STIFFNESSES OF ABUTMENTS ON PILES IN SEISMIC BRIDGE ANALYSES

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ABSTRACT

The characterization of bridge-abutment-backfill-pile interaction is an important component in seismic bridge response evaluations. This paper presents a relatively simple and realistic methodology to evaluate the nonlinear translational (longitudinal, and transverse) spring stiffnesses of abutments founded on pile foundation. Many important factors such as the abutment dimensions, nonlinear pile-soil interaction, superstructure loads, and difference in soil behavior under active and passive conditions have been accounted for. Only routinely used soil properties are required in the model. The paper describes a calibration study undertaken to ascertain the predictive capability of the proposed approach using large-scale field tests. A very good agreement exists between the predictions and the large-scale field tests results. Both longitudinal and transverse field tests were utilized in the calibration. This paper also presents the results of a parametric study in which the influence of pile fixity conditions, relative contribution of piles to abutment stiffness, and deck loads were investigated.

Key words: abutments on pile foundation, longitudinal abutment stiffness, nonlinear abutment stiffness, nonlinear soil behavior, seismic bridge design, seismic bridge response, transverse abutment stiffness (IGC: E2/E8/E12).

INTRODUCTION

Many past earthquakes around the world clearly demonstrated the existence of a significant interaction between the bridge superstructure, the abutments, the pile foundations, and the approach fill (bridge-abutment-backfill-pile interaction). Many experimental and analytical studies have demonstrated that this interaction plays a major role in the overall seismic behavior of the entire bridge system. Among the experimental studies, work by Douglas and Saiidi (1984), Crouse and coworkers (1985, 1987), Maragakis et al. (1992), and Eberhard et al. (1993) are a few examples of well-documented field tests which addressed the bridge-abutment-backfill-pile interaction. Many studies have concluded that, as a result of nonlinear hysteretic behavior of soils, the earthquake response of especially short-span highway bridges is highly nonlinear (Werner et al., 1990, 1994; Wilson and Tan, 1990). The bridge-abutment-backfill-pile interaction significantly influences the natural period and mode shapes of the bridge system, which in turn governs the magnitude and distribution of the inertial loads from earthquake excitation (Lam et al., 1991).

The importance of a dynamic analysis that allows for flexibility (springs) at the abutments (as opposed to

representing supports at abutments as rigid) has been incorporated into the seismic bridge design guidelines adopted by the American Association of State and Highway Transportation Officials (AASHTO, 1992). A quite involved and complicated dynamic analysis which integrates extensive seismological and geotechnical input along with the finite element (FE) modeling of the entire bridge-abutment-backfill system has been attempted (e.g., Ueng et al., 1994; McCallen and Romstad, 1994), but the adoption of such approaches is unattractive to practicing bridge design engineers.

A simpler, economical, and more appealing way to represent the abutments in dynamic response studies is to use translational and rotational springs. When seat-type abutments are considered, only the characterization of translational stiffnesses (longitudinal, vertical, and transverse) is required. A recent study by Siddharthan et al. (1995, 1996) focused on stiffnesses for abutments on spread footings.

The current widely used design procedure (e.g., Caltrans) consists of performing an iterative elastic analysis until convergence between the assumed and the computed secant abutment spring stiffnesses falls within a certain allowable limit. The abutment stiffness is assumed to be the sum of the contributions from the abutment (end

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wall) and piles. The *Caltrans Bridge Design Aids* (1989) recommends a linear behavior for the abutment contribution until an ultimate load is reached. The recommended value for the initial stiffness is 1.15×10^5 kN/m/m width of the wall for typical materials placed at bridge abutments in California. The ultimate abutment load is normally assumed to be limited by a maximum passive soil stress of 370 kPa (7.7 kips/ft²). The suggested lateral stiffness value for the pile foundation is 7005 kN/m per pile (40 kips/in for standard 45 ton, 70 ton, 0.4 m cast-in-drill-hole: CIDH piles). The major limitations of the Caltrans procedure are that (1) it is largely empirical (2) it does not explicitly account for many important factors such as abutment and pile dimensions and soil properties, and (3) it has the inherent limitation that it is generally applicable only to cases similar to those on which it is based.

The major objective of the research work presented here is to develop a simple procedure for estimating translational abutment stiffnesses for abutments on pile foundations. Among the three translational (longitudinal, transverse, and vertical) spring stiffnesses, only the first two are considered in this paper. Only these two springs (horizontal) by and large play the governing role in many seismic designs. The foundation soil can be either cohesive or cohesionless, but the abutment fill soil is considered cohesionless. This paper describes a simple analytical model that incorporates many of the important factors that govern abutment stiffness characteristics. Factors such as the abutment dimensions, the nonlinear pile-soil interaction, and the difference in soil behavior under active and passive conditions are accounted for. Special efforts were made to ensure that the analytical model is simple but yet realistic and requires a very few material parameters that may be obtained from routine geotechnical tests. Results of a calibration study undertaken to verify the applicability of the proposed analytical model are also presented. Recent large-scale field test results reported by Maroney et al. (1994) have been used in the calibration. Finally, the paper presents the results of a parametric study in which the influence of factors such as the pile head fixity, piles versus abutment (end wall) contribution to abutment stiffness, and superstructure deck load on abutment stiffnesses were investigated.

PROPOSED MODEL

Figure 1 shows the longitudinal spring (S_L) and transverse spring (S_T) connected to a bridge deck selected to represent the abutment in seismic bridge analysis. It should be noted that the springs will become effective only after the deck-abutment gap (if present) has been closed. Any one of the load-displacement relationships can be evaluated by estimating the force required to cause a given displacement. The development of longitudinal spring stiffness characteristics is presented first.

Longitudinal Abutment Stiffness (S_L) Evaluation

Figure 2 shows a sketch of a displaced abutment

Fig. 1. Representation of deck-abutment-backfill-pile interaction

Fig. 2. Sketch of displaced abutment caused by a longitudinal force

caused by a longitudinal force, P_L , applied at the abutment-bridge deck support. The abutment longitudinal displacement near the top is δ_L , and at the same time, it rotates through θ as shown. The movement of the abutment suggests that active and passive conditions can develop in front and at the back of the abutment as shown. At the bottom of the abutment, there is resistance for sliding and rotation. A close look at the figure suggests that, to estimate the force P_L for a given displacement δ_L , one is required to characterize (1) the lateral wall movement-earth pressure relationship under active and passive conditions and (2) the interaction between the abutment base, piles, and the foundation soil. Procedures adopted to establish these relationships are presented below.

Lateral Movement-Earth Pressure Relationship

Laboratory measurements and finite element studies (Clough and Duncan, 1991; Barker et al., 1991) have shown that (1) the active conditions in a soil deposit can

be mobilized with a much lower wall movement than the passive conditions and (2) the normalized wall movements (Δ/h), specified as a fraction of the wall height, h , can be used to represent the relationship between the wall movement and lateral pressure. This relationship is applicable for both the sliding and tilting movements of the abutment. Figure 3 presents a range for lateral earth pressure-wall movement relationships reported for compacted and uncompacted medium dense sand (Clough and Duncan, 1991). The vertical axis has also been normalized with respect to active and passive coefficients of earth pressure. It is seen from the figure that, for medium dense sand, the Δ/h ratio for the passive mobilization is as much as ten times that needed for active mobilization. The average curve shown in Fig. 3 was selected to represent the active and passive pressure-wall displacement relationships in the proposed model.

Resistance for Deformation at Abutment Base

The abutment base undergoes vertical and lateral movements and rotation. The resistance at the base is defined by a vertical reaction, F_v , a horizontal reaction, F_h , and a resistive movement, M_b (Fig. 2), all defined as per-unit width (or run). These base resistance components are derived from the pile-soil interaction.

For the case of pile foundations, a recent comprehensive study by Duncan et al. (1994) and Ooi and Duncan (1994) proposed a simple procedure to characterize pile-soil interaction under a variety of soil and loading conditions. They referred to their approach as characteristic load method, CLM. The CLM has been developed for single pile and pile group. It was developed by performing extensive nonlinear p - y analyses for a wide range of free-head and fixed-head piles and drilled shafts in clay and in sandy soil. Duncan and his co-workers represented the results in terms of dimensionless variables. Their method can be used to determine pile top deflections and rotation subjected to a lateral load and a moment. Duncan and his co-workers provided verification of their approach by comparing results obtained from two field tests in the case of single pile behavior, and four field tests in the case of pile groups. The CLM predictions

were good in all the field test comparisons, though the field cases represented many different loading and soil conditions. This extensive verification effort lends credibility to the application of the procedure to characterize pile foundation behavior.

The first step in the CLM is to define the characteristic load, P_s , and moment, M_s . These characteristic values for sandy soil have been defined by Duncan et al. (1994) as:

$$P_s = 1.57D^2[E_p R_I] \left[\frac{\gamma D \phi K_p}{E_p R_I} \right]^{0.57} \quad (1)$$

$$M_s = 1.33D^3[E_p R_I] \left[\frac{\gamma D \phi K_p}{E_p R_I} \right]^{0.40} \quad (2)$$

in which P_s =characteristic load for sandy soil (kN); M_s =characteristic moment for sandy soil (kN.m); D =pile diameter (m); E_p =pile modulus of elasticity (kN/m²); R_I =moment of inertia ratio, normalized relative to an equivalent solid circular cross section (dimensionless) I_p/I_{circular} ; γ =unit weight of soil (effective unit weight if submerged) (kN/m³); ϕ =effective stress friction angle of sand in degrees; and K_p =Rankine coefficient of passive earth pressure= $\tan^2(45 + \phi/2)$. More details on the guidelines to estimate these input parameters in typical field conditions are given by Duncan et al. (1994).

Figure 4(a) shows the recommended normalized relationship between pile top load and deflection for piles and shafts in sandy soils; the pile top load has been nor-

Fig. 3. Normalized lateral wall pressure-displacement relationship (after Clough and Duncan, 1991)

Fig. 4. Characteristic load moment curves for sandy soil (after Duncan et al., 1994): (a) Load-deflection relationships; (b) Moment-deflection relationships

malized using the characteristic load (P_s) and the pile deflection by its diameter. The relationships are given for both fixed-head and free-head pile foundation conditions. Figure 4(b) presents the normalized relationship between pile top moment and deflection for free head piles; the vertical axis gives the applied moment divided by the characteristic moment (M_T/M_s). Duncan et al. (1994) outlined an iterative procedure to estimate pile top deflection due to both pile top load and moment. The net effects of the load and moment are superimposed using,

$$Y_S = 0.5(Y_P + Y_M) \quad (3)$$

in which Y_S = estimated groundline deflection due to both load and moment; Y_P = groundline deflection due to the applied load plus an equivalent load; Y_M = groundline deflection due to the applied moment plus an equivalent moment.

Group Amplification Procedure

The group amplification factor was derived from the results of numerous analysis of pile groups using the CLM procedure and the Focht and Koch (1973) procedure (Ooi and Duncan, 1994). The basic steps in determining the pile group top deflection are to first estimate single pile deflection (Y_S) and then to use group influence factor (C_Y) to compute the pile group top deflection, Y_g as

$$Y_g = C_Y Y_S \quad (4)$$

After a thorough parametric study on a large number of pile groups with different number of piles, pile spacing and sizes, and soil stiffnesses, Ooi and Duncan (1994) recommended a simple relationship for C_Y as,

$$C_Y = \frac{A + N_{\text{pile}}}{B_0 \left(\frac{S}{D} + \frac{P_{\text{single}}}{CP_N} \right)^{0.5}} \quad (5)$$

in which $A=9$ for sand (dimensionless); N_{pile} = number of piles in group; $B_0=3$ for sand (dimensionless); S = average spacing of piles (m); D = diameter of single pile (m); $P_{\text{single}} = P_{\text{group}}/N_{\text{pile}}$ = average lateral load per pile (kN); P_{group} = total lateral load on the group of piles (kN); $C=16$ for sand (dimensionless); $P_N = K_P \gamma D^3$ for sand (kN). The group amplification procedure affords a simple means for quantifying the pile group interaction effects and, therefore, has been adopted in this paper.

Equilibrium of Displaced Abutment

Figure 5 shows the typical boundary forces and moments surrounding an abutment. The lateral stress at any point located in the front and back of the abutment is computed by multiplying the lateral pressure coefficient, k , from Fig. 3 by the vertical stress at that point. The net force on the wall is the integration of this stress over the height of the abutment. For example, the lateral force P_1 shown in Fig. 5 is

$$P_1 = \frac{1}{2} K \gamma h_1^2 \quad (6)$$

Fig. 5. Boundary forces and moments on the abutment

in which h_1 = the height of the passive zone in front of the abutment. This force is at an angle δ_b , which is the angle of friction at the vertical slip plane (Richards et al., 1993). Since the abutment undergoes rigid body movement, it is possible to evaluate the abutment displacement at any point in terms of δ_L and θ (see Fig. 2). The lateral passive and active earth pressure coefficients, K_P and K_a , required to carry out these calculations may be obtained based on Coulomb equations. As recommended in the design manuals (Barker et al., 1991), the Coulomb coefficient, K_P , modified to account for a more realistic failure surface (log spiral), has been used with Eq. (6) and Fig. 3.

It is necessary to find P_L for a given displacement, δ_L , near the top of the abutment (Fig. 2) such that force and moment equilibrium equations are simultaneously satisfied. A simplified step-by-step procedure that has been adopted to achieve this condition is as follows:

- (1) Assume a certain small starting value for δ_L , and increase the wall rotation, θ from zero.
 - (2) For the assumed value of θ , estimate the displacement of abutment in the front, back, and at the midpoint of the abutment base.
 - (3) Compute the lateral wall forces, P_1 and P_2 , the base resisting moment, M_b , and the lateral base resistance, F_h . The M_b , and F_h values that correspond to the deflection (midpoint of the base) and rotation of the abutment are computed based on the CLM described earlier.
 - (4) Use the horizontal force equilibrium condition to get an estimate of the force P_L .
 - (5) Check for moment equilibrium by taking moments about the midpoint of the abutment base. If the net (or residual) moment changes sign between two subsequent θ values, then the moment equilibrium is satisfied between these two values. If the net moment does not change sign, repeat the procedure from Step 2 with an increased value of θ .
 - (6) Repeat Steps 1 through 5 for the next δ_L value.
- It has been found that the proposed iterative procedure

converges very quickly. In the case of piles under fixed head conditions, only the horizontal force equilibrium is used (Step 4). The nonlinear longitudinal secant spring stiffness, S_L , is subsequently computed using $S_L = P_L / \delta_L$.

Transverse Abutment Stiffness (S_T) Evaluation

Figure 6(a) shows an abutment displaced transversely through a displacement δ_T when applied with a force P_T . The wing walls often provided to retain soil near the abutment, play a major role in providing resistance. Typically, only a part of the outer wing wall face is buried in the soil. However, the entire inside faces are buried. The displacement mechanism shown in Fig. 6(a) identifies the active and passive zones. Opposing types of failure zones exist across the wing walls. The resistances derived from these failure zones can be obtained from the procedures outlined in calculating the longitudinal stiffness. The portion of the wing wall buried in the soil varies with depth (see Fig. 1). Therefore, the resistance on the wing wall is computed using a number of small horizontal strips of varying widths. There is an additional component of resistance for deformation is present on the back of the abutment. Figure 6(a) shows this force as T_T .

The number of lanes supported by the abutment dictates the abutment width, W , in the transverse direction. For a two-lane highway, the width is typically 10.0 m. Since this is quite wide and the interface extends to the entire height of the abutment, the resistance (T_T) at the interface between the backfill and the abutment is an important resistance component.

The evaluation of T_T requires the characterization of lateral stress (q_h) versus displacement (δ_h) relationship at the abutment interface. The approach proposed here to obtain this relationship is based on Schmertmann's (1978) strain factor method. This method provides a logical means of accounting for the stress-dependent nonlinear stiffness of sand. The procedure is based on the assumption that the distribution of lateral (shear) strain in the backfill is compatible with linear elastic half space. General applicability of this assumption has been verified using the finite element studies and many field and laboratory observations (Schmertmann, 1970; Schmertmann et al., 1978). In using Schmertmann's method, the lateral displacement, δ_h caused by a lateral stress, q_h can be obtained by summing contributions from the all sublayers in the backfill as (Fig. 6(b))

$$\delta_h = \sum \gamma_h \Delta z = \sum \left(\frac{q_h I_h}{G} \right) \Delta z \quad (7)$$

in which γ_h = the shear strain in the sublayer, Δz = the thickness of the sublayer, I_h = the shear strain influence factor, and G = the secant shear modulus of the sublayer. The shear strain influence factor, which is similar to Schmertmann's vertical strain influence factor, can be deduced from the elastic half space solution (Schmertmann et al., 1978; Poulos and Davis, 1974). The maximum interface stress that can be mobilized at any point on the abutment-backfill interface depends on the lateral stress and thus on the depth from the surface of that

Fig. 6(a). Sketch of abutment movement for transverse stiffness evaluation

Fig. 6(b). Development of q_h versus δ_h relationship at the abutment interface

point. A simple iterative procedure adopted to obtain the nonlinear q_h versus δ_h relationship is presented below:

- (1) Divide the abutment interface into a number of smaller rectangular units and the soil adjacent to each of these units into a number of sublayers up to a lateral distance of about four times the width b of each unit (Fig. 6(b)). Also assume a certain value of q_h .
- (2) For each layer, assume the initial secant shear modulus, G , to be a fraction of $G_{\max}$. Here, $G_{\max}$ is the maximum shear modulus at a low shear strain; and it may be computed using the Seed and Idriss (1970) equation

$$G_{\max} = 218.8 (K_2)_{\max} (\sigma_m)^{1/2} \quad (8)$$

in which $(K_2)_{\max}$ = the constant that depends on the relative density of the soil and σ_m = the mean normal stress of the layer. Here, $G_{\max}$ and σ_m are given in kPa and σ_m is computed at the middle of the abutment unit using the overburden stresses.

- (3) Compute the shear strain, γ_h in each sublayer using the terms that are within the brackets in Eq. (7), and the corresponding secant shear modulus given for

this shear strain from the shear modulus reduction curve of Seed and Idriss (1970).

- (4) If the difference between the assumed (Step 2) and the computed (Step 3) sublayer shear moduli are not within a certain limit (say, 5%), repeat Steps 2 and 4. If convergence occurs in all sublayers, then compute the lateral displacement, δ_h , using Eq. (7).
- (5) Repeat Steps 1 through 4 for each of the abutment units.

The entire nonlinear q_h versus δ_h relationship can be obtained for each abutment unit by increasing the value of q_h . The limit to this relationship is imposed by the maximum interface friction stress given by the lateral stress multiplied by $\tan \delta_f$, in which δ_f is the interface friction angle between the abutment and the backfill. Estimation of lateral resistance of an abutment unit can be then obtained by multiplying the interface stress by the area of the unit.

If the wing walls lie close to each other, substantial overlap can occur between the failure zones in the soil confined within the wing walls. In such cases, it is necessary to reduce the lateral resistance to account for overlapping. Our calculations show that the influence of overlapping is evident only for taller abutments (more than 5 m) supporting two or fewer lanes (Siddharthan et al., 1996; El-Gamal, 1996). In other cases, the overlapping is minimal and can be neglected. Under an extreme case of a taller abutment with less than two lanes, the entire passive zone may not be able to be accommodated within the soil confined between the wing wall. In such cases, the entire soil mass within the wing wall along with the abutment can behave like a single rigid body.

The final task in the computation of the transverse stiffness is to undertake a similar iterative procedure as described previously in calculating the longitudinal stiffness to find P_T near the abutment top such that all equations of equilibrium are satisfied.

FIELD CALIBRATION

Large-scale field verification, though expensive, may be warranted for the validation of geotechnical engineering models, especially when nonlinear stress-dependent soil behavior is a governing factor. The calibration study reported here used abutment field test results recently reported by Maroney et al. (1994) and Maroney (1995).

Maroney et al. (1994) provided a detailed description of a well-instrumented testing program undertaken for Caltrans to characterize abutment behavior (Fig. 7). They tested to failure two large-scale abutments 1.7 m and 2.1 m high. While the taller (east) abutment was tested in the longitudinal and transverse directions, the shorter (west) abutment was tested only in the longitudinal direction. Both abutments were founded on 230 mm cast-in-drilled-hole (CIDH) piles and provided with wing walls as shown in Fig. 7. The east abutment sat on four piles, while the west abutment sat on three. The west abutment supported an embankment constructed with a low plastic clayey silt (Yolo loam) having a PL of 23.5

Fig. 7. Elevation and plan views of the large-scale field testing of abutment (after Maroney et al., 1994)

and a PI of 10.5. Maroney et al. (1994) suggested that the clayey silt exhibited substantial cohesive behavior. The eastern embankment consisted of poorly graded coarse clean sand compacted at 90% relative compaction, and its friction angle was estimated to be about 40° . These two materials are considered to represent the type of materials found behind abutments.

Cone penetration tests (CPT) conducted at the site prior to abutment construction indicated that the upper 5.5 m of soil consists of silty sand to sandy silt underlined by a 1.8 to 3.0 m thick layer of clayey soil followed by silty sand to sandy silt at greater depths. The water table was 9.1 m from the surface.

The testing program resulted in two longitudinal load responses against two different embankment materials (clayey silt and clean sand) and in one transverse load response against a clean sand embankment. In the longitudinal tests, the loads were applied at mid height on the abutments; while, in the transverse test, the load was applied near the top.

The loads against the abutments were applied in a cyclic manner with a steadily increasing magnitude. The entire testing was instrumented with an extensive network of pressure transducers, magnitudes, LVDTs, strain gages, and inclinometers. Since the approach to be calibrated is for granular material, only the results pertaining to the east abutment are presented below. For more details on the instrumentation and testing program, see Maroney et al. (1994) and Maroney (1995). The test results represent a unique situation in which extensively instrumented large-scale abutments were tested to failure in the field.

Figure 8 shows the field-measured longitudinal and transverse secant abutment stiffnesses of the east abutment as a function of abutment displacement. It is clear from the figure that the abutment behavior is nonlinear right from the start of the test. The longitudinal stiffness at a low displacement (around 0.9 mm) was measured to be around 1.24×10^5 kN/m, and it dropped by as much as a factor of 4.2 when the displacement increased to 38.0 mm. The longitudinal test was not continued fur-

Fig. 8. Field measured secant abutment stiffness: Sand embankment (after Maroney et al., 1994).

ther. On the other hand in the case of the transverse test, the stiffness at around 2.0 mm was about 10^5 kN/m and it dropped to 8.0×10^3 kN/m at a displacement of 105.0 mm. The reduction in stiffness is by as much as thirteen. The transverse test was continued until abutment failure was clearly evident.

Figure 9 shows the abutment dimensions used to evaluate the abutment stiffnesses utilizing the procedure described in this paper. The soil properties used with the proposed approach were either reported by Maroney et al. (1994) or interpreted from design manuals (NAVFAC, 1982; Barker et al., 1991). Table 1 gives the values used for the soil properties. The interface friction angles

Fig. 9. Abutment dimensions used in the validation study (Sand embankment)

Table 1. Soil properties used in the calibration study

Parameter	Value
Unit weight of soil, γ (kN/m ³)	18.5
Backfill friction angle, ϕ_b (°)	40
Backfill-abutment interface friction angle, δ_b (°)	15
Foundation soil friction angle, ϕ_f (°)	35
Foundation-base friction angle δ_f (°)	21

at the base and on the side of the abutment were selected to be 21° and 15° (Richards et al., 1993). These are consistent with values recommended for use for concrete cast on clayey silt soils and side friction against clean medium dense sand (NAVFAC, 1982). The relative compaction in the embankment is reported to be 90%, which is assumed to represent a relative density of about 60 to 70% (Byrne, 1987). This level of relative density corresponds to a $(K_2)_{\max}$ value of 55 (Seed and Idriss, 1970). As reported in the field test description, the longitudinal load was applied at mid depth while the transverse load was applied at 0.20 m from the top of the abutment in the computations (Fig. 9).

The procedures outlined in the paper can be used to obtain abutment stiffnesses under the condition of either fixed-head or free-head piles. In the case of fixed head piles, no rotation is allowed at the pile top. Such an example is representative of field condition in which the piles are provided with a large heavily reinforced pile cap and the cap is fully buried in soil (Ooi and Duncan, 1994). The pile cap provides a very effective rotational restraint against pile top rotation. In the field test reported by Maroney et al. (1994), the piles are directly connected to the abutment; and thus, it represents more closely the case of an abutment on free-head piles. Therefore, only the results computed for free-head piles are presented.

Figures 10(a) and 10(b) compare the predicted and measured secant abutment stiffnesses for the longitudinal and transverse directions. The stiffnesses have been divided by the width of the wall in the transverse direction (4.72 m) and presented in terms of kN/m/m of the wall

Fig. 10. Comparison between computed and measured abutment stiffnesses: (a) Longitudinal stiffness; (b) Transverse stiffness

width. The comparison between the predicted and reported stiffness values is very good. In the case of the longitudinal stiffness (Fig. 10(a)), there is an underprediction at the low displacement of about 20%, which is within the variation accepted in geotechnical engineering design. At other levels of displacement (more than 7 mm), the predicted stiffness values very closely match with those measured. In the case of transverse stiffness (Fig. 10(b)) also, the predicted values closely match those measured above a displacement of 2 mm.

The field testing program described by Maroney et al. (1994) also investigated the failure surface developed behind the abutment using foam columns installed within the abutment fill. They reported that the failure surfaces which developed behind the abutment were consistent with passive failure zones, and that the abutment movement was rigid body movement. These findings support and lend credibility to the assumptions used in the development of the proposed approach. The difference between the computed and measured stiffness values may be partly attributed to the presence of foam columns in the field tests.

ABUTMENT STIFFNESSES FOR TYPICAL CASES

The test results reported by Maroney et al. (1994) represent the largest abutment tested thus far in the field. The abutment height and base width in these tests are 2.1 and 0.51 m respectively. Since the abutment height and width of typical field abutments are much higher, the validity of the direct use of these field test results is questionable for many field cases. Furthermore, the abutments in the field carry superstructure load and many are founded on multiple rows of piles. In the parametric study presented below, the abutment stiffnesses have been investigated by using more typical field abutments.

In abutment stiffnesses estimation, the dimension of the abutment as well as number of piles, pile diameter and spacing must be known *a priori*. Figure 11 shows a typical seat-type abutment on a pile foundation along with various dimensions that define the abutment. Table

Fig. 11. Abutment dimensions

Table 2. Abutment dimension and soil properties used in the parametric study

Parameters	Value
Dimensions	
H (m)	4.5
H_p (m)	2.0
A_1, A_2, A_3, A_4 (m)	0.3, 1.0, 1.0, 0.5
(A_5, A_6) (m)	0.6, 1.3
(C_1, C_2, C_3) (m)	0.25, 1.2, 1.8
B (m)	3.2
W (m)	10.0
D_{pile} (m), R_f (Dimensionless)	0.4, 1.0
E_p (kN/m ²)	6.13E7
Pile spacing	3D
Deck Forces	
V_D (kN/m)	250.0
H_D (kN/m)	100.0
Soil Properties	
Unit weight of soil, γ (kN/m ³)	18.5
Backfill friction angle, ϕ_b (°)	35
Backfill-abutment interface friction angle, δ_b (°)	15
Foundation soil friction angle, ϕ_f (°)	35
Foundation-base friction angle, δ_f (°)	21

2 gives typical values of many abutment dimensions. The abutment considered here is a medium high ($H=4.5$ m) supporting a vertical (V_D) and a horizontal (H_D) deck load of 250 and 100 kN/m respectively (see Fig. 5). The abutment base rests on 3 rows of 0.4 m diameter piles. The backfill and the foundation soil are assumed to have a friction angle of $\phi=35^\circ$, and the interface friction angle at the foundation δ_f and on the side δ_b are assumed to be 21° and 15° respectively. These values are typical for medium dense sand and sandy silt mixtures (Barker et al., 1991; Richards et al., 1993). For transverse stiffness, the wing wall dimensions are required and the values are given in Table 2. The spacing between piles is very important in calculating the pile group effects. In this study, 3D pile spacing has been used. This means that, for the 10 m width abutment considered here, there are three rows of nine piles each (Fig. 11).

Figures 12(a), and 12(b) respectively present the longitudinal (S_L), and transverse (S_T) stiffnesses evaluated using the procedure outlined in the paper. All of the translational stiffnesses are presented as per unit width of the abutment in the translational direction (W).

A major reduction in longitudinal abutment stiffness (Fig. 12(a)) occurs within about 15 mm of abutment movement; and, after that, it remains unaffected by the abutment displacement. The reduction in stiffness as the abutment displacement increases from 1 to 15 mm is as much as 6.3 for the case of abutment on free head piles. However, this reduction is much lower, around 3.1, for the abutment on fixed head piles. The longitudinal stiffness in the case of fixed head piles is higher; at low displacements, the free-head piles case is about 60% of the fixed head piles case. But at larger displacements (above

Fig. 12. Translational stiffness for pile spacing = $3D$ and $V_D = 250$ kN/m: (a) Longitudinal stiffness; (b) Transverse stiffness

20 mm), this ratio drops to as low as 28%. Such results are consistent with the field test results with fixed head and free head piles (Ooi and Duncan, 1994). The observations relative to the transverse stiffness results shown in Fig. 12(b) are also similar except for the fact that the difference between fixed- and free-head piles at low displacements (less than 4 mm) is not high.

The use of the results presented above for field problems depends on the pile head fixity condition in the field. If the piles supplied with a heavily reinforced pile cap and it remains in full contact with the ground, the fixed-head condition may be appropriate. However, if the pile cap is not fully buried or if scour of the soil around the pile has reduced or eliminated cap-soil interaction, the case of free-head piles may be applicable. In any event, the free head piles case gives conservative estimates of abutment stiffness.

The conventional (e.g., Caltrans) method of evaluating the abutment stiffnesses consists of individually computing the stiffness contributions from end wall and piles and summing the results. In the approach presented here the stiffness contributions are evaluated in a coupled manner and, therefore, information on the relative contribution of, say, piles is not readily available. The pile contribution (as a percentage) of total abutment stiffness has been extracted and presented in Figs. 13(a) and 13(b) for the longitudinal and transverse cases respectively.

Figure 13(a) (longitudinal) reveals that the pile contribution increases proportionally with the abutment dis-

Fig. 13. Translational stiffnesses: Pile contribution: (a) Longitudinal stiffness; (b) Transverse stiffness

placement for both fixity conditions until a displacement of about 20 mm, beyond which it remains a constant. The pile contribution above 20 mm is about 75 and 50% for the abutments on fixed and free head piles respectively. On the other hand, the contribution of piles in the case of transverse stiffness (Fig. 13(b)) is very similar for the fixed and free head piles in all ranges of displacement. At large displacements (above 20 mm), the pile contribution is about 65% of the total for both fixity conditions. Since the pile contribution is a substantial portion of the total abutment stiffness (especially at higher displacements), care should be taken in designs to see whether the piles can take such displacements without failure.

Figures 14(a) and 14(b) show the influence of bridge deck loads on the abutment stiffness. The bridge deck loads, V_D and H_D , have been increased to 500 and 200 kN/m, while all the other input parameters were unaltered (Table 2). By comparing the results shown in the corresponding Figs. 12(a) and 12(b), the influence of the increase in deck load can be interpreted. A substantial difference between the stiffnesses is seen between the two deck load cases considered. The translational stiffnesses with the higher deck load are higher, indicating the importance of the deck load. It may be noted that the deck load components are included in the analysis when equilibrium (force and moment) equations are formulated.

Fig. 14. Translational stiffness for pile spacing = 3D and $V_D = 500$ kN/m: (a) Longitudinal stiffness; (b) Transverse stiffness

SUMMARY AND CONCLUSIONS

This paper presents an efficient approach, based on many well-established soil mechanics principles, to evaluate the nonlinear translational spring stiffnesses (longitudinal and transverse) to represent abutments founded on pile foundations in seismic designs. The applicability of previous studies is limited because such studies are incapable of handling many important factors such as the abutment dimensions, nonlinear soil behavior, nonlinear pile-soil interaction, and difference in soil behavior under active and passive conditions. The proposed approach accounts for all of these factors in a realistic manner. Only routinely used soil properties, such as the unit weight, angle of internal friction and interface friction angles are required in the model. The resistance for deformation at the base of the abutment on piles was modeled using the recent comprehensive procedure (CLM) developed by Duncan et al. (1994).

A calibration study to verify the predictive capability of the approach is described in the paper. This study utilized data from a recently conducted large-scale field test. Results from both longitudinal and transverse field tests were used. An very good agreement was observed between the proposed approach and the field test results.

In the second phase of the study, the proposed approach was used to investigate the influence of pile fixity conditions, contribution of piles, and deck load on abutment stiffnesses. The study revealed that (1) the abutment

stiffnesses are significantly affected by the pile fixity conditions, especially at large displacements where the stiffness of free-head pile case can be as low as about 28% of the fixed-head pile case, (2) pile contribution to abutment stiffness increases with displacement and after about 20 mm of displacement stabilizes; its contribution can be as much as 75% (longitudinal, fixed-head pile case) of the total abutment stiffness, and (3) an increase in superstructure load leads to increases in translational stiffnesses.

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