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CONTACT STRESSES AND GROUND MOTION GENERATED BY SOIL-STRUCTURE INTERACTION

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SUMMARY

A study has been made of the dynamic contact stresses that the foundation of a nine-storey reinforced concrete building exerts on the soil during forced vibration tests. The effects of the flexibility of the foundation on the contact stress distribution and on the force–displacement relationship for the foundation have been examined in an attempt at testing several simplifying assumptions commonly used in soil–structure interaction studies. Comparisons of calculated and observed ground displacements induced by soil–structure interaction in the immediate neighbourhood of the building have also been presented.

INTRODUCTION

Although forced vibration tests of structures are fairly common, very few of these tests have been focused on the study of the interaction between structures and the ground. In 1966, Kuroiwa and Jennings[1,2] conducted forced vibration tests of the nine-storey reinforced concrete Millikan Library Building located on the campus of the California Institute of Technology and measured the motion of the foundation as well as the motion on the nearby soil surface. The results of these tests demonstrated the possibility of performing full-scale soil–structure interaction experiments. In addition, Jennings[3] observed that the ground motion induced by the forced vibrations of the Millikan Library could be measured at distances up to 3 miles from the building.

Recently, a new and more comprehensive set of experiments designed to study the interaction between the Millikan Library Building and the surrounding soil as well as the resulting motion on the soil surface away from the building has been performed. The building was forced into its lowest resonance in both the N–S and E–W directions by means of a vibration generator mounted on the roof of the structure and the three components of motion were measured at fifty locations on each of six levels of the building including the basement. In addition, the displacement field was recorded at 100 locations in the near field corresponding to the soil in the immediate neighbourhood of the building, and at over 250 locations in the far field corresponding to a portion of the Pasadena area extending to a distance of up to 4 miles from the building.

The recording equipment consisted of three Ranger-type seismometers (moving coil, velocity-type transducers, with natural period in the vicinity of 1 sec), an Earth Sciences SC-201A signal conditioner and two Brush recorders. The signal from a Ranger-type seismometer, proportional to the relative velocity of the transducer mass, was first amplified 350,000 times by the SC-201A signal conditioner. The velocity proportional voltage was then attenuated and passed through a filter which had 6 dB per octave slope and 90 per cent phase shift in order that the recorded voltage output would be proportional to the relative displacement of the transducer mass. The results of these experiments have been reported by Foutch *et al.*[4] and Luco *et al.*[5]

It is one of the purposes of this study to evaluate the stress distribution at the contact between the foundation of the Millikan Library and the underlying soil and to compare the stresses thus computed with those resulting from the usual assumption of a rigid foundation slab. The data used in these computations

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correspond to the measured displacement field of the basement slab, the geometry of the foundation and some average properties of the soil. The results of this comparison should help in testing the adequacy of the rigid foundation slab assumption generally used in soil–structure interaction studies.

A second objective is to analyse the displacement field in the immediate neighbourhood of the building. In particular, comparisons will be made between theoretical values of the displacements in the near field calculated on the basis of the measured displacements of the foundation and the experimentally observed values. The reason for this comparison is the desire to test the existing analytical tools used to evaluate the soil displacements caused by soil–structure interaction. Such computations play an important role in the study of the interaction between adjacent structures.

DESCRIPTION OF THE BUILDING, FOUNDATION AND FOUNDATION MOTION

The R. A. Millikan Library is a nine-storey reinforced concrete building located on the campus of the California Institute of Technology. The library building is 69 × 75 ft in plan and stands 144 ft above grade and 158 ft above the basement level. The lateral loads in the transverse (N–S) direction are resisted primarily by 12-in reinforced concrete shear walls which are located at the east and west ends of the building. In the longitudinal (E–W) direction the 12-in reinforced concrete walls of the central core provide most of the lateral resistance (Figure 1).

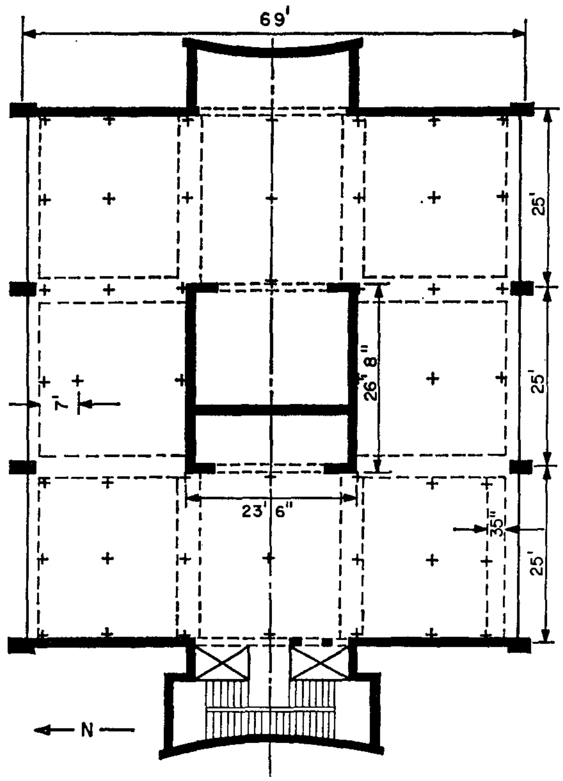

Figure 1. Typical floor plan

The foundation system of the library consists of a central pad 32 ft wide and 4 ft deep which runs in the E–W direction and extends from the east curved shear wall to the west curved shear wall (Figure 2). Also provided are beams 10 ft wide by 2 ft deep which run E–W beneath the rows of columns at the north and south ends of the building. These beams are connected to the central pad by stepped beams. The contact between the central pad and the underlying soil is approximately 23 ft below grade.

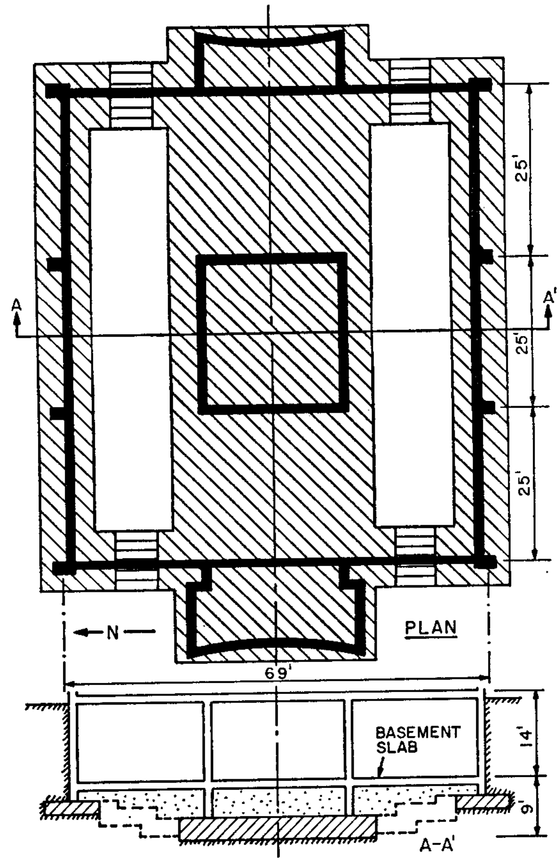

Figure 2. Foundation plan and a N–S section

The vertical, N–S and E–W components of motion at fifty locations in the basement of the Millikan Library have been recorded during the shaking of the building in both the N–S and E–W directions. The points where the measurements were made are indicated by crosses in Figure 1. The amplitudes of the three-dimensional motions at these locations are shown in Figures 3 and 4 for N–S and E–W excitations, respectively.

Inspection of Figures 3 and 4 indicates that the basement slab did not behave as a rigid plate and that its deformation patterns are dependent upon the superstructure above, on the properties of the soil beneath and on the direction of the excitation. While the building was forced into resonance in the N–S direction, the stiff shear walls on the east and west ends of the building caused an almost rigid translation of the basement slab in the N–S direction together with an almost uniform rotation about the E–W axis of symmetry of the base (Figure 3). Some deviations from this average rigid motion may be observed at the location of the central core and at the north and south ends of the slab. In this case, the deformation of the basement slab resembles that of a flexible rectangular plate with two rigid edges (east and west ends) vibrating on top of an elastic medium. For vibrations in the E–W direction most of the lateral loads are resisted by the stiff central core, and, consequently, large localized deformations of the basement slab are generated near its contact with the central core (Figure 4).

Most studies of the dynamic interaction between structures and the supporting soil are based on the assumption of a rigid foundation. The results presented above for the Millikan Library building indicate, on the other hand, that such an assumption does not hold for a rather common configuration of superstructure and foundation. In the light of these experimental observations it becomes important to compare the response that would be obtained by use of the rigid foundation assumption with that associated with a flexible foundation. In particular, it is interesting to study the effects of the flexibility of the foundation on the stress distribution at the contact between the foundation and the soil, and on the average translation and rotation of the foundation.

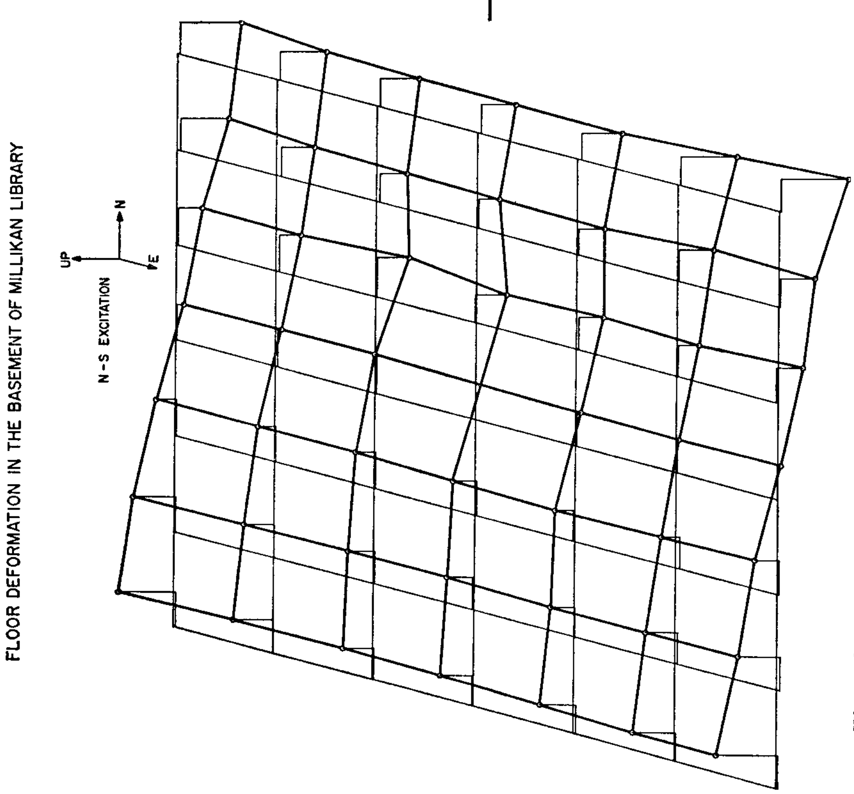

Figure 3. Basement floor deformation pattern for N–S excitation

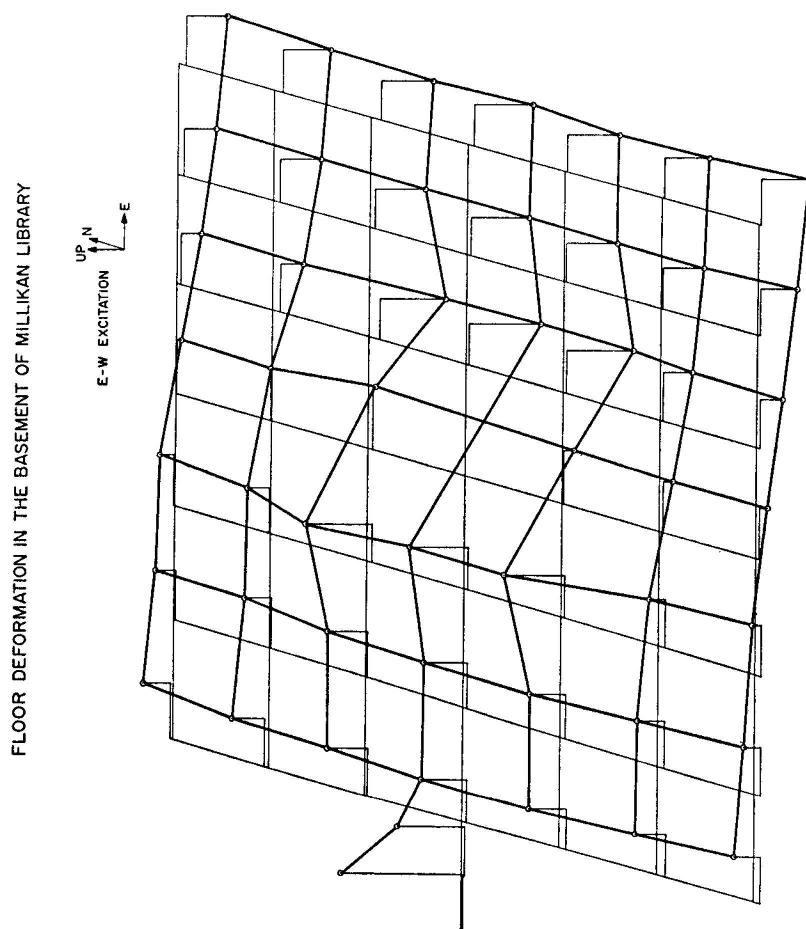

Figure 4. Basement floor deformation pattern for E–W excitation

CONTACT STRESSES

To estimate the dynamic stress distribution at the contact between the foundation of the Millikan Library and the underlying soil several simplifying assumptions need to be introduced. In the first place, it will be assumed that the motion of the foundation is represented by the recorded motion of the basement slab; second, the effects of the embedment of the foundation will be neglected, i.e. it will be assumed that the foundation may be represented by a flat plate placed on the soil surface; and third, the soil will be represented by a linearly elastic homogeneous half-space.

Referring to a Cartesian system of co-ordinates (x_1, x_2, x_3) with origin at the centre of symmetry of the basement slab and such that the axes x_1, x_2, x_3 are pointing east, north and up, respectively, it is possible to write the following integral equation for harmonic vibrations of frequency ω

$$u_i(x_1, x_2, 0) = \sum_{j=1}^{3} \int_S G_{ij}(x_1 - x_1', x_2 - x_2', 0; \kappa, \nu)\, \sigma_{j3}(x_1', x_2', 0)\, \mathrm{d}x_1'\, \mathrm{d}x_2' \quad (i = 1, 2, 3) \tag{1}$$

where $u_i(x_1, x_2, 0)\, \mathrm{e}^{i\omega t}$ and $\sigma_{j3}(x_i, x_2, 0)\, \mathrm{e}^{i\omega t}$ are respectively the displacement and stress components at the soil surface $(x_3 = 0)$. The contact stresses $\sigma_{j3}\, \mathrm{e}^{i\omega t}$ are generated by the forced vibrations of the building and do not include the static contact stresses associated with the weight of the superstructure.

The functions G_{ij} appearing in equation (1) are the Green's functions for harmonic vibrations of an elastic half-space.[6,7] The Green's functions are inversely proportional to the shear modulus μ of the soil and they also depend on $\kappa = \omega/\beta$, where β is the shear wave velocity for the soil, and on Poisson's ratio ν. The integrals appearing in equation (1) are evaluated over the surface S covered by the foundation.

Since the displacements $u_i(x_1, x_2, 0)$ have been measured, the evaluation of the contact stresses $\sigma_{j3}(x_1, x_2, 0)$ $(j = 1, 2, 3)$ reduces then to the solution of the integral equations (1). To solve these integral equations some further simplifications will be made. First, the transverse shear stresses σ_{31} and σ_{32} will be assumed to be zero for N–S and E–W vibrations, respectively; second, since the wavelengths of the waves which are generated in the soil are much longer than the geometrical dimensions of the foundation, the wave number κ will be set equal to zero; and, finally, the contact region S will be divided into 50 rectangular sub-regions S_k $(k = 1, 2, \ldots, 50)$ as shown in Figure 5 and the contact stresses will be assumed to have constant values $\sigma_{j3}^{(k)}$ within each sub-region. With these simplifications equation (1) reduces to

$$u_i(x_1, x_2, 0) = \sum_{k=1}^{50} \sum_{j} \bar{G}_{ij}^{(k)}(x_1 - x_1^k, x_2 - x_2^k, 0; 0, \nu)\, \sigma_{j3}^{(k)}$$

$$(i, j = 2, 3 \text{ for N–S vibrations}) \quad (i, j = 1, 3 \text{ for E–W vibrations}) \tag{2}$$

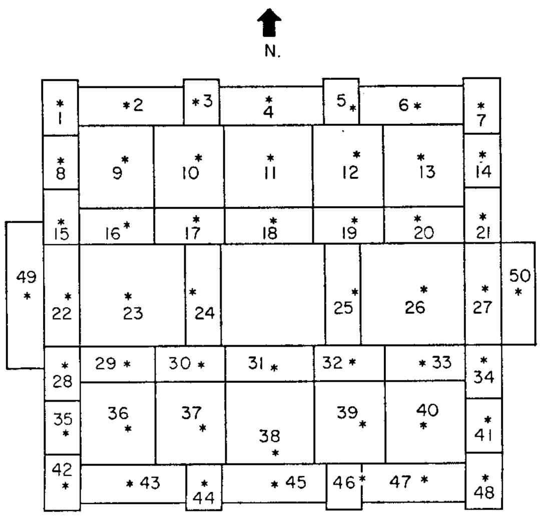

Figure 5. Discrete representation of the foundation plan in terms of rectangular sub-regions

where (x_1^k, x_2^k) corresponds to the centre of the rectangular region S_k, and

$$\bar{G}_{ij}^{(k)}(x_1-x_1^k, x_2-x_2^k, 0; 0, \nu) = \int_{S_k} G_{ij}(x_1-x_1', x_2-x_2', 0; 0, \nu)\,\mathrm{d}x_1'\,\mathrm{d}x_2' \tag{3}$$

The average Green's functions $\bar{G}_{ij}^{(k)}$ have been evaluated by one of the authors[6] for both the static ($\kappa = 0$) and the dynamic case ($\kappa \neq 0$).

Imposing equation (2) at the 50 locations (x_1^l, x_2^l) $(l = 1, 2, \ldots, 50)$ where the displacements were measured leads to the following system of linear algebraic equations for $\sigma_{j3}^{(k)}/\mu$

$$\sum_{k=1}^{50} \sum_j A_{lk}^{ij} \sigma_{j3}^{(k)}/\mu = u_i(x_1^l, x_2^l, 0) \quad (l = 1, 2, \ldots, 50)$$

$$(i,j = 2,3 \text{ for N–S vibrations}) \quad (i,j = 1,3 \text{ for E–W vibrations}) \tag{4}$$

where

$$A_{lk}^{ij} = \mu \bar{G}_{ij}^{(k)}(x_1^l - x_1^k, x_2^l - x_2^k, 0; 0, \nu) \quad (l, k = 1, 2, \ldots, 50) \tag{5}$$

It should be mentioned that the coefficients A_{lk}^{ij} depend only on the geometry of the problem and on the value of the Poisson's ratio ν for the soil. A value of $\nu = \frac{1}{3}$ is used here.

A few comments on the simplifications leading to equations (2) and (4) are in order. The first simplification, i.e. neglecting the effects of the transverse shear stress ($\sigma_{31} = 0$ for N–S vibrations and $\sigma_{32} = 0$ for E–W vibrations), is in line with the usual procedure of relaxing the mixed boundary conditions arising in this type of contact problem. The use of the static Green's functions is justified by the fact that the wavelengths involved are several hundred feet long whereas the linear dimensions of the foundation are less than 75 ft. This simplification implies that the displacements will be in phase or 180 degrees out of phase, a condition which is approximately satisfied by the observed displacements. Finally, the process of subdividing the foundation into sub-regions and assuming that the contact stresses are constant within each sub-region implies that only average values of the contact stresses within each sub-region will be obtained. These average values, however, are sufficient to obtain an estimate of the stress distribution on the contact between the foundation and the soil.

The system of linear equations (4) has been solved for the dimensionless stresses $\sigma_{i3}^{(k)}/\mu$ for excitation of the building in both the N–S and E–W directions. The values obtained for the normal stresses $\sigma_{33}^{(k)}$ and for the shear stresses $\sigma_{32}^{(k)}$ for N–S vibrations of the building are shown by double arrows in Figures 6(A) and 6(B), respectively. The normal stresses $\sigma_{33}^{(k)}$ and the shear stresses $\sigma_{31}^{(k)}$ generated by E–W vibrations of the building are shown by double arrows in Figures 7(A) and (B), respectively. For N–S vibrations of the building the larger contact stresses occur along the perimeter of the foundation, and, particularly at the corners as shown in Figure 6. Stresses with intermediate values may also be observed along the perimeter of the central core. For a shear modulus of the soil $\mu = 3{\cdot}3 \times 10^4$ psi, the maximum values of $\sigma_{33}^{(k)}$ and $\sigma_{32}^{(k)}$ are 5·0 and 1·2 psi, respectively. The larger normal stresses for E–W vibrations occur along the walls of the central core and at the salients located at the east and west ends of the foundation [Figure 7(A)]. The larger shear stresses generated by E–W vibrations of the building occur along the perimeter of the foundation as shown in Figure 7(B). The maximum values of $\sigma_{33}^{(k)}$ and $\sigma_{31}^{(k)}$ are in this case 3·3 and 0·7 psi, respectively, for a shear modulus $\mu = 3{\cdot}3 \times 10^4$ psi. It should be mentioned again that these 'dynamic' stresses do not include the static effects due to the weight of the building. The weight of the building distributed uniformly over the foundation area gives rise to a nominal static pressure of 35 psi. The value used for the shear modulus of the soil $\mu = 3{\cdot}3 \times 10^4$ psi is based on an estimated shear wave velocity of 1230 ft/sec and a unit weight of 100 lb/ft^3.

To estimate the degree by which the calculated stresses for a flexible foundation differ from those for a rigid foundation having the same shape and moving with the same average displacements and rotations it is only necessary to solve equation (4) with u_i given by

$$u_2 = \Delta_{N-S}, \quad u_3 = -\alpha_{N-S}\, x_2 \tag{6}$$

for N–S vibrations, and

$$u_1 = \Delta_{E-W}, \quad u_3 = \alpha_{E-W}\, x_1 \tag{7}$$

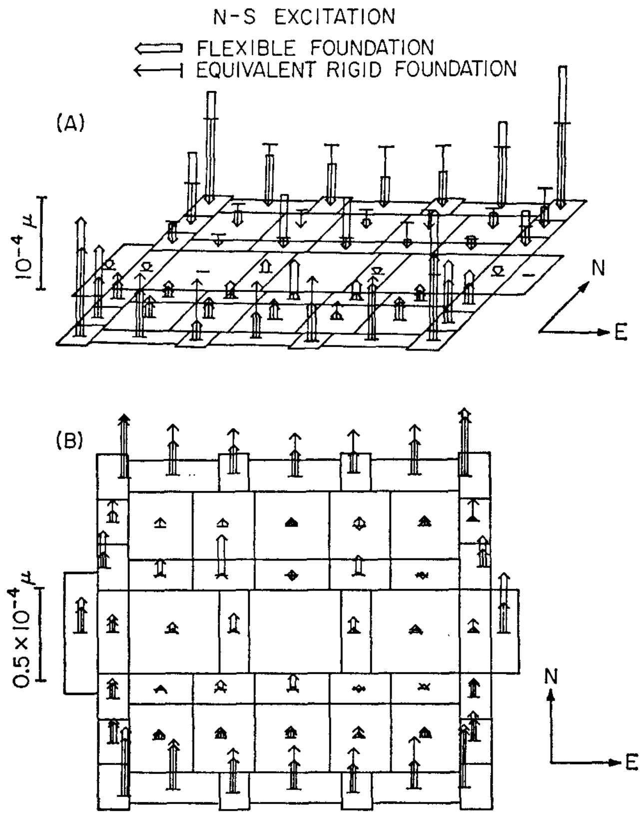

Figure 6. Contact stress distribution for N–S excitation

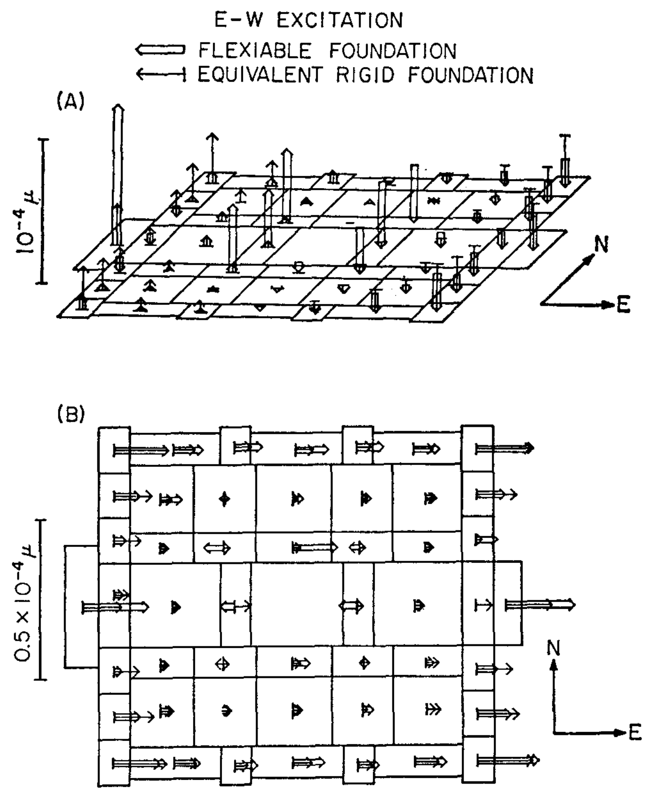

Figure 7. Contact stress distribution for E–W excitation

for E–W vibrations. In Equation (6), $\Delta_{N-S} = 3\times10^{-3}$ in and $\alpha_{N-S} = 1{\cdot}2\times10^{-5}$ rad are the average displacement and rotation of the flexible foundation for N–S excitation. Similarly, in equation (7), $\Delta_{E-W} = 1{\cdot}7\times10^{-3}$ in and $\alpha_{E-W} = 0{\cdot}45\times10^{-5}$ rad are the average displacement and rotation of the flexible foundation for E–W excitation.

The resulting contact stress distributions for the equivalent rigid foundation are also shown in Figures 6 and 7 by single arrows. It may be observed that the higher stresses for the equivalent rigid foundation are concentrated along the perimeter of the foundation, the values in the central part being much lower than those obtained for the flexible foundation [Figures 6(A) and 7(A)]. The results presented confirm the expectation that for a flexible foundation the higher contact stresses will occur in the neighbourhood of the stiffer structural elements while for a rigid foundation the higher stresses will concentrate along the perimeter irrespective of the location of the loads acting on the foundation. The flexibility of the foundation affects not only the contact stress distribution but also the relative deformation between major elements of the superstructure.

It is also of interest to evaluate the total forces and moments that the flexible foundation exerts on the soil and to compare these values with the corresponding results for the equivalent rigid foundation. Once the contact stresses are known the total force, H, and the total moment, M, acting on the soil are easily obtained by integration. The resulting values for the flexible and equivalent rigid foundations are listed in lines (1) and (2) of Table I, respectively. In computing these quantities a value for the shear modulus of the soil of $\mu = 3{\cdot}3\times10^4$ psi has been used. The forces and moments for the equivalent rigid foundation coincide with those for the flexible foundation except for the rocking moment for N–S vibrations that is 6 per cent larger for the rigid foundation. Thus, even though the contact stress distributions for the flexible and rigid models of the foundation are different the total forces and moments for both models are almost the same. This result indicates that the flexibility of the foundation has no major effect on the relationship between the total forces and moments acting on the foundation and its average motion.

Table I. Forces and moments acting on the soil

	N–S excitation (1.8 cps)		E–W excitation (1·3 cps)	
	H (lb) $\times 10^{-5}$	M (lb-ft) $\times 10^{-7}$	H (lb) $\times 10^{-5}$	M (lb-ft) $\times 10^{-7}$
(1) Flexible foundation ($\mu = 3{\cdot}3\times10^4$ psi)	2·08	1·63	1·22	0·69
(2) Rigid foundation ($\mu = 3{\cdot}3\times10^4$ psi)	2·08	1·73	1·22	0·69
(3) Flexible foundation + embedment effect	2·85	2·14	1·64	0·92
(4) Forces computed from superstructure	2·87	2·82	1·51	1·55

The results presented above are based on neglecting the effects of the embedment of the foundation. To obtain better estimates of the total forces and moments acting on the soil it is necessary to evaluate such effects. Based on Beredugo and Novak's[8] analysis of the force–displacement relationship for a rigid circular cylinder of radius a embedded to a depth h in an elastic half-space it is found that for low frequencies the horizontal force, $\hat{H}$, and the rocking moment, $\hat{M}$ (referred to the base of the cylinder), including the embedment effects may be obtained in terms of the force, H, and moment, M, for a surface foundation by means of the relationship (Poisson's ratio $\nu = \frac{1}{3}$)

$$\begin{Bmatrix} \hat{H} \\ \hat{M} \end{Bmatrix} = \begin{bmatrix} (1+0{\cdot}73\delta) & 0{\cdot}44\delta^2/a \\ 0{\cdot}37\delta^2 a & (1+0{\cdot}75\delta+0{\cdot}29\delta^3) \end{bmatrix} \begin{Bmatrix} H \\ M \end{Bmatrix} \tag{8}$$

where $\delta = h/a$ is the embedment ratio. Representing the foundation of the Millikan Library by an equivalent cylinder of radius $a = 40$ ft and effective embedment depth $h = 14$ ft leads to the forces and moments listed in line (3) of Table I. The effect of the embedment of the foundation corresponds to an increase of the total forces and moments of about 30 per cent. For convenience, the effective embedment depth was taken equal to the depth of the basement floor (14 ft).

The total force and moment that the foundation exerts on the soil may also be evaluated by considering the motion of the superstructure. Since the mode shapes, natural frequencies and mass distribution of the

Millikan Library are known,[1,2,4,9] the total force and moment acting on the soil may be easily computed leading to the results listed in line (4) of Table I. Comparison of the results presented in lines (3) and (4) of Table I indicates that for a value of the shear modulus of the soil $\mu = 3{\cdot}3 \times 10^4$ psi the horizontal forces computed from the motion of the foundation are in close agreement with those evaluated on the basis of the motion of the superstructure. Comparison between the corresponding moments however shows that the moments based on the motion of the foundation are somewhat lower than those obtained from the motion of the superstructure. Considering all the simplifying assumptions introduced the overall agreement may still be considered satisfactory.

NEAR FIELD SURFACE GROUND MOTION

The three-dimensional motion at 100 points located on the soil surface and in the immediate neighbourhood of the Millikan Library has been recorded for forced vibrations of the building at its resonant frequencies in both the N–S and E–W directions.[4,5] The field covered in this experiment extended approximately 400 ft from the library in both the east and west directions and 100 ft in the north and south directions as illustrated in Figure 8. In this figure, the foundation of the library corresponds to the rectangular region determined by points 30, 33, 69 and 66; the octagonal area marked by segmented lines corresponds to a one-storey structure adjacent to the library; and the rectangular area, also marked by segmented lines, corresponds to a shallow pond. The recordings at stations 1–10 and 91–100 were made along the arcades of surrounding buildings, also stations 30–33, 45–52 and 66–69 were located on the first floor of the Millikan Library. Most of the remaining stations were located on the soil surface. These measurements of the near field ground motion provide an excellent opportunity to test the methods available for the computation of the surface motion caused by soil–structure interaction. In addition, the data recorded allow the study of the effects of the embedment and flexibility of the foundation on surface ground motion.

Neglecting the effects that may be caused by the nearby buildings and those resulting from the embedment of the foundation, it is possible to analyse the displacement pattern on the soil surface by use of the same integral formulation employed to determine the contact stresses. Since the approximate distribution of contact stresses has been obtained, then equation (2) may be used directly to compute the quasi-static amplitudes of motion at the same locations where the ground motion was recorded.

The calculated values of the ground displacements generated by N–S and E–W vibrations of the building are compared in Figures 9 and 10 with the recorded values. In these figures, the x-axis is oriented E–W (positive to the east), while the y-axis is oriented N–S (positive to the north), both having for origin the centre of the foundation. In each of these figures the displacements are shown along E–W lines located at $y = 0$, 35, 48, 77 and 100 ft north of the E–W axis of symmetry of the foundation. Included in Figures 9 and 10 are the measured displacements together with curves representing the calculated values for both the flexible foundation (solid lines) and the equivalent rigid foundation (dotted lines). The notation used is such that 'FH' and 'FV' represent the horizontal and vertical displacement amplitudes for the flexible foundation, while 'RH' and 'RV' represent the horizontal and vertical displacement amplitudes for the equivalent rigid foundation. The displacement amplitudes shown in Figure 9 have been normalized by the amplitude of the N–S component of motion at station 18 in Figure 5; the recorded amplitude at that station was approximately 3×10^{-3} in. Similarly, the amplitudes shown in Figure 10 have been normalized by the amplitude of the E–W component of motion at station 25 of Figure 5; the recorded amplitude at that station was approximately $1{\cdot}7 \times 10^{-3}$ in.

The results presented in Figures 9 and 10 show that the calculated displacements for the flexible and rigid models of the foundation differ only for points located on the first floor of the building ($|x| \leqslant 37{\cdot}5$ ft, $|y| \leqslant 35$ ft). The flexibility of the foundation does not have any major effect on the calculated displacements for locations outside of this building. The computed and measured vertical components of motion follow the same trends and the agreement can be considered good in view of all the simplifying assumptions introduced. Comparisons of the computed and measured horizontal components of motion indicate some major differences. Both, for N–S and E–W excitations, the measured horizontal displacements at $y = 35$ ft,

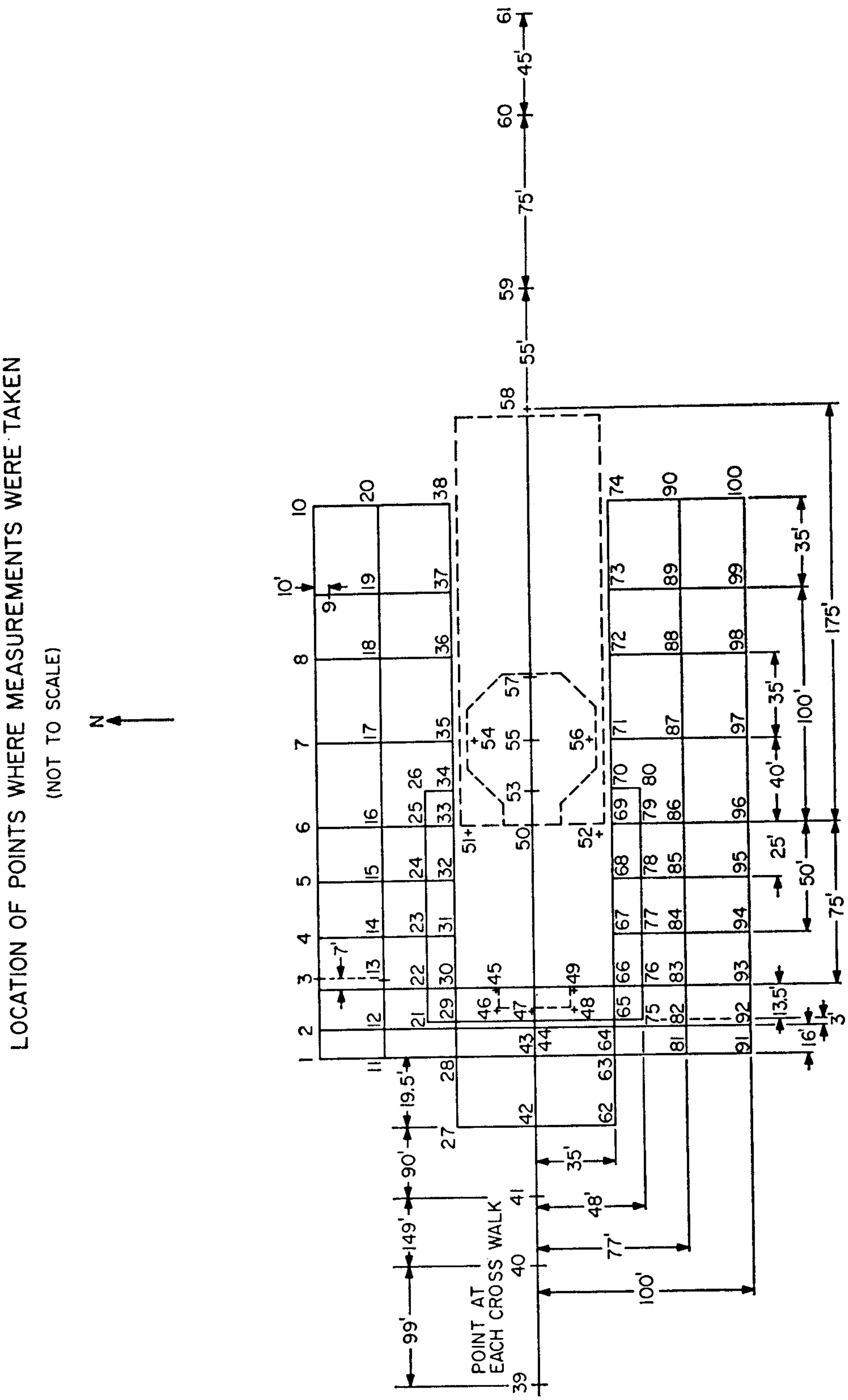

Figure 8. Location of points where near field displacements were measured

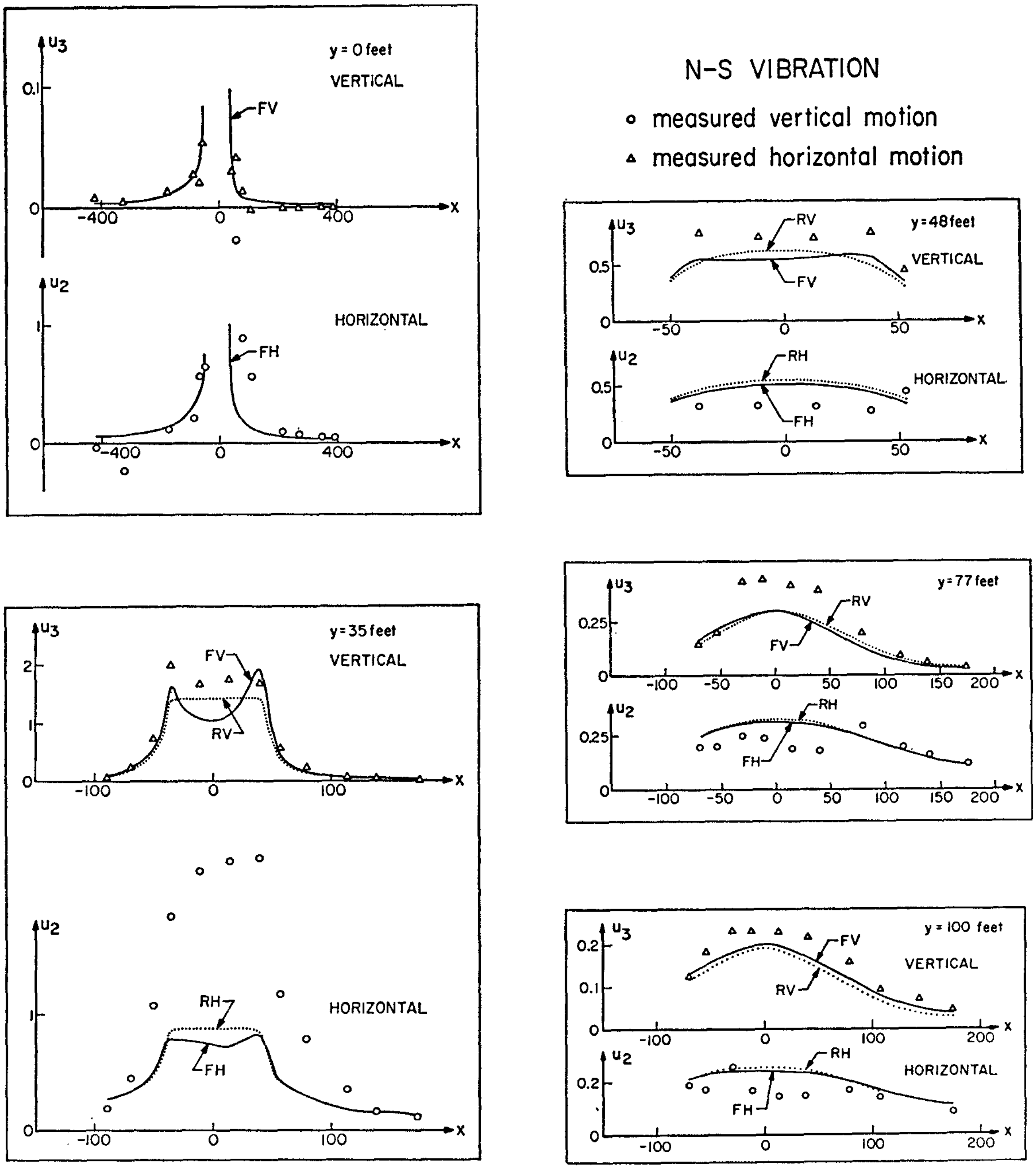

Figure 9. Ground displacements near the Millikan Library for N–S excitation

i.e. at the north end of the first floor of the building, are about two times larger than the calculated displacements. This difference is a direct result of neglecting the embedment of the foundation. Since the contact stresses were evaluated on the basis of the motion of the basement slab located 14 ft below the ground surface, then the calculated displacements within the building reflect the motion of the basement rather than that of the first floor. Due to rocking of the foundation and deformation of the basement walls the horizontal motion of the first floor is about twice the motion of the basement.

If the calculated horizontal displacements for E–W excitation are increased by 75 per cent to take into account the embedment of the foundation then a closer agreement with the observed data is obtained. The calculated displacements including this approximate correction for the embedment effect are shown in

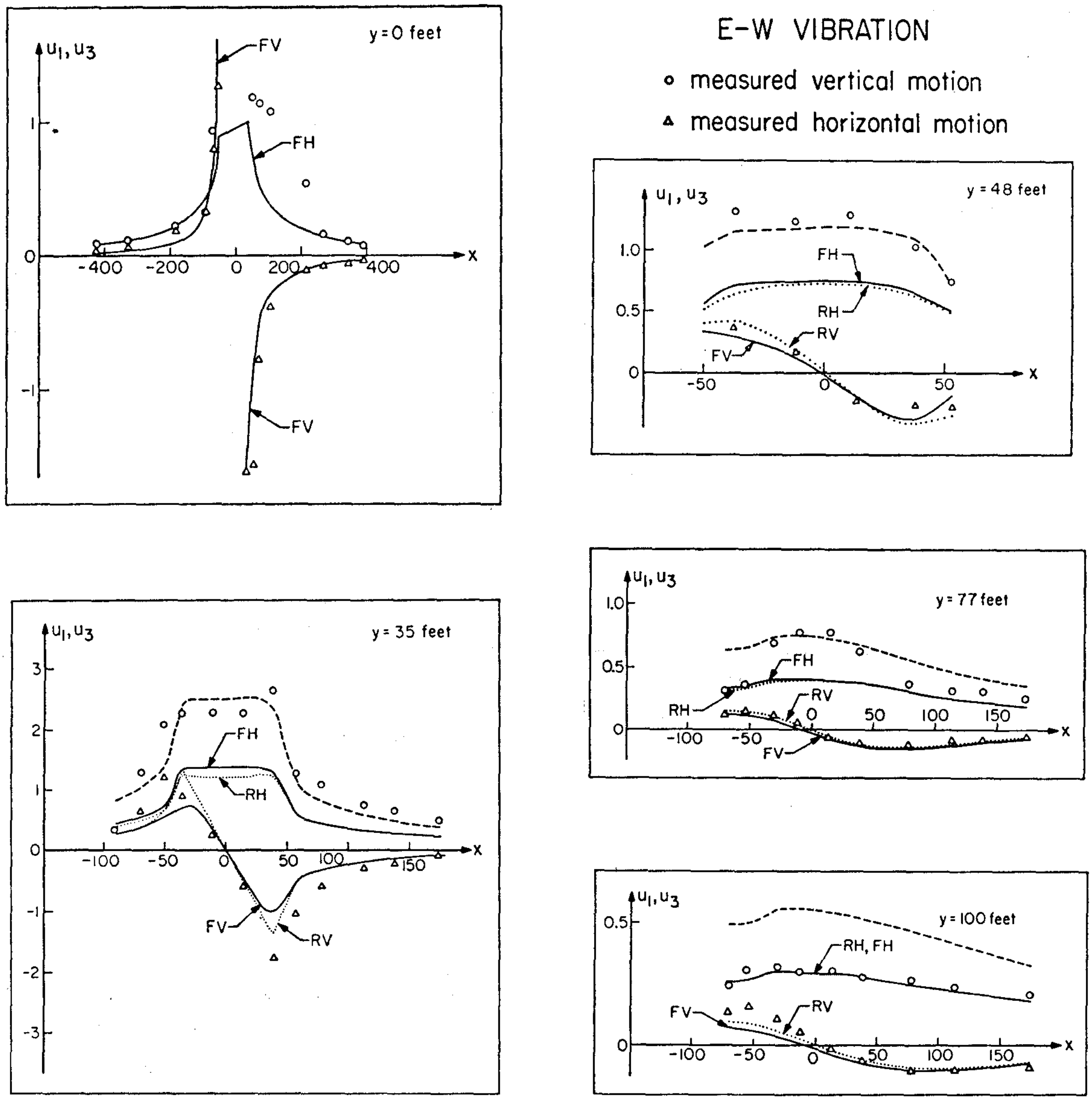

Figure 10. Ground displacements near the Millikan Library for E–W excitation

Figure 10 by segmented lines. In this figure it may be seen that the embedment correction is necessary to fit the observed horizontal displacements at locations near the building ($y = 48$ and 77 ft). However, for the line 100 ft north of the building the embedment correction does not seem to be necessary. This behaviour at $y = 100$ ft may also be a result of the presence of nearby buildings.

Although the embedment of the foundation seems to have an important effect on the horizontal motion generated by E–W vibrations of the building, the same effects are not observed for the case of N–S excitation as shown in Figure 9. For N–S excitation the calculated horizontal displacements of the soil surface away from the building (Figure 9, $y = 48$, 77 and 100 ft) coincide or are higher than the observed displacements. This difference in behaviour may be the result of unequal separation of the soil from the basement walls. Since rocking of the foundation is more pronounced for N–S vibrations it is possible that the soil in contact with the north and south basement walls might have experienced permanent deformation during past earthquakes leading to the absence of embedment effects for vibrations in the N–S direction. This might be in line with observed changes of the fundamental frequencies of vibration of the building after the San Fernando earthquake of 1971.

CONCLUSIONS

Estimates of the dynamic stresses that the Millikan Library Building exerts on the soil during forced vibration tests have been presented. The results obtained indicate that the higher contact stresses acting on the soil underneath the foundation concentrate in the neighbourhood of the stiffer elements of the superstructure. These results differ from those obtained on the basis of the usual assumption of a rigid foundation slab for which the higher stresses concentrate along the perimeter of the foundation. Even though the flexibility of the foundation has a major effect on the deformation and stress patterns at the soil–foundation contact, the relationship between the total forces acting at the contact and the average motion of the basement slab is practically independent of the flexibility of the foundation. This result implies that the rigid foundation slab assumption may be used in soil–structure interaction studies to obtain the overall motion of the superstructure. This assumption, however, would not reflect properly the relative deformation of structural elements close to the foundation level.

A simple method for evaluation of the near field ground motion caused by soil–structure interaction has been presented. Fair correlations between the computed and observed displacement fields in the immediate neighbourhood of the Millikan Library Building indicate that this approach could be used to obtain a first approximation of the interaction between adjacent buildings. Better estimates of the surface ground motion induced by soil–structure interaction will have to wait until the effects of the embedment of the foundation are properly accounted for.

ACKNOWLEDGEMENTS

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