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**SEISMIC SOIL-STRUCTURE INTERACTION ANALYSIS
OF
STRUCTURAL BASE SHEAR AMPLIFICATION**

by

MOHAMMAD ALI RAHGOZAR

B.Sc. (Eng.), Esfahan University of Technology, Esfahan, Iran

A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfilment of the requirements
for the degree of
*Master of Engineering**
Department of Civil and Environmental Engineering

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Ottawa, Ontario, Canada

August, 1993

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**SEISMIC SOIL-STRUCTURE INTERACTION ANALYSIS
OF
STRUCTURAL BASE SHEAR AMPLIFICATION**

submitted by **MOHAMMAD ALI RAHGOZAR**

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the degree of Master of Engineering

Thesis Supervisor

K. T. Law

Chair, Department of Civil and Environmental Engineering

A. G. Razaqpur

Carleton University

August, 1993

**To my mother, my wife
and
my father who is present everywhere**

Abstract

A study has been carried out on a site-specific basis to study ground motion amplification for three locations in Canada, namely: Ottawa, Vancouver and Prince Rupert. Soil models are so selected that they cover all of the soil conditions used to define the foundation factor, F , in the 1990 NBCC (National Building Code of Canada). 5-, 10-, and 20-storey buildings with moment-resisting frames are adopted as structural models. Three actual ground motion records are selected and then scaled to simulate the postulated seismic hazard parameters corresponding to the three mentioned locations. One of the intents of this investigation is to extend previous studies which are based on one-dimensional models for analyzing site response and structural base shear amplification. Therefore, the three-dimensional finite element computer program, FLUSH, is used to perform the numerical analysis of various soil-structure systems. In addition, this program is modified to compute time history of structural base shear.

Results from the current study indicate that all free field soil deposits amplify the bedrock motion with peak spectral ratio of at least 3.0, and as the soil depth increases, spectral ratio increases in soft clay deposits, decreases in dense sand deposits, and doesn't change significantly in firm clay deposits. The base shear amplification is most significant when the earthquake excitation has significant energy in the site period region. Long period sites (i.e., soft deep soil deposits) have the largest amplification when subjected to seismic motions with low ratio a/v (peak ground acceleration in g to peak ground

velocity in m/s); short period sites (stiff shallow soil deposits), on the other hand, have the largest amplification when subjected to the seismic motions with high a/v ratio. However, substantial amplification at structural periods near site periods occurs for any kind of seismic excitation. For short period sites, foundation factor decreases as the a/v of excitations decreases which proves that foundation factor is dependent on the frequency content of excitation. Most of the computed foundation factors that exceeded the NBCC values are for short period sites especially when they are subjected to high or intermediate frequency motions. Some modifications to the current provision of NBCC foundation factor are proposed to account for the above findings. Underestimations in base shear in NBCC is due to the underestimation of not only the foundation factor but also other parameters such as the seismic Response Factor, S . A new approach of defining soil conditions for NBCC foundation factor is proposed to improve the qualitative definitions used in current NBCC. The proposed approach is based on the thickness and shear wave velocity profile of the soil deposit.

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Notations

Symbol	Description
a , PHA	Peak ground acceleration
f'_c	Concrete ultimate compressive stress
g	Gravitational acceleration
h_{max}	Maximum thickness of soil layer
h_n	Height of building in feet
k	Square root of G_v/G_H
p	Power of depth in shear wave velocity variation with depth
r_o	Source distance
s	Frequency step
t	Time
t_i	Time at step i
v , PHV	Peak ground velocity
v_{bi}	The base input velocity at time t_i
v_{max}	Cut-off frequency
$v_{N/2}$	Folding frequency
v_s	Frequency

v_r	Shear wave velocity of rock
v_{ii}	The specified input velocity at time t_i
x	x-coordinate
y	y-coordinate
$\ddot{y}$	Rigid base acceleration
z	Soil depth
$A_{j\tau}$	The j th component of $\{A_\tau\}_\tau$
AS	Alluvial sand and silt
C	Linear regression coefficient
C_T	NEHRP fundamental period coefficient
D_r	Relative density
DS	Dense sand
E	Young modulus
F	Foundation factor
F _m	Mean value of foundation factor
$\bar{F}$	Proposed foundation factor
F_{max}	Maximum foundation factor proposed by Elhmadi et al., 1991
G	Secant shear modulus
G_H	Small strain shear modulus at rock boundary
G_o	Small strain shear modulus at soil surface
G_o	Small strain shear modulus
G^*	Complex shear modulus

H	Soil depth
H	Corresponds to high frequency motions
I	Importance factor
I	Corresponds to intermediate frequency motions
I_p	Plasticity index
L	The thickness of the soil slice
L	Corresponds to low frequency motions
M	Overturning moment
M, M_o	Earthquake magnitude
M_{mi}	i-node moment of element m
M_{mj}	j-node moment of element m
M_{ni}	i-node moment of element n
M_{nj}	j-node moment of element n
N	The number of digitized points
NC	Lightly to normally overconsolidated clay
OC	Heavily overconsolidated clay
P	Primary wave
R	Force modification factor
S	Secondary wave
S, S_N	Seismic response factor
S_c	Computed seismic response factor
T	Structural fundamental period

T	Total duration of control motion
T_s	Site period
U	Calibration factor
U_j	The jth component of {U_i}_i
V	NBCC design base shear force
V, V_s	Shear wave velocity
V_e	NBCC elastic base shear force
V_{eq}	Shear wave velocity at Z_{eq}
V_{mj}	j-node transverse shear of element m
V_{nj}	j-node transverse shear of element n
V_o	Shear wave velocity at soil surface
V_r	Computed base shear force when building is located on rock
V_r max	Maximum of V_r during time
V_s	Computed base shear force when building is located on soil surface
V_s max	Maximum of V_s during time
W	Weight of building
$\ddot{Y}_t$	Fast Fourier transformed rigid base acceleration
Z_a	Zonal acceleration
Z_{eq}	Equivalent depth
Z_v	Zonal velocity
β	Damping ratio
γ, γ_{xy}	Shear strain amplitude

γ_{eff}	Effective shear strain amplitude
γ_{max}	Maximum shear strain
ϵ_x	Axial strain in x-direction
ϵ_y	Axial strain in y-direction
ν	Poisson's ratio
ρ	Mass density of soil deposit
ρ_r	Mass density of rock
τ_i	Shear stress at the rock-soil interface
ω_s	Frequency at which amplification functions are to be determined
Δt	Time step
$\{m\}$	A vector related to $[M]$ and the direction of the rigid base acceleration $y(t)$
$\{u\}$	Displacements of nodal points relative to the rigid base
$\{\dot{u}\}$	The first derivative of $\{u\}$
$\{\ddot{u}\}$	The second derivative of $\{u\}$
$\{u\}_f$	Free field displacements
$\{\dot{u}\}_f$	Free field velocities
$\{A\}_s$	Amplification functions
$\{A_f\}_s$	A vector containing the amplification values from the rigid base acceleration to layer displacement
$[C]$	A simple diagonal matrix which depends on the properties of the free field and free field velocities

(F)	The forces which act at the ends of soil slice
(G)	Frequency-dependent stiffness matrix formed from complex moduli in free field
[K]	Plain strain stiffness matrix
[K]_f	Frequency-dependent stiffness matrix
[L]	Frequency-dependent boundary stiffness matrix
[M]	Plane strain mass matrix
[P]_f	Load vector corresponding to unit amplitude of rigid base motion
[R]	Frequency-dependent boundary stiffness matrix
(T)	The forces related to energy transmission
(U)_f	Fast Fourier transformed displacements
(U_f)_f	Fast Fourier transformed free field displacements
(V)	Forces originated from viscous boundaries

Chapter 1

INTRODUCTION

1.1 GENERAL

The general understanding of earthquake ground motions, the manner in which they propagate through the earth, description of their characteristics at a certain location and methods for incorporating this information into engineering designs have been the subject of considerable interest and research. The characteristics of earthquake shaking at a given site in a particular event will depend on a number of factors including:

1. The source mechanism of the earthquake
2. The distance of the site from the main zone of energy release
3. The geological characteristics of the rocks along the wave transmission path from source to site
4. The effects of wave interference
5. The local soil conditions at the site

The energy released from a source mechanism will travel in the form of seismic waves through the rock formation where some energy absorption takes place. The seismic waves

will be reflected and refracted as they approach the zone of special interest. The motions will be further modified as they pass from firm underlying rock formation through the different soil deposits to reach the ground surface where structures are founded. The significant characteristics of ground motions that affect the intensity of shaking and the extent of damage to structures include:

1. The maximum amplitude of ground acceleration, velocity and displacement
2. The frequency content of the excitation
3. The duration of strong-phase shaking

Analysis of the source mechanism and the effects of transmission path geology on earthquake waves is the main area of interest for seismologists. The effects of local site conditions on the characteristics of ground motions have been the focus of research by both seismologists and geotechnical engineers. This is because of the concentration of human population in sediment-filled valleys and because of common observation that ground motion at the surface of sediments is larger than on nearby hard rocks (e.g., Gutenberg, 1957). The aim of the seismological and geotechnical studies is the definition of the design earthquake input which is of main interest to the structural engineers. Once a suitable support-excitation history has been established, the calculation of the stresses and deflections in a given structure is a problem of structural dynamics and on occasion soil-structure interaction.

Site-response problems involve the determination of the effects of local soil conditions on the temporal and spatial variation of all the motions within a site from a specified control motion at some control point within the site, where an observed or

estimated motion is available. Soil parameters and geologic conditions that may cause significant effects on the characteristics of the ground motions at a site include:

1. Thickness of soil layers above the bedrock
2. Variation of soil type and properties (shear modulus, mass density and material damping) along the depth
3. Non-linear variation of soil properties under strong earthquakes
4. Lateral irregularity and surface topography at the site

Mathematically, the site-response problem is of wave propagation in a continuous medium. Soil amplification theories are useful as an engineering tool for obtaining reasonable estimates of motions, when applied with proper judgment. As in many other engineering problems, and particularly in the areas of geotechnology and earthquake engineering, there is a substantial amount of uncertainty in all phases of the analysis. Special consideration must be given to the selection of an appropriate control motion with characteristics consistent with the location where it is specified. However, the estimation of control motion and the wave pattern contributing to it is still a major problem to establish and far-reaching assumptions have to be made. Moreover, the estimation of soil properties and their variation with level of strains should be considered in any mathematical model.

1.2 STATEMENT OF PROBLEMS

There is increasing evidence that as the seismic motions propagate from bedrock to the ground surface, the soil tends to amplify the motions in frequency ranges in the

vicinity of the natural frequencies of the top layer (Duke, 1958; Medvedev, 1965; Seed, 1986; Heidebercht et al., 1990). In addition, a structure located on the surfacial layer may suffer more intense shaking due to the development of a state of quasi-resonance between structure and foundation soil. Numerous examples of recent earthquakes where site amplification were observed are available. For example, during the 1985 Mexico City earthquake, site amplification caused substantial damage and collapse of many buildings (Romo and Seed, 1986; Seed et al., 1987). Observations made after the destructive earthquakes have shown a correlation between damage and local geology, with the destruction being larger on soft soil than hard soil or rock (Duke, 1958; Medvedev, 1965; Seed, 1986).

Based on the observations during the destructive earthquake on 19 september 1985 in Mexico City, the structural design base shear force in NBCC (National Building Code of Canada) was increased, for example up to 33% for very soft and soft fine-grained soils with depth greater than 15 m. This was accomplished by replacing the foundation factor, F , from 1.5 to 2.0. Foundation factor is a parameter accounting for site conditions.

The problems start from here, first, how is this increase justified? Second, is this increase sufficient? Third, does the current provision of NBCC foundation factor accounts for quasi-resonance between structure and soil foundation? Fourth, are the effects of earthquakes with various frequency contents included in the specified foundation factors? Few studies were conducted to answer part of the mentioned questions such as Heidebercht et al., (1990); Henderson et al., (1990); Lo R.C. et al., (1991).

In addition, one-dimensional computational model was adopted for analysis in all

the previous studies as well as in the procedure of deriving current NBCC provisions of base shear and foundation factor, assuming that the structures are built on a rigid structural foundation and move with the ground surface motion. In other words, the effects of soil-structure interaction are neglected on the premise that, for most of the buildings addressed in the code, the effects lead to a reduction in base shear demand and thus a more conservative design (NBCC 90 Commentary). However, soil-structure interaction effects, being a function of the properties of the structure as well as the properties of the local soil deposit, may in some cases actually increase the lateral force demand during the seismic response (Seed, 1986).

In all of the previous studies, underestimation of the base shear is all attributed to low values of NBCC foundation factor. However the study in this thesis will show that such underestimation is due to low values of foundation factor as well as other parameters such as the seismic response factor, S .

Another problem, which is also one of main concerns of this study, is the qualitative definitions of soil conditions used in obtaining the foundation factor in the current NBCC. Such a qualitative definition is not adequate for geotechnical design.

1.3 OBJECTIVE AND SCOPE

The objective of this study is to answer the above questions by investigating seismic behaviour of soil-structure systems. The scope of the study is described as follows:

- 1) Evaluation of the previous studies on site response analysis, previous studies on

amplification of structural base shear and justifying the existence of foundation factor in design base shear. Study of the input motion records, frequency content of excitations and correlation between frequency content of excitations and site amplification.

2) Review of one-dimensional theories of wave propagation and site amplification and comparing them with the finite element computational model introduced in FLUSH program (Lysmer et al., 1975), which accounts for interaction effects between structure and soil foundation as two substructures of one system.

3) Gathering of soil sites to cover a wide range of soil foundations with different soil types and depths.

4) Study of the seismicity of different locations in Canada to choose locations with different earthquake source mechanisms and with a variety of seismic hazard parameters corresponding to the current NBCC base shear provision.

5) Gaining of insight into seismic soil-structure interaction effects on site response by studying the effects of soil and structural types on the response of soil-structure system.

6) Modification of FLUSH program to compute time history of the base shear of multi-storey buildings. By using the three-dimensional finite element computational model, foundation factor values of a wide combination of actual buildings and soil deposits are derived as measures of base shear amplifications. From this a number of phenomena are studied such as, (a) the effects of quasi-resonance between soil and structure on base shear and in turn on foundation factor, (b) the effects of earthquakes with different frequency contents on foundation factor, (c) the effects of other parameters

response factor, S , used in NBCC 90 on structural base shear.

7) Introduction of a new approach to define soil conditions for supplementing the current NBCC provisions for foundation factor.

1.4 THESIS ORGANIZATION

The thesis commences with a literature review in Chapter 2. Previous studies on the effects of site conditions on site amplification are reviewed. Existing one-dimensional as well as two- and three-dimensional engineering computational models are investigated. Acceleration time histories are analyzed as the only tool of simulating ground motions. Also, previous studies on base shear amplification, particularly those related to NBCC 90 provisions are discussed in this chapter.

In chapter 3, existing engineering computational models are described in great detail. The three-dimensional finite element computational method adopted in FLUSH program (Lysmer et al., 1975) is chosen for analysis in the current study.

Chapter 4 describes in details the soil sites and the input motions used in the current study. Types and strain dependent properties of soil deposits are explained in this chapter. In order to study the resonance phenomenon between soil and structure, the site periods are needed, and thus, they are computed using Closed Form Solutions (Dorby et al., 1976). After detailed considerations, three acceleration time histories compatible with the seismicity of three cities of Ottawa, Vancouver, and Prince Rupert are selected and then scaled based on the specified seismic hazard parameters of these three cities.

Chapter 5 introduces the applied soil-structure models. Then, the effects of soil and

structural types and ground excitation types on soil-structure response are investigated.

The effects of site amplification and soil-structure interaction on NBCC 90 provisions of base shear and foundation factor are studied, by means of the three-dimensional finite element model introduced in FLUSH program. First of all, a free-field response analysis is performed to study site amplification when there is no structure on top of soil deposit. In this part of study the effects of soil type, soil depth, and a/v of input motion as a measure of frequency content on site amplification are observed. Next, by assuming a building on top of soil deposit, time history of the structural base shear force is computed as a measure of structural response to ground motion. The computed maximum base shear forces are then used to derive foundation factors which are compared with current NBCC values. The comparison leads to new foundation factors being proposed. The correlation between the proximity of structural fundamental period to site period and foundation factor is also investigated in this chapter. Adequacy assessment of NBCC foundation factors as well as the effects of other parameters on NBCC underestimated base shear are evaluated. At the end of this chapter a new approach of defining soil conditions for supplementing the current NBCC provisions for foundation factors is proposed.

Chapter 7 gives major conclusions from this research and states contributions of the thesis. At last, suggestions on future work are given.

Chapter 2

LITERATURE REVIEW

2.1 EFFECTS OF SITE CONDITIONS ON GROUND AMPLIFICATION

Seismic waves propagating through near-surface soil layers can amplify and produce ground motions much larger and with different characteristics than those recorded at the rock base. The combined effects of earthquakes and local site conditions are commonly referred to as site effects. Numerous examples of recent earthquakes where site effects were observed are available. For example, during the 1985 Mexico City earthquake, site amplification caused substantial damage and collapse of many buildings (Romo and Seed, 1986; Seed et al, 1987). Records of damage during the 1989 Loma Prieta, California, earthquake also indicate that site effects were the major cause of failure of many facilities (Astaneh et al, 1990). Detailed studies of the relationship between building damage and soil conditions were provided by Seed (1986).

Observations made after destructive earthquakes have shown a correlation between damage and local geology, with the destruction being larger on soft soil than hard soil or rock (Duke, 1958; Medvedev, 1965). Although part of the explanation resides in excessive settlement, liquefaction or other failure phenomena occurring in soft or loose sediments during earthquakes, there is also evidence that the characteristics of ground shaking can be modified by the local soil conditions (Ohsaki, 1969; Seed, 1969). Some recent earthquakes have illustrated dramatically this effect, through selective damage and collapse of long-period buildings located in areas having softer or deeper soils: Mexico City 1957 and 1985 (Duke and Leeds, 1959; Romo and Seed, 1986; Seed et al., 1987).

2.2 SITE AMPLIFICATION

A study of site amplification usually consists of:

1. Determination of representative soil profile and dynamic soil properties to simulate typical conditions in the site,
2. Selection of input motions compatible with the local seismo-tectonic setting,
3. Dynamic response analyses for the selected sites using state-of-the-practice procedures (Naeim F., 1989).

In the last few years, the fast growth in the number of available strong motion records has allowed statistical studies of the characteristics of horizontal ground shaking, including the effect of local geology. Trifunac and Brady (1975) investigated 374 horizontal accelerograms corresponding to 187 records obtained in western United States between 1933 and 1971. Using as a basis for the comparison the Modified Mercalli

Intensity Scale, they obtained average values of peak acceleration which were somewhat lower for sites on alluvium than for sites on rock. Average peak velocities and displacements, on the other hand were higher for the alluvium. This suggest a shift to the long-period range caused by the presence of soil.

In 1976, two independent studies, one by Seed, Ugas, and Lysmer and the other by Mohraz, considered the influence of soil conditions on response spectra. The study by Seed used a total of 104 horizontal components of earthquake records with peak ground accelerations greater than 0.05g from 23 earthquakes. The records were divided into four categories: rock, stiff soils less than about 150 ft, and soft to medium clay and sand. The response spectra for 5% damping were normalized to the peak ground acceleration of the records and averaged at various periods. The average spectra for the four categories from their study are presented in Fig.2.1a. The ordinate in this plot represents the acceleration amplification. The figure shows that for periods greater than approximately 0.4-0.5 sec, the normalized spectral ordinates for rock are substantially below those for soft to medium clay and for deep cohesionless soils.

The study by Mohraz considered a total of 162 components of earthquake records divided into four soil categories: alluvium, rock, less than 30 ft of alluvium underlain by rock, and 30-200 ft of alluvium underlain by rock. To exclude the influence of the horizontal components of records with the smaller peak ground acceleration on amplification, Mohraz divided the response spectra for each soil category into three sets: horizontal components of records with the larger of the two peak ground accelerations, horizontal components with the smaller of the two peak ground accelerations, and vertical

components. Fig.2.1b shows the average acceleration amplification (ratio of spectral ordinates to peak ground acceleration) for 2% damping for the horizontal components with the larger of the two peak ground accelerations. Consistent with Seed's study, the figure shows that the soil conditions influence the spectral shapes to a significant degree. The acceleration amplification for alluvium extends over a larger frequency region than the amplifications for the other three soil categories. It is noted that the maximum acceleration amplification for less than 30 ft of alluvium on rock is approximately 40% greater than that for alluvium and about 33% greater than that for rock.

Based on the two preceding studies it is implied that tall buildings founded on deep or soft alluvium may be subjected to seismic forces several times larger than similar buildings on rock, if the peak ground acceleration is the same in both cases. Therefore, observed damage patterns and the statistical analysis recorded motions suggest that both stiffness and the thickness of soil should be considered in the seismic design of structures, especially in the long-period range.

The effects of site amplification were also observed during the 1989 Loma Prieta, California earthquake (Bensuka 1990). The structural damage pattern in the San Francisco Bay area was strongly influenced by local soil conditions, and the majority of the damage occurred on deep clay sites as a results of the significant amplification of the seismic motion at these sites.

2.3 SOME ENGINEERING COMPUTATIONAL METHODS AND PROGRAMS

Engineers have mainly relied on the one-dimensional analysis to predict surface motions at any site, overlooking the effects of interactions between soil and structure as the two substructures of one system (e.g., Heidebercht et al., 1990; Henderson et al., 1990; Elhmadi and Heidebercht 1991; Lo R. C. et al., 1991). Reviews of some existing one- and two-dimensional methods and programs are summarized below, although they are presented in more details in chapter 3.

The one-dimensional approach is based on Kanai's (1952) assumption that the main response in a soil deposit is caused by the upward propagation of shear waves from the underlying formation. Analytical procedures based on this concept incorporating non-linear soil behaviour in a linear iterative scheme in the frequency domain were introduced by Schnable et al., (1972) in the SHAKE program.

The computer program SIREN, developed by Pappin (Henderson et al., 1989a), solves the one-dimensional problems in the time domain using the finite difference method in a true non-linear model. In the SIREN program, the soil is represented as a series of lumped masses connected by non-linear springs. The non-linear springs are formulated as an assemblage of elastic-plastic springs which exhibit hysteretic damping in accordance with the Massing principles (Pyke, 1979). No viscous damping is included in this model. Transmission of energy at the rock soil interface is accomplished by adjusting the base input velocity in proportion to the shear stress existing at the interface (Heidebrecht et al., 1990; Henderson et al., 1990). SIREN program was used by Heidebrecht et al., (1990) and Henderson et al., (1990) for the analysis of site response

effects for structures located on clay sites and sand sites, respectively.

A finite-element computer program "FLUSH" was developed by Lysmer et al., (1975) to take into account the strong non-linear effects which occur in soil subjected to strong earthquake motions. This was achieved by the combination of equivalent linear method described by Seed and Idriss (1969) and the method of complex response in the frequency domain. The input excitation to the finite element model in FLUSH program is specified by an acceleration time history at the rigid bedrock which does not have to be horizontal. The computational method and assumptions made in FLUSH program are presented with more detailed explanation in chapter 3.

2.4 SEISMIC GROUND MOTIONS

Strong motion earthquake records are basic data for earthquake engineers. They serve as an important link between the fields of seismology and earthquake engineering. A study of such records can give insight into the nature of ground motion shaking.

When used as inputs for experiments or analysis, they provide realistic simulation to study the nature of structural responses due to seismic excitation. Earthquake ground motion records can be used to evaluate the performance of a particular structure, or assess the adequacy of building code requirements. However, the estimation of site specific ground motion parameters for seismic design studies is one of the most complex and challenging problems of earthquake engineering.

Since the first set of strong motion records obtained during the Long Beach earthquake of 1933 in California (Naumoski et al., 1988), thousands of strong motion

records have been recorded and become available from many countries including Japan, United States, Mexico, Canada among others.

Visual inspection of these records show that earthquake ground motions vary greatly from record to record. The intensity, duration of strong shaking, frequency content of any record depends on a number of factors, including the magnitude of the earthquake, the epicentral distance, the local geology and the site soil condition. These differences among records can lead to significant differences in structural responses, and in turn to design implications.

One major factor that has significant effect on structural responses is the frequency content of the record. It is believed that the ratio of peak ground acceleration, a (in g , the gravitational acceleration), to peak ground velocity, v (m/s), represent a measure of the frequency content of the seismic ground motion (Heidebrecht et al. 1983; Heidebrecht and Lu 1988). Naumoski et al., (1988) performed a statistical analysis on 45 selected earthquake records from the McMaster Database System. They divided the records into three sets with different a/v ratios, namely high a/v ($a/v > 1.2$), intermediate a/v ($1.2 > a/v > 0.8$), and low a/v ($a/v < 0.8$). There were fifteen records in each set. As a matter of fact, the acceleration time histories applied in the current study belong to these three sets (see section 4.5).

2.5 SEISMIC GROUND AMPLIFICATION INFLUENCES ON NBCC 90 BASE SHEAR AND FOUNDATION FACTOR

On 19 September 1985, a large earthquake ($M = 8.1$) occurred near the south coast of Mexico. Mexico City, situated at 300 - 400 km from the epicentre, sustained severe building damage and heavy casualties. Buildings that sustained moderate or higher levels of damage were restricted exclusively to the Lake Zone (EEFIT 1986). Dorby and Vucetic (1987) demonstrated that the immense amplification recorded in Mexico City were primarily due to the almost linear behaviour of Mexico City clays within the range of shear strains ($<0.3\%$) induced by earthquake. Taking into consideration that at such long epicentral distances, a substantial portion of the seismic energy at bedrock is associated with low frequencies, the levels of site amplification recorded in Mexico City should not be surprising. In fact, similar damage patterns were observed during the 1957 earthquake ($M = 7.5$). During that event, 96% of the damage sustained in Mexico City was restricted to the Lake Zone (Mitchell et al. 1986). Since soil deposits tend to focus the energy of the ground motions at frequencies in the neighbourhood of the natural frequency of the soil deposit, higher damage potential is expected to the structures of fundamental periods close to the site period. This argument is substantiated by the damage pattern observed during the 1985 earthquake for which the most affected structures were over 6 storeys in height (Milchell et al. 1986).

Following the 1985 earthquake, a presidential decree was published in Mexico giving emergency code changes for constructions in the federal District. The emergency changes involved increasing design forces by as much as 67% for buildings in the Lake Zone

(Mitchell et al. 1986). Based on observations during the 1985 event, a foundation factor (F) of 2.0 was specified in the NBCC (National Building Code of Canada) for very soft and soft fine-grained soils with depth greater than 15 m (NBCC 1990). This implies a 33% increase in design forces as compared to previous value of $F = 1.5$.

NBCC (1990) (Associate Committee on the National Building Code, 1990) provisions incorporate site effects in the specification of the design base shear through the use of the foundation factor, F, as follows:

$$V = (V_e/R)U$$

$$V_e = vSIFW$$

= base shear corresponding to elastic response

R = force modification factor

U = calibration factor based on experience = 0.6

v = zonal velocity

I = importance factor = 1.0 for structures of normal importance

F = foundation factor (function of both soil type and depth)

W = dead weight of the structure.

Foundation factor refers to the ratio of the computed base shear for structure situated on a soil deposit to the corresponding base shear for an identical structure assumed to be directly situated on bedrock and subjected to the same bedrock motions.

Heidebrecht et al., (1990) showed that the amplification levels associated with deep

soft clay sites and subjected to low intensity, very low a/v (ratio of peak ground acceleration to peak ground velocity) motions are unpredicted by the factor $F = 2.0$ provided in NBCC (1990).

In an effort to realize the engineering significance of the amplification effects observed in Mexico City, Hosni and Heidebrecht (1991) studied the dynamic response of bilinear single degree of freedom systems to investigate the adequacy of protection measures provided in Canadian and Mexico Codes and the significant differences between both codes in accounting for amplification effects of deep soft deposits. Following are the main conclusions of their studies:

1. $F = 2.0$ provided in NBCC 90 for deep soft clay deposits does not adequately account for site amplification effects similar to those recorded in Mexico City during the 1985 earthquake.

2. There is a need for adoption of period dependent force modification factors in NBCC. The range of structural periods for which the force modification factor is period dependent should be a function of the geological conditions at the site being considered.

To study the influence of site amplification of seismic ground motions on forces in building structures, Heidebrecht et al., (1991) selected nine sites to represent a wide range of foundations. The strong motion database at McMaster University was used to select input accelerograms. Three ensembles of actual strong motions records were selected, which correspond to the three ranges of a/v ratio in NBCC 1985; $Z_a > Z_v$, $Z_a = Z_v$, $Z_a < Z_v$, where Z_a and Z_v are zonal acceleration and velocity, respectively. They chose frame structures instead of wall structures as the structural buildings situated on

the soil deposit. Their justification was, except for long period regions, frame structures produce higher foundation factor.

For each of the sites considered, the dynamic foundation factor was computed. The results were compared with the corresponding parameters in NBCC 1990. On the basis of those results, the following are their conclusions:

1. The base shear amplification is most significant when the earthquake excitation has significant energy in the site period region. Long period sites (i.e., soft deep soil deposits with period greater than 2.0 sec) have the largest amplification when subjected to seismic motion with low a/v ratio. Short period sites (stiff shallow soil deposits with period less than 1.0 sec) also have the largest amplification when subjected to seismic motion with high a/v ratio. However, substantial amplification at structural periods near site periods occurs for any kind of seismic excitation.

2. The base shear amplification decreases as the intensity of excitation increases; this decrease is larger for long period sites than that for short period sites.

3. For long period sites, peak values of base shear ratios in the neighbourhood of site periods range from 4 to 6 for low intensity excitation ($v = 0.05$ m/s), and from 2 to 3 for high intensity excitation ($v = 0.4$ m/s). For short period sites, the peak values range from 2.5 to 4.5 for low intensity excitation to approximately 2 for high intensity excitation.

4. The NBCC 1990 foundation factors of 1.0, 1.3, and 2.0 are reasonably adequate for periods out of the site period range. However, these factors are inadequate for periods in the neighbourhood of the site period. In this period range, the computed dynamic foundation factors for almost all of the sites considered were larger than the NBCC 1990

foundation factors.

A site amplification study was carried out for the Fraser Delta of British Columbia (Lo R. C. et al., 1991). The delta is located in southwest corner of the British Columbia mainland within the Greater Vancouver Regional District, and is in one of the highest seismic areas in Canada. According to insitu borehole tests, the river sediments generally consist of unconsolidated fine-grained marine sediments (silts and clays) overlain by sand-sized marine and tidal flat deposits which are in turn overlain by fine silty overbank deposits. These postglacial deposits are up to about 200 m thick. The bedrock surface is estimated to occur at depths of 250 m or deeper under most of the delta (Blunden 1975). The foundation factors computed from these analyses were compared to the provisions in NBCC 1990. They concluded that for typical sites involving deep soil deposits in the Fraser Delta, the current code value of 2 as the foundation factor is generally adequate, except for the period range of about 0.5 to 2 sec depending on: the type of the building, the site conditions and the level of spectra considered. Most of the seismic activities affecting the Fraser Delta are associated with the interactions among the following plates: the North America plate, the Juan de Fuca plate, the Explorer plate and the Pacific plate further to the west (Rogers 1988). According to the seismicity zonation map of Canada, for the Fraser Delta peak ground velocity, v , is 0.21 m/s and peak ground acceleration is 0.21g. These values yield an a/v ratio of unity. Lo R.C. (1991) selected 22 records with $0.8 < a/v < 1.2$ to meet the above criteria. They used computer program SHAKE (Schnabel et al., 1972) to compute the response of the soil deposits to the selected input motions. They scaled the selected records to the same peak ground velocity for the area (0.21 m/s).

For the subduction earthquakes, the suite of records (100 to 250 km) were scaled to 0.2g, while the distant records (>250 km) were scaled to 0.05g. Heidebrecht and Stafford-Smith (1973) showed that the dynamic behaviour of most structural systems lies somewhere between uniform *frame* and *shear wall* type structures. Consequently, Lo R.C. et al., (1991) computed the base shear for those two types of structures using simple continuum models (Heidebrecht and Lu 1988). The base shear was obtained by combining the first five modal values using the square root of the sum of the squares method. According to their study, Lo R.C. 1991 concluded that for typical soil sites in the Fraser Delta the increase of the code foundation factor, F , from 1.5 (NBCC 1985) to the present value of 2, as a result of lessons learned from the 1985 Mexico earthquake, appears to be justified.

Elhmadi and Heidebrecht (1991) proposed a dynamic foundation factor for the NBCC. In this study they used the results of a parametric study done by Elhmadi et al., 1990. The main phases of their study are presented in the following.

Four categories of soil deposits were considered: (i) normally to lightly overconsolidated clay (NC); (ii) heavily overconsolidated clay (OC); (iii) alluvial sand and silt (AS); and (iv) dense sand (DS). For each category, four different site thickness, H , were considered, i.e., 5, 15, 40, 100 m. Fig.2.2 shows the small strain shear modulus, G_s , versus depth, for the four soil types. Fig.2.3 shows the variation of shear modulus and damping with shear strain for the soils used in that study. A detailed derivation of these relationships is described in Pappin et al., (1989), Heidebrecht et al., (1990), and Henderson et al., (1990). The soil deposits were modelled as one-dimensional systems with propagation of shear waves only in vertical direction. The computer program for the

nonlinear model known as SIREN, developed by Pappin at Ove Arup & Partners (Henderson et al., 1989a), was used for the analyses. Taking into account that the ratio of peak ground acceleration, a (in g , the gravitational acceleration), to peak ground velocity, v (m/s), represents a measure of the frequency content of the seismic ground motion (Heidebrecht et al., 1983; Heidebrecht and Lu 1988), NBCC 90 uses three ranges of a/v ratio to define different force coefficients in low period region (i.e., $T < 0.5$ sec). Elhmadi and Heidebercht (1991) also used frame structures to determine base shear. They drew the following conclusions. The NBCC 1990 foundation factor, F , underestimates the base shear force for many site types. Since the factor F is defined as a constant, the resonance effect at the site period, T_s , is not taken into account in NBCC 1990. They proposed a F -spectra which has a *trapezoidal* shape, with a maximum value in the neighbourhood of $T/T_s = 1.0$, to account for quasi-resonance between soil and structure. They divided possible soil conditions into five classes: (i) *deep cohesive(class 1)*, (ii) *deep cohesionless(class 2)*, (iii) *shallow cohesive and cohesionless (class 3)*, and (iv) *dense sand (class 4)*. Class 5 was reserved for ground conditions that would not produce amplification. Rock site was assumed in this class. The F spectra were independent of the a/v ratio of the earthquake motion, for both classes 1 and 2. For class 3 and 4, however, the F spectra were functions of the ratio a/v . The peak values, F_{max} , were equal to 2.5 for class 1 and to 1.5 for class 2. Values of F_{max} for both class 3 and class 4 were equal to 2.5 for high a/v ratio, 2.0 for intermediate a/v , and 1.5 for very low a/v . As it can be seen these F_{max} values are larger than those that one would obtain for the NBCC 1990 F factor (see Table.6.4).

An investigation for seismic response of structures located on soft clay sites was performed by Heidebrecht et al., (1990). The objectives were to determine the site effects at several soft soil sites in countries other than Mexico, to compare these with the Mexico City observations. In that study the soil deposits were modelled as one-dimensional layered systems with propagation of shear waves only in the vertical direction. The computer program SIREN was applied. To determine the influence of the characteristics of the exciting seismic motions, three ensembles of actual strong motions, each containing 15 time histories (recorded on rock or stiff soil sites) were used as input excitations to the soft soil sites, including Mexico City. They concluded that:

1. Spectral amplification is most significant when the seismic excitation has significant energy in the site period region; however, substantial amplification at structural periods near site periods are likely to occur for any kind of excitation.
2. The spectral amplification decreases as the intensity of excitation increases.
3. Peak values of spectral ratios in the neighbourhood of site periods are approximately 5 for velocity of 0.05 m/s and reduce to approximately 3 for velocity of 0.40 m/s.
4. The NBCC 1990 foundation factor of 2 is adequate for low- and medium-period input excitation , but is inadequate for high-period excitations at low levels of peak velocity; 0.05, 0.10 m/s.

The investigation of site response effects has been limited mainly to soft soil sites in view of the amplification observed in Mexico City during the september 1985 Mexico earthquake(Seed et al. 1987; Romo and Seed 1986). When sand sites are studied, the

analysis is usually based on the evaluation of the liquefaction potential (Seed 1979). Liquefaction has been the major concern at sand sites, as observed in the Niigata earthquake (Ohsaki 1986) and the 1964 Alaska earthquake (Seed 1968). During the 1971 San Fernando earthquake the damage to residential buildings correlated well with soil (sand) depths less than 18 m (Seed et al., 1987). This indicates that significant site response effects occurred, contrary to the belief that such sites vibrate in unison with the underlying rock or at worst amplify the rock motion only slightly. In a recent study (Balendra and Heidebrecht 1986), analytical predictions have shown that considerable amplification could be expected at sand sites.

To study the site response effects for structures located on sand sites, Henderson et al., (1990) considered several natural and artificial sand sites. They used the same nonlinear one-dimensional model and the same exciting seismic motions as Heidebrecht et al., (1990) did for soft soil sites. They used G/G_0 and damping ratios vs. shear strain in sands as illustrated in Fig.2.3.

According to the results of their study, they concluded that site amplification effects must be taken into account for the sand sites. The following are some of their conclusions:

1. Considerable spectral amplification occur for structural periods near the site period.
2. The spectral amplification decreases as the intensity of excitation increases.
3. Peak values of spectral ratios near the site periods are between 2 and 5.
4. The NBCC 1990 foundation factor of 1.0 and 1.3 are reasonably adequate for

high-period excitation. However, for low- and intermediate- period excitations, these factors are inadequate, especially for the shallower sites and low levels of peak velocity.

2.6 SUMMARY AND DISCUSSION

There is increasing evidence of the correlation between damage caused by strong-ground motions and the local geology. The destruction (site amplification) has been reported larger on soft clay or deep cohesionless soils than hard soils or rock. Previous studies have mainly focused on the amplification in soft media subjected to low frequency seismic ground motions. Such studies have overlooked the large amplifications in short period sites while subjected on high frequency seismic ground motions.

All computational models simulate the seismic motions in a soil deposit as the upward propagation of shear waves from underlying formation. Basic procedures of computations in one- and two-dimensional models introduced in SHAKE, SIREN, and FLUSH programs were considered, although they will be analyzed in more details in chapter 3.

Acceleration time histories are the only available way of simulating the ground motions and predicting soil-structure responses. They vary in their source mechanism, intensity, duration, and frequency content.

After Mexico City destructive earthquake (1985), NBCC increased the design base shear up to 33% especially for deep soft soils. Many studies were performed, to evaluate these increases, resulting in almost different conclusions of conservatism, sufficiency, or

even inadequacy in NBCC 90 foundation factor values. Some introduced foundation factors which were dependent on not only the frequency content of ground motion but also the proximity of the fundamental period of building and site period. One-dimensional approach of analysis was adopted in all the mentioned studies regardless of the interactions between soil and structure as the two substructures of one system. In addition, this question can arise that except for the different recommendations in previous studies regarding the increase of the design base shear via increase of the foundation factor, "is all of the underestimation in base shear in NBCC only due to low values of foundation factor or it can be due to the underestimation of else parameters, as well?".

In this study the main concerns would be first; study of site amplification resulting in seismic base shear and foundation factor for different soil types subjected to earthquakes with a wide range of a/v using two- and three-dimensional computational models. Secondly; study of seismic soil-structure interaction effects on site response. Thirdly; considering the validity of quasi-resonance between soil and structure in the proximity of fundamental period of building and site period. Also, the effects of other parameters whose functions may influence structural base shear, such as seismic response factor, S will be considered.

Fig.2.1a Average acceleration spectra for different site conditions (After Seed et al., 1976).

Fig.2.1b Average horizontal acceleration amplifications for four soil categories (After Mohraz 1976).

Fig.2.2 Small strain shear moduli profiles for NC, OC, AS, and DS sites (After Elhmadi and Heidebercht 1991).

Fig.2.3 Shear modulus and damping relationships
(After Elhmadi and Heidebercht 1991).

Chapter 3

ENGINEERING MODELS

FOR

DYNAMIC SITE RESPONSE ANALYSIS

3.1 INTRODUCTION

The investigation of the seismic site response requires an analytical model capable of accounting for the complexities involved in soil conditions and input rock motions. In general, site-response problems are ill-posed, and unique solutions can only be obtained by the introduction of restrictive assumptions leading to the development of the required models.

The analyses performed in this study are for soil-structure systems with horizontal soil layers overlying rigid rock, and subjected to horizontal seismic rock motions. Two of the existing engineering models will be presented in the next sections. The first model is based on the one-dimensional approach which has been used popularly among

practising engineers, in spite of the associated restrictions. The second is based on the finite-element method which provides fairly accurate analytical solution to the dynamic-response problem of the site and soil-structure system.

3.2 THE ONE-DIMENSIONAL MODEL

The current approach to most site-response problems is to use the one-dimensional model, for simplicity, regardless of how far the soil may extend horizontally. This model can be valid when lateral irregularities are far enough so that the effects of sloping rock boundaries or differences in soil stratifications can be neglected, or when the soil-structure interaction effects are not to be taken into consideration.

3.2.1 Assumptions

The following are the main assumptions used in the one-dimensional analysis:

1. The ground motions developed near the surface of a soil deposit during an earthquake may be attributed primarily to the upward propagation of shear waves from an underlying rock formation (Kanai, 1952; Idriss and Seed, 1968).

2. The ground surface, the rock surface and the boundaries between soil layers are horizontal and extended infinitely in the lateral direction.

In such cases, the ground motions induced by the seismic excitation at the base are only the result of shear deformations in the soil, and the deposit may be considered as a one-dimensional beam. The one-dimensional model can also predict the surface vertical response under the assumption of upward propagation of P-waves.

Further idealized conditions may be assumed to obtain a simple mathematical solution. These assumptions involve treating the soil continuum as a linear visco-elastic system consisting of layers, each of which has homogeneous and isotropic material properties (Schnabel et al., 1972).

3.2.2 Methods of Analysis

The one-dimensional model can be used to obtain the resulting motion at a point on the soil surface by considering the response of a soil column located underneath that point, as shown in Fig.3.1. The following two methods may be used for analysis:

a) The finite-difference approach (lumped masses model), suggested by Seed and Idriss (1969), where the soil column is discretized into lumped masses, one at each layer boundary, connected by shear springs. These springs represent the shear stiffness properties of the soil between any two discrete masses. The approach is idealized in Fig.3.2a.

b) The wave-propagation approach (continuous model) which is based on the solution of the wave equation and shown in Fig.3.2b. This approach was presented by Tsai and Housner (1970).

The computer program SIREN uses the first method. It was developed by Pappin (Henderson et al., 1989a) to solve the one-dimensional problem in the time domain using finite differences in a true non-linear model. In the SIREN program, the soil is represented as a series of lumped masses connected by non-linear springs. The non-linear springs are formulated as an assemblage of elastic-plastic springs which exhibit hysteretic

damping in accordance with the Massing Principles (Pyke, 1979). No viscous damping is included in this model. Transmission of energy at the rock soil interface is accomplished by adjusting the base input velocity in proportion to the shear stress existing at the interface (Papastamatiou 1973) using the formula

$$v_{bi} = v_{si} + \frac{\tau_i}{(v_r \rho_r)}$$

where v_{bi} is the base input velocity at time t_i , v_{si} is the specified input velocity at time t_i , τ_i is the shear stress at the rock-soil interface at time t_i , v_r is the shear wave velocity of the rock, and ρ_r is the density of the rock.

Analytical procedures in the SHAKE program are based on the concept in the second method incorporating non-linear soil behaviour in a linear iterative scheme in the frequency domain introduced by Schnable et al., (1972). The program is based on the continuous solution to the wave equation for vertical propagation of shear waves, as shown in Fig.3.2b. Hysteretic damping in the soil is included in the complex shear modulus terms. The system transfer functions are obtained in the frequency domain multiplied by the Fourier transform of the input bedrock motion to obtain the Fourier transform of the surface motion. An inverse transformation then gives the time history of the output motion. The non-linearity of the shear modulus and damping ratio is accounted for by the use of equivalent linear soil properties (Idriss and Seed, 1968 ; Seed and Idriss, 1970) using an iterative procedure to obtain values for shear modulus and damping compatible with effective strains in each layer.

3.3 FINITE ELEMENT MODEL

3.3.1 Basic Requirements for Finite Element Analysis

Analyses of soil-structure interaction effects during earthquakes are usually made by one of two methods: (a) a complete interaction analysis involving consideration of the variation of motions in the structure and in the adjacent soil, or (b) an inertial analysis in which the motions in the adjacent soil are assumed to be the same at all points above foundation depth. Like all analyses, they can be performed with different degrees of approximation or sophistication. Lysmer et al., (1975) present the following basic requirements of a good analytical procedure.

1. The analysis should consider the variation of soil characteristics with depth.
2. The analysis should consider the non-linear and energy-absorbing characteristics of the soils.
3. For embedded structures, the analysis should consider the variation of ground motions with depth.
4. The analysis should be capable of taking into account the three-dimensional nature of the problem.
5. The analysis should be capable of considering the effects of adjacent structures on each other.

In some cases, however, it may not be necessary to meet all of these requirements. For example, where a simple structure is involved, accurate evaluations of the motions at the base of a structure can be obtained using a two-dimensional analytical model. In general, all of the requirements listed above should be taken into account.

One of the primary arguments against the use of finite element methods of analysis is their high cost. This of course depends on the efficiency of the computer program used. In the recent past, analyses of this type have been substantially more costly than analyses using the inertial interaction approach in conjunction with half space theories. Although savings may be realized by virtue of lower degree of conservatism involved in the finite element approach, it is clearly desirable to reduce the analytical costs to the fullest extent possible.

3.4 FINITE ELEMENT METHOD USED IN FLUSH PROGRAM

3.4.1 General

A complete analysis of soil-structure interaction problem would involve the determination of the response of a structure when it is subjected to earthquake ground motions which vary from point to point in the soil and rock around and underlying the structure and travel in some unknown way across the base of the structure.

This admittedly complex problem is usually idealized for purpose of analysis so that motions in the vicinity of the structure are considered to be due to vertical propagation of body waves from underlying stiffer foundations. This is clearly an approximation and justified only on the ground that it is believed to be sufficiently accurate for engineering purposes.

On the basis of the above approximation, finite element analysis procedures satisfying the first three of the basic requirements for an adequate procedure have been available for several years. The basic method of approach is illustrated in Fig.3.3 (Lysmer et al., 1975). A control motion, specified at some point in the free-field soil profile, can

be deconvoluted to determine the corresponding motions at some depth, such as a soil-rock interface. One-dimensional amplification theory can be used for this purpose (Schnabel et al., 1972). Next the motion computed at this depth is used as input to a finite element model of the soil structure system, and the response computed at points of special interest (Seed et al., 1975). Another method of approach is to compute transfer functions relating the motions and forces at desired points in the soil or structure to the control motion applied at a point on the surface of the soil well away from the structure (Kausel, 1974). In either case, the analysis should be performed iteratively to allow for the strain dependent nature of the non-linear soil characteristics (Seed and Idriss, 1967; Schnabel et al., 1972). In each iteration the analysis is linear but the soil properties are adjusted from iteration to iteration until the computed strains are compatible with the soil properties used in the analysis.

Using this approach, different soil properties may be assigned to every element, if desired, so that there is no difficulty in considering the variation of soil characteristics with depth, while the iteration procedure permits consideration of the non-linear stress-strain and damping characteristics of the soils. In order to control the damping ratios to the desired values it has been found desirable to use the complex response method of analysis, and in the computer program FLUSH (Lysmer et al., 1975) has been widely used for analysis of this type.

FLUSH program is designed to solve both two- and three-dimensional soil-structure problems. To consider the three-dimensional features of a soil-structure system, Lysmer et al. (1975) used transmitting and viscous boundaries (Fig. 3.4). Viscous boundaries are

used along the planar surface of the slice of soil on which the building is located to simulate the radiation of seismic energy, away from the structure, in the third dimension. The thickness of this slice of soil is basically the length of the building in the third dimension. In addition, transmitting boundaries are specified at the vertical edges of the finite mesh to simulate the dynamic effects of the semi-infinite soil deposit beyond the finite element mesh. By making these assumptions, the structure and the underlying soil are confined within two transmitting boundaries and two viscous boundaries (Fig.3.4).

If the soil slice is made sufficiently long, then wave energy radiating along the axis of the slice will be absorbed by material damping while energy radiating in directions normal to the axis of the slice will be absorbed by viscous boundaries.

3.4.2 Computational Method

According to Lysmer et al. (1975), the computational model used in FLUSH program meets satisfactorily almost all of the mentioned basic requirements for an analytical model and has been checked for a large family of two- and three-dimensional seismic interaction problems. The analytical procedure is essentially two-dimensional and the equation of motion for a finite element representation of the system can be written:

$$[M] \{\ddot{u}\} + [K] \{u\} = -\{m\}\ddot{y} - \{V\} + \{F\} - \{T\} \quad [3-1]$$

where $\{u\}$ are the displacements of the nodal points relative to the rigid base, $[M]$ and

$[K]$ are the usual plane strain mass and stiffness matrices, respectively, of a slice of unit thickness and $\{m\}$ is a vector related to $[M]$ and the direction of the rigid base acceleration $\ddot{y}(t)$. Material damping can be included by forming $[K]$ from complex moduli.

The forces $\{V\}$ originate from the viscous boundaries on the planar sides of the slice.

$$\{V\} = \frac{1}{L} [C] (\{\dot{u}\} - \{\dot{u}\}_f) \quad [3-2]$$

where L is the thickness of the slice, $[C]$ is a simple diagonal matrix which depends on the properties of the free field, and $\{\dot{u}\}_f$ are the known free field velocities.

The forces $\{F\}$ act at the ends of the slice. They are merely the forces which act on a vertical plane in free field and they involve no horizontal transmission of wave energy. These forces are

$$\{F\} = [G] \{u\}_f \quad [3-3]$$

where $[G]$ is a simple frequency-independent stiffness matrix formed from the complex moduli in the free field.

The forces related to the energy transmission are

$$\{T\} = ([R] + [L]) (\{u\} - \{u\}_f) \quad [3-4]$$

where $[R]$ and $[L]$ are the frequency-dependent boundary stiffness matrices introduced by

Lysmer and Drake (1972) and Waas (1972). These matrices represent the dynamic effect of semi-infinite viscoelastic soil system, for example like those shown in Fig.5.5 and 5.6 at both ends of the model.

The equation of motion can be solved by the complex response method which assumes that the input motion can be written as a finite sum of harmonics, i.e, a truncated Fourier series

$$\ddot{y}(t) = \text{Re} \sum_{s=0}^{N/2} \ddot{Y}_s \cdot \exp(i\omega_s t) \quad [3-5]$$

where N is the number of digitized points in the input motion.

This implies that the response can also be written as Fourier series

$$\{u\} = \text{Re} \sum_{s=0}^{N/2} \{U\}_s \cdot \exp(i\omega_s t) \quad [3-6]$$

and

$$\{u\}_t = \text{Re} \sum_{s=0}^{N/2} \{U_t\}_s \cdot \exp(i\omega_s t) \quad [3-7]$$

The amplitudes $\ddot{Y}_s$ and $\{U_t\}_s$ can be found easily by the Fast Fourier Transform algorithm. Substitution of Eqs.[3-2] to [3-4] and corresponding terms of Eqs.[3-5] to [3-7] into the equation of motion, Eq.[3-1], yields:

$$([K] + [R])_s + [L]_s + \frac{i\omega_s}{L} [C] - \omega_s^2 [M] \{U\}_s =$$

$$- \{m\} \ddot{Y}_s + \{[G] + [R]_s + [L]_s + \frac{i\omega_s}{L} [C]\} \{U\}_s \quad [3 - 8]$$

which is a set of linear equations that determines the displacement amplitudes $\{U\}_s$ at each frequency ω_s , $s = 0, 1, \dots, N/2$. The equations can be solved by Gaussian elimination and the displacements in the time domain follow from Eq.[3-6] by the inverse Fast Fourier Transform. Further details of the solution procedure for Eq.[3-8] are given in section 3.5, NUMERICAL PROCEDURE.

3.4.3 Computational Model

The model consists of two types of real elements, displacement-compatible isoparametric quadrilateral elements (or solid elements) and linear bending elements (or beam elements). The latter are compatible only at their nodes where they have three degrees of freedom (two displacements plus one rotation) (see Figs.5.5 and 5.6).

The lower boundary of the model is assumed to be rigid and to translate horizontally or vertically according to some acceleration time history $\ddot{y}(t)$.

Transmitting boundaries must be vertical and the layer interfaces must coincide with the nodes on the boundary.

3.4.3.1 Mass Distribution

The mass matrix used for solid elements and in the free field computations in FLUSH program is the average of the lumped and consistent mass matrix. This distribution optimizes the ability of the element to transmit high frequencies (Lysmer et al., 1975).

Consistent mass matrices are used for beam elements. These matrices include the inertial effects of the rotational degrees of freedom.

In addition to the above distributed masses the finite element model may include concentrated masses at the nodal points.

3.4.3.2 Stiffness and Damping

All stiffness and boundary matrices in the complex equation of motion, Eq.[3-8], are formed using the complex shear moduli

$$G^* = G(1 - 2\beta^2 + 2i\beta\sqrt{1 - \beta^2}) \approx G \cdot \exp(2i\beta) \quad [3-9]$$

where β is the fraction of critical damping which may vary from element to element. It can be shown, by application to a simple damped oscillator, that for a system with uniform damping and no radiation damping (transmitting and viscous boundaries) the use of above complex moduli will lead to the same amplitudes as a modal analysis with the damping ratio β (Lysmer et al., 1975). A small error will occur in the phase of each mode but this is of minor importance for engineering applications (Lysmer et al., 1975).

Poisson's ratio is assumed to be real. This implies that P- and S-waves are assumed to have the same attenuation factor. This assumption may not be physically correct but for simplification has been made in FLUSH program.

3.4.3.3 Free Field Motions

The free field motions, $\{u\}_f$, appearing in Eqs.[3-2], [3-3], [3-4], and [3-7] are computed separately on the assumption that the free field consists of horizontal soil layers and that the seismic excitation consists of vertically propagating P- or S-waves. With these assumptions it is sufficient to consider a single column of elements because all points at the same level have identical motions. The computations are performed in the frequency domain in terms of the free field amplitudes (see Eq.[3-7]) which can be expressed as

$$\{U_f\}_s = \{A_f\}_s \cdot \ddot{Y}_s, s=0, 1, \dots, \frac{N}{2} \quad [3-10]$$

where $\{A_f\}_s$ is a vector containing the amplification values from the rigid base accelerations to layer displacements. The seismic input to the computer program FLUSH consists of a vertical or horizontal time history of acceleration digitized at N points at the constant time interval, Δt . This control motion can be specified to exist at any depth in the free field, say at the top of the jth soil layer. The program determines the rigid base accelerations from the inverse of Eq.[3-10], i.e.,

$$\ddot{Y}_s = U_{js} / A_{js}; s = 0, 1, \dots, \frac{N}{2} \quad [3-11]$$

where U_{js} and A_{js} are the j th components of $\{U_t\}_s$ and $\{A_t\}_s$, respectively. This process is known as deconvolution.

3.4.4 Numerical Procedure

3.4.4.1 The Method of Complex Response

Substitution of Eq.[3-10] into Eq.[3-8] gives the new equation of motion

$$[K]_s \{U\}_s = \{P\}_s \cdot \ddot{Y}_s \quad [3-12]$$

where $[K]_s$ is the frequency-dependent stiffness matrix

$$[K]_s = [K] + [R]_s + [L]_s + \frac{i\omega_s}{L} [C] - \omega_s^2 [M] \quad [3-13]$$

and

$$\{P\}_s = ([G] + [R]_s + [L]_s + \frac{i\omega_s}{L} [C]) \{A_t\}_s - \{m\} \quad [3-14]$$

is the load vector corresponding to unit amplitude of rigid base motion ($\ddot{Y}_s = 1$). In the above equations the subscript "s" indicates frequency dependence and the stiffness and damping matrices are formed from complex moduli to simulate viscous damping (Eq.[3-

9)).

The linear equations of Eq.[3-12] can be solved by Gaussian elimination. However, rather than solving Eq.[3-12] directly, it is convenient first to find the solution of

$$[K]_s \{A\}_s = \{P\}_s \quad [3-15]$$

where $\{A\}_s$ is the response corresponding to unit amplitude of rigid base motion. Since $[K]_s$ and $\{P\}_s$ are smooth functions of frequency, the components of $\{A\}_s$, called amplification functions, will also be smooth functions of $\omega = \omega_s$. After determining the amplification functions the complete response follows from Eq.[3-6].

$$\{u\} = \text{Re} \sum_{s=0}^{N/2} \{A\}_s \dot{Y}_s \exp(i\omega_s t) \quad [3-16]$$

which can be evaluated by the inverse Fast Fourier Transform algorithm.

3.4.4.2 The Frequency Domain

The frequencies, $\omega_s = 2\pi\nu_s$, at which the amplification functions have to be determined are defined by

$$\nu_s = \frac{s}{T}, s=1, \dots, \frac{N}{2} \quad [3-17]$$

where ν_s is the frequency in Hz and $T = N\Delta t$ is the total duration of control motion. In the computer program FLUSH the number, N , of discretized points in the control motion is implied to values which are power of 2 (Radix 2). This restriction has no inconvenience because it is always possible, and in fact desirable, to augment the earthquake by a string

of trailing zeros. This is so because the motion defined by Eq.[3-5] is periodic with the period $T = N.\Delta t$. Hence, in order to simulate the finite duration of actual earthquakes it is necessary to introduce a "quiet zone" at the end of each cycle to allow the viscous and radiation damping of the system time to attenuate the response from one cycle before the beginning of the next cycle. Fortunately, the damping of soils is high and the quiet zone usually needs to be only a few seconds long.

The highest frequency contained in the control motion is the "folding" frequency

$$v_{\frac{N}{2}} = \frac{1}{2\Delta t} \quad [3-18]$$

which, for a motion with say 2048 digitized points including trailing zeros and time interval 0.005, would be $1/(2*0.005) = 100$ Hz. Such high frequencies are usually not of interest and can be neglected by setting the high frequency terms of Eq.[3-15] equal to zero. This is done in FLUSH program by the introduction of a cut-off frequency, v_{max} . For example, for the mentioned hypothetical control motion and say $v_{max} = 20$ Hz, only $20*0.005*2048 = 205$ solutions of Eq.[3-15] are required.

3.4.4.3 The Equivalent Linear Method

The above solution procedure makes extensive use of superposition and is therefore, strictly speaking, applicable only to linear visco-elastic systems. However, the large shear deformations which occur in soils during strong earthquakes introduce significant nonlinear effects. This problem has been solved by Seed and Idriss (1969) by introduction of equivalent linear method. According to this method an approximate nonlinear solution

can be obtained by a linear analysis provided the stiffness and damping used in the analysis are compatible with the effective shear strain amplitudes at all points of the system. Data on strain-compatible soil properties are usually called *material curves* (e.g., Seed and Idriss 1970; Pappin et al., 1989). Any numbers of curves developed for specific materials may be input to program FLUSH which proceeds as follows:

A set of shear modulus and damping values is estimated for each soil element of the finite element model. This system is analyzed using these properties and the maximum shear strain time history is computed in each element of the model, Eq.[3-20]. From these time histories the effective shear strain amplitudes are estimated in each element, using Eq.[3-19], and the appropriate material curves are consulted to see if the strain level is compatible with the values of shear moduli and damping used in the response evaluation. If the soil properties are not compatible, the improved values of shear moduli and damping are entered for the next iteration and the process is repeated until convergence has occurred, usually within 3 to 5 iterations. The response from the last iteration is taken as being the nonlinear response.

3.4.4.4 Effective Shear Strain Amplitudes

The effective shear strain amplitudes used in the equivalent linear method are taken as

$$\gamma_{eff} = 0.65 * \max_c |\gamma_{max}| \quad [3-19]$$

The factor 0.65 in above equation is purely empirical. However, due to the relatively small slope of material curves the final motions are not sensitive to moderate variations

in this factor or in the estimate of the maximum shear strain. Time histories of ϵ_x , ϵ_y , and γ_{xy} and therefore $\gamma_{\max}$ are computed by the basic relation

$$\gamma_{\max}^2 = (\epsilon_x - \epsilon_y)^2 + \gamma_{xy}^2 \quad [3-20]$$

3.5 SUMMARY AND DISCUSSION

Three models of analyzing soil-structure systems were reviewed. The assumptions and methods of analysis of the three methods were discussed. There is a common assumption in all the models that vertical propagation of shear waves from bottom formation causes the surface motion. Two of the models are capable of solving only one-dimensional problems, whereas the third does take into account the two- and three-dimensional nature of problems. The simple one-dimensional approaches are based on uncoupled analysis of the soil-structure systems, i.e., neglecting soil-structure interaction effects.

Basic requirements of a computational method have been introduced and the model used in FLUSH program meets satisfactorily almost all of requirements. In this program energy absorbing nature of soil media is managed by introducing viscous and transmitting boundaries which confine the soil-structure system. FLUSH program takes into account the non-linear behaviour of soil imposed by strong earthquake motions. This is achieved by the combination of equivalent linear method and the method of complex response in frequency domain. The input excitation to the finite element model in FLUSH program is specified by an acceleration time history at rigid bedrock.

Fig.3.1 One - dimensional soil column

Fig.3.2 One - dimensional models for a system of N soil layers

(a) Soil Deposit Model

(b) Finite Element Model of Soil-Structure System

Fig.3.3 Schematic representation of soil-structure interaction analysis using finite element model
(After Lysmer et al., 1975).

Fig.3.4 Schematic view of a simplified 3-dimensional model with transmitting and viscous boundaries (After Lysmer et al., 1975).

Chapter 4

DESCRIPTION OF SITES AND INPUT MOTIONS

4.1 Introduction

The key parameters that are expected to govern the free-field seismic response of the soils overlying bedrock are presented in this chapter. These include soil type and site thickness, as well as the frequency content and intensity level of earthquake ground motions.

4.2 Soil Categories

Four categories of homogeneous soil deposits considered in this study are normally to lightly over-consolidated clay (NC), heavily over-consolidated clay (OC), alluvial sand and silt (AS), and dense sand (DS). For each soil model, two depths are included to represent shallow and deep soil deposits. Based on the study by Elhmadi and Heidebrecht (1990), it appears that the level of site effects is independent of the depth of a uniform soil deposit beyond a depth of 40 m. This is due primarily to the fact that as bedrock motions propagate upwards through the soil deposit, they are mainly influenced by the

low shear wave velocity of the upper layers of the deposit. As a result, 40 m is the depth chosen to represent deep soil deposits in the current study as, it maintains a reasonable bound on the size of the numerical model while providing a reliable representation of site effects for much deeper deposits. 10 m is the depth chosen to represent shallow soil deposits. This is based on the view that shallower soil deposits would not allow for the full development of soil-structure interaction effects due to the proximity of foundation and the bedrock (Wolf, 1985).

The soil properties required for site response analysis are based on measured data from actual sites located in eastern Canada, western Canada, and the United Kingdom (Elhmadi and Heidebrecht, 1990). The results of the measurements from the mentioned sites are summarized in Figs.2.2 and 2.3.

The soil properties required for dynamic soil-structure interaction analysis by any of the engineering models discussed before are:

- (a) Mass density of soil deposits, ρ
- (b) Initial shear modulus, G_0 , as a function of depth
- (c) Secant shear modulus, G , and hysteretic damping ratio β as a function of the shear strain, γ .

The mass density values ρ and G_0 profiles for the above four soil categories are shown in Fig.4.1. It should be noted that the zero G_0 at the ground surface of the (NC) profile makes the top soil layers very soft. This, in turn, sets up a resonant oscillation which is a function of the discretization and is not a function of the soil deposit. For this reason, a modification was made, by Elhmadi et al. (1990), for the G_0 profile in the upper

1 m, as shown in detail A in Fig.4.1, such that the profile is taken as constant and equal to G_0 at $z = 1$ m. Ground surface G_0 in cohesionless soils is theoretically supposed to be zero because of zero confined pressure at soil surface. On the other hand, due to impurity site sands as well as the crust effect, the G_0 may be measured nonzero. In addition, due to the effect building and its embedment, the G_0 at the soil-structure interface would be nonzero any way.

4.3 Strain Dependent Soil Properties

In addition to the soil bulk density, the soil-structure interaction analyses require a knowledge of the soil shear stiffness and energy dissipation or damping characteristics. In the following a discussion of these parameters for clays and sands is included.

Sun et al. (1988) have summarized recent research in the shear stiffness properties of clays and have presented various curves showing G/G_0 (the ratio of shear modulus G at shear strain amplitude γ to the small strain shear modulus G_0) plotted against shear strain amplitude for clays with varying plasticity indices (I_p). A typical set of curves is shown in Fig.4.2 and they indicate a clear trend of increasing G/G_0 with increasing I_p .

It follows that if G/G_0 vs. γ is dependent on I_p , then the internal hysteretic damping vs. γ relationship should also be dependent on I_p . Fig.4.3 shows results for clays from various sources. While it is seen that the Mexico City clay, which has an I_p of about 300%, has much lower damping, as expected, the other lines are all similar and are for I_p between 20 and 60%. As the I_p value reduces to 0% it is likely that the material will behave in the same way as a sand.

Figs.4.4 and 4.5 show the G/G_0 curves and the related hysteretic damping curves

used for the clay soil sites discussed in this study. A detailed derivation of these relationships is described in Pappin et al. (1989), Heidebercht et al. (1990), Henderson et al. (1990). According to Heidebrecht et al. (1990) the *clay* curves represent a curve suitable for a clay with an I_p between 30 and 50%.

Fig.4.6 shows a variety of G/G_0 versus strain amplitude curves that have been measured for sands. Bolton and Wilson (1989) have recently shown that sands exhibit a purely hysteretic response when tested in a resonant column apparatus. They have found that the behaviour of sand to cyclic loading is frequency or rate independent and that all energy absorption can be demonstrated to originate from hysteretic damping. Given the variability in Fig.4.6, this implies that the G/G_0 versus γ relationship should be chosen to be compatible with the observed damping.

Seed et al. (1984) concluded that for sands the damping can be represented purely as a function of shear strain amplitude and Fig.4.7 shows their recommended function and various results. Recent work (Stokoe et al., 1986) has shown that the damping may be lower and the lower bound line may be more appropriate, as shown in Fig.4.7. Figs.4.4 and 4.5 show the G/G_0 curve and the related hysteretic damping curve used for the sand soil sites discussed in this study. A detailed derivation of these relationships is described in Pappin et al. (1989), Heidebercht et al. (1990) and Henderson et al. (1990).

4.4 Estimating Low-strain Site periods, T_s , Using Closed Form Solutions

A study was made of simplified procedures for estimating the fundamental period, T_s , of a linear or equivalent linear model of a soil profile (Dorby et al. 1976). Closed-

form-solutions for computing the first period of an elastic shear-beam representation of the profile were found to give reliable values. Closed-form-solutions for homogeneous soil deposits are presented in chart form for the following cases:

(a) Shear wave velocity increasing as a power of depth ($V = V_o Z^{p^2}$). AS and DS soil profiles are included in this category (see Fig.4.1).

(b) Shear modulus increasing or decreasing linearly with depth. NC and OC soil profiles belong to this category (see Fig.4.1).

Fig.4.8 shows the corresponding small-strain shear wave velocity plotted against the depth for the soil deposits discussed in the current study.

Deposits With $V = V_o Z^{p^2}$. This case (Fig.4.9) was presented for constant ρ (soil density) through the depth. It is representative of normally consolidated uniform soil deposits; values of ρ between 0.5 and 1.0 are typically used (Dorby et al., 1976). The fundamental period, T_s , is expressed as $T_s = 4H/V_{eq}$, where V_{eq} is the value of shear wave velocity at the "equivalent depth", Z_{eq} . The solution for this case is plotted in Fig.4.9 as a graph of Z_{eq}/H versus ρ . Table 4.1 shows the results for the AS and DS soil profiles.

Deposits with G_o varying linearly with depth. This case was presented again for constant ρ . In this case G_o is increasing with depth (Fig.4.10). Fundamental period can be derived as $T_s = 4H/V_{eq}$ and V_{eq} is the value of shear wave velocity at the "equivalent depth", Z_{eq} . Based on the chart in Fig.4.10, Z_{eq} is derived where $k = (G_o/G_H)^{1/2}$. G_o and G_H are the moduli at the top and base of the layer. Table.4.2 shows the results for NC and OC soil profiles.

Results of T_s for the four soil categories are listed in Table.4.3.

4.5 Input Records In Rock Boundary

The parameters of the seismic excitation input in rock are of main concern in this study. They include the ratio of peak acceleration to peak velocity, a/v , and the peak velocity, v , respectively giving a measures of the frequency content and the intensity of motion in rock. The peak velocity of a record can be used as an indicator of the motion intensity level (Dorby et al., 1976), for given duration and frequency content of ground motions.

Despite all the improvements in the methods of analysis, the uncertainty in the input motions is still a major problem for the dynamic analysis. The same control motion acting in the same control point can arise from different types of seismic waves. In general, identifying the wave pattern and its contribution to a given control motion is not yet satisfactorily understood. Hence, it will be assumed sufficiently accurate for engineering purposes to select the control motion as a given accelerogram representing a horizontal component of excitation and acting along the rock boundary.

Two of the significant features of earthquake motions in rock are the frequency content and the peak amplitude of motion which together can be presented by means of a response spectrum. Previous investigations (Idriss, 1978) have shown a decrease in the ratio of peak acceleration, a (in g , gravitational acceleration), to peak velocity, v (in m/s), with increasing epicentral distance. This is due to the differences in the attenuation of acceleration and velocity ground motions. Hence, a/v is a measure of the frequency content of a recorded acceleration time history (Heidebrecht et al., 1983; Heidebrecht and Lu, 1988), where the high frequency components are filtered out as the waves travel

through the ground.

Based on the ratio a/v , a database of strong ground motions that include 45 horizontal accelerograms recorded on rock or stiff soil (Naumoski et al., 1988) were chosen from the McMaster University Seismological Executive (MUSE) Database System. They were classified into three ensembles each contains 15 records. Records with $a/v > 1.2$ belong to the high ensembles (H a/v). The second ensemble is termed intermediate (I a/v) and includes records with $0.8 < a/v < 1.2$. The third ensemble, termed very low (L a/v), contains records with $a/v < 0.6$. It should be mentioned that the 1990 edition of the National Building Code of Canada (NBCC 1990) uses three ranges of a/v ratio to define base shear coefficients:

H	High	a/v ratios ($Z_a > Z_v$)
I	Intermediate	a/v ratios ($Z_a = Z_v$)
L	Very Low	a/v ratios ($Z_a < Z_v$)

in which Z_a and Z_v represent the acceleration and velocity related seismic zones for a particular location, respectively.

The current study is carried out on a site-specific basis for three locations in Canada, namely Ottawa, Vancouver, and Prince Rupert. In other words, the applied acceleration time histories are so scaled that they satisfy the seismicity requirements, frequency contents and peak velocities at a 10 percent in 50 years probability of exceedance, of those locations according to seismic zoning maps of Canada (NBCC 90)

(see Figs.4.11 to 4.13 and Table.4.4).

Seismic hazard in Eastern Canada is associated with intraplate faulting within the American plate (Weichert, 1990). However, seismic hazard in Western Canada is dominated by either interplate faulting between the Pacific and American plates along the Queen Charlotte Transform or the subduction of the Juan de Fuca plate beneath the American plate (Milne et al, 1978). It is important to distinguish between these three major sources of seismic hazard in establishing site-specific ground motion simulations because of observed differences in earthquake ground motion attenuation rates in Eastern and Western Canada.

In addition, there exist significant differences in the strain release patterns, and consequently the maximum earthquake magnitudes that can occur, for the two sources of seismic hazard in western Canada. The three locations included in the current study are selected such that the seismic hazard for each location is associated with a different source.

PHA and PHV are defined to be the peak horizontal ground acceleration and velocity for the bedrock motions, expressed as non-dimensional ratios of 1 g and 1 m/s, respectively. Zoning of PHA and PHV is based on determination of the values at a 10 percent in 50 years probability of exceedance. In view of the significance of the PHA/PHV ratio on the response of structures and soil deposits, the selected locations also represent three combinations of Z_a and Z_v , namely $Z_a > Z_v$, $Z_a = Z_v$, and $Z_a < Z_v$. Table.4.4 provides a listing of the seismic hazard parameters associated with the three locations included in the current study. Information on the seismic hazard computations,

used in the development of the seismic zoning maps of Canada, for the three locations is provided by Basham (1985). Based on this information, the earthquake magnitude (M_o) and source-distance (r_o) combinations dominating the seismic hazard, at the three locations, are also listed in Table.4.4.

For each city, one ground motion time history is so selected that it produces PHA/PHV (i.e., a/v) ratio as specified in the seismic hazard zoning maps of Canada (NBCC 90). It is then scaled to satisfy the required seismic hazard parameters of the city. Hence, the intensities, measured in terms of PHV, of the modified time histories match the respective code values listed in Table.4.4.

The description for the selected accelerograms are presented in Table.4.5 and Figs.4.11 to 4.13 show the related acceleration time histories selected for the three locations. It should be mentioned that the real duration of Ottawa-related acceleration time history is 50.96 seconds, but due to the limitation in the number of data points in terms of printer plotting (almost only 1500 data points) this particular time history is shown only for its first 30 seconds (Fig.4.11), while the actual duration of time history was used in the analysis.

The response spectrum of a single-degree-of-freedom system with 5% damping, subjected to the above three cities-related acceleration time histories are illustrated in Fig.4.14. These spectra are all scaled for the peak velocity of $v = 1$ m/s.

These acceleration time histories are used as horizontal input excitations in the rock boundary, and they cover a wide frequency ranges of earthquake records.

Soil Type	Depth (m)	ρ	z_{eq}/H	V_{eq} (m/s)	T_s (sec)
AS	10	0.74	0.615	141.4	0.28
	40	0.74	0.615	193	0.83
DS	10	0.54	0.63	287	0.14
	40	0.54	0.63	410	0.39

Table.4.1

Soil Type	Depth (m)	$G_{(H=0)}$ (MPa)	$G_{(H=H)}$ (MPa)	k	z_{eq}/H	V_{eq} (m/s)	T_s (sec)
NC	10	1.2	18	0.19	0.6	93	0.43
	40	1.2	65	0.14	0.6	152	1.01
OC	10	40	55	0.85	0.685	152	0.26
	40	40	108	0.61	0.66	229	0.70

Table.4.2

Soil Type	$T_{s(H=10)}$ (sec)	$T_{s(H=40)}$ (sec)
NC	0.43	1.01
OC	0.26	0.70
AS	0.28	0.83
DS	0.14	0.39

Table.4.3

Site	PHA	PHV	Source	M _o	r _o (km)
Ottawa	0.20	0.10	Intraplate	6.6	65
Vancouver	0.21	0.21	Subduction	7.0	60
Prince Rupert	0.13	0.27	Interplate	7.9	200

Table.4.4

Record	Site	Date	Magnitude	Max. Acc. (g)	Max. Vel. (m/s)	a/v	Soil Condition
H a/v	Lytle Greek	Sep. 12 1970	5.4	0.198	0.096	2.03	Rock
I a/v	Japan	Nov. 16 1974	6.1	0.70	0.072	0.97	Stiff Soil
L a/v	California	Feb. 9 1971	6.4	0.101	0.193	0.52	Stiff Soil

Table.4.5

Fig.4.1 Small strain shear moduli profiles for NC, OC, AS, and DS sites

Fig.4.2 Variation of G/G_0 for clays with different plasticity
(After Sun et al., 1988).

Fig.4.3 Damping ratios for clays (After Heidebecht et al., 199X).

Fig.4.4 Shear modulus variation with shear strain for various soils

Fig.4.5 Damping ratio variation with shear strain for various soils

Fig.4.6 Variation of G/G_0 with shear strain amplitude for sands
(After Henderson et al., 1990).

Fig.4.7 Damping ratios for sands (After Henderson et al., 1990).

Fig.4.8

2

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

Fig.4.9 z_{eq} for profiles with velocity increasing as a power of depth
(After Dorby et al., 1976).

Fig.4.10 z_{eq} for profiles with linear variation of shear modulus with depth
(After Dorby et al., 1976).

Fig. 4.11 High frequency seismic motion compatible Ottawa-related seismicity

Fig.4.12 Intermediate frequency seismic motion compatible with Vancouver-related seismicity

Fig.4.13 Low frequency seismic motion compatible with Prince Rupert-related seismicity

Fig.4.14 Acceleration response spectra of the three input motions

Chapter 5

SEISMIC SOIL-STRUCTURE INTERACTION EFFECTS ON SITE RESPONSE

5.1 QUALITATIVE CONSIDERATION OF SOIL-STRUCTURE INTERACTION EFFECTS

The seismic response analysis of a soil-structure system, with taking the flexibility of the underlying soil into account, is a considerably complicated problem. An exact solution is indeed impossible. The important aspects of soil structure interaction can, however, be analyzed using an appropriate model, such as the one is used in FLUSH program (Lysmer et al., 1975). To illustrate the interaction between soil and structure, the seismic response of a structure on rock is compared to that of the same structure, but founded on soil surface. In order to simulate the effects of the earthquake a horizontal motion that propagates vertically is selected. It is assumed that the incident seismic waves arriving from the source of the earthquake are the same for both structures.

For the structure on the rock, this (horizontal) motion can be applied directly to the base of the structure. The input acceleration resulting in the applied horizontal inertial

loads will be constant over the height of the structure (Wolf, 1985). During the earthquake, an overturning moment and a transverse shear at the base develop. As the rock is very stiff, these two stress resultants will not lead to any rocking motion (Dynamic rotation) at the base. The resulting horizontal displacement of the base is thus equal to the control motion. In other words, for a given control motion, the seismic response of the structure depends only on the properties of the structure.

However, for the structure founded on soft soil, the motion of the base of the structure at point O (Fig.5.1) will be different from the control motion (bedrock motion) because of the coupling of the soil-structure system. To gain insight into how the soil affects the dynamic response of the structure, it is worthwhile to distinguish the three following effects. First, the motion of the site in the absence of the structure and any excavation, what is called the free field response, is modified (Fig.5.1). The presence of soil layer will reduce the motion in the bedrock. This wave will propagate vertically through the soil layer, resulting in motions in O' and ground surface which differ from that in bedrock (Fig.5.1). Point O' is in the free field and will lie on the soil-structure interface (the base) when the structure has been built. In general, the motion is amplified (depending on its frequency content), thus resulting in horizontal displacements that increase toward the free surface of the site. Second, excavating and embedding the rigid base into the site will modify the motion. The rigid base will experience some average (horizontal) displacement and a rocking component. This rigid body motion will result in accelerations (leading to inertial loads) which will vary over the height of the structure, in contrast to the applied accelerations in the case of a structure founded on rock (Wolf,

1985). This geometric averaging of the seismic input motion will be the result of the so-called *kinematic-interaction* part of the analysis (Wolf, 1985). Third, the inertial loads applied to the structure will lead to an overturning moment and a transverse shear acting at point O (see Fig.5.1 and 6.19). These will cause deformations in the soil and, thus, once again, modify the motion at the base. This part of the analysis will, therefore, be referred to as *inertial analysis* (Wolf, 1985).

Taking soil-structure interaction into account will result in the following main effects. First, the seismic-input motion acting on the structure-soil system will change. Because of the amplification of the site (free-field response), the translational component will in many cases be larger than the control motion and , in addition, a significant rocking component will arise for an embedded structure. Each frequency component of the motion is affected differently, resulting in, for example, an acceleration time history which is quite different from the control motion. This amplification of the seismic motion is held responsible for the fact that structures founded on a deep soft-soil site have been damaged more severely in actual earthquakes than have neighbouring structures founded on rock. Second, the presence of the soil in the final dynamic model will make the system more flexible, decreasing the fundamental frequency to a value which will, in general, be significantly below that applicable for the fixed-base structure. The implication of this reduction will depend on the frequency content of the seismic-input motion. In certain cases, the fundamental frequency will be moved below the range of high seismic excitation, resulting in a significantly smaller seismic input "felt" by the structure (Wolf, 1985). The shape of vibrational mode will also be changed. The introduced rocking of the

base will affect the response, specially at the top of a tall structure. Third, the radiation of energy of the propagating waves away from the structure will result in an increase of the damping of the final dynamic system. For a soil site approaching an elastic-half space, this increase will be significant, leading to a strongly reduced response (Wolf, 1985). For a soil site with shallow thickness, it is possible that no waves propagate away from the structure. In this case only the material damping of the soil will act. In any soil-structure analysis, it is very important to determine for a specific site whether the loss of energy by radiation of waves can actually take place. In essence, the soil structure interaction increases the more flexible the soil is and the stiffer the structure is. On the other hand, it will be negligible for a flexible structure founded on firm soil.

It has been questioned whether or not the interaction effects will increase the seismic response. If, however, the first effect discussed above (the change of seismic-input motion) is neglected when the interaction analysis is performed, the response will in many circumstances be smaller than the fixed base response obtained from an analysis that neglects all interaction effects (Wolf, 1985). Neglecting the interaction effects implies that the structure is founded on a rigid base moving with the ground surface motion. Real foundations have both flexibility and damping capacity, which change structural response. The flexibility of foundation increases the fundamental period of a soil-structure system and damping capacity arises from dissipation of energy by radiation away from the structure and the hysteretic damping in the foundation, therefore increasing the effective damping of the structure. This, however, will result in a conservative design (Wolf, 1985).

5.2 STRUCTURAL MODELS

The type of framing and the relative contribution of the higher modes of vibration are the basic parameters affecting the seismic response of structures (Fenves and Newmark, 1969). Accordingly, it is believed that sufficiently reliable results, representative of the response of actual structures, can be obtained by studying the response of two extremal structural models, so-called the *moment-resisting frames* and *shear walls* (Heidebrecht and Stafford-Smith, 1973).

On the other hand, a study was conducted using simple continuum models for both frame and wall structures to determine the base shear force, V_s , for the motion at the surface (Heidebrecht and Stafford Smith, 1973). Five modes and 5% modal damping were used in the analysis. It was observed that the frame structures results exceeded the values computed for wall structures, especially in the low and intermediate a/v regions (Heidebrecht et al., 1990). Based on the foregoing, the moment-resisting frames are expected to represent the extreme of the dynamic response of regular multi-storey buildings. For this reason, concrete moment-resisting frames are considered as structural systems in the current study.

The response of symmetrical one-bay frames is considered satisfactory approximation to the response of actual multi-bay frames subjected to dynamic or static loads (Council on Tall Buildings and Urban Habitat, 1979). According to the shortage of computer time and space, it is considered appropriate to adopt one-bay frame models in this study.

To cover a wide range of actual multi-storey buildings with various fundamental

periods, it was decided to use 5-, 10-, and 20-storey buildings. The 5-storeys represent a regular building, the 10-storeys can be representative of a relatively tall building, and the 20-storeys as a high rise building. The span of these frame models is assumed to be 10 m. The storey height is taken as 3.60 m except for the first floor which is 4.5 m tall. The structural frames are assumed to be 6.0 m centre-to-centre in plan. The size of the structural models in the third direction was considered to be 30 m; the longer dimension of building in plan, in other words, 6 frames in the third direction.

The cross-section of columns was chosen identical for every two-sequential storeys. The cross-section dimensions in both x and y directions are reduced 5 cm in every two storey from bottom to top. In other words, for the first two storeys in 20-storey building a cross section of 100x100 cm and for the last two storeys, 19th and 20th, 55x55 cm (Razaqpur, 1993). The column size of 5- and- 10-storey buildings are identical to the top 5 and 10 storeys of the 20-storey building, respectively. In other words, in the 10-storey building the column size in the first two floors are 75x75 cm and ends to 55x55 cm for the 9th and 10th. In the 5-storey building, the first column size is 65x65 cm, the second and the third are 60x60 cm, and finally 55x55 cm for the fourth and fifth.

The beam length is 10 m and its cross-section is 100x50 cm in all floors and in all structural models considered in the current study. A 15-cm-thick-concrete slab covers all the floors and connects the frames. The buildings are located on the ground surface over a concrete mat foundation which is 1 m thick and is extended 2.5 m from the axes of columns (Razaqpur, 1993).

For convenience of classifying the soil-structure models, it seems appropriate to

name the models. All the model names start with "B" and a number which is 5, 10, or 20. "B" is the abbreviation of "Building" and the number immediately following "B" is the total number of storeys. Next, there is a dash, -, to separate the structural part from the soil part in the model name. After the dash, the foundation soil type appears, which is one of NC, OC, AS, and DS. If the building is located on bedrock and there is no foundation soil, then after dash there is a "ON.R" which is the abbreviations of ON ROCK. The last number in the soil-structure name is the depth of soil overlying bedrock, in meters. For example, B10-DS40, means that a 10 storey building is located on 40 m dense sand overlying bedrock. Figs.5.2 through 5.4 show the finite element models for 5-, 10-, and 20-storey buildings located on bedrock. Also, Figs.5.5 and 5.6 show two of the finite element models for a 5-storey building located on 10 m of soil and a 20-storey building located on 40 m of soil, respectively. The finite element characteristics of all of the models used in the current study are shown in Table.5.1. It is worth noting that the connections between the foundation and the column in the first floor are assumed to be fixed, i.e., no rotation. It should be mentioned that throughout dynamic soil-structure analysis, shear deformations in structural elements (beams and columns) were not taken into account due to their negligible effects on the final results.

Vertical loading was assumed to be 5 KPa, for all floors in all buildings, as dead load and no live load was considered. One of the major goals of this study is the study of "Foundation Factor" which is defined as the ratio of the computed base shear for the structure situated on a soil deposit to the corresponding base shear for an identical structure assumed to be directly situated on bedrock and subjected to the same bedrock

motions. According to this definition, vertical loading due to the weight of structure does not have a significant influence on the foundation factor, which is just a ratio of two horizontal base shear forces corresponding to an identical building.

The following is the characteristics of the concrete of which the structural elements and the mat foundation are made (Razaqpur, 1993; Humar, 1993).

$$\text{Poisson's ratio, } \nu = 0.18$$

$$\text{Young's modulus, } E = 5000(f'_c)^{1/2}$$

$$f'_c = 35 \text{ MPa}$$

$$G = E/(2(1 + \nu))$$

$$\text{Unit weight} = 24 \text{ KN/m}^3$$

$$\text{Damping ratio} = 0.05$$

5.3 SOIL MODELS

As it was mentioned in chapter 4, two soil depths are considered for the soil-structure models in the current study, 10 and 40 m, to simulate shallow and deep foundations. The soil deposits are assumed to be composed of horizontal layers with infinite length. To analyze the seismic response of a soil-structure system, by using FLUSH program, the soil deposit should be divided into finite elements. Also, to do free field analysis, the soil medium has to be divided into finite layers. In order to have more precise results, Lysmer et al. (1975) recommended that the vertical dimension of soil elements (or soil layers in the free field analysis) not to exceed $h_{\max}$, where

$$h_{max} = Vs/(5v_{max})$$

where V_s is the shear wave velocity of that particular soil element or soil layer in the last iteration and v_{max} is the highest frequency of analysis. In this study the highest frequency of analysis was considered to be 15 Hz. Typical values for an earth dam is 8 Hz and for a nuclear power plant 25 Hz (Lysmer et al, 1975). This restriction on the height of the soil element can be justified on the basis of accurately representing the wave propagation process. As the waves travel vertically in the soil deposit, they are largely influenced by the low shear wave velocity of the upper layers of the deposit. If the thickness of these layers are too large, some of high frequency components of motions may be artificially filtered out particularly in soft soils, such as the NC soil profile in this study. Therefore, the finite element meshes for all soil types are established based on the required sizes for NC, which is the softest soil in this study. In other words, if the element heights satisfy the above criterion for NC profile, they will be applicable for OC, AS, and DS soil profiles, as well. In order to determine the finite element mesh before knowing what is the value of shear wave velocity in the last iteration, Lysmer et al. (1975) suggested to perform a preliminary free field computations using programs SHAKE or FLUSH, the latter on a single column of elements representing the soil layers. To follow this recommendation, free field analyses were performed on 10 and 40 m deep deposits of normally to lightly overconsolidated clay, NC soil profile. In the case of the 40 m soil model, the thickness of the top layer, which is the thinnest, was considered to be 1.0 m and the bottom layer which is the thickest, was taken as 2.5 m. In the case of the 10 m

soil model, the thickness of top layer was taken as 1.0 m and the bottom layer as 1.70 m. The horizontal dimension of the elements can usually be chosen several times of the vertical dimension (Lysmer et al., 1975). This is so because the major part of the motion consists of vertically propagating wave with horizontal wave fronts. For all of the soil elements in both the 10 and the 40 m soil models, the horizontal dimension is considered to be 2.5 m. Figs.5.5 and 5.6 show two of the finite element meshes for the 10 and the 40 m deep soil models. Viscous boundaries are used along the planar surfaces of the slice of soil on which the building is located to simulate the radiation of seismic energy, away from the structure. In the third dimension, the thickness of the slice of soil is basically the width of the building in the third direction, specified as being 30 m in the current study. In addition, Lysmer-Wass transmitting boundaries are specified at the vertical edges of the finite element mesh to simulate the dynamic effects of the semi-infinite soil deposit beyond the finite element mesh. The FLUSH program uses 2-D plane strain elements to model the soil deposits and foundations.

5.4 INFLUENCE OF SOIL CONDITIONS AND STRUCTURAL TYPE ON THE SOIL RESPONSE

The dynamic soil properties and soil behaviour under different seismic excitations exert main effects on the response of soil-structure system. In the following the dynamic properties of the soils applied in the current study as well as the implications of their seismic behaviour are presented.

(1) The profile of initial shear modulus with depth. The initial shear modulus as

shown in Fig.5.7 is the modulus at very low shear strain amplitudes ($< 0.0001\%$). There are in situ and laboratory tests to determine such profiles. In the field, the cross-hole test or shear wave velocity test is normally used. In laboratory, resonant-column test using undisturbed samples may be used.

Fig.4.1 shows the G_0 versus depth for the soil deposits considered in this study. with dense sand, DS, having the highest values of shear modulus and normally overconsolidated clay, NC, the lowest. Alluvial sand, AS, and heavily overconsolidated clay, OC, have almost the same values of shear moduli. NC and OC soil profiles have an almost linear variation of G_0 with depth while AS and DS soil profiles do not follow such a trend.

(2) *Damping degradation with shear strain amplitude.* This is usually shown in plots whose x coordinate is in log-scale (shear strain-related axis). These curves usually are derived from conducting of a resonant-column test on an undisturbed sample in laboratory. Fig.4.5 shows the curves that are derived for the soil deposits used in the current study. According to these curves, maximum damping ratio, which is defined as the fraction of critical damping (or hysteretic damping), is 25% which occurs beyond shear strain amplitude of 1% for NC, AS, and DS soil materials and beyond shear strain of 1.5% for OC soil material. According to these curves, for the same shear strain amplitude, NC has the lowest damping ratio, AS and DS have the highest value of damping ratio, and OC is in between. Except for the NC profile, the damping ratios in the last iteration during analysis with FLUSH program, rarely exceeded 15% even when they are subjected to the Ottawa-related earthquake, H_a/v . This is because of the higher

initial shear moduli of OC, AS, and DS soil profiles, compared to NC profile, that result in lower shear strain in the last iterations, and in turn, lower damping. Subjected to low frequency Prince Rupert-related acceleration time history, in all soil profiles the strain level is lower than that of while subjected to the high or intermediate frequency motion, with shear strain usually less than 0.1%. In this strain range, NC has lower damping than the other soil types. In the next sections of this chapter, it is shown that stiff soils amplify the high and intermediate frequency motions more than the low frequency motions, whereas, soft soils (specially soft clay) amplify the low frequency motions more than high or intermediate frequency motions. One of the reasons can be due to the lower material damping in the last iteration in each case than that of the other soil types. Lower damping corresponds to lower energy dissipation, and in turn, to higher amplification.

When NC profile is subjected to H a/v motion (Ottawa site), a damping of even 20% may be encountered. This is because of high strain level (up to 0.8%) developed in the top layers with low shear modulus. Based on the effect of soil damping on amplification discussed in the preceding paragraph, this can be a reason why soft clay does not amplify a high frequency motion.

(3) *Shear modulus degradations, G/G_0 , with shear strain.* G is the secant shear modulus and is a function of shear strain (Fig.5.7). Fig.4.4 shows G/G_0 versus shear strain curves taken for the soil deposits used in the current study (Pappin et al., 1989). According to these curves, the maximum value of G/G_0 is unity at very low shear strains (< 0.0001 %) and the minimum is less than 0.06 at 10% or higher shear strains. According to these curves, for the same shear strain amplitude, NC has the highest value

of G/G_0 , AS and DS have the lowest value, and OC profile is in between. This means that with increasing shear strain the cohesionless soils, AS and DS, lose their rigidity more rapidly than the cohesive soils, NC or OC.

Poisson's ratio of the soil profiles is also used in the analysis but not as a dynamic property, because FLUSH program does not iterate this parameter during dynamic analysis. The applied Poisson's ratios in the current study were 0.45, 0.35, 0.20, 0.30 for NC, OC, AS, and DS, respectively (Law K.T., 1993).

5.4.1 Influence of The Soil Conditions on The Soil Response

For the soil-structure models applied in the current study, there are two soil depths (10 and 40 m), four soil types (NC, OC, AS, and DS) and three structural types (5-, 10-, and 20-storey buildings). These soil-structure models are subjected to the three related-acceleration time histories of the cities of Ottawa (H a/v), Vancouver (I a/v), Prince Rupert (L a/v). Therefore, a total of 72 dynamic analyses of soil-structure interaction are conducted. In order to be concise, the results of the study in this section are plotted only for the building types of 20- and 5-storey since these are the limits of building heights in the current study.

Figs.5.8 to 5.13 show the results of dynamic response analyses for the soil-structure models including the 20-storey building and Figs.5.14 to 5.19 illustrate the results for the cases including the 5-storey building. Part (a) of these figures show the variation of maximum acceleration with depth along the centreline (e.g., see Figs.5.1, 5.2 and 5.6). Part (b) of the figures show the response acceleration of a single-degree-of freedom

system located at the center of the foundation-soil interface (Fig.5.1).

Figs.5.8 to 5.10 show the results for the cases of the 20-storey building over 10 m deep foundation with different soil types subjected to H, I, and L a/v earthquakes, respectively. Applying H a/v (Ottawa site), Fig.5.8 shows that NC, OC, and AS have almost the same periods at their peak responses, whereas dense sand, DS profile, has shorter periods than that of the rest. It means that the existence of such a tall building, 20-storey, affects the response of DS less than that of the other soils. Also, OC which is firm clay has the highest response. According to part (a) of this figure, all soil types have amplified the bedrock motion with the NC profile yielding the lowest amplification. This is because in the NC soil profile, the top layers are too weak to transfer the high frequency components of the bedrock motion. The AS and OC profiles under high frequency excitations behave similarly.

Fig.5.9 shows the results when the 20-storey building is located on 10 m deep soil with a variety of soil types and subjected to I a/v earthquake. In this case the periods at the maximum responses are different for the soil types, so that DS due to its high stiffness has the shortest period and NC has the highest period because of its softness. Also, the response acceleration in the DS profile is the highest (Fig.5.9b). Fig.5.9a shows that all soil types have amplified the bedrock motion (I a/v) with DS more so than the other soil types.

The Prince Rupert-related earthquake (L a/v) has been amplified by all soil profiles (Fig.5.10a). NC has the highest maximum acceleration at foundation-soil interface. This is due to the well known fact that soft soils amplify low frequency ground motions. As

in the case of I a/v ground motion, part (b) of this figure shows that, NC and DS have the extreme periods at their maximum response, this can be an indication of softness and stiffness. AS and OC have, again, nearly the same period (Fig.5.10b), but OC has amplified the bedrock motion more than AS. This is due to the higher damping and lower G/G_0 in the AS profile while both they both have almost the same initial shear modulus profiles with depth.

Figs.5.11 to 5.13 show the results of dynamic soil-structure interaction analysis when the 20-storey building is located on 40 m deep foundation. Subjected to different a/v earthquakes, NC, OC, and AS have almost the same period at their maximum responses in each case. Although the DS soil profile has the same period at its maximum response as the rest of the soil profiles while subjected to high frequency motion (Ottawa site), it has a shorter period when subjected to I, L a/v compared to NC, OC, and AS. This can be due to the increase in soil depth which increases the site period and causes the DS profile to vibrate under H a/v motions more smoothly. Fig.5.11a shows that under H a/v ground motion the maximum acceleration versus depth has substantial oscillations for NC, OC, and AS but not for DS. This can be justified with the fact that high strain level in the relative soft soils NC, OC, and AS (compared to DS) leads to higher damping, and then to filtering out the high frequency components of the ground motion. In other words, wherever there is a breakage in the plots, it can be a concentration of high strain level, and in turn, energy dissipation. Therefore, the oscillations in the profile of maximum acceleration versus depth can be an indication of energy dissipation. For DS (Fig.5.11a), the damping and/or energy dissipation is low due to low strain level and,

thus, the curve is more smooth. It should be mentioned that, this is true only when the applied acceleration time history has a high a/v ratio. Fig.5.12a shows that the curves are more smooth for I a/v motion and DS has amplified the bedrock motion more than the rest with NC just behind it. For L a/v ground motion, Fig.5.13a shows a similar situation for OC and AS to those of Fig.5.12a, but in contrast to the case of I a/v , NC has the highest amplification and DS follows next. This is because a deep profile of soft clay has the lowest energy dissipation under low frequency excitations, due to its lower damping (see Fig.4.5).

Figs.5.14 to 5.16 show the results when a 5-storey building is located on 10 m of different homogeneous soil deposits, subjected to H, I, and L a/v earthquake, respectively. Figs.5.17 to 5.19 show the results when the 5-storey building is located on 40 m of different homogeneous soil deposits, subjected to H, I, and L a/v earthquake, respectively. The results are generally similar to those of the 20-storey building analyzed previously, except for Vancouver-related earthquake cases (I a/v). In these cases the NC soil profiles, both of 10 and 40 m deep, have the highest amplification and the DS profiles have the second highest (Figs.5.15a and 5.18a), while in the 20-storey cases, the reverse is true in that DS had the highest amplification. This shows that existing 5-storey buildings will experience higher amplification than 20-storey buildings on the NC profile subjected to Vancouver-related rock motion (I a/v). This clearly shows the soil-structure interaction and its effects on the response of the soil-structure system. The details of influence of structural type on the soil response is presented in the upcoming section.

5.4.2 Influence of The Structure Type on The Soil Response

Soil medium and structure are considered the two substructures in the soil-structure interaction analysis. One will interact with the other and vice versa. Based on the dynamic finite element analysis in the current study, the following discusses the influence of the structure as one of the substructures, on the soil as being the other substructure.

In order to be concise, the discussion is only on two soil categories ; normally to lightly overconsolidated clay, NC, and dense sand, DS. Since these two soil types are considered to be the extremes of softness and stiffness for the four soil types considered in this study.

Variation of maximum acceleration in soil medium along the centreline and the response acceleration at the center of soil-structure interface are assumed to show some important aspects of the soil response with different structural types. These results are also compared with the free field response of the soil corresponding to the same elevation as the foundation interface (Fig.5.1).

Figs.5.20 to 5.25 show the results for the NC soil type and Figs.5.26 to 5.31 show the results for the DS soil type. Part (a) of these figures show the variation of the maximum acceleration along the centreline and part (b) plot the response spectra for the node at the intersection of the centreline and the foundation interface.

Figs.5.20 to 5.22 show the results for the NC soil profile and 10 m deep and supporting the 5-, 10- and 20-storey buildings, and subjected to the H, I, and L a/v motions, respectively. As the frequency level of excitations changes from H to L a/v, the soil response becomes close to free-field response which proves that under the low

frequency earthquake (Prince Rupert site) the effect of a shallow NC soil profile is more pronounced to amplify the rock motions than the building height. Figs.5.20 to 5.22 also show that for shorter buildings the responses are higher and closer to the free-field response. As a matter of fact, the responses of soil with 5-and 10-storey buildings subjected to H and I a/v are even higher than the free-field response. This is true, according to the resonance phenomenon between structure and soil that occurs in the vicinity of $T/T_s = 1.0$, in which T is the fundamental period of building and T_s is the site period. As presented in chapter 4, T_s for 10 m of NC soil profile is about 0.43 second. The fundamental period, for instance, of the 5-storey building is approximately 0.5 second as given by $T = 0.1N$ (NBCC 90) where N is the total number of storeys. In this case $T/T_s = 1.16$ which is close to unity. The reason why the maximum response occurs while the soil-structure system is subjected to low frequency motions, is that many dynamic loads excite only certain modes of structures, mostly the low frequency modes (Wolf, 1985). Therefore, the proximity between the structural natural frequency, which belongs to the lowest frequency mode, and the predominant frequency of the applied dynamic loading can cause resonance phenomenon.

Figs.5.23 to 5.25 show the results for the 40 m of NC soil profile supporting the 5-, 10-, and 20-storey buildings, and subjected to H, I, and L a/v earthquakes. Occurrence of resonance between soil and structure under low frequency motion (Prince Rupert L a/v) in the NC profile is also shown in these cases. Consequently, the 10-and 20-storey buildings have high maximum acceleration at foundation interface. The 20-storey building has the highest value of response acceleration and the 10-storey has the second highest.

It shows that the deep NC soil amplifies the low frequency bedrock motions more for tall buildings than for the short buildings. The resonance phenomenon is also illustrated for 5- and 10-storey buildings, Figs.5.24 and 5.25, while they are subjected to I and L a/v earthquakes, respectively. Therefore, it is concluded that a deep soft clay foundation amplifies the high frequency motions while supporting short buildings (e.g., 5-storey), the intermediate motions while supporting relatively tall buildings (e.g., 10-storey), and low frequency motions while supporting high rise buildings (e.g., 20-storey). Another important observation in seismic behaviour of deep soft clay, which is almost structural height-independent, is that, amplification is concentrated within the top 5 m for H a/v, the top 10 m for I a/v, and the top 30 m for L a/v (see Figs.5.23a to 5.25a). This is the reason why the deepest foundation was considered as being 40 m, because beyond this depth amplification is not significant, even under low frequency motions. This is in agreement with the statement in Elhmadi and Heidebercht (1990). They state that it appears that the level of site effects is independent of the depth of a homogeneous soil deposit beyond a depth of 40 m. The other worthwhile point to note in Figs.5.23a to 5.25a is that, as the input motion changes from H to L a/v, the dynamic effect of the structural interaction on the soil response goes deeper. For example, under H a/v, the variation of maximum acceleration along the centreline in the bottom 35 m of soil deposit, is more and less similar to the free field response. Within the top 4 or 5 m the effects of structures and the separation of the responses from the free field response becomes apparent (Fig.5.23a). However, while subjected to low frequency motion (Fig.5.25a) it goes even 30 m deep or more.

Figs.5.26 to 5.31 show the results for the dense sand, DS. Figs.5.26 to 5.28 show the response of 10 m deep dense sand supporting the 5-, 10-, and 20-storey buildings and subjected to H, I, and L a/v earthquakes. In this case, although the effect of different buildings show the same trend due to the high stiffness of DS, the amplification is the highest for the DS soil supporting 5-storey building and subjected to H a/v, or the 10 storey building and subjected to I a/v, or 20 storey building and subjected to L a/v. In essence, the softer soil-structure systems (having higher fundamental period) amplify the low frequency bedrock motions and the harder soil-structure systems (having lower fundamental period) amplify the high frequency motions. The important observation in the behaviour of dense sand is that the amplification starts from the deeper levels compared to that of NC profile (see Figs.5.29a and 5.31a). This means that the DS profile has capability of transmitting most of the high frequency components. On the whole, in DS media, both for shallow and deep profiles, the building height does not make much difference in the soil response. In other words, the DS soil amplifies all types of motions and this amplification increases as the a/v or frequency of excitations decreases.

5.5 SUMMARY AND CONCLUSION

A structure excited by a dynamic load (e.g., a seismic motion) interacts with the surrounding soil. In contrast to the structure, the soil is an unbounded domain whose radiation condition has to be taken into account in the dynamic model.

Soil-structure interaction analysis consists of two distinct parts: determining the free-

field response of the site and calculating that modification of seismic motion which results from the actual interaction when the structure is inserted into of the seismic environment of the free-field. The former part is known as kinematic interaction and the latter as inertial interaction.

One-bay moment resisting frames were chosen as structural models. 5-, 10-, and 20-storey buildings were considered as the structural models to cover a wide range of actual buildings with various fundamental periods.

Soil models applied in this study included two depths of 10 and 40 m to represent shallow and deep soil deposits, respectively. Four soil types of lightly to normally overconsolidated clay, NC, heavily overconsolidated clay, OC, alluvial sand and silt, AS, and dense sand, DS were included.

Soil-structure models were analyzed using the FLUSH program. Results are as follows:

1) All soil types amplify the bedrock motion but with different degrees. Soil-structure systems with lower damping in the final stage of dynamic analysis, will have higher response, and higher amplification of the bedrock motion than the other cases. In other words, those combinations of soil conditions, structural models and seismic excitation, that lead to lower effective damping, will amplify the bedrock motion most significantly.

2) Soil-structure models including soft clay, NC, and dense sand, DS, have the extreme periods at their maximum responses. DS-related models yield the shortest periods and NC-related models the highest periods.

3) Alluvial sand, AS, and heavily overconsolidated clay, OC behave similarly while they are subjected to the same seismic excitations.

4) The oscillations in the profile of maximum acceleration versus depth is an indication of energy dissipation and the turning points show the concentration of high shear strain levels.

5) Quasi-resonance between soil and structure was observed when the fundamental period of the building was in the proximity of site period. This phenomenon is more pronounced when the soil-structure system is subjected to low frequency motions than when subjected to high frequency motions. This is because, it is shown that dynamic loads excite only certain modes of structure, mostly those of low frequency.

6) The depth from the soil surface where site amplification and soil-structure interaction effects start to appear in homogeneous soil media, goes deeper as the frequency content of excitations changes from H a/v to L a/v .

7) Short period soil-structure systems significantly amplify the H a/v motions while long period systems significantly amplify the L a/v motions.

Soil Structure Models	Number Of Solid Elements	Number Of Beam Elements	Number Of Nodal Points	Degrees Of Freedom	Number Of Iterations	Number Of Strain Dependent Materials	Number Of Semi-Infinite Layers	Iteration Limit (%)	Highest Frequency Of Analysis
B5-ON.R	6	15	36	58	1	0	1	5.0	15.0
B10-ON.R	6	30	56	98	1	0	1	5.0	15.0
B20-ON.R	6	60	96	178	1	0	1	5.0	15.0
B5-Soil10	72	15	112	206	4	1	9	5.0	15.0
B5-Soil40	152	15	202	386	4	1	19	5.0	15.0
B10-Soil10	72	30	132	246	4	1	9	5.0	15.0
B10-Soil40	152	30	222	426	4	1	19	5.0	15.0
B20-Soil10	72	60	172	326	4	1	9	5.0	15.0
B20-Soil40	152	60	262	506	4	1	19	5.0	15.0

Table.5.1 Finite Element and Computational Characteristics of Soil-Structure Models

Fig.5.1 Schematic view of soil-structure system including transmitting boundaries, foundation interface and the centerline along which the maximum accelerations are computed.

Fig 5.2 Finite element model of 5-storey building located on rock

Fig.5.3 Finite element model of 10-storey building located on rock

Fig.5.4 Finite element model of 20-storey building located on rock

Fig.5.5 Finite element model of 5-storey building located on 10 m deep foundation

Fig.5.6 Finite element model of 20-storey building located on 40 m deep foundation

Fig.5.7 Typical shearing stress versus shearing strain relationship for a soil.

Fig.5.8 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 20-storey building located on 10 m of various soil types subjected to H a/v.

Fig.5.9 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 20-storey building located on 10 m of various soil types subjected to I a/v.

Fig.5.10 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 20-storey building located on 10 m of various soil types subjected to L a.v.

Fig.5.11 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 20-storey building located on 40 m of various soil types subjected to H a/v.

Fig.5.12 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 20-storey building located on 40 m of various soil types subjected to I a/v.

Fig.5.13 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 20-storey building located on 40 m of various soil types subjected to L a/v.

Fig.5.14 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 5-storey building located on 10 m of various soil types subjected to H a/v.

Fig.5.15 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 5-storey building located on 10 m of various soil types subjected to I a/v.

Fig.5.16 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 5-storey building located on 10 m of various soil types subjected to L a/v.

Fig.5.17 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 5-storey building located on 40 m of various soil types subjected to H a/v.

Fig.5.18 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 5-storey building located on 40 m of various soil types subjected to I a/v.

Fig.5.19 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 5-storey building located on 40 m of various soil types subjected to L a/v.

Fig.5.20 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 10 m of NC soil type carrying various buildings and subjected to H a/v.

Fig.5.21 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 10 m of NC soil type carrying various buildings and subjected to I a/v.

Fig.5.22 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 10 m of NC soil type carrying various buildings and subjected to L a/v.

Fig.5.23 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 40 m of NC soil type carrying various buildings and subjected to H a/v.

Fig.5.24 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 40 m of NC soil type carrying various buildings and subjected to I a/v.

Fig.5.25 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 40 m of NC soil type carrying various buildings and subjected to L a/v.

Fig.5.26 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 10 m of DS soil type carrying various buildings and subjected to H a/v.

Fig.5.27 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 10 m of DS soil type carrying various buildings and subjected to I a/v.

Fig.5.28 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 10 m of DS soil type carrying various buildings and subjected to L a/v.

Fig.5.29 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 40 m of DS soil type carrying various buildings and subjected to H a/v.

Fig.5.30 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 40 m of DS soil type carrying various buildings and subjected to I a/v.

Fig.5.31 a) Maximum acceleration profile along centerline, b) response spectra for foundation interface for 40 m of DS soil type carrying various buildings and subjected to L a/v.

Chapter 6

SITE AMPLIFICATION

AND

SOIL-STRUCTURE INTERACTION IMPLICATIONS

ON

NBCC 1990 BASE SHEAR AND FOUNDATION FACTOR PROVISIONS

6.1 STUDY OF SITE AMPLIFICATION

A study of site amplification usually consists of:

- (1) Determination of representative soil profile and dynamic soil properties to simulate typical conditions in the site.
- (2) Selection of input motions compatible with the local seismo-tectonic setting, and
- (3) Dynamic response analysis for the selected sites using state-of-practice procedures (Seed et al., 1976; Mohraz 1976; Naeim 1989).

Dynamic soil properties used in the current study and the input motions compatible

with the seismicity of the three cities of Canada: Ottawa, Vancouver, and Prince Rupert have been discussed in Chapter 4. In addition, implications of some dynamic behaviour of these particular soils subjected to the above seismic ground excitations were presented in chapter 5. In the upcoming section the results of the third part of site amplification study, response spectral analysis for the selected sites using the FLUSH program, are summarized.

6.2 FREE FIELD SEISMIC RESPONSE ANALYSIS

A soil deposit subjected to earthquake ground motions has a tendency 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 soil deposit. This behaviour of soil deposit is termed site amplification. To evaluate such behaviour, a free field response analysis is performed in the current study and the results are presented in this section. In this study spectral ratio is chosen to demonstrate the magnitude of amplification. The spectral ratio is defined as the ratio of the response acceleration of a single-degree-of-freedom (SDOF) system located on the ground surface to that of the same (SDOF) system located on bedrock. In this study, the spectral ratios are plotted versus the periods of (SDOF) vibrations.

On the basis of pervious studies (Seed et al., 1976; Mohraz 1976), the major parameters that contribute to site amplification are soil type, soil depth, and frequency content of bedrock motions. Accordingly, in the following free field response analysis, the effects of these parameters on site amplification are studied. To do this, soil models

are developed to represent homogeneous deposits of normally to lightly overconsolidated clay, NC; heavily overconsolidated clay, OC; alluvial sand and silt, AS; and finally dense sand, DS. For each soil model two depths, 10 and 40 m, are considered to represent shallow and deep deposits. These soil deposits are subjected to the acceleration time histories scaled based on the seismicity of Ottawa (H a/v), Vancouver (I a/v) and Prince Rupert (L a/v).

6.2.1 Effects of Soil Type on Site Amplification

Figs.6.1 to 6.3 show the variation of free field spectral ratio versus period for the four soil deposits of 10 m deep subjected to the H, I, and L a/v earthquakes. Fig.6.1 shows that in Ottawa (H a/v), the NC soil profile has amplified the bedrock motion, more than the other soil types, with spectral ratio of up to 6.0. The OC and AS soil profiles have almost the same responses, with spectral ratios of up to 4.0. The DS profile has amplified the motion but not as much as the others.

Subjected to the Vancouver-related earthquake (I a/v) and the Prince Rupert-related ground motion (L a/v), the situations are almost the same (Figs.6.2 and 6.3). The highest amplification belongs to the NC and DS profiles with the spectral ratio up to 5.5. This is true because the NC soil profile amplifies the low frequency motions due to its higher G/G_0 and lower damping than those of the other soils. For the DS profile this is because of its high values of initial shear modulus. For the AS and OC profiles the bedrock motion has been amplified with a spectral ratio of up to 4.0.

Figs.6.4 to 6.6 illustrate the variation of free field spectral ratio versus period for

the four soil deposits of 40 m deep. In all three cities (H, I, and L a/v) the maximum spectral ratio belongs to the normally to lightly overconsolidated clay, NC soil profile, with values ranging from 7.5 to 8.5. This range of spectral ratio has also been reported by Seed et al. (1987) for earthquakes with peak velocity in the neighbourhood of 0.10 m/s. For the AS and OC profiles, like the 10 m models, have almost the same maximum spectral ratios of up to 5.0. The results of this study are presented in Table.6.1.

6.2.2 Effects of Soil Depth on Site Amplification

Figs.6.7 to 6.18 demonstrate the effects of depth on site amplification for the four soil categories subjected to the H, I, and L a/v motions. The results are also summarized in Tables 6.1 and 6.2. Figs.6.7 to 6.9 show that, for the NC profile, there is always an increase of peak spectral ratio as depth increases from 10 to 40 m. The highest increase (62%) is when the soil is subjected to the Vancouver-related earthquake (I a/v) (Fig.6.8 and Table 6.2).

For the alluvial sand profile, AS, although there is an increase of 27 percent under the H and I a/v, there is a decrease of 15 percent under the L a/v (Figs.6.13 to 6.15 and Table 6.2). This means that increasing the depth of alluvial sand causes the low frequency motion to be damped faster than H or I a/v motions.

For the OC soil deposit, except for a slight difference under the H and L a/v motions, there is no effect of increasing depth on the site amplification (Figs.6.10 to 6.12 and Table 6.2).

Apart from a slight increase of 6 percent in spectral ratio when applying the H a/v,

increasing the depth from 10 to 40 m in fact reduces severely the maximum spectral ratio up to 34 percent in the DS soil profile while subjected to the Vancouver-related earthquake ($I a/v$) and 27 percent while subjected to the Prince Rupert-related earthquake ($L a/v$). This means that the L and $I a/v$ motions cause more severe excitations in shallow deposits of dense sand than in deep deposits.

Nevertheless, It is worth to note that all types of soil deposits do amplify ground motions, but with different degrees. This is implied in all Figs.6.7 through 6.18, because not only the maximum spectral ratio is always more than 3.0 but also in all period levels this value is in excess of unity.

6.3 SEISMIC STRUCTURAL BASE SHEAR FORCE

To design an earthquake-resistance structure, one needs to know the characteristics and probability of occurrence of the "design" seismic ground motion, the characteristics of the structure and foundation, the allowable stresses in materials of construction, including the foundation soils, and the amount of damage that is tolerable. NBCC 90 has provided seismic zoning maps for peak horizontal ground acceleration and velocity, based on an statistical analysis of earthquakes that have been experienced in different locations in Canada. The probability of exceedance of 10 percent in 50 years for the ground motion parameters is considered which is mathematically equivalent to a probability of exceedance of 0.0021 per annum. There is, and always will be, an inherent uncertainty in the appropriate seismic ground motions to be used in earthquake-resistance design. Because of the nature of earthquakes, their size and location and the ground motion

effects, that they will produce, cannot be accurately defined.

6.3.1 The Structural Response to Ground Motion

The elastic response of a single-degree-of-freedom system to ground motion depends on the fundamental period and damping characteristics of the system, and on the frequency content and amplitude of ground motion. The base shear which can be used as a measure of this response is expressed as the product of the mass of the system and the spectral acceleration as given by the response spectrum. The acceleration response spectrum reflects the dependence on the natural period, T , the damping of the system and also the characteristics of the ground motion. Details of its derivation can be found in (Housener et al., 1953). This concept of calculating base shear can be applied to multi-storey buildings having many modes of vibration. For usual buildings of low or moderate height, the principal earthquake response is due to the fundamental mode of vibration. For taller structures some allowance for contribution of the higher modes is made in the base shear calculations (Wolf, 1985). This method has also been adopted in NBCC 90.

Due to inelastic behaviour of material, the base shear induced by earthquake is reduced as compared to that of elastic behaviour. Also, because of ductility effect of a structure: the capability to deform beyond the yield point, the design base shear of a structure will be lower (Clough et al., 1966; Penzien 1962; Uzumeri et al., 1978).

6.3.2 Computation of Elastic Base Shear

Soil-structure models described in chapter 5 were used to analyze the elastic base shear, using the FLUSH program, subjected to Ottawa, Vancouver, and Prince Rupert seismicity related earthquakes. The FLUSH program does not give the base shear of a multi-storey building directly. This program was modified to compute the base shear by virtue of static equilibrium of the structure at each time step (see Fig.6.19). This figure shows the freebody-diagram of the first floor columns, m and n, of a multi-storey building. At each time step the static equilibrium is so assumed that the base shear is equal to the transverse shears of the first floor columns. The transverse shear in the columns is the ratio of the sum of nodal moments, $M_{mi} + M_{mj} + M_{ni} + M_{nj}$, to the column height, l (Fig.6.19). It should be mentioned that FLUSH program is able to compute the time history of nodal moments.

To compute the foundation factor, structural elastic base shears should be computed for the building located on bedrock, V_r , and for the same building but located on soil surface, V_s . Figs.6.20 to 6.22 show the time histories of absolute elastic base shear, V_r , for the 5-, 10-, and 20-storey buildings located on rock subjected to the three acceleration time histories of H, I, and L a/v . The time history of elastic base shear, V_s , for all the combinations of the three mentioned types of buildings, and two foundation depths, 10 and 40 m, and the four soil types, NC, OC, AS, and DS, subjected to the three H, I, and L a/v acceleration time histories were derived. Maximum base shear in all cases happened in the first 30 seconds. Therefore, the time histories of base shear are plotted just for their first 30 seconds. The maximum values of these 72 time histories, $V_s \text{ max}$, are presented

in Table.6.3. It is noteworthy that the FLUSH program gives all force outputs for unit length of the structure in the third direction,i.e., kN/m.

6.3.3 NBCC 1990 Base Shear

Based on NBCC 90 a structure should carry a minimum base shear, V , given by

$$V = (V_e/R)U$$

$$V_e = vSIFW \quad (\text{Equivalent lateral elastic seismic base shear})$$

$$v = \text{peak horizontal velocity (m/s)}$$

$$S = \text{Seismic response factor}$$

$$I = \text{Seismic importance factor of structure (considered to be 1.0 for normal structures)}$$

$$F = \text{Foundation factor}$$

$$W = \text{Dead load (plus part of live load according to NBCC 90)}$$

$$R = \text{Force modification factor that reflects the capability of structure to dissipate energy through inelastic behaviour (for concrete moment resisting frames, NBCC 90 specifies } R = 4)$$

$$U = \text{Calibration factor (} U=0.6).$$

NBCC 90 presents the seismic response factor, S , values in a plot form as Fig.6.23. This factor is a function of the fundamental period, T , of the structure, and the relative values of the velocity-related and acceleration-related seismic zones, Z_v and Z_a , pertaining to a particular location (or frequency content of the applied earthquakes). This factor

represents the idealized elastic response of 5 percent damped multi-degree-of-freedom systems, for unit values of the zonal velocity, v , and weight, W (Heidebrecht et al., 1988). The force modification factor, R , accounts for the energy-absorption capability of structural system by damping and inelastic behaviour of structure. Types of construction that have performed well in earthquakes are simply assigned higher values of R .

F is the foundation factor which represents the site amplification effects on the design base shear. NBCC 90 specifies $F = 1.0$ for rock or very stiff soil sites and $F = 1.3$ - 2.0 for other site conditions (see Table.6.4). Note that F is defined such that the product of FS does not exceed a specified limit, namely,

$$FS < 4.2, \quad \text{when } Z_a > Z_v$$

$$FS < 3.0, \quad \text{when } Z_a = Z_v \text{ or } Z_a < Z_v$$

Figs.6.24 to 6.31 show the NBCC 90 foundation factors, F , versus fundamental period of structure for the four site categories of I, II, III, and IV with respect to the definitions of S and the limitation on FS product.

6.4 FOUNDATION FACTOR

It is shown that soil conditions exert a major influence on the type and amount of structural damage resulting from an earthquake (Seed et al., 1972; Finn et al., 1988; and Mitchell et al., 1986). Accordingly, two major parameters in defining the foundation factor in NBCC 90 are soil type and depth (see Table.6.4). As the motions propagate from

the bedrock to the ground surface, the soil tends to amplify the motions in frequency ranges in the vicinity of the natural frequencies of the top layer. In addition, a structure located on the surficial layer, may suffer more intense shaking due to the development of a state of quasi-resonance between structure and foundation soil. Analysis of these phenomena by suitable computational models has been explained in chapter 3. These include lumped masses wave propagation, two-dimensional finite element, or even approximate three-dimensional finite element models. In all above models, the earthquake motions are modeled as being vertical propagation of shear waves. This simplified model of wave propagation ignores the source mechanism of the earthquake, the geology of the travel path and the effects of surface waves (Pyke 1979; Heidebrecht et al. 1990; Henderson et al., 1990). Most codes, including NBCC 90 and NEHRP (American Code), present a rough allowance for site effects and this is due to the uncertainties and complexities in the realistic estimation of site effects on the seismic response of structures and ground.

The seismic design procedures presented in NBCC 90 (e.g., design base shear, V) are on the basis of the assumption that the structures are founded on a rigid base moving with the ground surface motion (NBCC Commentary). Real foundations have both flexibility and damping capacity, which change structural response. The flexibility of foundation increases the fundamental period of a structure, and the damping capacity arises from dissipation of energy by radiation away from the structure and by hysteretic damping in the foundation, therefore increasing the effective damping of the structure. These effects are described as soil-structure interaction and are neglected in the current

NBCC provisions on the premise that, for most of the buildings addressed in the code, these effects lead to a reduction in base shear demand and thus a more conservative design (NBCC 90 Commentary). However, soil-structure interaction effects, being a function of the properties of the structure as well as the properties of the local soil deposit, may in some cases actually increase the lateral force demand during the seismic response (Seed, 1986).

6.4.1 Foundation Factor Computation Results

Analytically, the foundation factor, F , refers to the ratio of the maximum base shear of the structure situated on a soil deposit, V_{smax} , to the corresponding maximum base shear for an identical structure assumed to be situated directly on bedrock and subjected to the same bedrock motions, V_{rmax} .

$$F = \frac{V_{smax}}{V_{rmax}}$$

For all the above mentioned 72 models in section 6.3.2, foundation factor values were computed using above simple relationship. These values are presented in Table.6.5. NBCC 90 classifies different soil conditions into four categories and for each presents a value for foundation factor (Table.6.5). The soil models applied in this study cover all four categories so that, 10 m profiles of DS and OC fall in category I, 10 m profiles of NC and AS and 40 m profiles of OC and DS belong to category II, 40 m profile of AS

lie on category III, and 40 m profile of NC in category IV. The NBCC 90 foundation factors for the above soil models are brought in Table.6.5. The computed values of foundation factor having a star , *, are those which exceed the corresponding NBCC 90 values. 27 cases out of the total of 72 cases have values exceeding the NBCC 90 specified values. 25 of these belong to categories I and II. These results suggest that the code values are mostly conservative in categories of III and IV, and inadequate in categories of I and II. More details on this topic will be presented in section 6.4.4.

NBCC 90 claims that the code values are conservative due to neglecting soil-structure interaction effects. However, accounting for soil-structure interaction effects in the current study has resulted in values in excess of NBCC values. This is in agreement with Seed (1986) that soil-structure interaction effects may increase lateral force demand during seismic response, in contrast to the NBCC claim.

Fig.6.32 shows the time histories and computed maximum elastic base shears for two of the cases whose computed foundation factors exceed the NBCC specified values, B20-DS40 subjected to I a/v, and B5-OC10 subjected to H a/v. The time histories of elastic base shear for the other 25 cases are presented in Appendix B.

6.4.2 Approximate Fundamental Period of Multi-Storey Buildings, T

There are empirical correlations to compute period of the fundamental mode of buildings with different load-resisting systems. The following is usually used for moment-resisting space frames.

$$T = 0.1N$$

This relationship is recommended by many codes including NBCC 90 and NERHP (American Seismic Design Code), where N is the total number of storeys.

In the American code, they also recommend using either of the above relationship or the following correlation.

$$T = C_T h_n^{3/4}$$

Where, h_n is the height of the building in feet and for a concrete moment-resisting frame $C_T = 0.030$. They suggest using of $T = 0.1N$ for buildings with 12 storeys or less with the storey height of 10-12 feet. The results of using either of the correlations are in reasonable agreement for the buildings in the current study.

Period calculations based on the preceding correlations give values which are mostly in reasonable agreement with measured values. Occasionally, slightly lower value may be obtained which will lead to a conservative design, at least in computing the elastic base shear. This is so because smaller fundamental periods are associated with lower energy dissipation, hence leading to higher design base shears. Anderson et al. (1952) and Clough et al., (1966) explained in great details the foregoing argument. Table 6.5 presents the applied periods of fundamental mode for the 5-, 10-, and 20-storey buildings in the current study which are computed based on the NBCC 90 recommended correlation.

6.4.3 Validity of Foundation Factor Period-Dependence

There is increasing evidence that as the motion propagate from bedrock to ground surface, the soil tends to amplify the motions in frequency ranges in the vicinity of the natural frequencies of the top layer, T_s . In addition, a structure located on the surficial layer, with natural period, T , in the neighbourhood of that of the soil layer, may suffer more intense shaking due to the development of a quasi-resonance between structure and foundation soil (Seed et al., 1972 and Finn et al., 1988).

In order to clarify the foregoing argument, the computed foundation factors were plotted versus T/T_s for each soil type (see Figs.6.33 to 6.36), where T_s is the site period and T is the fundamental period of structure. Table.6.5 presents the values of T/T_s for all the soil-structure models considered in this study. For each soil type, there are three sets of line-points corresponding to H, I, and L a/v . The T/T_s axis is in logarithmic scale. Connecting points by straight lines does not mean that a random T/T_s between each two-sequential plotted points should lie exactly on the connecting line. These lines are used just to separate points from different frequency content responses, although they do show a rough trend of foundation factor versus T/T_s .

According to these plots, the above-unity values of foundation factors are in the range of $1.0 < T/T_s < 10$, which is in good agreement with Elhmadi and Heidebrecht (1991) who stated this range as $0.1 < T/T_s < 10$. Also, the peak values are mostly in the range of $1.0 < T/T_s < 4.0$. Elhmadi and Heidebrecht stated this range as being $1.0 < T/T_s < 2.0$. It is interesting to observe the similarity between performance of AS and OC soils (Figs.6.34 and 6.35). In both cases, the responses to H a/v are higher than those of the

I and L a/v , and the responses to L a/v are relatively smooth and mostly below unity (i.e., no amplification), except for T/Ts in the proximity of unity (Figs.6.34 and 6.35). Fig.6.33 shows that the maximum value of foundation factor for the NC profile belongs to the L a/v response corresponding to B5-NC40 at $T/T_s = 0.5/1.01 = 0.5$. In addition, the peak values mostly belong to L a/v and in the vicinity of $T/T_s = 1.0$. Fig.6.36 shows clearly the quasi-resonance between structure and foundation soil in the neighbourhood of $T/T_s = 1.0$ in the DS medium. All responses for the DS profile follow almost the same trend specially about the proximity of T to Ts. The highest value of computed foundation factor, 3.125, occurs in this soil medium and under the Vancouver-related earthquake, I a/v motion.

6.4.4 Adequacy Assessment of NBCC 90 Foundation Factor

To assess the adequacy of the specified code F values, these are compared with the computed values, based on dynamic analysis of soil-structure systems, in Figs.6.24 to 6.31. In these figures the NBCC 90 values are plotted versus period, considering the limitation in the FS product and S period dependence.

Figs.6.24 and 6.25 show the code F value in category I, $F = 1.0$. Foundation factor in this category is independent of period and frequency content of excitations. However, the computed values for H a/v (Ottawa site) are well above code values with the mean value of $F_m = 1.48$ for DS10 and $F_m = 2.0$ for OC10 soil models. This is clearly seen that as the a/v of records goes from low to high, the computed values become far from the code value (Figs.6.24 and 6.25), in other words higher amplification of base shear.

This supports Heidebrecht et al. (1991) who stated that the long period sites have the largest amplifications when subjected to L a/v, whereas the largest amplification on the short period sites are associated with H a/v. This is to be expected, since the low a/v records are predominantly long period in character, whereas the reverse is true for the H records.

Figs.6.26 to 6.29 show the NBCC 90 F values for category II, $F = 1.3$, and the computed values simultaneously, for relatively soft soils of shallow depth, AS10 and NC10, and for relatively stiff soils of large depth, OC40 and DS40.

According to Table.6.5 and Fig.6.29, most of the exceeded code values in this category belong to the DS40 soil model. The site period for this soil model is $T_s = 0.39$ sec. Another important observation is that for this soil model, three of the exceeded values correspond to H a/v, two to I a/v, and only one to L a/v. This is in good agreement with both the preceding conclusion category I: amplification of H a/v by short period sites, and the following statement of Heidebrecht et al. (1991). They stated that the amplification of base shear due to site effects is not restricted to soft soils with relatively long periods and subjected to L a/v. The sites with periods less than 1.0 second may also have significant amplification.

For the NC10 soil model, all the corresponding H a/v foundation factors are below the code values, $F_m = 1.0$, whereas L a/v values are reasonably in agreement with the code values. This is to be expected since NC profile tends to filter out high frequency components and amplify the low frequency motions. There is a jump for the case of B20-NC10 when subjected to I a/v, $F = 1.792 > 1.3$ (see Fig.6.26).

Fig.6.27 shows that, for the OC40 soil model, the computed results are mostly lower than the code values implying conservatism in NBCC 90, except for the case of B10-OC40 while subjected to H a/v in which $F = 2.414 (> 1.3)$. This may be due to resonance between soil and structure. As a matter of fact, the fundamental period in this case, $T = 1.0$ is fairly close to the site period, $T_s = 0.7$. It should be noted that, again this underestimation happens when the H a/v motion is applied.

Fig.6.28 compares the code values with the results for the AS10 corresponding to category II in NBCC 90. Computed values corresponding to L a/v are well below the code values, i.e., conservatism in NBCC 90. However, three cases have values exceeding the code values: two correspond to H a/v and one to I a/v. This again reminds of the inadequacy of NBCC 90 for relatively high frequency motions.

Figs.6.30 and 6.31 show the comparison of the computed foundation factors and NBCC 90 values for categories III and IV, respectively. In most cases NBCC 90 offer conservative values.

In the case of AS40 soil model (category III; $F = 1.5$), values corresponding to H a/v, like the preceding cases, have the highest mean value of foundation factors ($F_m = 1.27$). In addition to this, for B10-AS40 model, the computed value is $F = 1.93 (> 1.5)$.

In the case of NC40 soil model (category IV; $F = 2.0$), L a/v motions causes the highest values of foundation factor with the computed foundation factor for B5-NC40 model equal to $2.446 (> 2.0)$. This is to be expected, based on the well known fact that deep soft soils amplify the low frequency motions.

6.5 PROPOSED MODIFICATIONS ON NBCC 90 FOUNDATION FACTOR

An overall observation on the preceding section reveals that there are some inadequacies in NBCC 90 foundation factor values, particularly for stiff soils, DS and OC. Except for NC soil type, foundation factor is underestimated for all soil types when they are subjected to the Ottawa-related earthquake ($H a/v$). The inadequacy of the NBCC 90 values under very high a/v records is not unexpected because they are satisfactory for a/v of 1.5 or less. This is to be expected, since the short period values of the seismic response factor, S , in NBCC 90 is set on the basis of Z_a being one zone higher than the Z_v , i.e., a/v about 1.5 (Heidebrecht et al., 1991). It should be noted that the Ottawa seismicity-related earthquake which is applied in the current study has an $a/v = 2.03$.

To demonstrate the conservatism and inadequacy of NBCC 90 F values, compared to the computed values, a linear regression analysis was performed to derive the coefficients of conservatism and inadequacy for each soil type. Figs.6.37 to 6.40 show the results for NC, OC, AS, and DS soil types. The regression coefficient, C , is the ratio of dependent variable, y , to independent variable, x , in other words $y = Cx$. In this study, computed foundation factor is taken as the dependent variable and NBCC 90 values as the independent values. If the regression line is close to the $y = x$ line, the computed data are in good agreement with the code values, with regression coefficient of about unity. The lines below $y = x$ imply that the code values are conservative, or $C < 1.0$. Whereas the lines above the $y = x$ show, the NBCC 90 values are inadequate, or $C > 1.0$. Statistically, as the number of data points increases the regression analysis will become more reliable. Table.6.6 presents the regression coefficients, as well as the proposed

modified foundation factors for each category of NBCC 90. The proposed modified foundation factor, F^* , equal to the multiplication of the regression coefficient, C , and the current specified NBCC 90 F value, $F^* = C \times F$. It is worth to note that this proposed foundation factor is dependent on the frequency content of bedrock motion, in contrast to the NBCC 90 F values. Also, as it was previously mentioned in section 6.4.3, this proposed values are recommended only for $1.0 < T/T_s < 4.0$. For T/T_s out of this range, the former code values seem to be sufficient. It must be mentioned that for proposed values of foundation factor, F^* , regression coefficients less than unity, $C < 1.0$ are assumed to be unity, $C = 1.0$. This means using of the current specified NBCC values. As it can be seen from Figs.6.37 through 6.40 and Table.6.6 the highest value of regression coefficients belong to stiff soil, DS and OC, particularly for higher a/v motions.

Another attention that should be paid is that these values are proposed just for the three cities of Ottawa, Vancouver, and Prince Rupert and does not cover all other locations in Canada. They do, however, cover three different ground motions that may likely hit major Canadian cities.

6.6 PROPOSED MODIFICATIONS ON NBCC 90 SEISMIC RESPONSE

FACTOR, S

NBCC 90 defines seismic response factor, S , as a function of the fundamental period of building, T , and in the case of short period buildings ($T < 0.5$ sec), also a function of frequency content (a/v) of the applied seismic ground motions (Fig.6.23).

$$S = \frac{2.5}{\sqrt{T}} \quad (\text{if } T > 0.5 \text{ sec})$$

However, the results of the current study show that this factor, S , is underestimated or at least the values offered in NBCC 90 does not correspond to the cases analyzed in present study. As a matter of fact, seismic response factor, S , is found not only dependent on the frequency content of the applied ground motion even for long period buildings ($T > 0.5$ sec), but also this factor seems underestimated for this period range. It might be questioned that the computed values of base shear are derived based on only three earthquake records and these underestimations may be due to the effects of these particular motions, i.e., if we use another set of records the results may differ. It should be mentioned that this argument does not change the facts that, first the applied records in this study are chosen based on the seismicity of some real locations, secondly the result shows deficiencies in predicting factor S at least for these three sample records studied here. Nevertheless, results of this study are presented as follows.

NBCC 90 values of elastic base shear forces, V_e , were derived for all of the 24 soil-structure models subjected to the three input motions of H, I, and L a/v (Table.6.3). A linear regression analysis for the total of 72 cases was performed using the computed maximum elastic base shear forces, V_{smax} , as dependent variable and NBCC 90 values, V_e , as independent variable (Fig.6.41). Regression coefficients corresponding to three levels of a/v ratios are shown in Fig.6.41 and Table.6.6.

In addition, the analyses were performed to compute the ratios of computed

foundation factor and NBCC 90 corresponding values (regression coefficients) for all the 72 cases (Fig.6.42).

NBCC 90 elastic base shear is given by:

$$V_N = v_N \cdot I_N \cdot F_N \cdot S_N \cdot W_N$$

where index N corresponds to *NBCC 90 values*. The computed elastic base shear may be written in the same form as in NBCC 90. Therefore,

$$V_c = v_c \cdot I_c \cdot F_c \cdot S_c \cdot W_c$$

where index c corresponds to *computed values*. Since $v_c = v_N$, $I_c = I_N$ and $W_c = W_N$ as both buildings and ground motions are the same for the two cases, the ratio V_c/V_N can be written as:

$$\frac{V_c}{V_N} = \frac{F_c}{F_N} \cdot \frac{S_c}{S_N}$$

V_c/V_N and F_c/F_N can be derived for different a/v ratios on the basis of the forgoing regression analyses (see figs.6.41 and 6.42) so that,

$$\begin{aligned} V_c/V_N &= 1.62 & \text{for} & \quad \text{H } a/v \\ &= 2.02 & \text{for} & \quad \text{I } a/v \\ &= 1.14 & \text{for} & \quad \text{L } a/v \end{aligned}$$

and

$$\begin{aligned}
 F_d/F_N &= 1.2 && \text{for } H \text{ a/v} \\
 &= 1.0 && \text{for } I \text{ a/v} \\
 &= 0.86 && \text{for } L \text{ a/v}
 \end{aligned}$$

These lead to

$$\begin{aligned}
 S_d/S_N &= 1.35 && \text{for } H \text{ a/v} \\
 &= 2.02 && \text{for } I \text{ a/v} \\
 &= 1.33 && \text{for } L \text{ a/v}
 \end{aligned}$$

That the values of S_d/S_N are more than unity shows that the NBCC 90 values are underestimated. In addition, the differences in these values imply that in the case of long period buildings ($T > 0.5$ sec), seismic response factor, S , is also dependent on the frequency content (a/v) of seismic ground motions, whereas NBCC 90 considers this only for short period buildings ($T < 0.5$ sec).

6.7 A NEW APPROACH OF DEFINING SOIL CONDITIONS FOR NBCC

FOUNDATION FACTOR

One of the most important aims of a building code is to offer safe as well as economic criteria for structural design. In addition, these criteria and procedures should be clear and easy to understand for the users, such as construction and design engineers. Accordingly, there should be as low ambiguity as possible for engineers to distinguish

between different cases, and in turn to make decision.

It seems that this essential feature is lacking in defining different soil conditions in NBCC 90 for values of foundation factor, F (Table.6.4). In describing different soil conditions, NBCC 90 repeatedly uses qualitative words such as *dense*, *very dense*, *stiff*, *very stiff*, *hard*, *compact*, *firm*, *very firm*, *loose*, *very loose*, *soft*, *very soft* etc. Geotechnically, these words have certain definitions. For example, Terzaghi and Perk (1948) classified different clays based on their unconfined compressive strength as follows:

<i>Consistency</i>	<i>Unconfined Compressive Strength (kg/cm²)</i>
very soft	< 0.25
soft	0.25 - 0.5
medium	0.50 - 1.0
stiff	1.0 - 2.0
very stiff	2.0 - 4.0
extremely stiff	> 4.0

In addition, Bell (1992) classifies granular soils based on their relative density, D_r , as follows:

<i>Description</i>	<i>Relative Density, D_r</i>
very loose	< 0.2
loose	0.2 - 0.4
medium dense	0.4 - 0.6
dense	0.6 - 0.8

very dense

> 0.8

These classifications may be useful, but they are not unique. Many different classifications exist such as AASHTO (ASTM designation D-3282; AASHTO method M145) or Unified Systems (ASTM designation D-2487). In addition to this, engineers would rather deal with quantitative than qualitative definitions for different soil types, at least in the case of choosing the real site-related foundation factor.

Seismic forces have considerable influence on design and, in turn, on the costs of a structure. The foundation factor, F , is directly related to NBCC 90 design seismic base shear force. An overestimation of this factor based on an uncertain choice of soil categories listed in Table.6.4 may lead to overdesigned forces and in turn uneconomic design. On the other hand, underestimation of this factor will result in designing a structure which may not resist the actual seismic forces.

According to the foregoing, defining more clear soil conditions for NBCC 90 foundation factor, F , seems to be essential. This problem can be also seen in building codes of other countries such as NEHRP (American seismic design code).

One of the soil parameters that represents well the dynamic and amplification characteristics of soils is shear wave velocity, V_s . This is consistent with the fact that the main cause of seismic site vibration and amplification is vertical propagation of shear waves (Kanai, 1952; Idriss and Seed, 1968). In other words, shear wave velocity of a soil represents the characteristics of a soil in passing the shear waves.

In order to offset the unclear definitions of soil conditions in NBCC 90 provisions of foundation factor, a new soil condition definition is proposed here. Based on shear

wave velocity and depth, soil categories considered in the current study can be divided into six soil condition zones (see Fig.6.43). It should be mentioned that the boundary between shallow and deep soil depths considered as being 15 m in accordance with NBCC 90. These zones are defined as follows:

Zone 1 Shallow soft clay

Zone 2 : Shallow loose sand and firm clay

Zone 3 : Shallow dense sand

Zone 4 : Deep soft clay

Zone 5 : Deep loose sand and firm clay

Zone 6 : Deep dense sand

For depths beyond 40 m, the zones would be the same as that of 40 m. This is on the basis of the work of Elhamedi et al. (1990) and the observations in Chapter 5 that increasing depth beyond 40 m does not have significant effects on site amplification.

The proposed foundation factors, F^* , are presented in Table.6.7 in terms of the proposed soil condition zonation introduced in this section.

6.8 SUMMARY AND CONCLUSION

The design implications of site amplification and soil-structure interaction due to seismic ground motions are investigated in this chapter. Four soil types with shallow and deep depths were selected to represent a wide range of foundation conditions. Three strong motions of H, I, and L a/v were selected and then scaled to conform with the

Ottawa-, Vancouver-, and Prince Rupert-related seismicities, respectively. These three records correspond to the three ranges of a/v ratio in NBCC 90.

For each of the sites, free-field response analysis was performed to study the effects of soil type, soil depth, and frequency content of excitations on site amplification. The following are the main conclusions of this part of study.

1. Site amplification in NC soil profiles is the highest, compared to the other soil types, with spectral ratio of up to 8.3, while DS profiles have the lowest values of spectral ratios. Site amplifications in firm clay, OC, and alluvial and silt, AS are fairly similar, in fact, the peak spectral ratios almost for all the cases are interestingly close to each other.

2. All soil types amplify bedrock motions, as a matter of fact the peak spectral ratios are always in excess of 3.3.

3. Increasing the depth from 10 to 40 m in NC profile always results in increasing spectral ratio. The amplification does not change considerably in firm clay deposits, OC, when the depth increases from 10 to 40 m. Surficial excitation response in shallow deposits of dense sand, DS, is much more severe than that of deep deposits, as reflected by the dramatic decrease of spectral ratios of up to 34% when the depth increases from 10 to 40 m.

4. Based on the results of this study, there is always a decreasing trend of peak spectral ratios particularly in deep deposits of NC, OC, and AS when the a/v of the ground motions goes from H to L. The reverse is true for dense sand, DS.

Previous studies on base shear amplification and on defining the NBCC foundation factor have used mostly one-dimensional analysis and neglected soil-structure interaction

effects. In the current study the three-dimensional model used in the FLUSH program was applied. This program was modified to compute structural base shear by virtue of static equilibrium at each time step. For the three buildings of 5-, 10, and 20-storey considered in the current study the time histories of absolute base shear were derived both for the building located on top of soil deposit and for the same building located on bedrock. The foundation factor is defined as the ratio of the computed maximum base shear for the structure situated on the soil deposit to the corresponding base shear for the same structure assumed to be situated directly on bedrock and subjected to the same bedrock motions. On the basis of these results, the following are the main conclusions that should be considered in the aseismic design of structures located on soil deposits:

1. The base shear amplification is most significant when the earthquake excitation has significant energy in the site period region. Long period sites (i.e., soft deep soil deposits) have the largest amplification when subjected to seismic motions with low a/v ratio. Short period sites (stiff shallow soil deposits), on the other hand, have the largest amplification when subjected to seismic motions with high a/v ratio. However, substantial amplification occurs at structural periods near site periods for any kind of seismic excitation.

2. For short period sites foundation factor decreases as the a/v of excitations decreases, indicating that the foundation factor is dependent on frequency content of excitation, in contrast to the NBCC values which are frequency independent.

3. 27 out of 72 cases of the computed foundation factors exceeded the NBCC values. 25 out of these 27 cases correspond to the NBCC soil categories of I and II. This

reveals the inadequacy of NBCC foundation factors in these two categories. Most of the exceeded values belong to the soil-structure models whose structural fundamental period and site period are close to each other.

4. Analysis showed that the most of the exceeded code values are in short period sites such as DS soil profiles, particularly when the sites are subjected to high or intermediate frequency motions.

5. Dynamic soil-structure interaction analysis performed in this study reveals the existence of a quasi-resonance between soil and structure. Therefore, the above unity values of foundation factors are in the range of $1.0 < T/T_s < 10$ and the peak values of foundation factors fall in the range of $1.0 < T/T_s < 4.0$, in which T and T_s are fundamental period of structure and site period, respectively.

6. In contrast to the NBCC foundation factor values which are independent of frequency content of seismic excitation, the computed values not only are dependent on the frequency content of motion, but also sometimes they are in excess of the code values. Therefore, modifications to the current provision of NBCC foundation factor are proposed to account for the above observations. These modifications are based on a linear regression analysis between the computed and the code values. The proposed values of foundation factor, F^* , are the multiplication of the regression coefficient, C , and the current NBCC values of foundation factor with C not less than 1.0. The proposed F^* is applicable for $1.0 < T/T_s < 4.0$, and for T/T_s out of this range the NBCC values are adequate.

7. Results of this study show that the underestimation in base shear may be not only

due to the underestimation of the foundation factor but also to the underestimation of seismic response factor, S . In contrast to NBCC 90 claim that S is independent of a/v of excitations for fundamental periods of $T > 0.5$ sec, results show that S is dependent on a/v of excitations and the computed values are in excess of the code values.

8. A new approach of defining soil conditions for NBCC foundation factor is proposed to remedy the difficulty in defining soil categories in NBCC 90. The proposed approach is based on the thickness and the shear wave velocity profile of the soil deposit. Six shear wave velocity zones are introduced to clearly define different types of soil deposits.

Soil Type	Depth (m)	Ottawa (H a/v)	Vancouver (I a/v)	Prince Rupert (L a/v)
NC	10	6.0	5.0	5.3
	40	8.3	8.1	7.1
OC	10	4.7	4.1	4.2
	40	5.0	4.1	3.9
AS	10	4.1	3.3	4.5
	40	5.2	4.2	3.8
DS	10	3.3	5.0	5.5
	40	3.5	3.3	4.0

Table.6.1 Maximum Spectral Ratios

Soil Type	Ottawa (H a/v)		Vancouver (I a/v)		Prince Rupert (L a/v)	
	Inc. %	Dec. %	Inc. %	Dec. %	Inc. %	Dec. %
NC	38		62		34	
OC	6		0			7
AS	27		27			15
DS	6			34		27

Table.6.2 Percentile Increase or Decrease of Maximum Spectral Ratio Due to Increasing Depth From 10 to 40 m

NIBCC 90 Categories	Soil Models	Soil Structure Models	NIBCC 90 Elastic Base Shear (KN/m)			Computed Elastic Base Shear (KN/m)		
			II a/v	I a/v	I. a/v	II a/v	I a/v	I. a/v
I	OC10	B5-OC10	50.95	109.18	140.38	171.17	456.97	167.11
		B10-OC10	72.10	154.5	198.65	195.38	279.03	381.85
		B20-OC10	101.91	218.37	280.73	187.44	380.49	212.91
	DS10	B5-DS10	50.95	109.18	140.38	147.55	381.84	190.92
		B10-DS10	72.10	154.5	198.65	134.96	255.21	403.92
		B20-DS10	101.91	218.37	280.73	134.58	322.21	132.45
II	NC10	B5-NC10	66.24	141.93	182.49	68.55	294.52	257.24
		B10-NC10	93.73	200.85	258.25	88.88	199.25	147.55
		B20-NC10	132.48	285.74	367.33	119.86	455.72	311.75
	OC40	B5-OC40	66.24	141.93	182.49	108.44	326.85	217.45
		B10-OC40	93.73	200.85	258.25	171.92	338.67	485.83
		B20-OC40	132.48	285.74	367.33	119.08	318.14	181.63
	AS10	B5-AS10	66.24	141.93	182.49	108.05	312.72	160.52
		B10-AS10	93.73	200.85	258.25	152.97	150.65	220.55
		B20-AS10	132.48	285.74	367.33	146.00	480.41	169.14
	DS40	B5-DS40	66.24	141.93	182.49	300.91	848.12	495.70
		B10-DS40	93.73	200.85	258.25	100.55	364.03	492.02
		B20-DS40	132.48	285.74	367.33	260.05	793.90	266.83
III	AS40	B5-AS40	76.43	154.5	198.65	85.20	239.14	215.90
		B10-AS40	108.16	231.75	297.98	138.25	132.64	237.59
		B20-AS40	109.93	327.56	421.10	118.50	352.81	190.92
IV	NC40	B5-NC40	100.94	154.5	198.65	81.52	341.96	441.29
		B10-NC40	144.21	309.00	397.31	118.12	189.57	173.72
		B20-NC40	203.81	136.74	561.46	118.89	440.52	384.17

Table 6.3

Categories	Soil Condition	F
I	Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils; compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 15 m deep	1.0
II	Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15 m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15 m deep	1.3
III	Very loose and loose coarse-grained soils with depth greater than 15 m	1.5
IV	Very soft and soft fine-grained soils with depth greater than 15 m	2.0

Table.6.4 NBCC 90 Foundation factors

NIBCC 90 Categories	Soil Models	Soil-Structure Models	T / Ts (sec / sec.)	NIBCC 90 Foundation Factor			Computed Foundation Factor		
				II aN	I aN	I. aN	II aN	I aN	I. aN
I	OC10	B5-OC10	0.5/0.26	1.0	1.0	1.0	1.508°	1.532°	0.934
		B10-OC10	1.0/0.26	1.0	1.0	1.0	2.727°	0.957	0.997
		B20-OC10	2.0/0.26	1.0	1.0	1.0	1.763°	1.497°	0.981
	DS10	B5-DS10	0.5/0.14	1.0	1.0	1.0	1.30°	1.28°	1.067°
		B10-DS10	1.0/0.14	1.0	1.0	1.0	1.884°	0.875	1.055°
		B20-DS10	2.0/0.14	1.0	1.0	1.0	1.266°	1.267°	0.61
II	NC10	B5-NC10	0.5/0.43	1.3	1.3	1.3	0.604	0.988	1.438°
		B10-NC10	1.0/0.43	1.3	1.3	1.3	1.241	0.684	0.358
		B20-NC10	2.0/0.43	1.3	1.3	1.3	1.127	1.792°	1.436°
	OC40	B5-OC40	0.5/0.70	1.3	1.3	1.3	0.956	1.096	1.215
		B10-OC40	1.0/0.70	1.3	1.3	1.3	2.414°	1.162	1.269
		B20-OC40	2.0/0.70	1.3	1.3	1.3	1.12	1.251	0.837
	AS10	B5-AS10	0.5/0.28	1.3	1.3	1.3	0.952	1.039	0.897
		B10-AS10	1.0/0.28	1.3	1.3	1.3	2.135°	0.517	0.576
		B20-AS10	2.0/0.28	1.3	1.3	1.3	1.373°	1.89°	0.779
	DS40	B5-DS40	0.5/0.39	1.3	1.3	1.3	2.652°	2.844°	2.771°
		B10-DS40	2.0/0.39	1.3	1.3	1.3	1.403°	1.249	1.285
		B20-DS40	1.0/0.39	1.3	1.3	1.3	2.446°	3.123°	1.229
III	AS40	B5-AS40	0.5/0.83	1.5	1.5	1.5	0.751	0.802	1.206
		B10-AS40	1.0/0.83	1.5	1.5	1.5	1.93°	0.455	0.621
		B20-AS40	2.0/0.83	1.5	1.5	1.5	1.115	1.398	0.88
IV	NC40	B5-NC40	0.5/1.01	2.0	1.41	1.41	0.718	1.147	2.446°
		B10-NC40	1.0/1.01	2.0	2.0	2.0	1.649	0.651	0.453
		B20-NC40	2.0/1.01	2.0	2.0	2.0	1.12	1.73	1.77

Table 6.5

Categories	Soil Models	NBCC 90 Foundation Factor, F	Regression Coeff., C			$F^2 = C \times F$		
			II a/v	I a/v	L a/v	II a/v	I a/v	L a/v
I	OC10	1.0	1.58	1.11	0.91	1.6	1.2	1.0
	DS10	1.0	1.58	1.5	1.13	1.6	1.5	1.2
II	NC10	1.3	0.67	0.78	0.89	1.3	1.3	1.3
	OC40	1.3	1.58	1.11	0.91	2.0	1.4	1.3
	AS10	1.3	1.0	0.74	0.59	1.3	1.3	1.3
	DS40	1.3	1.58	1.5	1.13	2.0	2.0	1.5
III	AS40	1.5	1.0	0.74	0.59	1.5	1.5	1.5
IV	NC40	2.0	0.67	0.78	0.89	2.0	2.0	2.0

Table.6.6 Proposed Modifications on NBCC 90 foundation factors for Ottawa, Vancouver, and Prince Rupert

Categories	Soil Type	Depth (m)	Shear Wave Velocity zone	F [*]		
				H a/v	I a/v	L a/v
I	OC	< 15	2	1.6	1.2	1.0
	DS	< 15	3	1.6	1.5	1.2
II	NC	< 15	1	1.3	1.3	1.3
	OC	> 15	5	2.0	1.4	1.3
	AS	< 15	2	1.3	1.3	1.3
	DS	> 15	6	2.0	2.0	1.5
III	AS	> 15	5	1.5	1.5	1.5
IV	NC	> 15	4	2.0	2.0	2.0

Table.6.7 Proposed foundation factors for NBCC including shear wave velocity zonation for Ottawa, Vancouver, and Prince Rupert

Fig.6.1 Free field spectral ratio of 10 m deep of various soil deposits subjected to H a/v.

Fig.6.2 Free field spectral ratio of 10 m deep of various soil deposits subjected to 1 a/v.

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MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

Fig.6.3 Free field spectral ratio of 10 m deep of various soil deposits subjected to L a/v.

Fig.6.4 Free field spectral ratio of 40 m deep of various soil deposits subjected to H a/v.

Fig.6.5 Free field spectral ratio of 40 m deep of various soil deposits subjected to I a/v.

Fig.6.6 Free field spectral ratio of 40 m deep of various soil deposits subjected to L a/v.

Fig.6.7 Free field spectral ratios of 10 and 40 m of NC soil profile subjected to H a/v.

Fig.6.8 Free field spectral ratios of 10 and 40 m of NC soil profile subjected to I a/v.

Fig.6.9 Free field spectral ratios of 10 and 40 m of NC soil profile subjected to L a/v.

Fig.6.10 Free field spectral ratios of 10 and 40 m of OC soil profile subjected to H a/v.

Fig.6.11 Free field spectral ratios of 10 and 40 m of OC soil profile subjected to 1 a/v.

Fig.6.12 Free field spectral ratios of 10 and 40 m of OC soil profile subjected to L a/v.

Fig.6.13 Free field spectral ratios of 10 and 40 m of AS soil profile subjected to H a/v.

Fig.6.14 Free field spectral ratios of 10 and 40 m of AS soil profile subjected to I a/v.

Fig.6.15 Free field spectral ratios of 10 and 40 m of AS soil profile subjected to L a/v.

Fig.6.16 Free field spectral ratios of 10 and 40 m of DS soil profile subjected to H a/v.

Fig.6.17 Free field spectral ratios of 10 and 40 m of DS soil profile subjected to I a/v.

Fig.6.18 Free field spectral ratios of 10 and 40 m of DS soil profile subjected to L a/v.

$$V = V_{ij} + V_{ij}$$

$$V_{ij} = \frac{M_{mi} + M_{mj}}{l}$$

$$V_{ij} = \frac{M_{ni} + M_{nj}}{l}$$

$$V = \frac{M_{mi} + M_{mj} + M_{ni} + M_{nj}}{l}$$

Fig 6.19 Schematic view of free-body diagram of a multi-storey building.

Fig.6.20 Base shear time histories, V_r , of 5-storey building located on rock and subjected to various ground motions.

Fig.6.21 Base shear time histories, V_r , of 10-storey building located on rock and subjected to various ground motions.

Fig.6.22 Base shear time histories, V_r , of 20-storey building located on rock and subjected to various ground motions.

Fig.6.23 NBCC 90 Seismic Response Factor, S .

Fig.6.24 Computed as well as NBCC foundation factors for OC10 soil model, falling in NBCC category I.

Fig.6.25 Computed as well as NBCC foundation factors for DS10 soil model, falling in NBCC category I.

Fig.6.26 Computed as well as NBCC foundation factors for NC10 soil model, falling in NBCC category II.

Fig.6.27 Computed as well as NBCC foundation factors for OC40 soil model, falling in NBCC category II.

Fig.6.28 Computed as well as NBCC foundation factors for AS10 soil model, falling in NBCC category II.

Fig.6.29 Computed as well as NBCC foundation factors for DS40 soil model, failing in NBCC category II.

Fig.6.30 Computed as well as NBCC foundation factors for AS40 soil model, falling in NBCC category III.

Fig.6.31 Computed as well as NBCC foundation factors for NC40 soil model, falling in NBCC category IV.

Fig.6.32 Two examples of NBCC exceeded foundation factor values.

Fig.6.33 Computed foundation factors versus T/T_s for NC soil type subjected to various ground motions.

Fig.6.34 Computed foundation factors versus T/T_s for OC soil type subjected to various ground motions.

Fig.6.35 Computed foundation factors versus T/T_s for AS soil type subjected to various ground motions.

Fig.6.36 Computed foundation factors versus T/T_s for DS soil type subjected to various ground motions.

Fig.6.37 Foundation factor regression analysis results for NC soil profile and different ground motions.

Fig.6.38 Foundation factor regression analysis results for OC soil profile and different ground motions.

Fig.6.39 Foundation factor regression analysis results for AS soil profile and different ground motions.

Fig.6.40 Foundation factor regression analysis results for DS soil profile and different ground motions.

Fig.6.41 Base shear regression analysis results for total existing soil-structure models and different ground motions.

Fig.6.42 Foundation factor regression analysis results for total existing soil-structure models and different ground motions.

Fig 6.43 Proposed soil condition zonation for NBCC foundation factors based on shear wave velocity profile of soil deposit

Chapter 7

SUMMARY AND CONCLUSION

7.1 GENERAL

To investigate the site amplification and soil-structure interaction implications on the current NBCC provisions of design base shear force, V , and in turn, foundation factor, F , soil-structure models were developed. The FLUSH program was adopted for numerical analysis. In addition this program was modified to compute the time history of the base shear of a multi-storey building.

The soil types considered are: normally to lightly overconsolidated clay, NC, heavily overconsolidated, OC, alluvial sand and silt, AS, and dense sand, DS. Two soil depths of 10 and 40 m have been considered to represent shallow and deep foundation soils. To cover a wide range of actual structures with various fundamental periods, three one-bay moment-resisting frame buildings of 5-, 10-, and 20-storey are included.

To study the effects of earthquake motions with different frequency contents on the response of soil deposits as well as soil-structure systems, three acceleration time histories with completely different frequency contents have been selected. These time histories are

histories are scaled to be compatible with the seismicity of three locations in Canada, namely: Ottawa, Vancouver, and Prince Rupert. These three records correspond to the three ranges of a/v ratio in NBCC 90.

7.2 CONTRIBUTIONS AND CONCLUSIONS OF THESIS

By using three-dimensional computational model incorporated in the FLUSH program which takes into account for interaction between soil and structure and energy absorbing nature of soil medium, the effects of soil conditions and structural types, as two substructures of one system, on soil-structure response have been analyzed. This is in contrast to the previous studies whose analytical model was one-dimensional and thus neglected the interaction effects. The following are the primary conclusions of this part of study.

- 1) All soil types amplify the bedrock motion to different degrees. Soil-structure systems with lower damping in the final stage of dynamic analysis, have higher response, and amplify the bedrock motion more than the other cases. In other words, those combinations of soil condition, structural model and seismic excitation, which lead to lower effective damping, will amplify the bedrock motion more significantly.
- 2) Soil-structure models involving soft clay, NC, and dense sand, DS, have the longest and the shortest periods at their maximum responses, respectively.
- 3) Alluvial sand, AS, and heavily overconsolidated clay, OC behave similarly when they subjected to the same seismic excitations.
- 4) The oscillations in the variation of maximum acceleration versus depth is an indication

of energy dissipation and the turning points represent the concentration of high shear strain levels.

- 5) Quasi-resonance between soil and structure has been observed when the fundamental period of the building is in the proximity of the site period. This phenomenon is more pronounced when the soil-structure system is subjected to low frequency motions than to high frequency motions. Because it is shown that dynamic loads excite only certain modes of structure, mostly those of low frequency.
- 6) The depth from the soil surface within which the site amplification and soil-structure interaction effects start to appear in homogeneous soil media increases as the frequency content of excitations changes from H a/v to L a/v.
- 7) Short period soil-structure systems significantly amplify the H a/v motions while long period systems significantly amplify the L a/v motions.

The design implications of site amplification and soil-structure interaction due to seismic ground motions have been investigated. First, for each of the sites, free-field response analysis was performed to study the effects of soil type, soil depth, and frequency content of excitations on site amplification. The following are the main conclusions of this part of study.

- 1) Site amplification in NC soil profiles is the highest, compared to the other soil types, with spectral ratio of up to 8.3, while DS profiles have the lowest values of spectral ratios. Site amplifications in firm clay, OC, and alluvial and silt, AS are fairly similar,

in fact, the peak spectral ratios almost for all the cases are interestingly close to each other.

- 2) All soil types amplify bedrock motions, as a matter of fact the peak spectral ratios are always in excess of 3.3.
- 3) Increasing the depth from 10 to 40 m in NC profile always results in increasing spectral ratio. The amplification does not change considerably in firm clay deposits, OC, when the depth increases from 10 to 40 m. Surface excitation response in shallow deposits of dense sand, DS, is much more severe than that of deep deposits, as reflected by the dramatic decrease of spectral ratios of up to 34% when the depth increases from 10 to 40 m.
- 4) Based on the results of this study, there is always a decreasing trend of peak spectral ratios particularly in deep deposits of NC, OC, and AS when the a/v of the ground motions goes from H to L. The reverse is true for dense sand, DS.

Time history of structural base shear force for all the mentioned soil-structure models were computed. Then, foundation factors, for all the cases considered, were computed as being the ratio of the computed maximum base shear for the structure situated on a soil deposit to the corresponding base shear for an identical structure assumed to be situated directly on bedrock and subjected to the same bedrock motions. Based on the results of these analyses, the following are the main conclusions:

- 1) The base shear amplification is most significant when the earthquake excitation has significant energy in the site period region. Long period sites (i.e., soft deep soil deposits) have the largest amplification when subjected to seismic motions with low

a/v ratio. Short period sites (stiff shallow soil deposits), on the other hand, have the largest amplification when subjected to seismic motions with high a/v ratio. However, substantial amplification occurs at structural periods near site periods for any kind of seismic excitation.

- 2) For short period sites foundation factor decreases as the a/v of excitations decreases, indicating that the foundation factor is dependent on frequency content of excitation, in contrast to the NBCC values which are frequency independent.
- 3) 27 out of 72 cases of the computed foundation factors exceeded the NBCC values. 25 out of these 27 cases correspond to the NBCC soil categories of I and II. This reveals the inadequacy of NBCC foundation factors in these two categories. Most of the exceeded values belong to the soil-structure models whose structural fundamental period and site period are close to each other.
- 4) Analysis showed that the most of the exceeded code values are in short period sites such as DS soil profiles, particularly when the sites are subjected to high or intermediate frequency motions.
- 5) Dynamic soil-structure interaction analysis performed in this study reveals the existence of a quasi-resonance between soil and structure. Therefore, the above unity values of foundation factors are in the range of $1.0 < T/T_s < 10$ and the peak values of foundation factors fall in the range of $1.0 < T/T_s < 4.0$, in which T and T_s are fundamental period of structure and site period, respectively.
- 6) In contrast to the NBCC foundation factor values which are independent of frequency content of seismic excitation, the computed values not only are dependent on the

frequency content of motion, but also sometimes they are in excess of the code values. Therefore, modifications to the current provision of NBCC foundation factor are proposed to account for the above observations. These modifications are based on a linear regression analysis between the computed and the code values. The proposed values of foundation factor, F^* , are the multiplication of the regression coefficient, C , and the current NBCC values of foundation factor with C not less than 1.0. The proposed F^* is applicable for $1.0 < T/T_s < 4.0$, and for T/T_s out of this range the NBCC values are adequate.

- 7) Results of this study show that the underestimation in base shear may be not only due to the underestimation of the foundation factor but also to the underestimation of seismic response factor, S . In contrast to NBCC 90 claim that S is independent of a/v of excitations for fundamental periods of $T > 0.5$ sec, results show that S is dependent on a/v of excitations and the computed values are in excess of the code values.
- 8) A new approach of defining soil conditions for NBCC foundation factor is proposed to remedy the difficulty in defining soil categories in NBCC 90. The proposed approach is based on the thickness and the shear wave velocity profile of the soil deposit. Six shear wave velocity zones are introduced to clearly define different types of soil deposits.

7.3 SUGGESTIONS FOR FUTURE WORK

- 1) As is mentioned repeatedly throughout this thesis, the analyses and, therefore, the resulting conclusions are made only for three a/v ratios corresponding to three specific

locations in Canada, namely: Ottawa, Vancouver, and Prince Rupert. To cover all locations in Canada each with its own specific seismic hazard parameters, work should be performed using more acceleration time histories to account for those a/v ratios which are in between the ones used in the current study. This will have at least two benefits. First, more locations will be covered. Secondly, more data will be available for statistical analyses such as the linear regression analysis to yield more precise results.

- 2) Future studies should be conducted to derive representative values for the seismic response factor, S , for a wide variety of a/v ratios.
- 3) The soil condition zonation proposed for current NBCC provisions of foundation factor should be tested for more shear wave velocity profiles of actual sites.
- 4) Subjected to strong seismic excitations, soils behave strongly nonlinearly. True nonlinear analysis are, however, not yet possible. The fact that only linear or equivalent linear analyses are possible, like the computational method adopted in the FLUSH program, shows that present state of the art of soil-structure interaction for seismic excitation is still in its primary stage. Non-linear methods should be developed, which will eventually allow permanent settlements to be calculated. This can be accomplished by developing nonlinear methods which account for nonlinearity in soil behaviour when excited by strong motions. Some promising nonlinear constitutive models for soils do exist and future development is needed for practical applications.

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Appendix A:

A.1 Input Data of 10-storey Building Located on Bedrock

```

1
SOIL-STRUCTURE INTERACTION -- 10 STOREY Bldg. ON ROCK
6 30 0 56 50 24 1 0 1 98.43
2 1
BOTH LEFT
16 8 0.02 0.0 .2
HORI
15.0 3
5.0 4
10.0 8
15.0 16
2 1 1 1 5 0

15.0

0.05
1 1
1 1 30 51 2 1 0 0.18 150.0 261774.00 261774.00 0.050
2 2 51 52 4 1 0 0.18 150.0 261774.00 261774.00 0.050
3 4 52 53 5 1 0 0.18 150.0 261774.00 261774.00 0.050
4 5 53 54 6 1 0 0.18 150.0 261774.00 261774.00 0.050
5 6 54 55 47 1 0 0.18 150.0 261774.00 261774.00 0.050
6 47 55 56 49 1 0 0.18 150.0 261774.00 261774.00 0.050
7 2 3 7 8 -2 0.18 150.0 261774.00 261774.00 0.050
0.3691 0.1860 0.0
8 7 8 45 46 -2 0.18 150.0 261774.00 261774.00 0.050
0.78 0.6728 0.0
9 7 8 9 -2 0.18 150.0 261774.00 261774.00 0.050
0.3691 0.1860 0.0
10 9 10 43 44 -2 0.18 150.0 261774.00 261774.00 0.050
0.78 0.6728 0.0
11 9 10 11 12 -2 0.18 150.0 261774.00 261774.00 0.050
0.3215 0.1410 0.0
12 11 12 41 42 -2 0.18 150.0 261774.00 261774.00 0.050
0.78 0.6728 0.0
13 11 12 13 14 -2 0.18 150.0 261774.00 261774.00 0.050
0.3215 0.1410 0.0
14 13 14 39 40 -2 0.18 150.0 261774.00 261774.00 0.050
0.78 0.6728 0.0
15 13 14 15 16 -2 0.18 150.0 261774.00 261774.00 0.050
0.2772 0.1050 0.0

```

16	15	16	37	38	-2	0.18	150.0	261774.00	261774.00	0.050
	0.78		0.6728	0.0						
17	15	16	17	18	-2	0.18	150.0	261774.00	261774.00	0.050
	0.2772		0.1050	0.0						
18	17	18	35	36	-2	0.18	150.0	261774.00	261774.00	0.050
	0.78		0.6728	0.0						
19	17	18	19	20	-2	0.18	150.0	261774.00	261774.00	0.050
	0.2362		0.0760	0.0						
20	19	20	33	34	-2	0.18	150.0	261774.00	261774.00	0.050
	0.78		0.6728	0.0						
21	19	20	21	22	-2	0.18	150.0	261774.00	261774.00	0.050
	0.2362		0.0760	0.0						
22	21	22	31	32	-2	0.18	150.0	261774.00	261774.00	0.050
	0.78		0.6728	0.0						
23	21	22	23	24	-2	0.18	150.0	261774.00	261774.00	0.050
	0.1985		0.0540	0.0						
24	23	24	29	30	-2	0.18	150.0	261774.00	261774.00	0.050
	0.78		0.6728	0.0						
25	23	24	25	26	-2	0.18	150.0	261774.00	261774.00	0.050
	0.1985		0.0540	0.0						
26	25	26	27	28	-2	0.18	150.0	261774.00	261774.00	0.050
	0.78		0.6728	0.0						
27	27	28	29	30	-2	0.18	150.0	261774.00	261774.00	0.050
	0.1985		0.0540	0.0						
28	29	30	31	32	-2	0.18	150.0	261774.00	261774.00	0.050
	0.1985		0.0540	0.0						
29	31	32	33	34	-2	0.18	150.0	261774.00	261774.00	0.050
	0.2362		0.0760	0.0						
30	33	34	35	36	-2	0.18	150.0	261774.00	261774.00	0.050
	0.2362		0.0760	0.0						
31	35	36	37	38	-2	0.18	150.0	261774.00	261774.00	0.050
	0.2772		0.1050	0.0						
32	37	38	39	40	-2	0.18	150.0	261774.00	261774.00	0.050
	0.2772		0.1050	0.0						
33	39	40	41	42	-2	0.18	150.0	261774.00	261774.00	0.050
	0.3215		0.1410	0.0						
34	41	42	43	44	-2	0.18	150.0	261774.00	261774.00	0.050
	0.3215		0.1410	0.0						
35	43	44	45	46	-2	0.18	150.0	261774.00	261774.00	0.050
	0.3691		0.1860	0.0						
36	45	46	47	48	-2	0.18	150.0	261774.00	261774.00	0.050
	0.3691		0.1860	0.0						
1		00.00		4.100						
2		8.202		4.100						
3		8.202		4.100	5					
4		16.41		4.100						
5		24.61		4.100						
6		32.81		4.100						
7		8.202		19.18		1.680				
8		8.202		19.18	4					
9		8.202		30.99		1.680				
10		8.202		30.99	4					
11		8.202		42.80		1.680				
12		8.202		42.80	4					

13	8.202	54.61		1.680
14	8.202	54.61	4	
15	8.202	66.420		1.680
16	8.202	66.420	4	
17	8.202	78.230		1.680
18	8.202	78.230	4	
19	8.202	90.04		1.680
20	8.202	90.04	4	
21	8.202	101.85		1.680
22	8.202	101.85	4	
23	8.202	113.66		1.680
24	8.202	113.66	4	
25	8.202	125.47		1.680
26	8.202	125.47	4	
27	41.01	125.47		1.680
28	41.01	125.47	4	
29	41.01	113.66		1.680
30	41.01	113.66	4	
31	41.01	101.85		1.680
32	41.01	101.85	4	
33	41.01	90.04		1.680
34	41.01	90.04	4	
35	41.01	78.230		1.680
36	41.01	78.230	4	
37	41.01	66.420		1.680
38	41.01	66.420	4	
39	41.01	54.61		1.680
40	41.01	54.61	4	
41	41.01	42.80		1.680
42	41.01	42.80	4	
43	41.01	30.99		1.680
44	41.01	30.99	4	
45	41.01	19.18		1.680
46	41.01	19.18	4	
47	41.01	4.100		
48	41.01	4.100	5	
49	49.22	4.100		
50	00.00	00.00		
51	8.202	00.00		
52	16.41	00.00		
53	24.61	00.00		
54	32.81	00.00		
55	41.01	00.00		
56	49.22	00.00		

CONTROL MOTION

-0.000284-0.000440-0.000189 0.001094 0.001546-0.001720-0.002011-0.000473

1	1				
2					
1	0	0	5	0	2
7	1				
36	1				
0					

A.2 Input Data of 10-storey Building Located on 10 m of

Alluvial Sand and Silt Deposit Overlying Bebrock

```

1
SOIL STRUCTURE INTERACTION-- 10 STOREY Bldg. ON 10m ALLUVIAL SAND
72 30 0 132 124 24 4 1 9 98.43
10 10
BOTH LEFT
16 9 0.02 0.0 0.2
HORI
15.0 3
5.0 4
10.0 8
15.0 16
2 1 9 1 5 0
15.0 15.0 15.0 15.0
0.10 100.00 50
0.05
38 1
39 111
40 111
41 1
42 1
43 1
44 1
45 1
46 1
1 1 2 11 10 1 1 0.20 125.0 247.01 92.5 0.057
2 2 3 12 11 2 1 0.20 125.0 320.01 130.5 0.065
3 3 4 13 12 3 1 0.20 125.0 392.01 172.5 0.063
4 4 5 14 13 4 1 0.20 125.0 473.01 211.5 0.055
5 5 6 15 14 5 1 0.20 125.0 561.74 253.5 0.046
6 6 7 16 15 6 1 0.20 125.0 662.01 302.5 0.040
7 7 8 17 16 7 1 0.20 125.0 747.01 343.5 0.036
8 8 9 18 17 8 1 0.20 125.0 853.010 384.5 0.031
9 9 124 125 18 9 1 0.20 125.0 980.010 420.5 0.031
10 10 11 21 19 1 0 0.18 150.0 261774.00 261774.00 0.050
11 11 12 22 21 2 1 0.20 125.0 320.01 130.5 0.100
12 12 13 23 22 3 1 0.20 125.0 392.01 172.5 0.100
13 13 14 24 23 4 1 0.20 125.0 473.01 211.5 0.100
14 14 15 25 24 5 1 0.20 125.0 561.74 253.5 0.100
15 15 16 26 25 6 1 0.20 125.0 662.01 302.5 0.100
16 16 17 27 26 7 1 0.20 125.0 747.01 343.5 0.100

```

17	17	18	28	27 8	1	0.20	125.0	853.010	384.5	0.100
18	18	125	126	28 9	1	0.20	125.0	980.010	420.5	0.100
19	19	21	30	29 1	0	0.18	150.0	261774.00	261774.00	0.050
20	21	22	31	30 2	1	0.20	125.0	320.01	130.5	0.100
21	22	23	32	31 3	1	0.20	125.0	392.01	172.5	0.100
22	23	24	33	32 4	1	0.20	125.0	473.01	211.5	0.100
23	24	25	34	33 5	1	0.20	125.0	561.74	253.5	0.100
24	25	26	35	34 6	1	0.20	125.0	662.01	302.5	0.100
25	26	27	36	35 7	1	0.20	125.0	747.01	343.5	0.100
26	27	28	37	36 8	1	0.20	125.0	853.010	384.5	0.100
27	28	126	127	37 9	1	0.20	125.0	980.010	420.5	0.100
28	29	30	39	38 1	0	0.18	150.0	261774.00	261774.00	0.050
29	30	31	40	39 2	1	0.20	125.0	320.01	130.5	0.100
30	31	32	41	40 3	1	0.20	125.0	392.01	172.5	0.100
31	32	33	42	41 4	1	0.20	125.0	473.01	211.5	0.100
32	33	34	43	42 5	1	0.20	125.0	561.74	253.5	0.100
33	34	35	44	43 6	1	0.20	125.0	662.01	302.5	0.100
34	35	36	45	44 7	1	0.20	125.0	747.01	343.5	0.100
35	36	37	46	45 8	1	0.20	125.0	853.010	384.5	0.100
36	37	127	128	46 9	1	0.20	125.0	980.010	420.5	0.100
37	38	39	48	47 1	0	0.18	150.0	261774.00	261774.00	0.050
38	39	30	49	48 2	1	0.20	125.0	320.01	130.5	0.100
39	40	41	50	49 3	1	0.20	125.0	392.01	172.5	0.100
40	41	42	51	50 4	1	0.20	125.0	473.01	211.5	0.100
41	42	43	52	51 5	1	0.20	125.0	561.74	253.5	0.100
42	43	44	53	52 6	1	0.20	125.0	662.01	302.5	0.100
43	44	45	54	53 7	1	0.20	125.0	747.01	343.5	0.100
44	45	46	55	54 8	1	0.20	125.0	853.010	384.5	0.100
45	46	128	129	55 9	1	0.20	125.0	980.010	420.5	0.100
46	47	48	98	96 1	0	0.18	150.0	261774.00	261774.00	0.050
47	48	49	99	98 2	1	0.20	125.0	320.01	130.5	0.100
48	49	50	100	99 3	1	0.20	125.0	392.01	172.5	0.100
49	50	51	101	100 4	1	0.20	125.0	473.01	211.5	0.100
50	51	52	102	101 5	1	0.20	125.0	561.74	253.5	0.100
51	52	53	103	102 6	1	0.20	125.0	662.01	302.5	0.100
52	53	54	104	103 7	1	0.20	125.0	747.01	343.5	0.100
53	54	55	105	104 8	1	0.20	125.0	853.010	384.5	0.100
54	55	129	130	105 9	1	0.20	125.0	980.010	420.5	0.100
55	96	98	107	106 1	0	0.18	150.0	261774.00	261774.00	0.050
56	98	99	108	107 2	1	0.20	125.0	320.01	130.5	0.100
57	99	100	109	108 3	1	0.20	125.0	392.01	172.5	0.100
58	100	101	110	109 4	1	0.20	125.0	473.01	211.5	0.100
59	101	102	111	110 5	1	0.20	125.0	561.74	253.5	0.100
60	102	103	112	111 6	1	0.20	125.0	662.01	302.5	0.100
61	103	104	113	112 7	1	0.20	125.0	747.01	343.5	0.100
62	104	105	114	113 8	1	0.20	125.0	853.010	384.5	0.100
63	105	130	131	114 9	1	0.20	125.0	980.010	420.5	0.100
64	106	107	116	115 1	1	0.20	125.0	247.01	92.5	0.100
65	107	108	117	116 2	1	0.20	125.0	320.01	130.5	0.100
66	108	109	118	117 3	1	0.20	125.0	392.01	172.5	0.100
67	109	110	119	118 4	1	0.20	125.0	473.01	211.5	0.100
68	110	111	120	119 5	1	0.20	125.0	561.74	253.5	0.100
69	111	112	121	120 6	1	0.20	125.0	662.01	302.5	0.100
70	112	113	122	121 7	1	0.20	125.0	747.01	343.5	0.100

71	113	114	123	122	8	1	0.20	125.0	853.010	384.5	0.100
72	114	131	132	123	9	1	0.20	125.0	980.010	420.5	0.100
73	19	20	56	57	-2		0.18	150.0	261774.00	261774.00	0.050
	0.3691	0.1860	0.0								
74	56	57	94	95	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
75	56	57	58	59	-2		0.18	150.0	261774.00	261774.00	0.050
	0.3691	0.1860	0.0								
76	58	59	92	93	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
77	58	59	60	61	-2		0.18	150.0	261774.00	261774.00	0.050
	0.3215	0.1410	0.0								
78	60	61	90	91	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
79	60	61	62	63	-2		0.18	150.0	261774.00	261774.00	0.050
	0.3215	0.1410	0.0								
80	62	63	88	89	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
81	62	63	64	65	-2		0.18	150.0	261774.00	261774.00	0.050
	0.2772	0.1050	0.0								
82	64	65	86	87	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
83	64	65	66	67	-2		0.18	150.0	261774.00	261774.00	0.050
	0.2772	0.1050	0.0								
84	66	67	84	85	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
85	66	67	68	69	-2		0.18	150.0	261774.00	261774.00	0.050
	0.2362	0.0760	0.0								
86	68	69	82	83	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
87	68	69	70	71	-2		0.18	150.0	261774.00	261774.00	0.050
	0.2362	0.0760	0.0								
88	70	71	80	81	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
89	70	71	72	73	-2		0.18	150.0	261774.00	261774.00	0.050
	0.1985	0.0540	0.0								
90	72	73	78	79	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
91	72	73	74	75	-2		0.18	150.0	261774.00	261774.00	0.050
	0.1985	0.0540	0.0								
92	74	75	76	77	-2		0.18	150.0	261774.00	261774.00	0.050
	0.78	0.6728	0.0								
93	76	77	78	79	-2		0.18	150.0	261774.00	261774.00	0.050
	0.1985	0.0540	0.0								
94	78	79	80	81	-2		0.18	150.0	261774.00	261774.00	0.050
	0.1985	0.0540	0.0								
95	80	81	82	83	-2		0.18	150.0	261774.00	261774.00	0.050
	0.2362	0.0760	0.0								
96	82	83	84	85	-2		0.18	150.0	261774.00	261774.00	0.050
	0.2362	0.0760	0.0								
97	84	85	86	87	-2		0.18	150.0	261774.00	261774.00	0.050
	0.2772	0.1050	0.0								
98	86	87	88	89	-2		0.18	150.0	261774.00	261774.00	0.050
	0.2772	0.1050	0.0								

99	88	89	90	91	-2	0.18	150.0	261774.00	261774.00	0.050
	0.3215		0.1410	0.0						
100	90	91	92	93	-2	0.18	150.0	261774.00	261774.00	0.050
	0.3215		0.1410	0.0						
101	92	93	94	95	-2	0.18	150.0	261774.00	261774.00	0.050
	0.3691		0.1860	0.0						
102	94	95	96	97	-2	0.18	150.0	261774.00	261774.00	0.050
	0.3691		0.1860	0.0						
1	SEED AND IDRISS (1972)					--	SAND			
1.0	0.98	0.97	0.93	0.85	0.70	0.52	0.25	0.15	0.10	0.04
0.02	0.30	0.70	1.36	2.65	5.00	8.75	14.50	20.0	22.0	24.0
1		00.00		32.81						
2		00.00		30.07						
3		00.00		27.34						
4		00.00		24.61						
5		00.00		21.87						
6		00.00		18.23						
7		00.00		14.58						
8		00.00		10.93						
9		00.00		5.47						
10		8.202		32.81						
11		8.202		30.07						
12		8.202		27.34						
13		8.202		24.61						
14		8.202		21.87						
15		8.202		18.23						
16		8.202		14.58						
17		8.202		10.93						
18		8.202		5.47						
19		16.41		32.81						
20		16.41		32.81	5					
21		16.41		30.07						
22		16.41		27.34						
23		16.41		24.61						
24		16.41		21.87						
25		16.41		18.23						
26		16.41		14.58						
27		16.41		10.93						
28		16.41		5.47						
29		24.61		32.81						
30		24.61		30.07						
31		24.61		27.34						
32		24.61		24.61						
33		24.61		21.87						
34		24.61		18.23						
35		24.61		14.58						
36		24.61		10.93						
37		24.61		5.47						
38		32.81		32.81						
39		32.81		30.07						
40		32.81		27.34						
41		32.81		24.61						
42		32.81		21.87						
43		32.81		18.23						

44	32.81	14.58		
45	32.81	10.93		
46	32.81	5.47		
47	41.02	32.81		
48	41.02	30.07		
49	41.02	27.34		
50	41.02	24.61		
51	41.02	21.87		
52	41.02	18.23		
53	41.02	14.58		
54	41.02	10.93		
55	41.02	5.47		
56	16.41	47.89		1.680
57	16.41	47.89	4	
58	16.41	59.70		1.680
59	16.41	59.70	4	
60	16.41	71.51		1.680
61	16.41	71.51	4	
62	16.41	83.32		1.680
63	16.41	83.32	4	
64	16.41	95.13		1.680
65	16.41	95.13	4	
66	16.41	106.94		1.680
67	16.41	106.94	4	
68	16.41	118.75		1.680
69	16.41	118.75	4	
70	16.41	130.56		1.680
71	16.41	130.56	4	
72	16.41	142.37		1.680
73	16.41	142.37	4	
74	16.41	154.18		1.680
75	16.41	154.18	4	
76	49.22	154.18		1.680
77	49.22	154.18	4	
78	49.22	142.37		1.680
79	49.22	142.37	4	
80	49.22	130.56		1.680
81	49.22	130.56	4	
82	49.22	118.75		1.680
83	49.22	118.75	4	
84	49.22	106.94		1.680
85	49.22	106.94	4	
86	49.22	95.13		1.680
87	49.22	95.13	4	
88	49.22	83.32		1.680
89	49.22	83.32	4	
90	49.22	71.51		1.680
91	49.22	71.51	4	
92	49.22	59.70		1.680
93	49.22	59.70	4	
94	49.22	47.89		1.680
95	49.22	47.89	4	
96	49.22	32.81		
97	49.22	32.81	5	

98	49.22	30.07
99	49.22	27.34
100	49.22	24.61
101	49.22	21.87
102	49.22	18.23
103	49.22	14.58
104	49.22	10.93
105	49.22	5.47
106	57.42	32.81
107	57.42	30.07
108	57.42	27.34
109	57.42	24.61
110	57.42	21.87
111	57.42	18.23
112	57.42	14.58
113	57.42	10.93
114	57.42	5.47
115	65.62	32.81
116	65.62	30.07
117	65.62	27.34
118	65.62	24.61
119	65.62	21.87
120	65.62	18.23
121	65.62	14.58
122	65.62	10.93
123	65.62	5.47
124	00.00	00.00
125	8.202	00.00
126	16.41	00.00
127	24.61	00.00
128	32.81	00.00
129	41.02	00.00
130	49.22	00.00
131	57.42	00.00
132	65.62	00.00

CONTROL MOTION

-0.000284-0.000440-0.000189 0.001094 0.001546-0.001720-0.002011-0.000473

1	111
2	111
3	111
4	1
5	1
6	1
7	1
8	1
9	1
10	111

2					
1	0	0	5	0	2

73	1
102	1
0	

Appendix B:

Fig.B.1

Fig.B.2

Fig.B.3

Fig.B.4

Fig.B.5

Fig.B.6

Fig.B.7

Fig.B.8

Fig.B.9

Fig.B.10

Fig.B.11

Fig.B.12

END

2 0-0 7-9 4

FIN