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Base shear provisions for soil-structure systems

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ABSTRACT: Numerical models of soil-structure systems are used to study the influence of site effects and soil-structure interaction on seismic base shear demand in 20 storey reinforced concrete buildings situated on soil deposits. Soil models representative of shallow and deep dense sand and soft clay deposits are developed in this study. Results are presented based on ground motion simulations developed for Prince Rupert in British Columbia. The common practice of neglecting soil-structure interaction in aseismic design is investigated. The NBCC 90 design base shear provisions for structures located in Prince Rupert are evaluated.

INTRODUCTION

Past observations of structural damage patterns during earthquakes underscored the influence of local soil conditions on the seismic response of engineering structures. In general, a soil deposit tends to focus the seismic energy associated with the bedrock motions within a narrow band of frequencies in the neighbourhood of the natural frequency of the deposit. As a result, the free field motions will have a higher content of low frequencies and thus pose a significantly higher seismic hazard to most engineering structures as compared to the corresponding bedrock motions. This behaviour of soil deposits is termed site effects. Site effects are customarily incorporated in the development of various aseismic codes with the first such attempt dating back to the 1932 Chilean code (Seed 1986).

Current NBCC (National Building Code of Canada) regulations incorporate site effects in the specification of design base shear through the use of the foundation factor (F). Soil-structure interaction effects, however, are not taken into consideration. These interaction effects, being a function of the structural characteristics as well as the properties of the local soil deposit, may increase, decrease or have no effect on the lateral forces induced in the structure during seismic response (Seed 1986).

Recent studies, based on uncoupled analyses of soil-structure systems, have shown the NBCC 90 (Associate Committee on the National Building Code 1990) provisions for site effects to be inadequate, especially for structures having predominant periods in the neighbourhood of the site period (Elhmadi et al. 1990; Hosni and Heidebrecht 1991). For such structures, however, soil-structure interaction effects are expected to bring about a reduction in the induced base shear (Seed 1986). Consequently, an evaluation of the current NBCC provisions for base shear in structures situated on soil deposits should be based on coupled, rather than uncoupled, analyses of soil-structure systems. The current study aims at investigating the influence of site effects and soil-structure interaction on seismic base shear demand in 20 storey reinforced concrete buildings. Results are compared to the current NBCC provisions for structures located in Prince Rupert, British Columbia. Seismic hazard for Prince Rupert is associated with the seismically active Queen Charlotte Transform, source of the largest earthquakes in Western Canada (Milne et al. 1978). This study provides an assessment of the assumption, underlying current NBCC provisions, that neglecting soil-structure interaction leads to conservative estimates of the design base shear. Results presented herein are part of a larger study involving six soil types and ground motions developed for three cities in Canada (Hosni 1992).

GROUND MOTION SIMULATIONS

In the current study, the scheme proposed by Hosni and Heidebrecht (1992) for ground motion scaling is used to develop site-specific ground motion simulations that are consistent with the seismological model of Canada underlying the current NBCC provisions.

Seismic hazard for Prince Rupert is associated with shallow interplate faulting along the Queen Charlotte Transform, where most of the strain release occurs during large earthquakes (Milne et al. 1978).

Seismic hazard for Prince Rupert , measured in terms of PHA and PHV (peak horizontal ground acceleration and velocity respectively), at the probability of exceedence of 10 percent in 50 years adopted in NBCC 90, is associated with a 7.9 magnitude event at a source-distance of 200 km (Basham 1990). Since this transform is tectonically similar to the San Andreas fault system (Milne et al. 1978) for which recorded ground motions are available, a set of 10 time histories recorded in California are selected as the initial ground motion data set. These are then scaled according to the scheme proposed by Hosni and Heidebrecht (1992) to be consistent with the magnitude and source-distance combination characterizing seismic hazard for Prince Rupert. In order to maintain the intensity levels associated with these scaled ground motions consistent with that assigned to Prince Rupert in NBCC 90, the scaled time histories are further modified such that the peak ground velocity for each time history is equal to the PHV value of .27 m/s assigned to Prince Rupert in NBCC 90.

Based on the final set of scaled time histories, the mean and M+SD (mean plus one standard deviation) pseudovelocity response curves, at 5 percent damping, are shown in Figure 1. The high content of low frequencies, observed in this figure, is characteristic of ground motions recorded at such long distances from the fault.

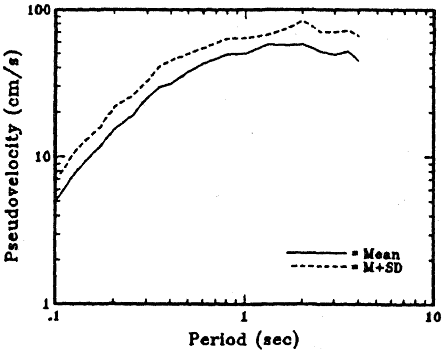

Figure 1. Pseudovelocity spectra based on the ground motions developed for Prince Rupert.

SOIL MODELS

In the current study, artificial soil models are developed to represent dense sand and soft clay deposits. Results of the parametric study by Elhmadi et al. (1990) had shown that for homogeneous deposits, the level of soil amplification of the structural response is independent of soil depth for deposits exceeding 40 m in depth. The study had also shown significant differences between soil amplification levels for 15 and 40 m deposits. In the NBCC 90 provisions for site effects, 15 m is the depth used to distinguish between shallow and deep soil deposits. For these reasons, both 15 and 40 m soil models are developed for each soil type in the current study. These represent shallow and deep soil deposits respectively.

Based on the ground motion data set developed for Prince Rupert, Table 1 shows the mean fundamental period associated with each of the soil models. S15 and S40 refer to the 15 and 40 m dense sand models. Similarly, C15 and C40 refer to the soft clay models. The fundamental periods, at low strain, for these soil models are also shown. Low strain refers to shear strains smaller than .0001 percent and represents almost linear response of the deposit.

Table 1. Fundamental periods for soil models

Period (sec)	S15	S40	C15	C40
Low strain	0.22	0.46	0.52	1.00
Mean	0.46	1.00	0.99	1.82

It is worth noting that the mean site periods, listed in Table 1, are about double the corresponding low strain values. This is expected given the fact that Prince Rupert lies in a zone associated with high ground motion intensity.

STRUCTURAL MODELS

In spite of possible deficiencies in NBCC 90 provisions for structures situated on soil deposits, it is possible that the inherent overstrength in most engineering structures may outweigh these deficiencies. The ratio of the actual strength of a structure to its design strength is the factor used as a measure of this overstrength. The current study is restricted to long period structures as these are usually associated with the lowest overstrength factors (Tso and Naumoski 1991).

In this study, distinction is made between frame and wall structural systems. A reason for this is that differences in dynamic properties between both systems entail differences in the levels of soil-structure interaction and induced base shear, even under the same ground motion excitation. Another reason is that both systems constitute the extremes of dynamic response of regular multi-storey buildings (Fenves and Newmark 1969).

Ductile moment-resisting space frames

The response of symmetrical one-bay frames is considered a satisfactory approximation to the response of actual multi-bay frames subjected to dynamic or static loading (Council on Tall Buildings and Urban Habitat 1979). In view of this observation and the fact that computer time and space are both a function of the frame size, one-bay frame models are used in this study. These frame models have a span of 10 m and are assumed 6 m centre-to-centre in plan. Storey heights are 3.6 m, with the total height of the frame being 72 m.

Design base shear, as specified in NBCC 90, incorporates a force modification factor (R) to reduce the design base shear for a structural system from a level associated with elastic response to a level consistent with the energy dissipation capacity of the system as it undergoes inelastic deformations. For the current study, frame models are designed to represent ductile moment-resisting frames which are associated with the largest R value of 4.

Ductile flexural walls

Isolated wall models are used to represent ductile flexural walls. The wall models are 10 m wide, 72 m high and 5 m centre-to-centre in plan. The factor R associated with this structural system is 3.5. In the design of these wall models, the principal source of energy dissipation is assumed to be the development of a plastic hinge at the base of the wall.

The design and detailing of both the frame and wall models conforms to the special provisions for seismic design in CAN3-A23.3-M84 (Canadian Standards Association 1984). For both models, 5 m deep foundation walls are designed so as to restrict seismic energy dissipation to the superstructure.

FINITE ELEMENT MODEL

Computations in the current study are carried out using computer program FLUSH (Lysmer et al. 1975). The adopted soil-structure model, for a 40 m deep deposit, is shown in Figure 2. The deposit is modelled using 19 layers overlying a rigid base. Corresponding 15 m deposits are modelled using only the upper 9 layers overlying the rigid base. Layer depths are computed to ensure that these do not artificially filter out frequency components of the ground motion that are significant to the response of the soil-structure system. In FLUSH, rigid base motions are assumed to be vertically incident body waves. Since computations in FLUSH are carried out in the frequency domain, soil nonlinearity can only be accounted for through a process of iterations. The program, however, does not account for nonlinearities in the structural model. Due to the influence of the proximity of structural and site periods on the soil amplification potential, it is necessary to make some allowance in the structural model for member cracking during seismic response. For this reason, factors are used to reduce member properties to reflect the expected state of cracking (Goodsir et al. 1983).

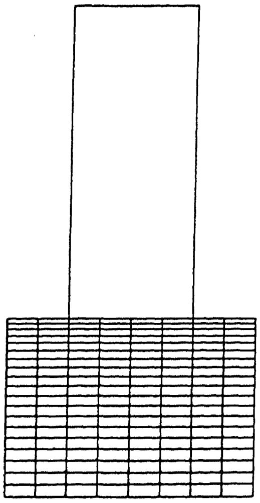

Figure 2. Schematic diagram of the finite element model. (not to scale)

FLUSH employs transmitting boundaries at the vertical edges to model the effects of the semi-infinite soil layers beyond the finite element mesh. A distance of one single element between the foundation wall and the transmitting boundary is deemed sufficient in the current study as the resulting surface motions at the boundary are found to closely resemble the free field motions. Due to symmetry of the soil-structure model, only half of it is used in the computations with appropriate boundary conditions being enforced at the centreline nodes. In FLUSH, the soil deposit is modelled using two-dimensional finite elements while viscous boundaries, in the third dimension, are used to model the actual three dimensional effects.

For the frame model, the maximum base shear during seismic response is computed using the bending moment time histories at both ends of the base columns. For the wall model, this base shear is computed using the shear stress time histories for the base elements. In

FLUSH, Fourier amplification functions from rigid base accelerations to nodal displacements are computed. These are used to determine the fundamental periods listed in Tables 1 and 2.

DISCUSSION OF RESULTS

Site effects

Figure 3 shows the base shear ratio results for the frame and wall models, based on uncoupled analyses of the soil-structure systems. The base shear ratio is the ratio of computed base shear for the structure situated on a soil deposit to the corresponding base shear for the same structure situated directly on the rigid base. This base shear ratio is directly comparable to the foundation factor (F) specified in NBCC 90. Both figures show the mean values as well as the M+SD and M-SD (mean plus and mean minus one standard deviation values respectively). Also shown are the maximum and minimum values based on the 10 time histories developed for Prince Rupert. The corresponding F values, specified in NBCC 90, are superimposed on Figure 3.

Figure 3 indicates that the value of F=1 underestimates the amplification potential of dense sand deposits. For both S15 and S40 soil models, the computed base shear ratios exceed F=1 for all 10 time histories. A similar conclusion, also based on uncoupled analyses but using simple uniform structural models, was reached by Henderson et al. (1990) for an actual 7 m dense sand site in the United Kingdom. The value of F=2 assigned to deep soft clay deposits appears quite reasonable based on results for the frame model, while being close to the lower end of results for the wall model. In this case, the higher amplification associated with results for the wall model may be attributed to the fact that the fundamental period for the wall model, as compared to that for the frame model, is closer to the mean fundamental period for the C40 soil model. Similar to the case of dense sand, base shear ratio results for the 15 m soft clay model exceed the corresponding value of F=1.3 specified in NBCC 90.

Soil-structure interaction

As stated earlier, evaluation of code provisions for soil-structure systems should

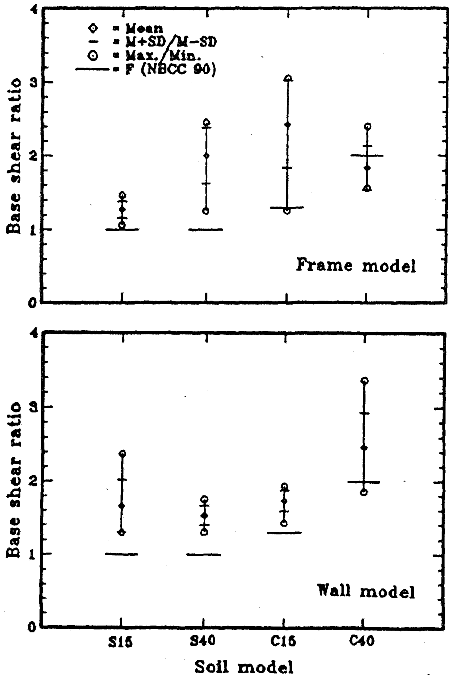

Figure 3. Base shear ratio results based on uncoupled analyses of the soil-structure systems.

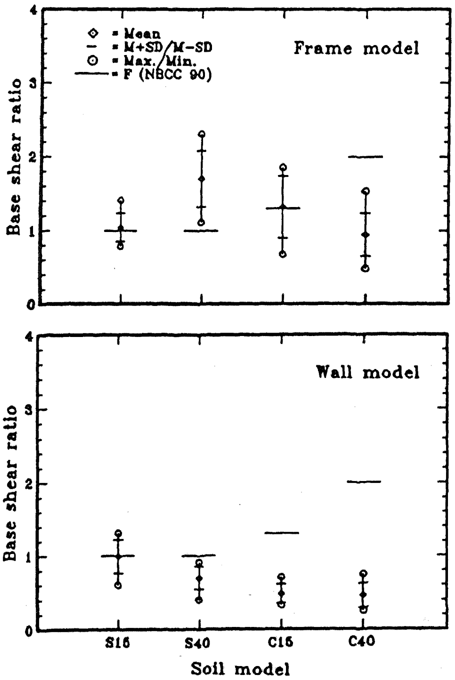

Figure 4. Base shear ratio results based on coupled analyses of the soil-structure systems.

be based on coupled analyses. Figure 4 shows the base shear ratio results for the frame and wall models, based on coupled analyses which, contrary to uncoupled analyses, allow for modelling soil-structure interaction effects. Since NBCC 90 does not provide special provisions for interaction effects, base shear ratio results shown in Figure 4 shall again be directly compared to the F values specified in the code.

Figure 4 indicates that mean base shear ratios using coupled analyses are consistently lower than corresponding values based on uncoupled analyses and shown in Figure 3. This is in direct agreement with the assumption underlying NBCC provisions that soil-structure interaction effects will invariably lead to a reduction in base shear demand. The reduction in the base shear ratio from uncoupled to coupled analysis is higher for wall models than for frame models. This is mainly attributed to the more significant interaction effects associated with the wall models. The rocking component of the interaction effects results in an increase in the soil-structure system period as compared to the corresponding structural period. This shift in system period away from the site period accounts for the decrease in base shear ratio associated with interaction effects. Table 2 lists the mean fundamental periods for the different soil-structure systems, along with the fundamental period for the structural models assumed to be directly situated on bedrock. As can be seen from this table, the period shift is more significant for the wall models and consequently results in a higher drop in the base shear ratio due to soil-structure interaction effects.

Table 2. Fundamental periods for soil-structure systems.

Period (sec)	Bedrock	S15	S40	C15	C40
Frame	2.93	3.14	3.16	3.83	4.12
Wall	1.46	2.05	2.15	3.18	3.65

Based on coupled analyses, the F values appear to be adequate with the exception of F=1 assigned to deep dense sand deposits in the case of the frame model. This exception may be due to the relatively small difference between the soil-structure system period and the structural period in the case of frame models underlain by dense sand deposits, as shown in Table 2. In this case, the drop in base shear demand associated with soil-structure interaction effects is less significant and results based on coupled analyses become essentially the same as those based on uncoupled analyses. The value of F=2, introduced in NBCC 90 for deep soft clay deposits and based on observations of the response of soft clay soils in Mexico City during the 1985 earthquake, appears to be too conservative when soil-structure interaction effects are taken into consideration.

NBCC 90 design base shear

The elastic design base shear, as specified in NBCC 90 for Prince Rupert, is compared to the computed M+SD base shear in Figure 5. In this figure, the base shear coefficient refers to the base shear normalized to the dead weight of the structure. Results for the frame and wall models are shown corresponding to their respective structural periods of 2.93 and 1.46 sec. M+SD results are chosen for comparison to the code design base shear because this probability level is customarily accepted in engineering practice as the design probability level (Newmark et al. 1973). Base shear coefficients, based on NBCC 90, are shown for F=1.0, 1.3 and 2.0.

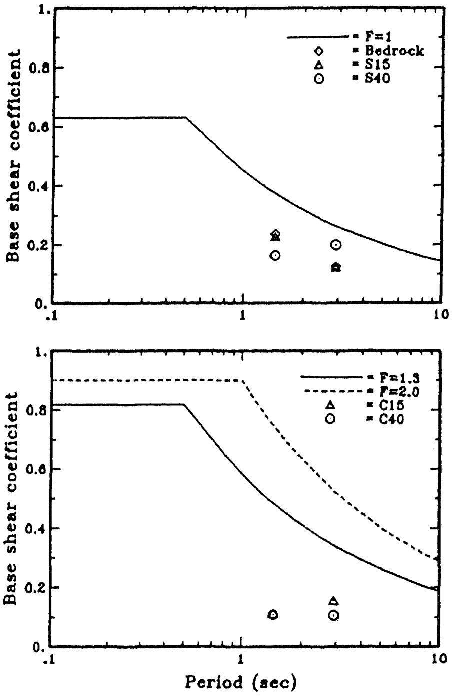

Figure 5. Comparison of the computed M+SD base shear to the NBCC 90 design base shear.

Figure 5 indicates the NBCC provisions for design base shear to be quite conservative for the different soil-structure systems. This is true even in the case of the frame model situated on the deep dense sand deposit, a case for which the F=1 value specified in NBCC 90 is shown, in Figure 4, to underestimate the combined influence of site effects and soil-structure interaction. This is due to two main reasons. First, the NBCC design base shear for structures situated on bedrock is significantly higher than that based on the ground motions developed in the current study. This is evident from comparing the M+SD base shear coefficient for both the frame and wall models, assumed to be directly situated on bedrock, to the NBCC design base shear corresponding to F=1. Second, the reduction in base shear demand, associated with soil-structure interaction effects, is not taken into consideration in the code provisions.

Results are compared to the elastic base shear specified in NBCC 90 because FLUSH does not allow for modelling of the hysteretic energy dissipation in the superstructure. As a result, any reduction of the specified elastic base shear would be unrealistic.

CONCLUSIONS

Following are the main conclusions based on the current study:

1. In view of the significant influence of soil-structure interaction effects on base shear demand in soil-structure systems, these should be incorporated in evaluations of the code provisions for structures situated on soil deposits.
2. The assumption underlying current NBCC provisions, that neglecting soil-structure interaction effects results in a conservative design, appears to be justified on the basis of the results presented herein. In some cases, however, the degree of conservatism appears to be excessive and not economically feasible.
3. There is a need to review code provisions for site effects associated with dense sand deposits.
4. NBCC 90 design base shear for Prince Rupert appears to provide a high degree of protection to 20 storey reinforced concrete buildings situated on bedrock or on deposits of dense sand or soft clay.

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