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Effects of soil-structure interaction on the seismic performance of a concrete frame-wall structure

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ABSTRACT

The effects of the soil and of the soil-structure interaction on the seismic performance of a six-storey reinforced concrete office building is studied. The frame-wall structural system is designed according to the latest Canadian codes to possess nominal ductility. Three cases are studied corresponding to three different thicknesses of the underlying soil. The finite element soil-structure system is first reduced by means of derived Ritz vectors. Step-by-step analyses of the reduced linear system are then performed to obtain horizontal and vertical acceleration responses at the soil-structure interface. These modified motions are then used as input motions for the non-linear analyses of the frame-wall structure alone. The performance of the structure is then evaluated in terms of lateral displacement, drift index and base shear.

INTRODUCTION

Soil effects and soil-structure interaction are important parameters on the behaviour of building structures under seismic loading. The true response of the structure can only be obtained if the interaction of the vibrating structure and the underlying soil is considered and properly modelled. There is a clear distinction between site effects and soil-structure interaction. Site effects are the modification of the motion at the free surface after propagation through soil layers. Soil structure interaction depends not only on the free surface motion but on the mechanical properties of the structure and the underlying soil. Seed (1975) has shown that a quasi-resonance between the structure and the soil layer can increase significantly the seismic forces acting on the structure and therefore their damage potential. Seed (1975), Balendra and Heidebrecht (1987), among others, suggested the use of a period-dependent foundation factor to account for these effects.

The Uniform Building Code (UBC 1988) includes the soil effect in the minimum lateral load through the factor S, which depends on the height and the type of soil only. In the 1982 edition (UBC 1882) however, the factor S was also a function of the ratio of the period of the structure to the period of the soil layer.

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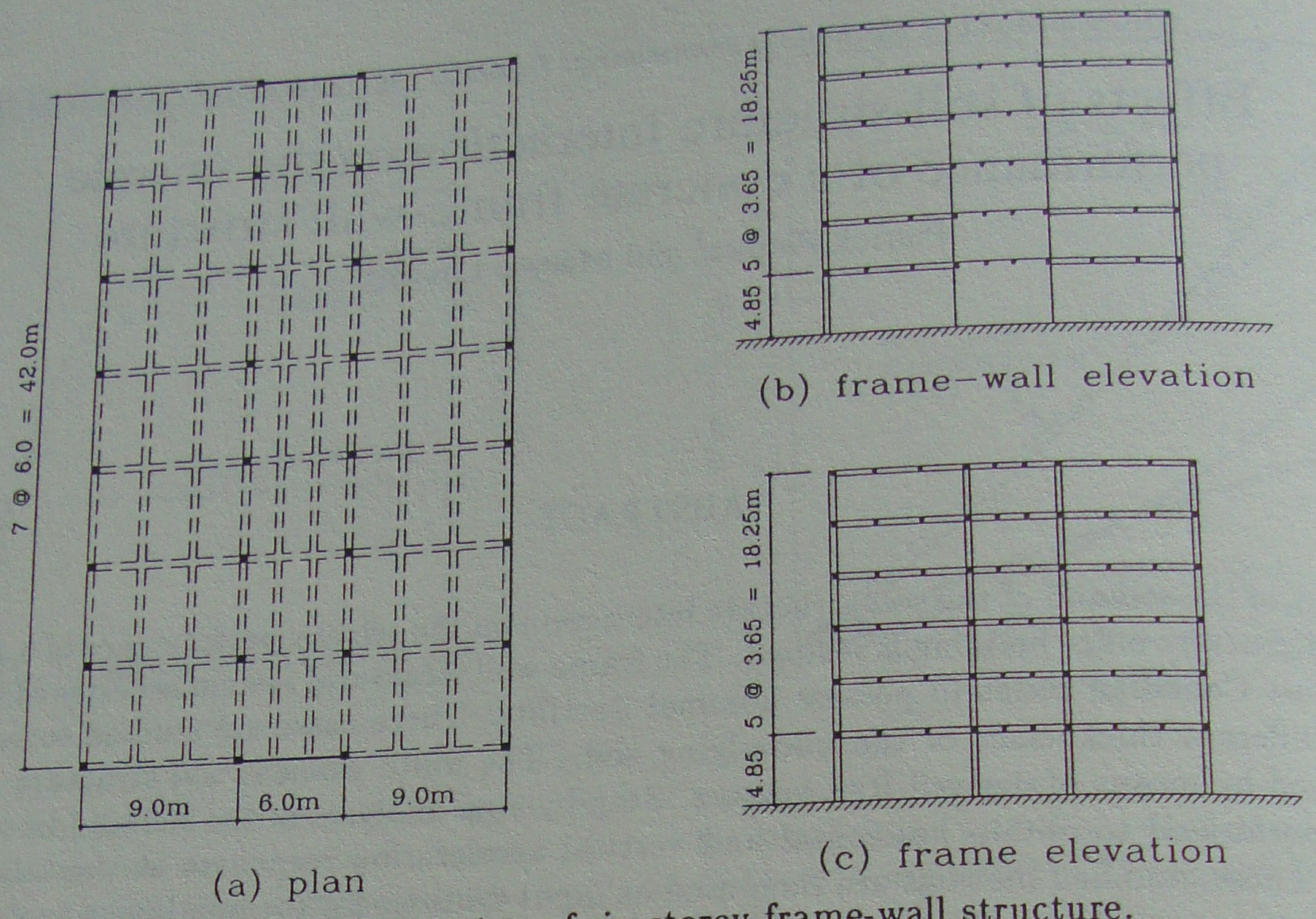

Figure 1. Plan view of six-storey frame-wall structure.

The NEHRP (NEHRP 1986) accounts for the height and the type of soil in the equation of the minimum lateral load. Supplementary equations are given to account for soil-structure interaction.

The National Building Code of Canada (NBC 1990) includes a foundation factor in the equation of the minimum lateral load. This factor, F, depends on the thickness and type of soil. No particular procedure is recommended to account for soil-structure interaction.

DESCRIPTION OF STRUCTURE AND SUPPORTING SOIL

In order to study the adequacy of the F factor in the National Building Code of Canada, a six-storey office building was designed for a Montréal area. The plan view of the building is shown in Fig. 1. The reinforced concrete frame-wall building has 7 - 6 m bays in the longitudinal (N-S) direction and 3 bays in the transverse (E-W) direction, consisting of a central corridor bay and 2 - 9 m external office bays. The storey height is 4.85 m for the ground level and 3.65 m for all the other levels. The structural system is made of 6 internal moment-resisting frames and two external bents consisting of two perimeter frames attached to a central wall. The design is according to the 1990 NBC and the 1984 CSA Standard for the design of concrete structures for buildings with $R = 2.0$. This implies that the walls possess nominal ductility and take 100% of the lateral load and the frames carry only gravity loads. It is assumed that the central roof bay supports machinery. The specified yield stress for the steel is 400 MPa and the concrete strength is 30 MPa. The calculated natural period of the building is 1.1 s.

The site has been chosen from boring data on the island of Montréal to correspond to a foundation factor $F = 1.3$. The soil medium is made up of granular material of average density.

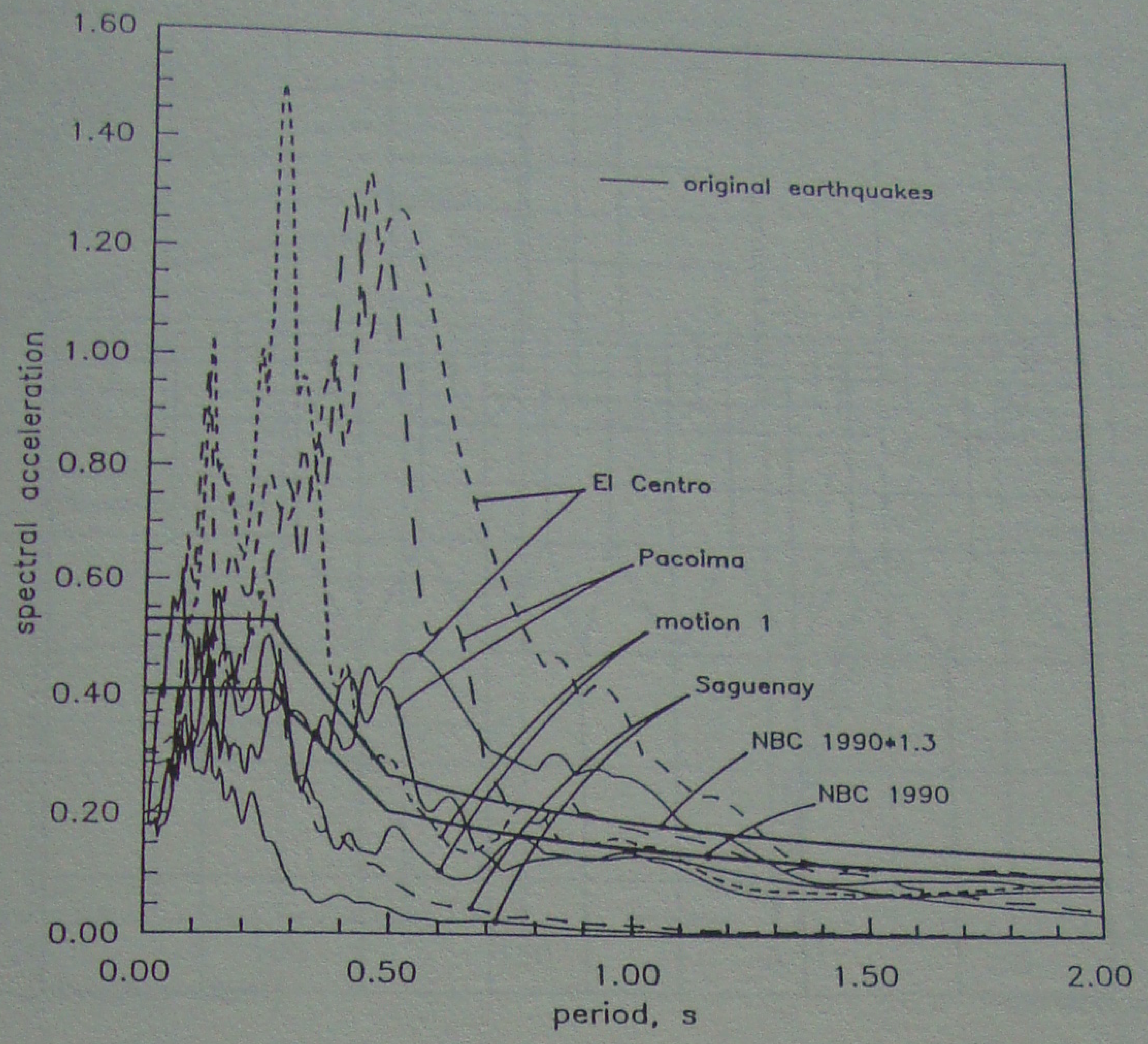

Figure 2. Response spectra of earthquake motions used.

The properties of the soil are derived after equations proposed by Seed et al. (1986). The initial values for the shear modulus, the mass density and the Poisson ratio, are 128 MPa, 2080 kg/m^3 and 0.35, respectively. Three different thicknesses of soil are studied to appreciate the site effect and the effects of soil-structure interaction on the nonlinear response of the building.

EARTHQUAKE MOTIONS

The use of several accelerograms is necessary to assess adequately the nonlinear performance of structures under seismic loading. In this study a set of four earthquakes is used, consisting of three historic recordings and a motion artificially generated with SIMQKE (1976). The artificially-generated accelerogram has a spectrum compatible with the spectra given in the Commentary of NBC (1980) with the velocity bound adjusted by multiplying by the ratio of the peak horizontal ground velocity, v, over the peak horizontal ground acceleration, a. The characteristics of the four earthquakes are presented in Table 1. Figure 2 compares the response spectra for the four motions with the NBC 1990 equivalent spectral acceleration.

Table 1. Earthquakes selected

Earthquakes	Date	Location	Component	PGA, g	PGV, m/s	a/v
Imperial Valley	05/18/40	El Centro	S00E	0.348	0.335	1.04
San Fernando	02/09/71	Pacoima Dam	S74W	1.075	0.577	1.86
Saguenay	11/25/88	Chicoutimi	Long.	0.106	0.015	7.02
Motion 1				1.000	0.857	1.17

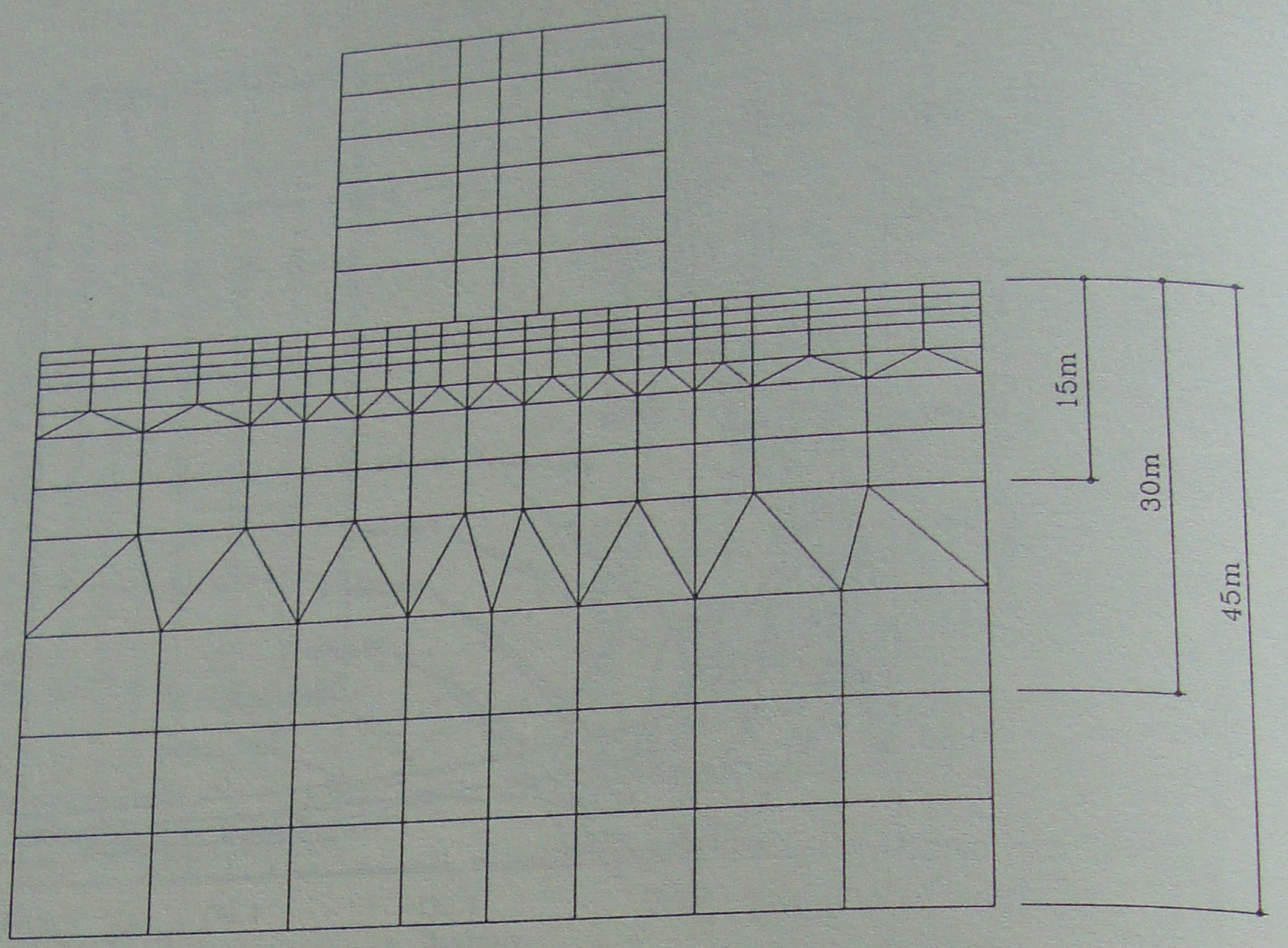

Figure 3. Finite element model of soil and structure.

All the ground motions were scaled to the desired maximum peak ground accelerations of $0.078\,g$, $0.18\,g$ and $0.27\,g$ and are referred to as "low", "intermediate " and "high", respectively. The "low level" earthquakes were assumed to have a maximum acceleration having a probability of exceedance of about 40% in 50 years (100-year return period). The "intermediate level" earthquakes correspond to peak horizontal acceleration having a probability of exceedance of 10% in 50 years (about 500-year return period). The peak horizontal acceleration used for the "high level" earthquakes corresponds to a probability of exceedance of about 5% in 50 years (1000-year return period) as determined from the data base of the Geological Survey of Canada. The Saguenay earthquake was also scaled according to the velocity because the period of the structure studied was in the velocity bound of the spectrum. This scaling resulted in maximum peak ground accelerations of $0.22\,g$, $0.68\,g$ and $1.02\,g$ for the "low", "intermediate" and "high" levels of earthquake, respectively.

ACCOUNTING FOR SOIL-STRUCTURE INTERACTION

It is known that the predicted nonlinear response of certain classes of structures is approximately equal to the predicted linear response for the same motion. Based on that fact, the effects of soil-structure interaction on the nonlinear seismic response of a structure can be studied in two steps. First, a linear analysis of the soil and the structure under seismic loading is carried out. The resulting motion at the soil-structure interface, which includes the interaction effects, is then used as the input motion in a nonlinear analysis of the structure alone.

The program SHAKE (Schnabel et al. 1972) was used to analyze the response of the soil layers. This program calculates the response of a layered infinite soil medium and accounts for the nonlinearity of the shear modulus and the damping. Response curves given by Seed et al. (1986)

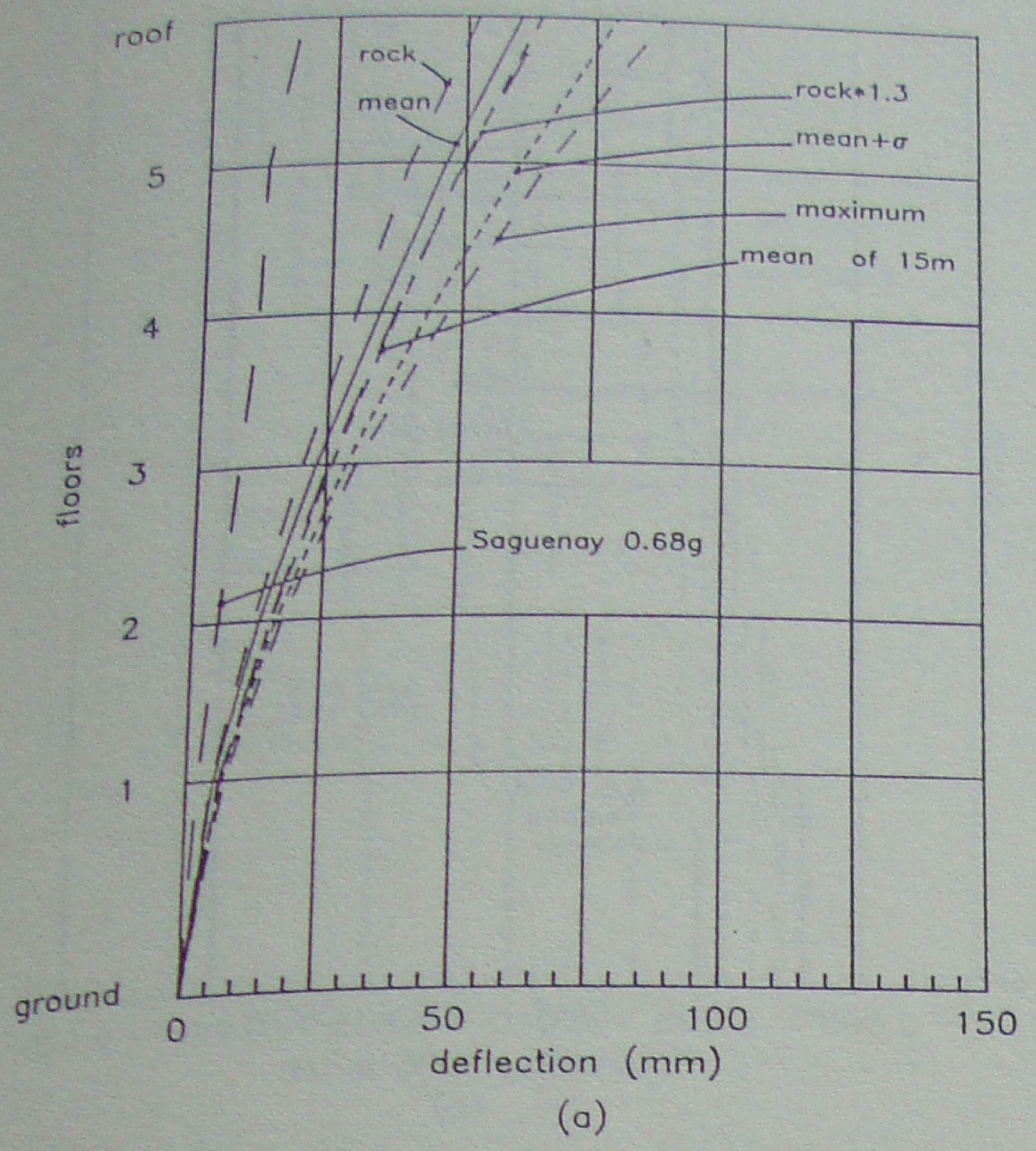

Figure 4. Maximum lateral displacements for structure subjected to ground motions with maximum accelerations of (a) $0.18\,g$ and (b) $0.27\,g$.

were used to find the variation of the shear modulus and the damping with respect to the rate of shear deformation.

To study the soil-structure interaction, different models can be used. In this study the structure and the soil were modelled with beam and plane strain finite elements respectively, and analysed with the program CAL (Wilson 1986). The rigid foundation was modelled through the use of transfer matrices. The extent of soil modelled was approximately three times the width of the building with viscous boundaries, as proposed by Lysmer and Kuhlemeyer (1969) (see Fig 3).

The variations of the shear modulus and the damping of the soil were determined with the program SHAKE for each level of excitation. The shear modulus varied from 14 MPa to 128 MPa, with an average value of 72 MPa. The average damping for the soil was 11.5% of critical. The period of the combined soil-structure varied from 1.12 s to 1.34 s for a soil thickness of 15 m to 45 m.

The largest combined soil-structure finite element model had 411 degrees of freedom. These models were reduced with 35 load dependent Ritz vectors. The response of the reduced model was obtained by step-by-step integration. A measure of the soil-structure interaction is the acceleration time histories obtained at the soil-structure interface. Figure 2 shows the soil-structure interaction effects on the original motions for a peak ground acceleration of $0.18\,g$ and 5% of critical damping. Also shown in Fig. 2 is the NBC 90 equivalent spectral acceleration times the foundation factor $F = 1.3$.

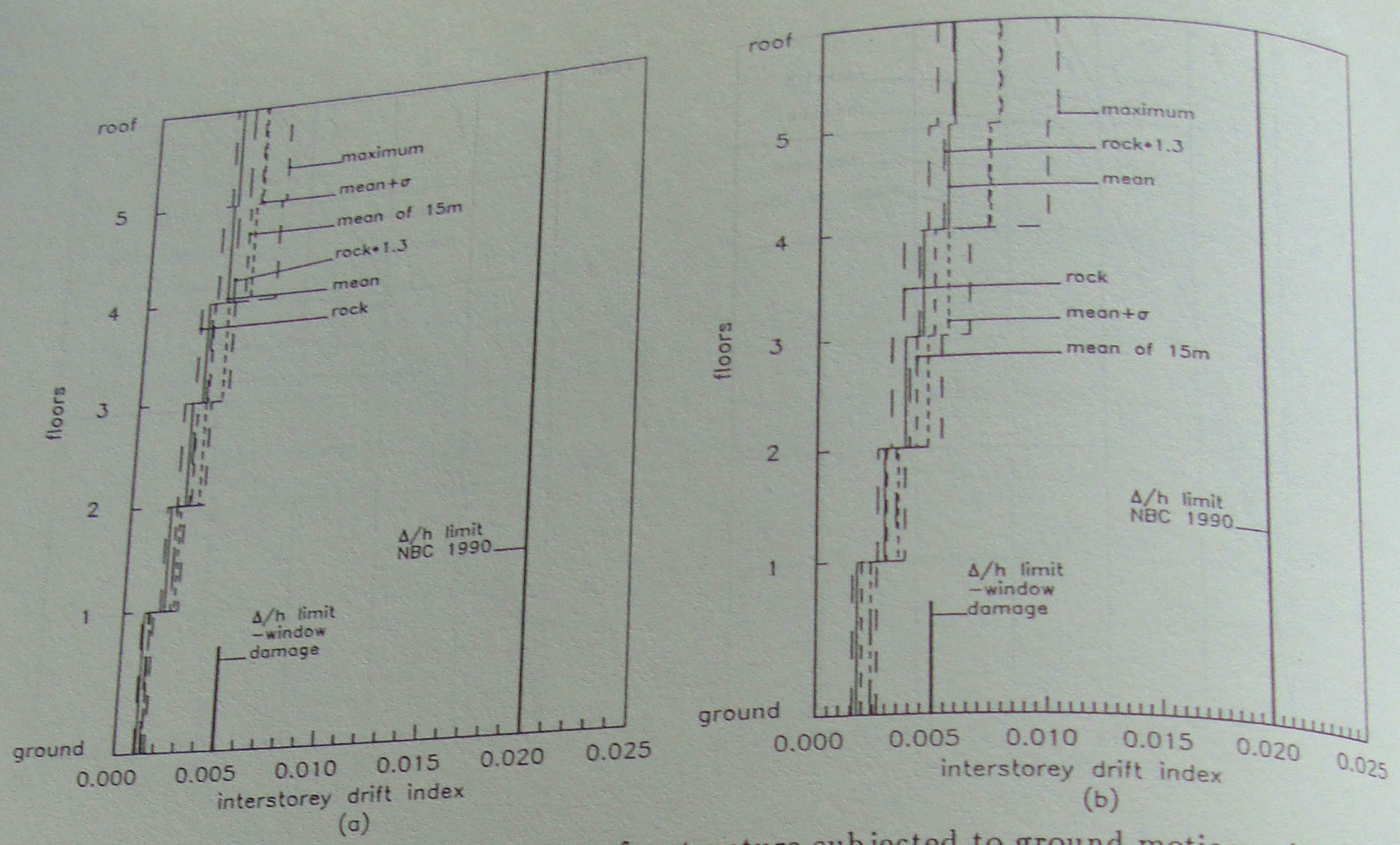

Figure 5. Maximum interstorey drift indices for structure subjected to ground motions with maximum accelerations of (a) $0.18\,g$ and (b) $0.27\,g$.

PREDICTED NONLINEAR DYNAMIC RESPONSES

The resulting horizontal acceleration time histories at the soil-structure interface was used as input motion in the nonlinear analyses of the structure alone. To predict the nonlinear dynamic responses of the six-storey frame-wall structure, the general purpose nonlinear dynamic analysis program DRAIN-2D (Kanaan and Powell 1975) was used. It is interesting to note that the displacements calculated in the nonlinear dynamic analyses were similar to those calculated in the corresponding dynamic linear analyses. This justifies the procedure that was followed to study the dynamic soil-structure interaction effects.

Figure 4 shows the maximum lateral displacements for the four different earthquakes scaled to $0.18\,g$ and $0.27\,g$. It is interesting to note that the predicted deflections for the "base rock" motion multiplied by 1.3 is approximately equal to the average maximum deflection predicted with 15 m of soil. As can be seen from Fig 4 b, the Saguenay earthquake, even scaled to $1.02\,g$, causes very small deflections due to its high frequency content.

Figure 5 shows the averages of the maximum interstorey drift indices (i.e., the ratio of the interstorey drift, Δ, to the storey height, h) for the "intermediate level" and the "high level" earthquakes. The interstorey drift ratio, $\Delta/h = 0.005$, corresponding to likely window damage, is also shown. The maximum predicted interstorey drift indices are well below the 0.02 limit given in the NBC (1990) for structures other than post-disaster buildings. However, due to large interstorey drift ratios, significant nonstructural damage is expected in the higher storeys.

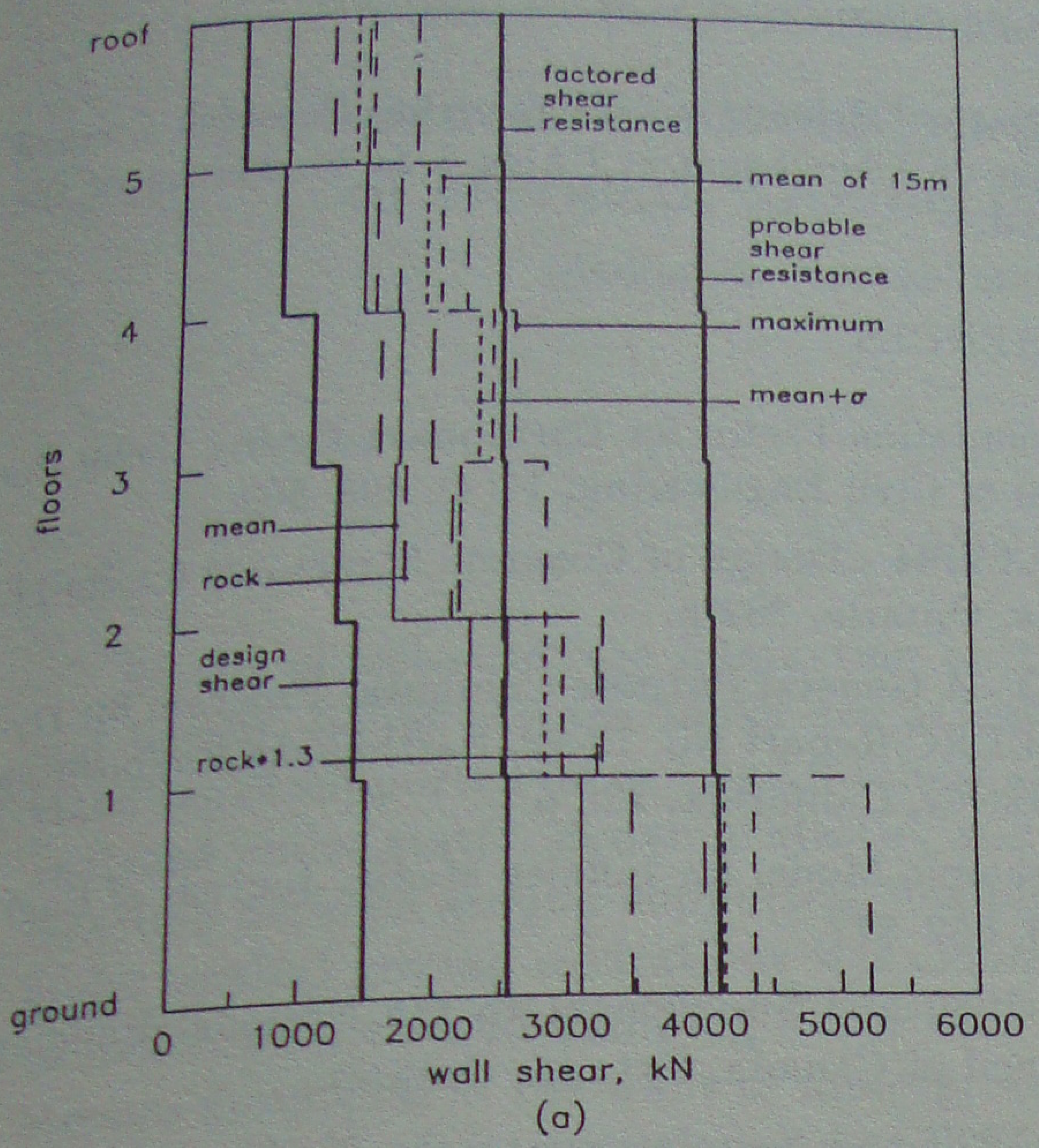

Figure 6. Maximum predicted wall shear with code design shear and shear resistances for structure subjected to ground motions with maximum accelerations of (a) $0.18\,g$ and (b) $0.27\,g$.

Figure 6 illustrates the predicted wall shear for the "intermediate level" and the "high level" earthquakes. Also shown are the design wall shear. The factored and the probable resistance shown in Fig. 6 illustrate the range of wall shear capacities available. The shear capacities of the walls were calculated assuming axial loads corresponding to 0.85% of dead load. As expected, the nonlinear dynamic analyses give wall shears larger than the computed design shears for the "intermediate level" and "high level" earthquakes. An important fact is that the predicted shears at the first storey are in the majority of the cases larger than the probable resistance of the walls. The underestimation of the maximum shear is due to the fact that the same lateral force distribution is used for both flexural and shear design. In fact, in this case, the maximum moment and the maximum shear in the wall do not occur simultaneously. It should be noted that the predicted column shears are well below their nominal capacities; the lateral load is almost entirely taken by the walls.

CONCLUSION

Predicted displacements using the foundation factor are close to the ones obtained from a full soil-structure interaction study. This can be explained from the good agreement between the different spectra and the Code spectra in the frequency range of interest. It is also evident that the code procedure can significantly underestimate the response in the short period range. It is not clear, at least with the motions used, whether the large differences observed in the short period range can be attributed to the interaction problem or to an underestimation of the seismic response.

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