismic code used for the analysis, for a bridge built in seismic zone III characterised by a peak ground acceleration a_0=0.24g. A behaviour factor q=1.00 is adopted to facilitate the parametric studies of the effects of SSI. Detailed calculations of abutment stiffness can be found in [Karantzikis 1997].

Parametric studies are conducted for three different soils (loose-medium-dense). Results from the analysis are presented in Tables 2 through 4. In Tables 2 through 4 G_0 indicates the shear modulus of the soil for small strains. Results with the proposed procedures, which consider the abutments nonlinearity caused by backfill soil yielding, are compared with the results from analysis that ignores it. The comparison clearly demonstrates that SSI plays a major role in bridge seismic response.



Superstructure: Prestressed concrete box girder
$A=11.427m^2$
$I_{yy}=4.54m^4$
$I_{zz}=565.76m^4$
$I_t=1.00m^4$

Bents: Three circular columns, D=1.40m
$A=1.54m^2$
$I_{yy}=I_{xx}=0.188m^4$
$I_t=0.38m^4$
Foundation:Footing

Abutments:
Height: 2.50m
Foundation: 23 piles
Pile diameter D=0.50m

Figure 10

Some of the most important observations drawn from Tables 2 through 4 include: In the longitudinal direction, soil capacity has been exceeded only for the loose backfill soil. However, in the transverse direction soil capacity has been exceeded for all types of soil. The proposed procedures lead to greater forces and moments at the bents as well as to larger displacements attributed to the fact that soil capacity has been exceeded and backfill stiffness at the abutments has been reduced.

Table 2: Monolithic abutments, Loose soil

LOOSE SOIL , Go=70000kPa			
-Longitudinal Earthquake-	Bent #2# M_y (kNm)	Bent #3# M_y (kNm)	Abutment #1# displacement δ_x(cm)
Analysis without SSI	-2138	-2003	1.323
Proposed analyses	-2825	-2528	1.640
	+32%	**+26%**	**+24%**
-Transverse Earthquake-	Bent #2# M_x (kNm)	Bent #3# M_x (kNm)	Abutment #1# displacement δ_y(cm)
Analysis without SSI	3148	3223	1.753
Proposed analyses	4976	5062	3.030
	+58%	**+57%**	**+73%**

Table 3: Monolithic abutments, Medium soil

MEDIUM SOIL , Go=140000kPa			
-Transverse Earthquake-	Bent #2# M_x (kNm)	Bent #3# M_x (kNm)	Abutment #1# displacement δ_y(cm)
Analysis without SSI	2101	2181	0.928
Proposed analyses	2876	2957	1.430
	+37%	**+36%**	**+54%**

Table 4: Monolithic abutments, Dense soil

DENSE SOIL , Go=280000kPa			
-Transverse Earthquake-	Bent #2# M_x (kNm)	Bent #3# M_x (kNm)	Abutment #1# displacement δ_y(cm)
Analysis without SSI	1651	1735	0.588
Proposed analyses	2263	2346	0.920
	+37%	**+35%**	**+56%**

CONCLUSIONS

Two procedures to consider non-linear soil-abutment interaction under seismic loads have been developed. The first through iterative linear dynamic response analyses, and the second through non-linear static analysis. The procedures are relatively simple and easy to apply for bridge design. However one of the greatest uncertainties in applying these procedures is the determination of an appropriate value of the soil shear modulus, G_0. Determination of soil shear modulus with in-situ measurements at several bridge sites would be a valuable contribution in this area. Incorporation of abutment stiffness in design and retrofit analysis of highway bridges leads to a more reliable estimation of the overall seismic load level and distribution of seismic loads among bents and abutments. More importantly, it leads to better estimation of displacements. Parametric studies demonstrate that, if the bridge is analysed with the proposed methodology instead of a simple procedure that ignores backfill stiffness reduction, the calculated forces and moments at the piers are greater by 25%-60% and the displacements by 25%-75%, depending on soil properties.

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