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LETTERS TO THE EDITOR

ADEQUACY OF THE PARMELEE MODEL TO REPRESENT OPEN PLANE FRAMES ON ISOLATED FOOTINGS UNDER SEISMIC EXCITATION

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1. INTRODUCTION

There are occasions, such as multi-storey buildings founded on soft soil, when it becomes necessary to consider the effects of deformability of the foundation. These effects are generally referred to as soil–structure interaction.

The flexibility of the shallow foundation may best be represented by frequency-dependent impedance [1–3]. However, some studies [4–7] have shown that frequency-independent impedance is adequate to simulate the soil–structure interaction phenomenon. For this reason, several soil–structure interaction studies have represented the foundation reaction by the lumped parameter frequency-independent system [8–11]. Although the interactive system does not possess classical normal modes, satisfactory results have been obtained by assuming that the damping matrix satisfies modal orthogonality conditions [2, 7, 9]. The inertia effect of the soil beneath the foundation is invariably ignored in such studies owing to the complexities involved in estimating the effective mass of the soil [1].

While the dynamic analysis of framed structures on isolated footings can be done on the basis that the superstructure possesses continuously distributed properties, the analysis is complex and practicable only in the case of very simple structures. More conveniently, a finite element approach can be adopted in which framed structures are discretized into segments and the displacements of the interconnecting nodes constitute the generalized co-ordinates (or dynamic degrees of freedom) of the structure. The number of one-dimensional elements selected depends on the physical arrangement of the structure. Kinematic constraints are commonly adopted to reduce the degrees of freedom in order to save computational effort without significant loss of accuracy.

As an extreme, for framed structures it is usually assumed that the floor slabs have considerable in-plane rigidity and that the columns are in-extensible. The mass is assumed to be concentrated at the floors and to possess only translatory degrees of freedom. An $\mathbf{n}$ storey frame has only n degrees of freedom along its plane. When the foundation is shallow and flexible, translation and rotation of the footing are included. Thus, an $\mathbf{n}$ storey structure yields an $\mathbf{n}+2$-degree-of-freedom system with soil–structure interaction. This discretization may be called as the Parmelee model.

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The adoption of the Parmelee model for the analysis of tall framed structures with soil–structure interaction and subjected to horizontal seismic excitation is very common in practice and in the literature [1–3, 6–9, 12–14].

Studies using the Parmelee model to represent tall framed structures subjected to horizontal seismic excitation have indicated that soil–structure interaction results in a reduction of the natural frequencies of the structure [1, 6, 12]. The maximum response in terms of displacement of the top storey relative to the foundation largely depends on the frequency content of the excitation. Thus, foundation displacements do not influence the elastic forces developed in the inter-storey columns. On the other hand, if the physical arrangement of the superstructure is retained in the model adopted for the analysis under dynamic loads, the foundation displacements can influence the elastic forces developed in the inter-storey columns. Further, such a model is more consistent with the model used for analyzing the structure under static loads.

In this letter, we examine the appropriateness of representing open-plane frames on isolated footings and subjected to horizontal seismic excitation by the Parmelee model.

2. FORMULATION OF EQUATIONS OF MOTION FOR SOIL–STRUCTURE INTERACTION

In the Parmelee model of a multi-storey structure (Figure 1(a)), the mass is lumped at the floor levels and is assigned a translatory degree of freedom. For computing the inter-storey stiffness k_i all the j columns are added together, $k_i = 12E\sum_{m=1}^{j} I_m/h^3$, where h is the inter-storey height and I_m is the moment of inertia of the mth column.

For the frame-type discretization (Figure 1(b)), nodes can be located at column–beam junctions with each node being assigned two translatory (one horizontal and one vertical) degrees of freedom and a rotational degree of freedom. For simplicity, the portion of the structure surrounding the node can be treated as a rigid body and its mass and mass moment of inertia can be computed, so that a diagonal mass matrix is obtained for the entire structure. For obtaining the relevant stiffness matrix, the structural portion between nodes is treated as a uniform beam segment. While the element is axially extensible, shear distortion can be neglected and the element stiffness matrix can be obtained using cubic Hermetian polynomials to represent the shapes developed in the beam element subjected to nodal displacements.

For either representation of the entire structure, the dynamic reaction of the underlying soil can be approximated by frequency-independent springs k_{hs}, k_{vs} and $k_{\theta s}$, so that the forces applied to the footing are related to the footing displacements by

$$\begin{Bmatrix} P_h \\ P_v \\ P_\theta \end{Bmatrix} = \begin{bmatrix} k_{hs} & 0 & 0 \\ 0 & k_{vs} & 0 \\ 0 & 0 & k_{\theta s} \end{bmatrix} \begin{Bmatrix} v_b \\ v_v \\ \theta \end{Bmatrix}, \tag{1}$$

where P_b, P_v and P_θ are the interaction horizontal, vertical and rotational forces applied to the rigid foundation and v_b, v_v and θ are the corresponding displacements.

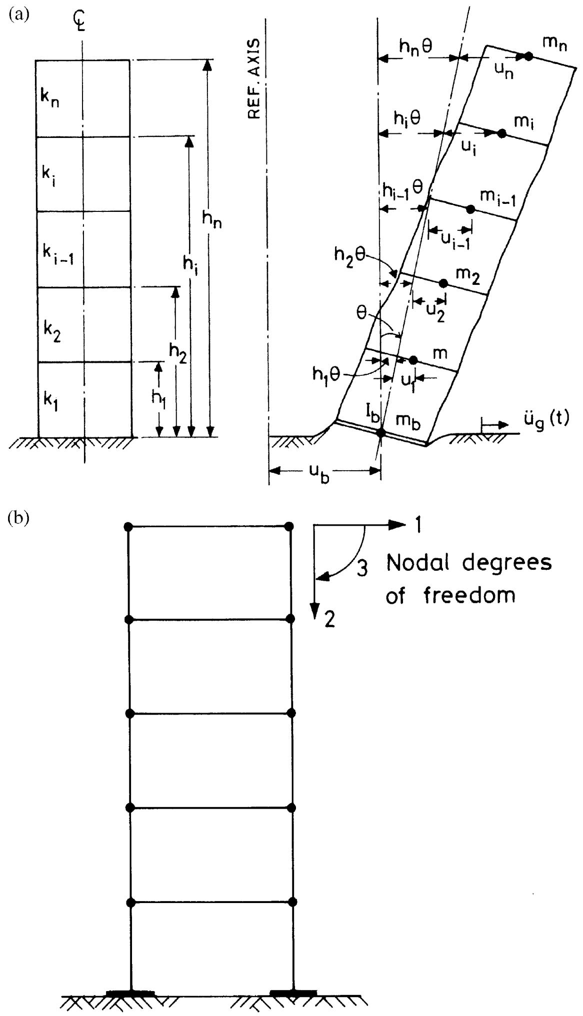


Figure 1. (a) Parmelee model idealization of a typical open-plane frame. (b) Frame model discretization.

In the manner of Parmelee *et al.* [5], Perelman *et al.* [4], and Rainer [6], it can be further assumed that there is no coupling between the foundation degrees of freedom. A further approximation is that energy dissipation in the elastic medium beneath the footing base can be accounted for by assuming a damping matrix proportionate to the stiffness and mass matrices of the total structure. This assumption permits the de-coupling of the equations of motion. While the stiffness coefficients in equation (1) are actually frequency dependent, they can be approximated by frequency-independent ones. In the current study those used by Pais and Kausel [15] have been used. These are suitable for circular and rectangular foundations.

When the structure subjected to horizontal ground motion is idealized by the Parmelee model, the foundation base is permitted only horizontal translation and rocking motion. The equations of motion for an **n** storey structure can be written as (referring to Figure 1(a))

$$\begin{bmatrix} [m] & \{m\} & \{mh\} \\ \{m\}^{\mathrm{T}} & m_b + \sum_1^n m_i & \sum_1^n m_i h_i \\ \{mh\}^{\mathrm{T}} & \sum_1^n m_i h_i & I + \sum_1^n m_i h_i^2 \end{bmatrix} \begin{Bmatrix} \ddot{u} \\ \ddot{u}_b \\ \ddot{\theta} \end{Bmatrix} + [C] \begin{Bmatrix} \dot{u} \\ \dot{u}_b \\ \dot{\theta} \end{Bmatrix} + \begin{bmatrix} [k] & \{0\} & \{0\} \\ \{0\}^{\mathrm{T}} & k_{hs} & 0 \\ \{0\}^{\mathrm{T}} & 0 & k_{\theta s} \end{bmatrix} \begin{Bmatrix} u \\ u_b \\ \theta \end{Bmatrix} = \{P\}, \tag{2a}$$

where I total mass moment of inertia $= \sum_1^n I_i + I_b$, $[m]$ = diagonal mass matrix listing the floor masses $m_1 m_2 \cdots m_n$, $\{m\} = [m_1\, m_2 \cdots m_n]^{\mathrm{T}}$, $\{mh\} = [m_1 h_1\, m_2 h_2 \cdots m_n h_n]^{\mathrm{T}}$, $\{u\} = [u_1\, u_2 \cdots u_n]^{\mathrm{T}}$, $[k]$ = stiffness matrix of the superstructure modelled as a shear building and k_{hs}, $k_{\theta s}$ are the equivalent soil springs corresponding to the horizontal translation and rocking motion of the foundation base. When the structure has more than one footing (as in the present study), a single degree of freedom each describes the horizontal and rocking motions. Thus, k_{hs} is the sum of all the p separate horizontal soil springs acting on the p footings, and likewise for $k_{\theta s}$.

$$\{P\} = -\left[\,\{m\}, \quad m_b + \textstyle\sum_1^n m_i, \quad \sum_1^n m_i h_i\right]^{\mathrm{T}} \ddot{u}_g(t).$$

For the frame model (Figure 1(b)), the corresponding equations of motion under horizontal seismic excitation permitting three degrees of freedom at each footing base are

$$\begin{bmatrix} [m] & 0 \\ 0 & [m_f] \end{bmatrix} \begin{Bmatrix} \ddot{u} \\ \ddot{u}_b \end{Bmatrix} + [C] \begin{Bmatrix} \dot{u} \\ \dot{u}_b \end{Bmatrix} + \begin{bmatrix} [k_s] & [k_{sf}] \\ [k_{fs}] & [k_f + k_{ss}] \end{bmatrix} \begin{Bmatrix} u \\ u_b \end{Bmatrix} = -\begin{bmatrix} [m] & [0] \\ [0] & [m_f] \end{bmatrix} \{r\} \ddot{u}_g(t), \tag{2b}$$

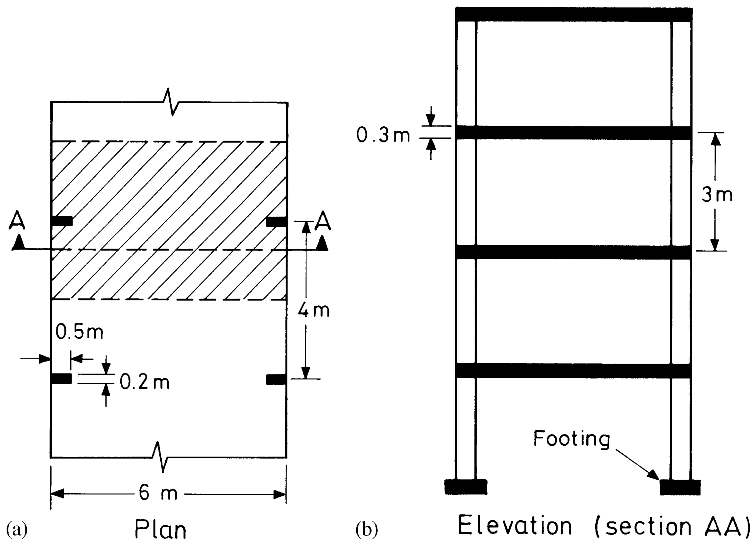
where $\{r\}$ is a vector of 1 and 0 s to account for the degrees of freedom influenced by the horizontal ground acceleration $\ddot{u}_g(t)$. The diagonal matrix $[m]$ contains the mass and mass moment of inertia associated with the superstructure degrees of freedom, and the diagonal matrix $[m_f]$ refers to the footing degrees of freedom. The matrix $[k_s]$ contains the stiffness elements associated with the superstructure and the matrix $[k_{fs}]$ refers to the degrees of freedom shared by the foundation and superstructure elements. The matrix $[k_f + k_s]$ which pertains to the foundation degrees of freedom incorporates the effect of the soil–structure interaction.

3. FRAMES ADOPTED AND RANGE OF SOIL PROPERTIES

A 1-bay 1-storey and a 1-bay 4-storey open-plane frame of flat slab construction were adopted for the study. Figure 2(a) and 2(b) show the plan and elevation of the 1-bay 4-storey frame. For both frames a bay span of 6 m and a uniform storey height of 3 m is considered. The slab is 0·3 m thick (Figure 2(b)) and the column dimensions are 0·2 m × 0·5 m (Figure 2(a)). The inter-frame spacing is 4 m. In the transverse section, the slabs with columns constitute a flexible frame as shown shaded in Figure 2(a).

The material properties of structural members used for the linear analysis of these frames are modulus of elasticity of the concrete, E_c = 22 GPa and the mass density of the concrete, ρ = 2400 kg/m^3.

To permit soil–structure interaction in the 1-storey and 4-storey frames adopted in the study, rigid bases of concrete of size 1·0 m × 0·5 m and 0·3 m thick were selected as



(a) (b)

Figure 2. Typical open-plane frame on isolated footings.

footings. For both non-interactive and interactive systems, a constant modal damping ratio of 5% was adopted.

To render the results of the interactive study realistic, the shear modulus of soil G_s was varied from 10 to 150 Mpa, so that the results are representative of medium to hard soils where isolated footings are used to support light to medium weight structures. A value of 0·3 was adopted for the Poisson ratio of the soil, μ_s.

4. DETAILS OF DYNAMIC LOADS SELECTED AND THE ANALYSIS

The first 30 s of the horizontal acceleration components of five earthquakes were used as seismic loading for evaluating the effect of soil–structure interaction over the range of G_s adopted. The details of these excitations are listed in Table 1. There is wide variation in their intensity as defined by Housner's response spectrum intensity (SI), evaluated for 5% critical damping ratio (ξ). The period of the damped simple oscillator which yields the maximum pseudo-spectral velocity for each excitation is also indicated in the table. The range of this period for these excitations fairly covers the fundamental period range of the structures adopted in this study.

The analysis consisted of determining the eigenvalues and eigenvectors for the undamped system using the Jacobi method. For each frame, the response of both the Parmelee model and the frame model representation to all the seismic excitations was obtained in the time domain (using modal analysis) by evaluating the Duhamel integral, and the results are reported in the form of peak shears in the lowest storey column. The damping ratio was constant for all the modes of vibration (5%).

5. RESULTS AND DISCUSSIONS

5.1. EIGENVALUES AND EIGENVECTORS

In Table 2(a) are indicated the six lowest frequencies of the 1-storey frame represented by the frame model over a range of soil shear modulus G_s values. Also included are the six

TABLE 1

Details of earthquakes selected

Description of earthquakes: Name	Symbol	Maximum acceleration: Value (m/s^2)	Maximum acceleration: Time (s)	Spectral pseudo-velocity, S_v (for $\xi = 5\%$): Max. S_v (m/s)	Spectral pseudo-velocity, S_v (for $\xi = 5\%$): Period (s)	Response spectrum intensity, SI (for $\xi = 5\%$), (m)
Uttarakashi 1991 (N15W)	Q1	−2·372	6·22	0·464	0·249	0·432
UK (Abhat) 1991 (N85E)	Q2	2·484	4·26	0·541	0·887	0·697
Eurake 1954 (N46W)	Q3	1·973	7·10	0·697	1·413	1·038
El Centro 1940 (S90W)	Q4	2·101	11·44	0·724	2·067	1·119
El Centro 1940 (S00E)	Q5	3·417	2·12	0·802	0·990	1·356

natural frequencies, when soil–structure interaction is not permitted. The natural frequencies for the interactive frame decrease as the shear modulus of soil G_s decreases, but the effect is more pro-nounced for the fundamental frequency. All the natural frequencies of the Parmelee model are indicated in Table 2(b) which also includes the natural frequency of the shear building when soil–structure interaction is not permitted. As reported in the literature, soil–structure interaction results in reduction of natural frequencies with reduction of G_s. More importantly, for all G_s, the fundamental frequency of the Parmelee model is significantly less than that for the frame model. For low G_s, the fundamental frequency for the Parmelee model is just 1/3 of that yielded by the corresponding frame model. It has been earlier observed [16] that a shear building yields a fundamental frequency 10–20% larger than that of the corresponding frame model. In the current study, the shear building yields a natural frequency 13·3% larger than the fundamental frequency of the frame model. The floor slab to column stiffness ratio is 2·16. The span to thickness ratio of the floor slab of 30 is considered adequate for deflection control in reinforced concrete members as per CP 100 [17]. For hypothetical floor slab to column stiffness ratios of 6·75 and 8·0 achieved without changing the mass of the structure, the fundamental frequency of the frame model was 43·294 and 45·902 rad/s, respectively, so that the natural frequency of the shear building is correspondingly only 9·8 and 3·63% higher. Very stiff floors are thus needed if a shear building model is to represent an open-plane frame. Such large stiffness ratios in practice cannot be attained without increasing the overall mass of the structure and affecting the storey height.

When soil–structure interaction is permitted, increases in the floor slab to column stiffness ratio results in a widening of the difference between the fundamental frequencies of the frame model and the Parmelee model. Thus, for $G_s = 10$ MPa, the fundamental frequency of the frame model is 19·78 and 20·29 rad/s for slab–column stiffness ratios of 6·75 and 8·0 respectively.

For the 4-storey structure, the six lowest undamped frequencies of the frame model are indicated in Table 3(a). The table also includes the six lowest frequencies yielded by the non-interactive frame. All the natural frequencies of the corresponding Parmelee model are listed in Table 3(b) along with the four natural frequencies of the non-interactive shear building. For this structure also, the Parmelee model yields a much lower fundamental

TABLE 2

Natural frequencies (rad/s) of 1-bay 1-storey frame

	Shear modulus of soil, G_s (MPa)					
Mode	10	30	50	90	150	Non-interactive
(a) *Frame model*						
1	19·311	24·526	27·256	30·593	33·417	41·976
2	47·487	64·477	65·313	66·219	66·974	69·541
3	49·337	71·782	82·497	90·452	94·223	100·595
4	62·016	83·165	104·618	133·710	161·649	285·500
5	106·546	115·639	126·107	147·143	171·419	290·128
6	178·953	286·376	362·449	475·983	596·012	989·972
(b) *Parmelee model*						
1	6·36	10·87	13·84	18·08	22·46	47·57
2	65·19	84·29	90·88	97·50	103·65	—
3	245·31	333·14	404·34	519·20	655·26	—

frequency than the corresponding frame discretization for any G_s (nearly 1/5 the value for $G_s = 10$ MPa). The frequencies for higher modes obtained with the Parmelee model are larger than those yielded by the frame model. The sharp reduction in the fundamental frequency is perhaps due to the mass and mass moment of inertia corresponding to the foundation degrees of freedom being enhanced by the values for the different floors (equation 1(a)).

In Figure 3(a) are shown the first two modes of the frame model of the single-storey structure in the absence of soil–structure interaction. Mode 1 is primarily a sway mechanism, while in mode 2 counter rotation of the beam–column joints dominate over all the other displacements. Examination of the other four modes showed that the third mode is similar to mode 1 except that the joint rotations are more pronounced. The fourth mode is essentially of column axial deformation type with both columns undergoing elongation or shortening. In the fifth mode one column undergoes extension while the other shortens. In the sixth mode, the beam undergoes axial deformation. In Figure 3(b) are shown the first two modes, when soil–structure interaction is permitted for two values of G_s. These mode shapes resemble those for the structure without soil–structure interaction except for the modification due to footing displacements. In both modes, it is seen that footing displacements and rotations tend to relieve the elastic forces developed in the columns due to distortion of the frame. In both modes, decrease in G_s is found to result in greater footing displacements and rotations.

In Figure 3(c) are shown the first two modes of the Parmelee model of the same structure for two values of G_s along with the solitary mode of the non-interactive structure (shear building). For the Parmelee model, footing rotation dominates in the first mode. Decrease in G_s results in footing translation of similar magnitude as that for the storey mass. In the second mode, the footing rotation opposes the displacement of the storey mass. In this mode, a decrease in G_s results in footing translation of the same order as the storey mass. For both modes, while the footing displacement and rotation influences the storey displacement, these displacements do not affect the elastic force developed in the storey column as this depends on the relative storey displacement.

Inspection of the eigenvectors of the 4-storey structure modelled as a frame without soil–structure interaction indicated that the first and second mode shapes resemble the

TABLE 3

Natural frequencies (rad/s) of 1-bay 4-storey frame

Mode	Shear modulus of soil, G_s (MPa) 10	30	50	90	150	Non-interactive
(a) *Frame model*						
1	6·35	8·43	9·31	10·20	10·82	12·22
2	22·00	27·42	29·58	31·62	33·01	36·21
3	24·65	40·43	49·65	55·99	56·91	59·79
4	43·56	50·59	54·40	60·98	65·17	65·19
5	54·05	55·49	58·05	64·95	70·47	75·41
6	65·10	65·14	65·15	65·16	70·89	79·35
(b) *Parmelee model*						
1	1·31	2·25	2·89	3·83	4·86	16·34
2	33·77	40·18	41·85	43·09	43·79	46·96
3	68·08	69·16	69·49	69·778	70·01	71·74
4	79·70	82·75	83·99	85·14	85·93	87·77
5	90·12	90·44	90·74	91·36	92·34	—
6	239·07	330·84	403·09	518·63	654·98	—

modes shapes of a cantilever. With soil–structure interaction, these two mode shapes are modified by footing displacements and rotations which tend to attenuate the elastic forces produced in the first floor columns by the deformation of the frame. Increase in G_s value was seen to result in reduction of the displacements and rotations of the footings. Inspection of the eigenvectors for the Parmelee model of the 4-storey structure showed that footing displacement and rotation had the same effect as seen for the single-storey structure.

5.2. RESPONSE TO SEISMIC EXCITATION

Table 4(a) presents the maximum absolute shear and the instant of seismic excitation at which it occurs in the column of the single-storey structure when it is modelled as a frame. The results for the non-interactive case are also included. It is seen that the response of both the non-interactive system and the system with soil–structure interaction is governed by the first mode of vibration (not less than 97·5% of the total response is contributed by the first mode). It is also seen that the response is governed by the frequency content of the seismic excitation used since there is no clear trend with respect to G_s for any of the five seismic excitations used. When the structure is represented by the Parmelee model, it is seen that the first mode of vibration is not always the dominant mode (Table 4(b)). It may contribute as little as 69% of the total response. When the second mode is also considered, nearly 99% of the total response is obtained. The results yielded by the two models do not concur over the entire G_s range as well as for any of the five seismic excitations used.

In the absence of soil–structure interaction, for the single-storey frame model, it was found that for excitation Q5 the maximum column shear was 51·04 and 66·55 kN when the floor slab to column stiffness ratio was increased to 6·75 and 8·0 respectively. The shear building model and the frame model can thus yield comparable responses to horizontal seismic excitation at uneconomically high floor slab to column stiffness ratios.

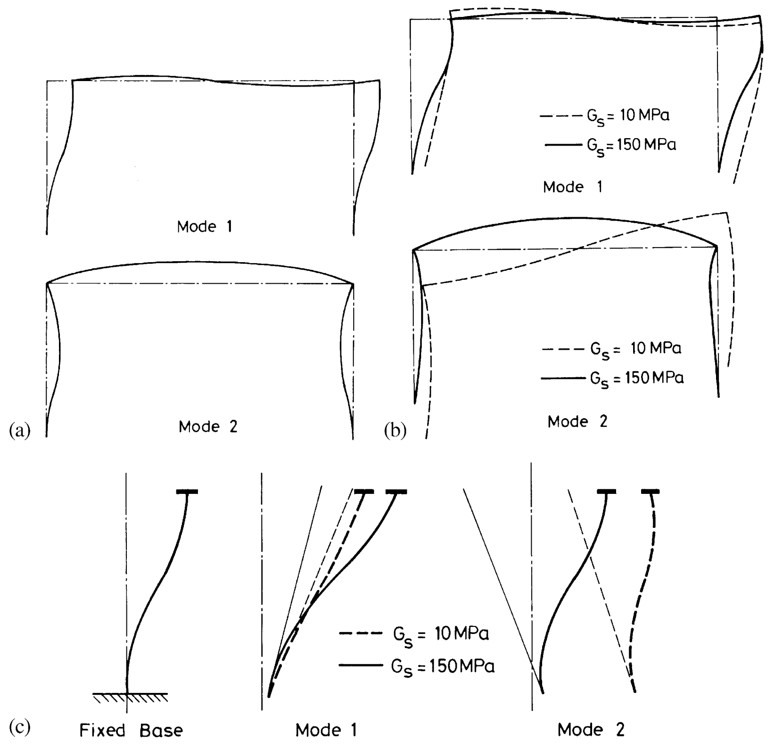


Figure 3. (a) Two lowest mode shapes of non-interactive single-storey frame model. (b) Two lowest mode shapes of single-storey frame model with soil-structure interaction. (c) Two lowest mode shapes of single-storey parmelee model.

When soil–structure interaction is permitted, increasing the floor slab to column stiffness ratio does not result in the maximum column shear values yielded by the frame model and the Parmelee model concurring. Maximum column shear values of 62·75 and 62·57 kN are obtained for stiffness ratios of 6·75 and 8·0 respectively, for excitation Q5 and $G_s = 10$ MPa. The Parmelee model yields a maximum column shear of 38·28 kN.

For the 4-storey structure, it is seen that the first mode of the frame model yields more than 93% of the total response (Table 5(a)) regardless of the phenomenon of soil–structure interaction. As in the case of the 1-storey structure, there is no relationship between the maximum shear in the first storey column and G_s for any seismic excitation. As seen for the 1-storey Parmelee model, for the 4-storey structure also the first mode of excitation generally does not contribute the bulk of the response (Table 5(b)). Of the six possible modes of vibration, the first two contribute over 98% of the maximum response. The maximum response yielded by the frame model (with or without soil–structure interaction) rarely agrees with that yielded by the Parmelee model. Further, for both the 1-storey and for the 4-storey structures it is seen that the first mode of the frame model contributes over 95% of the response. This makes it especially convenient to use design spectra for the structure with soil–structure interaction when the frame model is adopted. For the Parmelee model of a multi-storey structure, at least two modes are needed for the same purpose. The accuracy in this case is affected, since for both the modes, the peak response occurs at a different moment of excitation.

TABLE 4

Maximum absolute shear (kN) at first floor of 1-storey frame

Earthquake type	No· of modes	Soil shear modulus, G_s (MPa) 10	50	90	150	Non-interactive case
(a) *Frame model*						
Q1	1	55·85 (4·74)†	87·54 (6·38)	68·10 (6·10)	59·32 (4·06)	76·64 (4·02)
	4	55·13 (4·74)	87·73 (6·38)	68·50 (6·10)	59·32 (4·06)	77·38 (4·02)
	6	55·11 (4·74)	87·74 (6·38)	68·50 (6·10)	59·32 (4·06)	77·38 (4·02)
Q2	1	39·47 (7·32)	50·29 (4·32)	46·84 (7·56)	60·18 (4·30)	46·92 (4·26)
	4	38·59 (7·32)	49·89 (4·32)	46·81 (7·56)	60·01 (4·30)	48·48 (4·26)
	6	38·57 (7·32)	49·91 (4·32)	46·81 (7·56)	60·01 (4·30)	48·48 (4·26)
Q3	1	24·58 (7·14)	25·02 (8·62)	30·36 (7·02)	32·21 (7·16)	32·24 (7·12)
	4	25·07 (7·12)	25·15 (8·62)	30·61 (7·02)	32·69 (7·16)	33·11 (7·12)
	6	25·14 (7·12)	25·15 (8·62)	30·61 (7·02)	32·70 (7·16)	33·11 (7·12)
Q4	1	38·13 (4·64)	46·44 (2·94)	48·50 (2·90)	41·23 (4·60)	31·09 (4·18)
	4	38·15 (4·64)	46·58 (2·94)	48·77 (2·90)	41·78 (4·60)	31·17 (4·18)
	6	38·20 (4·64)	46·59 (2·94)	48·78 (2·90)	41·78 (4·60)	31·17 (4·18)
Q5	1	57·92 (2·64)	65·23 (2·52)	57·27 (2·50)	56·77 (2·26)	45·97 (3·54)
	4	61·18 (2·64)	65·57 (2·52)	57·08 (2·50)	57·36 (2·26)	47·27 (3·54)
	6	61·28 (2·64)	65·55 (2·52)	57·09 (2·50)	57·36 (2·26)	47·27 (3·54)
(b) *Parmelee model*						
Q1	1	7·50 (4·68)†	36·31 (5·84)	46·73 (4·76)	70·39 (6·44)	57·23 (3·92)
	2	12·31 (6·22)	33·42 (5·84)	45·90 (4·94)	69·88 (6·44)	—
	3	12·20 (6·22)	33·29 (5·84)	46·02 (4·94)	69·79 (6·44)	—
Q2	1	20·17 (6·50)	34·33 (4·60)	28·22(4·34)	65·62 (7·26)	44·33 (4·24)
	2	19·93 (6·58)	34·79 (4·60)	31·00 (4·34)	67·28 (7·26)	—
	3	19·99 (6·58)	34·76 (4·60)	31·26 (4·34)	66·79 (7·26)	—
Q3	1	26·25 (7·34)	19·99 (7·06)	22·12 (7·18)	15·93 (7·88)	29·79 (7·10)
	2	26·24 (7·36)	23·27 (7·06)	24·76 (7·18)	16·99 (7·08)	—
	3	26·25 (7·36)	23·44 (7·06)	25·23 (7·18)	16·49 (7·88)	—
Q4	1	18·40 (12·22)	40·39 (2·04)	29·50 (25·06)	30·86 (4·38)	29·56 (1·92)
	2	19·95 (12·20)	43·47 (2·04)	32·55 (11·46)	31·87 (4·38)	—
	3	19·92 (12·20)	43·60 (2·04)	33·13 (11·46)	31·51 (4·38)	—
Q5	1	35·39 (4·38)	59·10 (5·06)	44·74 (2·66)	56·25 (2·58)	65·95 (2·62)
	2	38·35 (4·34)	61·52 (5·06)	50·12 (2·66)	61·50 (2·58)	—
	3	38·28 (4·34)	61·62 (5·06)	50·50 (2·66)	61·27 (2·58)	—

†The excitation instant at which maximum value of shear occurs is given in brackets.

6. CONCLUSIONS

(1) The Parmelee model yields a fundamental frequency much lower than that yielded by the frame model.

(2) Soil–structure interaction results in lowering of the fundamental frequency with G_s decrease for both the models. The footing displacements and rotations brought about by soil–structure interaction in the frame model can attenuate the shear forces developed in the columns. On the other hand, these movements do not directly influence the shear developed in the columns of the Parmelee model.

(3) The lack of agreement between the two models in terms of fundamental frequency and maximum response indicates that caution needs to be exercised when discretizing an open-plane frame for seismic load analysis, when soil–structure interaction is to be

TABLE 5

Maximum absolute shear (kN) at first floor of 4-storey frame

Earthquake type	No· of modes	Soil shear modulus, G_s (MPa)				Non-interactive case
		10	50	90	150	
(a) *Frame model*						
Q1	1	34·80 (4·68)†	89·20 (6·26)	107·39 (6·22)	96·05 (6·74)	106·62 (6·70)
	2	67·97 (6·72)	98·05 (5·98)	92·38 (5·96)	95·15 (6·74)	114·60 (6·70)
	4	67·97 (6·72)	98·15 (5·98)	92·44 (5·96)	94·43 (6·74)	115·09 (6·70)
	6	67·77 (6·72)	98·44 (5·98)	92·61 (5·96)	94·07 (6·74)	115·04 (6·70)
Q2	1	93·39 (6·50)	130·76 (4·84)	159·79 (6·62)	167·48 (5·88)	177·37 (5·76)
	2	109·48 (7·54)	126·33 (4·82)	153·38 (4·76)	179·47 (6·20)	181·87 (5·76)
	4	109·47 (7·54)	126·66 (4·82)	152·54 (4·76)	179·04 (6·20)	182·47 (5·76)
	6	109·49 (7·54)	127·00 (4·82)	152·45 (4·76)	179·10 (5·90)	181·58 (5·76)
Q3	1	122·01 (7·34)	121·72 (7·22)	119·77 (8·10)	131·74 (8·06)	110·11 (8·78)
	2	118·43 (7·86)	129·81 (7·22)	128·84 (7·20)	132·32 (7·18)	121·25 (7·14)
	4	118·43 (7·86)	130·06 (7·22)	129·58 (7·20)	133·39 (7·18)	123·22 (7·14)
	6	118·59 (7·86)	130·32 (7·22)	129·75 (7·20)	133·60 (7·18)	123·91 (7·14)
Q4	1	85·70 (12·22)	132·69 (12·00)	170·81 (11·96)	186·23 (11·90)	196·11 (2·10)
	2	85·50 (12·26)	145·84 (12·00)	186·86 (11·96)	190·93 (11·92)	202·55 (2·10)
	4	85·50 (12·26)	146·14 (12·00)	187·49 (11·96)	191·73 (11·92)	203·71 (2·10)
	6	85·53 (12·26)	146·37 (12·00)	187·63 (11·96)	192·00 (11·92)	203·99 (2·10)
Q5	1	164·59 (4·38)	220·38 (2·26)	265·38 (2·22)	284·64 (2·20)	265·10 (2·16)
	2	174·20 (4·36)	208·27 (2·24)	261·01 (2·22)	289·78 (2·20)	283·07 (2·16)
	4	174·21 (4·36)	207·41 (2·24)	259·78 (2·20)	288·49 (2·20)	286·62 (2·16)
	6	174·28 (4·36)	206·56 (2·24)	259·45 (2·20)	287·94 (2·20)	287·74 (2·16)
(b) *Parmelee model*						
Q1	1	1·68 (15·68)†	7·27 (5·92)	10·78 (5·92)	33·15 (5·76)	179·50 (5·80)
	2	53·39 (04·06)	58·05 (4·02)	53·72 (3·94)	60·35 (3·92)	187·57 (5·78)
	4	53·26 (04·06)	59·39 (4·02)	54·19 (3·94)	61·35 (3·92)	188·72 (5·78)
Q2	1	4·46 (8·12)	21·22 (8·10)	27·50 (5·66)	40·13 (9·18)	119·83 (4·56)
	2	52·34 (4·28)	42·93 (5·78)	46·00 (5·78)	48·91 (8·48)	122·16 (5·70)
	4	52·92 (4·28)	43·73 (5·78)	46·79 (5·78)	49·24 (8·48)	122·71 (5·70)
Q3	1	5·02 (19·54)	28·27 (8·70)	68·47 (9·20)	96·83 (8·14)	96·48 (6·72)
	2	25·93 (07·16)	26·67 (7·70)	67·75 (9·14)	102·40 (8·18)	100·41 (6·72)
	4	26·32 (07·16)	26·35 (7·70)	67·65 (9·14)	102·65 (8·18)	101·49 (6·72)
Q4	1	19·29 (4·56)	57·08 (11·86)	44·35 (4·22)	86·48 (12·40)	162·46 (11·50)
	2	47·69 (4·32)	69·31 (11·82)	64·92 (4·18)	88·45 (12·42)	167·38 (11·50)
	4	47·97 (4·32)	69·67 (11·82)	66·26 (4·18)	88·12 (12·42)	168·63 (11·50)
Q5	1	8·74 (28·60)	54·27 (5·56)	56·00 (6·18)	71·87 (6·04)	210·00 (4·78)
	2	47·79 (2·36)	75·51 (5·36)	68·71 (9·54)	82·03 (3·08)	212·60 (4·78)
	4	48·04 (2·36)	75·63 (5·52)	69·84 (9·54)	81·54 (3·08)	214·13 (4·78)

† The excitation instant at which maximum value of shear occurs is given in brackets.

considered. A frame model is perhaps more suited, since it is also the choice for the static load analysis of such structures.

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