Models for seismic response of highway bridge abutments

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ABSTRACT: A simplified equivalent spring model representing the dynamic stiffness of a highway bridge abutment system is developed for use in seismic response studies. It is based on linear elastic two-dimensional plane strain analyses of typical bridge abutment cross-sections. The model consists of two massless translational springs; one spring in the transverse direction and one in the vertical direction. The springs are then incorporated as boundary conditions in a finite element model of the Meloland Road Overpass bridge. Dynamic bridge responses computed from the finite element model show good qualitative agreement with strong-motion records obtained on the bridge during the 1979 Imperial Valley earthquake.

INTRODUCTION

Prior to the 1971 San Fernando earthquake most seismic damage to highway bridges was associated with foundation failures caused by soil liquefaction, excessive settlement or slope instability. The San Fernando earthquake showed however, that significant damage could also result from the dynamic behaviour of the bridge superstructure and its interaction with the earth abutments at either end of the bridge deck. The importance of a dynamic analysis which includes allowance for the abutment response is recognized by current seismic design guidelines for bridges (AASHTO 1983; ATC-6 1981), although little information is provided as to exactly how this may be accounted for in a seismic bridge analysis and few studies have been done to estimate an equivalent stiffness for abutment boundary elements. Douglas and Reid (1982) conducted an experiment on a full scale bridge to determine the bridge dynamic properties and they concluded that simple boundary elements springs were adequate to approximate the soil-structure interaction effect. Douglas and Norris (1984) studied a procedure to estimate the foundation dynamic stiffness of the pile foundations using geotechnical considerations. The only recently published study known to the authors on the dynamic modelling of a bridge abutment system is by Werner, et al (1985), and a related study by Scott and Levine (1986).

The objective of the research reported in this paper is to establish general simplified models for evaluating the dynamic stiffness characteristics of highway bridge abutment systems. In this analysis, linear elastic 2-D plane strain models of typical bridge abutment cross-sections are used to estimate a single equivalent abutment stiffness. Separate spring values are developed for motions in both vertical and transverse directions and are applied to the Meloland Road Overpass (MRO) bridge in Southern California.

ABUTMENT MODEL

In most highway bridges the earth fill which forms the approach to the bridge is much longer than either the height or the width of the abutment, and the approach slope is usually quite small. Therefore, it seems reasonable to represent the 3-D abutment fill with a 2-D plane strain model as shown in Figure 1-a. The development of this model to predict an equivalent dynamic abutment stiffness is described in the following paragraphs and its application in the dynamic analysis of a two-span highway bridge is illustrated. In this development it is assumed that: (1) the inclined faces of the abutment wedge are flat surfaces, (2) for dynamic response, the abutment wedge is considered to behave like a pure shear beam and (3) the shear modulus of the soil mass is constant.

Free Vibration Analysis

Initially, a simplified uniform shear beam is considered (Figure 1-b) having a height and top width equal to that of the wedge in Figure 1-a, with fundamental frequency

$$f_{sb} = \frac{1}{4H}\sqrt{\frac{Gg}{\gamma}} \qquad (1)$$

where, G = shear modulus, γ = weight density, H = height of the beam, and g = acceleration due to gravity. For most standard highway bridges, the soil parameter G is usually estimated with only a limited

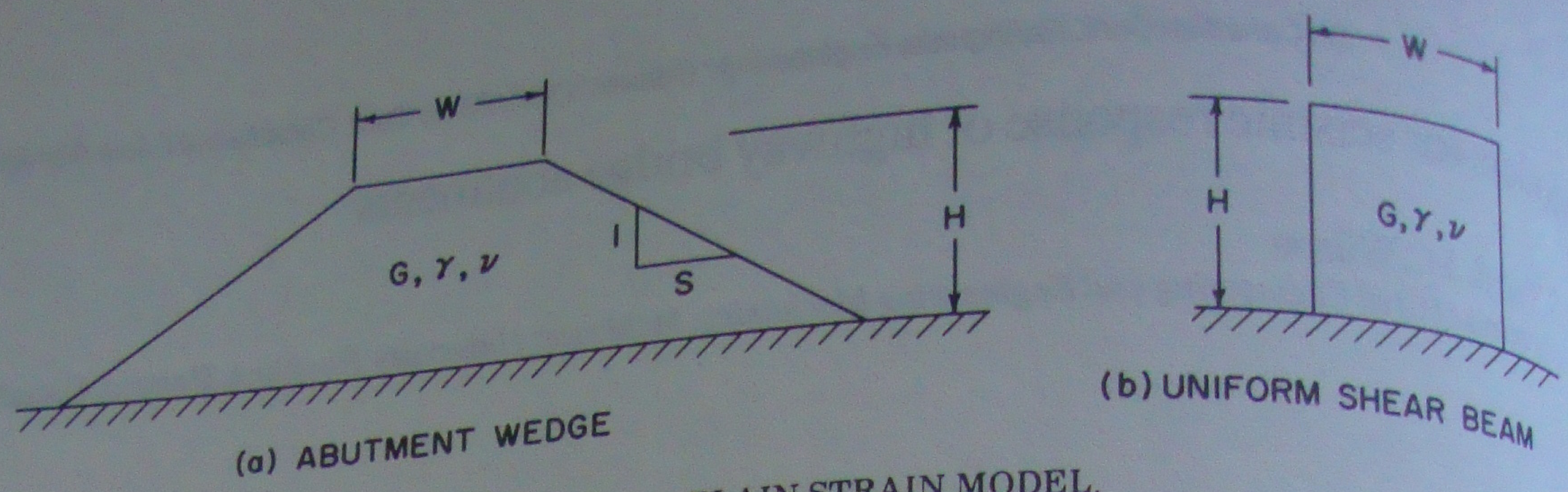

FIGURE 1. PLAIN STRAIN MODEL.

know ledge of the actual soil properties at the bridge site.

A program to compute the fundamental frequency of the abutment wedge (Figure 1-a), was set up using 3-node triangular plane strain elements (2 d.o.f per node), with the finite element grid shown in Figure 2. This program was then used in a parametric study in which frequencies of the uniform shear beam and the finite element wedge were computed for various heights, widths and side slopes. From this study, the following empirical relationship between the shear beam and wedge frequencies was established:

$$f_n^2 = 1.722\,[H/W]^{.172} f_{sb}^2 \tag{2}$$

where, f_n = frequency of the abutment wedge, H = height of the wedge, W = top width of the wedge, and f_{sb} = frequency of the corresponding uniform shear beam from Eq. (1). The above equation was found to be generally applicable for all heights and widths, but there is a slight variation for different side-slopes. However, the variation within the practical range of earth fill side-slopes of 1:1 to 1:3 was found to be small (<10%), and the relationship in Eq. (2) may be assumed to be applicable in most practical situations. Results obtained using Eq. (2) agree quite well with predictions of the more complicated truncated wedge theory established by Ambraseys (1960). The Ambraseys' wedge theory has been used to predict fundamental frequencies of earth fill structures, such as earth dams, with reasonable results (Abdel-Ghaffar, 1979). As long earth fill abutments may be considered physically similar to the earth dam structures, the agreement seems to provide encouraging support of the plane-strain model for abutments with geometric features similar to the MRO.

Equivalent Spring Analysis

Having established the plane strain model, we proceed to develop an equivalent spring model for the abutment based on plane strain analysis. To do this, transverse and vertical static loads F_T are applied at

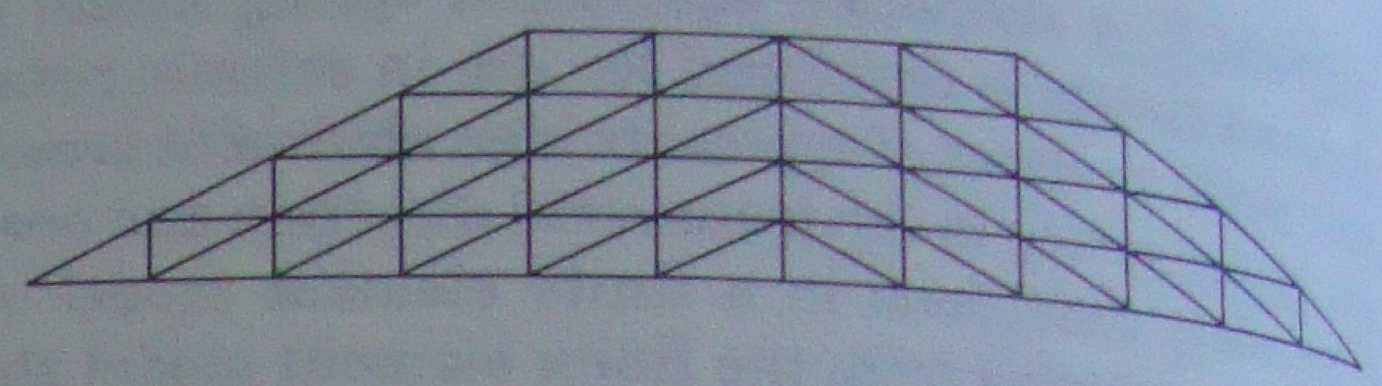

FIGURE 2. FINITE ELEMENT GRID.

the top of abutment wedge as show in Figure 3-a and 3-b, respectively. The top deflection stiffness, K_T, is obtained using the relationship, $K_T = F_T/\Delta_T$; where F_T is the top applied load and Δ_T is the resulting top deflection. The abutment wedge, being a plane strain model, is assumed to be a unit section of a long abutment soil mass. The unit stiffnesses are:

$$k_t = \frac{2\,S\,G}{\ell n(1 + 2\,S\,H/W)} \tag{3}$$

$$k_v = \frac{2\,S\,E}{\ell n(1 + 2\,S\,H/W)} \tag{4}$$

where, k_t = transverse stiffness per unit length along the abutment, k_v = vertical stiffness per unit length along the abutment; H, W and S are the height, width and side slope, respectively, of the abutment wedge; and G and E are the shear and elastic modulus of the abutment soil. Separate springs are developed for the transverse and vertical directions only, as responses in these directions are usually the major contributors to the overall earthquake response of typical highway bridges.

The static stiffnesses developed above are taken to be values for equivalent springs describing abutment dynamic stiffness. These stiffnesses are then incorporated as boundary conditions for the analysis of the bridge superstructure. To obtain the total abutment stiffness, the stiffness of a unit slice of the abutment fill is multiplied by the length of the abutment wingwall. To use the wingwall length as the multiplying factor is physically reasonable, since the soil contained

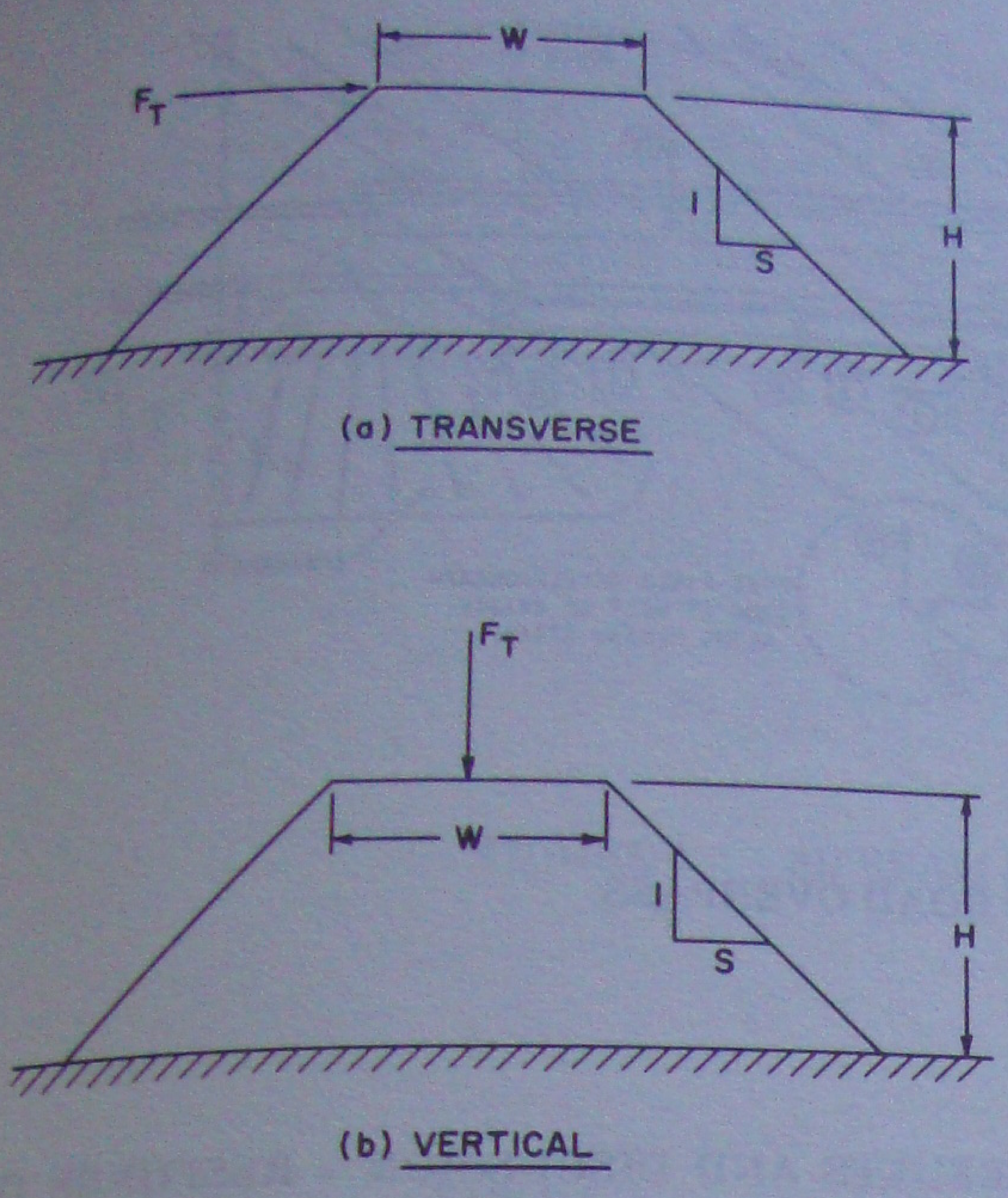

FIGURE 3. EQUIVALENT SPRINGS.

within the vicinity of the wingwalls is likely to have the greatest influence on the abutment stiffness. This is especially true for the transverse spring, but even for the vertical spring, the wingwall length should represent the influence length reasonably well.

The same top stiffnesses, $K_T = F_T/\Delta_T$, for the abutment wedges, were computed using the finite element grid shown in Figure 2 to examine the practical suitability of Eqs. (3) and (4). Several computations of K_T for different combinations of G, H, W and S were done and it was found that for both transverse and vertical springs, the theoretical values (Eqs. (3) and (4)) and finite element values were similar. Thus, for the subsequent analyses of the abutment models, the theoretical stiffnesses in Eqs. (3) and (4) are used as their computation is straightforward.

In summary, the proposed abutment model consists of replacing the abutment soil fill with massless transverse and vertical boundary springs, the stiffnesses of which can be established using unit values from Eqs. (3) and (4), multiplied by the length of the wingwall. The wedge shaped cross-section of the soil abutment at the ends of bridge is used for the abutment dimensions in Eqs. (3) and (4).

MELOLAND ROAD OVERPASS (MRO)

The general applicability and usefulness of the abutment model developed above is used examined using the seismic response of the Meloland Road Overpass. The MRO, located in the Imperial Valley in Southern California, is a modern reinforced concrete bridge comprised of two continous ~32 m spans with monolithic abutments and a central single column. The bridge was instrumented with 26 accelerometers (Figure 4) which were triggered during the 1979 Imperial Valley earthquake. The simplicity of the bridge and the availability of the measured earthquake response of the abutment and bridge superstructure make this structure an ideal example case.

MRO – Soil Parameters

Results of test borings indicate that the soil at the MRO site is mostly clay with traces of silt and sand. The Standard Penetration Test (SPT) resistance measured approximately 46 blows/m (14 blows/ft). Using this information Scott and Levine (1986) suggest that the soil is probably a medium to stiff clay. Since no information is avalable on the abutment fill, it is assumed to have the same soil properties as the soil below the bridge. For a medium to stiff clay, an elastic modulus of $E = 20\ MN/m^2$ (400 $kips/ft^2$), and Poisson's ratio of $\nu = 0.30$, are assumed. It is felt that these values provide a reasonable description of soils at the MRO site. The following abutment wedge dimensions (see Figure 1-a) are used for the MRO soil mass: height, H = 7.3 m; top width, W = 14.6 m; and side-slope, S = 2. These dimensions were taken from the structural drawings of the MRO bridge (Caltrans, 1969). An estimated soil weight density of $\gamma = 15.7$ kN/m^3, $G = 7.7\ MN/m^2$ [$G = E/2(1+\nu)$], and $\nu = 0.3$, gives a shear wave velocity for the abutment soil of 67 m/sec.

MRO – Comparing Frequencies

Using the estimated soil parameters and the above abutment dimensions; a fundamental transverse frequency of f = 2.83 Hz [Eq (2)] was calculated for the MRO bridge abutment (using a uniform shear beam frequency of f = 2.29 Hz [Eq (1)] as the value for f_{sb}). This frequency is compared to observed response of the MRO bridge, as deduced from an analysis of the recorded abutment earthquake motion using a single input-single output system identification methodology (SYSID) developed by Beck (1978). In this approach the recorded free field acceleration (Ch. 24 on Figure 4) was taken as input and the abutment motions (Ch. 26 and Ch. 11) were used as outputs for the system identification. The system identification indicated a change in frequencies over the duration of the earthquake, however during the initial few seconds of response, an abutment frequency of ~2.4 Hz was observed, which corresponds reasonably well to the computed 2.83 Hz values.

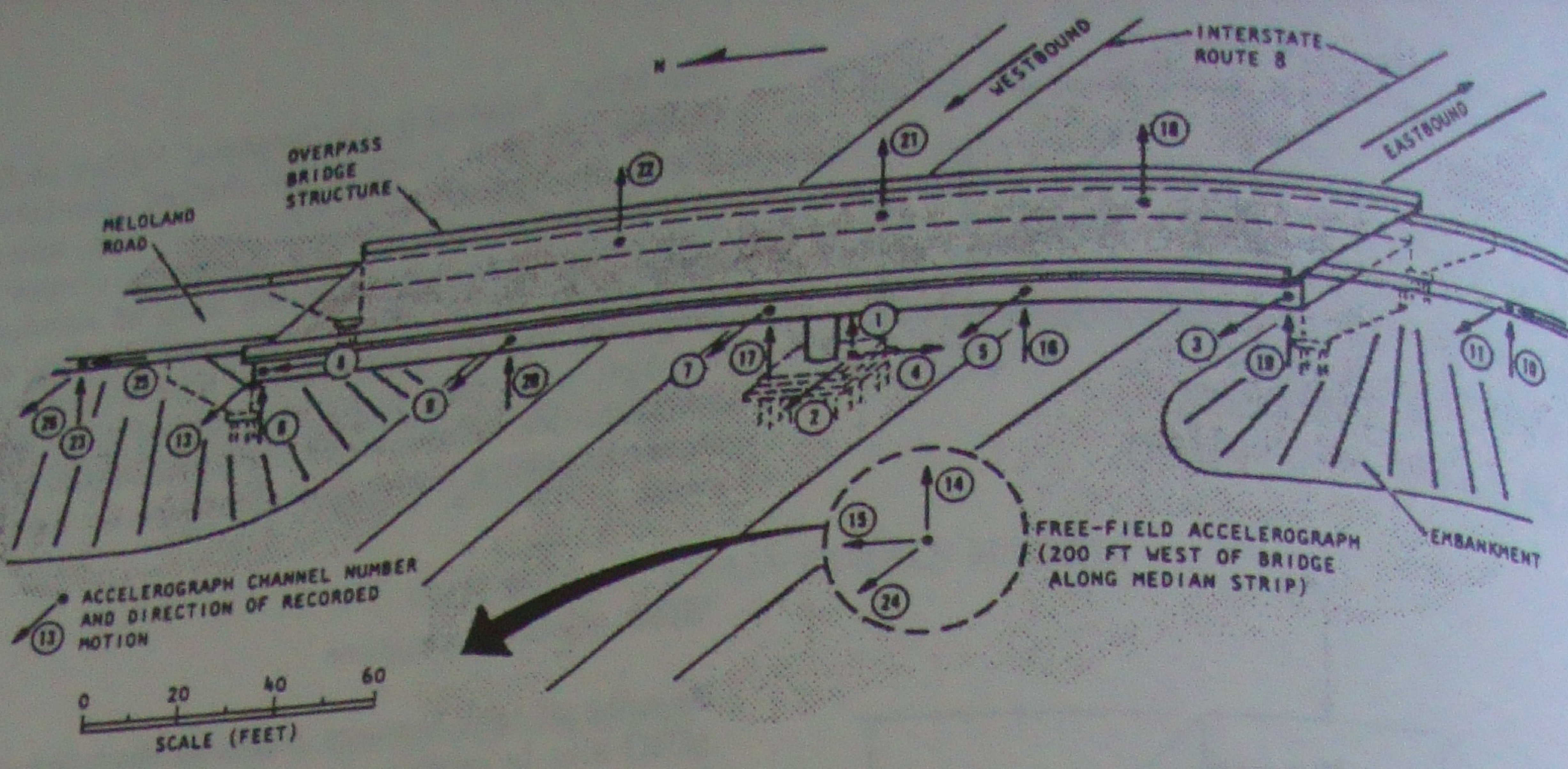

FIGURE 4. MELOLAND ROAD OVERPASS.

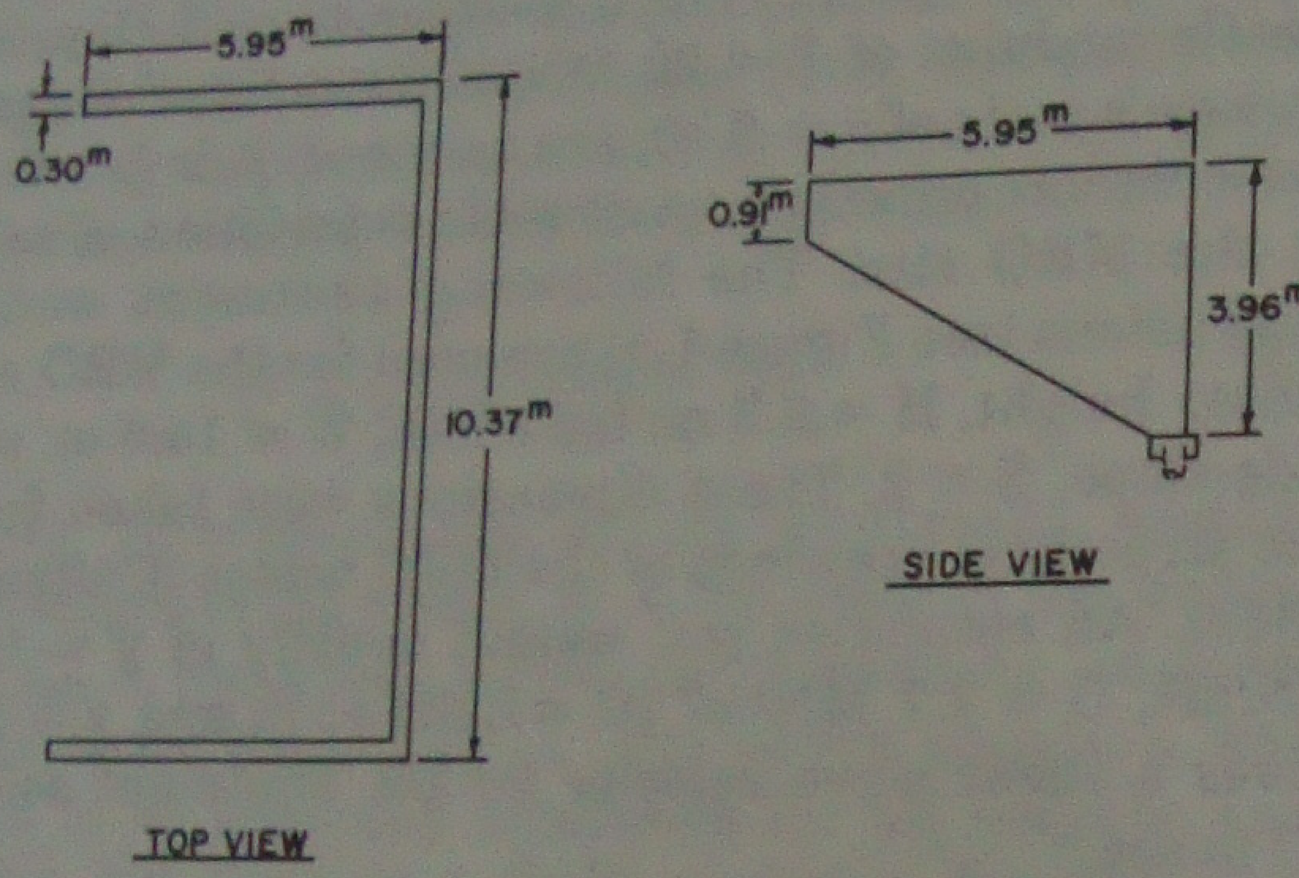

FIGURE 5. ABUTMENT WINGWALL.

MRO – Equivalent Springs

Using the abutment dimensions and the estimated soil parameters stated earlier, unit stiffnesses, k_t = 28 MN/m^2 [Eq. (3)] and k_v = 73 MN/m^2 [Eq. (4)], were evaluated for the MRO abutment.Considering a 5.95 m length of the MRO wingwall, shown in Figure 5, results in total equivalent abutment stiffnesses of K_t = 170 MN/m and K_v = 430 MN/m. Since abutments at both ends of the bridge are similar in size and are assumed to have the same soil properties, it is appropriate to use the same stiffnesses for boundary springs at each end of the finite element model of the bridge.

RESULTS AND DISCUSSION – RESPONSE OF MRO BRIDGE

A finite element model was developed for the MRO bridge, incorporating the calculated abutment stiffnesses as boundary spring elements at the ends of the deck. The bridge deck and the column are modeled with 3-D beam elements; 8 elements along the deck and 1 along the column (Figure 6). Each node is free to translate and rotate in all three axes, except for the bridge deck end nodes where translation in X-direction (longitudinal) is restrained. The column base is supported by a pile foundation and, for the purpose of this study, it is felt that reasonable foundation stiffnesses can be obtained by assuming only rotational degrees of freedom at the base. Two rotational springs (rocking stiffnesses) are placed at the column base with numerical values calculated using techniques of Poulos and Davis (1980).

Results of a free vibration analyses of the finite element bridge model are compared with two other analyses of the recorded earthquake motions. The first analysis is a single input-single output system identification (SYSID; Beck,1978) where the recorded motion at freefield (Ch. 24) was taken as input and at bridge deck (Ch. 5, Ch. 7 or Ch. 9) as output. The second analysis used for comparison was a multiple input-multiple output system identification (MODE-ID), taken from a report on the MRO strong motion records by Werner et al. (1985). Frequencies and mode shapes for two of cases are compared in Table 1.

The first three frequencies determined from the bridge finite element model are quite close to values from Werner et al. A fundamental frequency of f = 2.44 Hz using SYSID is also similar to the others. Both the finite element model (F.E.M.) and the MODE-ID procedure identified the fundamental mode as a symmetric transverse vibration of the bridge. The second mode was identified by the F.E.M. as an

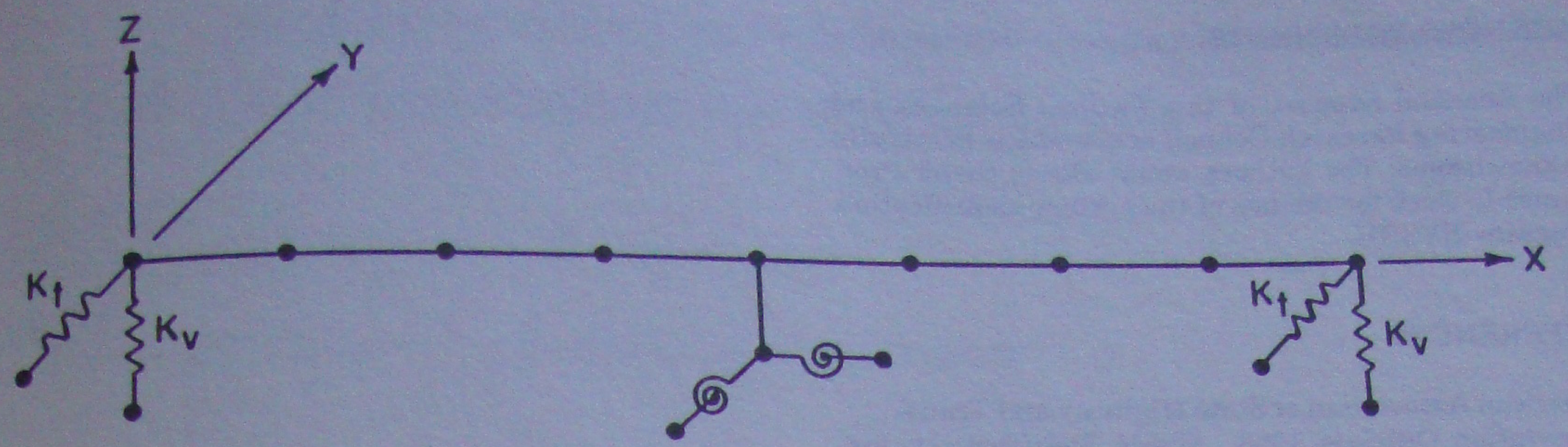

FIGURE 6. 3D BEAM MODEL OF THE MRO BRIDGE.

TABLE 1

	MODE FREQUENCIES AND SHAPES		
	First	Second	Third
MODEL (This study)	Transverse f = 2.57 Hz	Vertical f = 3.28 Hz	Vertical f = 4.55 Hz
MODE-ID (Werner et al)	System Transverse f = 2.55 Hz	Deck Transverse f = 3.27 Hz	Deck Vertical f = 4.57 Hz

antisymmetric vertical mode but the MODE-ID procedure identified it as a deck tranverse mode. The F.E.M.'s third mode is a symmetric vertical mode where small translations at deck ends are observed. The third mode of the MODE-ID is solely a deck vertical mode. Although the frequencies are identical, reasons for the discrepancy in mode shapes have not been identified at the present time.

The close agreement between the fundamental mode shapes and frequencies of the finite element model and the observed data represents a significant and encouraging step in the development of a simplified procedure for modelling the stiffness of bridge abutments for seismic dynamic analysis. Differences that seem to exist for higher modes may be due to limitations of the capability of the simplified model to represent a complicated soil-structure system. Research work is continuing to investigate these problems.

CONCLUSIONS AND RECOMMENDATIONS

The results of this study indicate promising developments in finding simple equivalent models for approximating the dynamic stiffness of highway bridge abutment systems. The proposed model for bridge abutment stiffness, based on an analysis of the dynamic response of typical abutment systems appears to produce reasonably good agreement with system identification analysis of the measured earthquake response of the Meloland Road Overpass bridge.

Several improvements and additions to the model may be considered for the future, including development of simplified methods to include masses and rotational springs in the model. For the MRO abutments, short time windows (4 sec intervals) of the earthquake record showed some lengthening of the period of the abutment system occurred during the excitation, but the stiffness seemed to be restored after the shaking. Therefore, a linear analysis of the MRO seems to be a reasonable approach to take in the initial development of these models but non-linear analysis might warrant investigation at a later date.

As pointed out earlier, one of the greatest uncertainties in the analysis is estimating the soil properties (i.e., G). For the benefit of future research developments it would be highly desirable to obtain shear wave velocity measurements on the soil mass of a few selected bridge abutments. This data would help to reduce uncertainties in the soil parameters and assist in the development and improvement of the abutment models.

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