Site-specific ground response analysis

L. GovindaRaju[1,*], G. V. Ramana[2], C. HanumanthaRao[2] and T. G. Sitharam[1]

[1]Department of Civil Engineering, Indian Institute of Science, Bangalore 560 012, India
[2]Department of Civil Engineering, Indian Institute of Technology, Delhi 110 016, India

The local soil conditions have a profound influence on ground response during earthquakes. The recent destructive earthquakes have again demonstrated that the topography, nature of the bedrock and nature and geometry of the depositional soils are the primary factors that influence local modifications to the underlying motion. We highlight the engineering importance of site-specific ground response analysis and difficulties faced in conducting a complete ground response analysis. Steps to be followed in conducting a meaningful site amplification study are explained. Difficulties/uncertainties in choosing an input ground motion are discussed and the various methods currently available for site amplification study are summarized. A case study on ground response analysis of a site in Ahmedabad City during the Bhuj earthquake is presented. The study shows that the varying degree of damage to multistorey buildings in the close proximity of Sabarmati river area in Ahmedabad was essentially due to amplification of the ground. Amplification was attributed to the fact that the natural frequencies of the building and frequency content of the ground motion recorded on the ground floor of the Regional Passport Staff Quarters building are in close proximity.

THE very basic problem to be solved by geotechnical engineers in regions where earthquake hazards exist is to estimate the site-specific dynamic response of a layered soil deposit. The problem is commonly referred to as a site-specific response analysis or soil amplification study (although ground motions may be de-amplified). This is generally the beginning point for most aseismic studies and a solution to this problem allows the geotechnical engineer to:

- Calculate site natural periods.
- Assess ground motion amplification.
- Provide structural engineers with various parameters, primarily response spectra, for design and safety evaluation of structures.
- Evaluate the potential for liquefaction.
- Conduct first analytical phase of seismic stability evaluations for slopes and embankments.

Soil conditions and local geological features affecting the ground response are numerous. Some of the more important features are horizontal extent and depth of the soil deposits overlying bedrock, slopes of the bedding planes of the soils overlying bedrock, changes of soil types horizontally, topography of both bedrock and deposited soils and faults crossing the soil deposits.

A complete ground response analysis

Ideally, a complete ground response analysis should take into account the following factors:

- Rupture mechanism at source of an earthquake (source).
- Propagation of stress waves through the crust to the top of bedrock beneath the site of interest (path).
- How ground surface motion is influenced by the soils that lie above the bedrock (site amplification).

In reality, several difficulties arise and uncertainties exist in taking account the above listed factors:

- Mechanism of fault rupture is very complicated and difficult to predict in advance.
- Crustal velocity and damping characteristics are generally poorly known.
- Nature of energy transmission between the source and site is uncertain.

In professional practice, the following procedures are usually adopted to make the process tractable and overcome the above difficulties:

- Seismic hazard analyses (probabilistic or deterministic) are used to predict bedrock motions at the location of the site.
- Seismic hazard analyses rely on empirical attenuation relationships to predict bedrock motion parameters.
- Ground response problem becomes one of determining responses of soil deposit to the motion of the underlying bedrock.

Steps in site-specific ground response analysis

The following are the sequence of steps (Figure 1) to be followed to modify the earthquake motions in the bedrock to account for the effects of soil profile at a site.

Characterization of the site

Based on the results of the geophysical as well as geotechnical investigations and laboratory testing, one or more

*For correspondence. (e-mail: lgr@civil.iisc.ernet.in)

idealized soil profiles must be selected for the site of interest. In this context, complete dynamic site characterization includes the following:

- Shear wave velocity profile with depth (through geophysical testing method such as Spectral Analysis of Surface Wave (SASW) method).
- Variation of shear modulus with strain (or modulus reduction curve).
- Variation of damping with strain (or damping ratio curve).

Selection of rock motions

Appropriate rock motions (either natural or synthetic acceleration time histories) are selected to represent the design rock motion for the site. The rock motion should be associated with the specific seismotectonic structures, source areas or provinces that would cause most severe vibratory ground motion or foundation dislocation capable of being produced at the site under currently known tectonic framework. Here an interaction with a seismologist is required. If natural time histories are used, it is preferable to use a set of natural time histories that have ground motion characteristics similar to those estimated for the design rock motions. That means the selected histories should have:

- Peak ground motion parameters
- Response spectral content and
- Duration of strong shaking

In the absence of natural motions, artificial motions can be generated using the concept of 'spectrum compatible time histories'. For this problem several procedures are available such as time domain, frequency domain generation, empirical Green's function technique, ARMA modeling, etc.

Ground response analysis and design spectra

Ground response analysis, usually in the form of one-dimensional analysis (linear, equivalent linear or nonlinear) are performed for the site specific profiles using the rock motions as input motion, to compute the time histories at the ground surface. Response spectra of calculated ground surface motions are statically analysed or interpreted in some manner to develop 'design spectrum' for the site. The time histories from the ground response analysis can be used directly to represent the ground surface motions or artificial time histories can be developed to match the design spectrum.

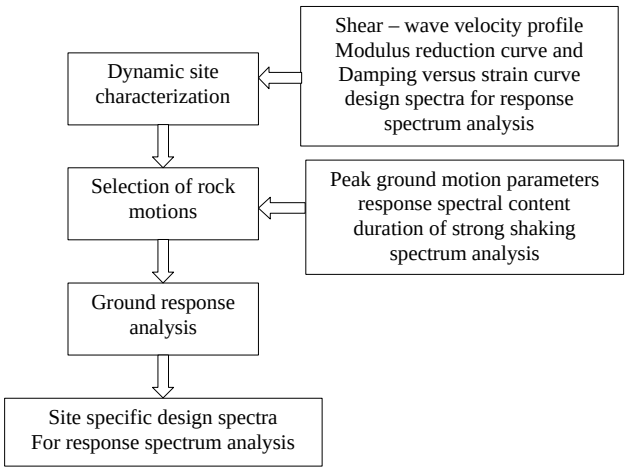

Figure 1. Site-specific ground response analysis.

Wave propagation analysis/site amplification

During earthquakes, the ground motion parameters such as amplitude of motion, frequency content and duration of the ground motion change as the seismic waves propagate through overlying soil and reach the ground surface. The phenomenon, wherein the local soils act as a filter and modify the ground motion characteristics, is known as 'soil amplification problem'. Physically, the problem is to predict the characteristics of the seismic motions that can be expected at the surface (or at any depth) of a soil stratum. Mathematically, the problem is one of the wave propagation in a continuous medium.

Excitation of a compliant medium (for example, a soil deposit or an earth dam) is not instantly felt at other points within the medium. It takes time for the effects of the excitation to be felt at distant/different points. The effects are felt in the form of waves that travel through the medium. The manner in which these waves travel is a function of the stiffness and attenuation characteristics of the medium and will control the effects they produce. Usually, the geological materials are treated as continua and the dynamic response of these materials to dynamic/transient loading such as earthquakes, blasts, traffic-induced vibrations, etc. are evaluated in the context of one or two or three-dimensional wave propagations depending on the geometry and loading conditions.

One-dimensional wave propagation analyses

These are widely used for 'ground response analysis' or 'soil amplification studies' as:

- they are believed to provide conservative results
- a large number of commercial programs with different soil models are available for use on personal computers
- they are time tested, i.e. most design projects in the past designed using this methodology survived the earthquakes.

Assumptions in one-dimensional ground response analysis

- The soil layers are horizontal and extend to infinity.
- The ground surface is level.

- The incident earthquake motions are spatially-uniform, horizontally-polarized shear waves, and propagate vertically.

Justification for using one-dimensional analysis

- In the areas of strong earthquake motion, the stress waves, from the earthquake focus are propagating nearly vertically when they arrive at the earth's surface. Wave velocity generally decreases from the earth's interior towards the surface, and hence stress waves from the focus are bent by successive refractions into a nearly vertical path.
- Even if the waves within the firm ground are propagating in a shallow inclined direction, the waves set up within the soil by refraction at the interface between the firm ground and soil will propagate nearly vertically (by Snell's law of refraction).
- Vertical ground motions are generally not as important from the standpoint of structural design as horizontal ground motions.
- Soil properties generally vary more rapidly in the vertical direction than in the horizontal direction.

In reality, a complete ground response analysis must take into account the various factors mentioned before including the additional factors such as rupture mechanism at the origin of earthquake, propagation of seismic waves through the crust to the top of bedrock. These factors are difficult to quantify and hence a complete ground response analysis becomes highly complicated. Therefore, one-dimensional ground response analyses are used extensively due to their simplicity.

Methods of analysis

A number of techniques are available for 'ground response analysis'. The methods differ in the simplifying assumptions that are made, in the representation of stress–strain relations of soil and in the methods used to integrate the equation of motion. The development of existing methods of dynamic response analysis has been a gradual evolutionary process stimulated by changing needs of practice and the increasing knowledge about the fundamental behaviour of soils under cyclic loading derived from field observations and laboratory testing. The method can be broadly grouped into the following three categories:

- Linear analysis.
- Equivalent linear analysis.
- Nonlinear analysis.

Linear analysis. Linear analysis, because of its simplicity, has been extensively used to study analytically the dynamic response of soil deposits. Closed form analytical solutions have been derived for idealized geometries and soil properties, e.g. by assuming that the deposit consists of one uniform layer with soil stiffness either constant or varying with depth in a way which can be expressed by simple mathematical functions. In general, however, soil does not behave elastically and its material properties can change in space. In such situations, no analytical solutions are possible and numerical techniques such as finite element or finite difference method are used.

Equivalent linear analysis. Schnabel *et al.*[1] addressed nonlinear hysteretic stress–strain properties of sand by using an equivalent linear method of analysis. The method was originally based on the lumped mass model of sand deposits resting on rigid base to which the seismic motions were applied. Later, this method was generalized to wave propagation model with an energy-transmitting boundary. The seismic excitation could be applied at any level in the new model.

Nonlinear analysis. A nonlinear analysis is usually performed by using a discrete model such as finite element and lumped mass models, and performing time domain step-by-step integration of equations of motion. For nonlinear analysis to give meaningful results, the stress–strain characteristics of the particular soil must be realistically modeled.

Ground response in Ahmedabad during Bhuj earthquake – A case study

The devastating earthquake that struck the Kutch area in Gujarat at 8.46 AM (IST) on 26 January 2001 was the most damaging one in the last 50 years and had a magnitude of M_w 7.7. The epicenter of the earthquake was located at 23.4°N, 70.28°E and at a depth of about 25 km to the north of Bacchau town. A number of medium to high rise residential apartments having ground plus four floors and a few with ground plus ten floors particularly of reinforced concrete suffered extensive damage and collapse in Ahmedabad city which is nearly 300 km away from the epicenter. The city is founded over deep deposits of cohesionless soils. The random distribution of such damage has been recorded from a number of localities scattered on the left and right banks of Sabarmati River. Figure 2 shows the distribution pattern of collapsed buildings in Ahmedabad[2]. Nearly half of the collapsed buildings are located in five clusters A to E (dark circles) each of one kilometer diameter and three-fourths of the collapsed buildings are confined to three linear bands I, II and III. This clearly indicates the need for the study of site effect due to seismic disturbance in the zone of interest.

Ground motion characteristics

Strong motion records. Figure 3 shows the acceleration time histories for the three components of the strong mo-

tion records at ground floor of the Passport Office building in Ahmedabad. Peak ground accelerations of 0.106 g, 0.08 g and 0.07 g were observed for longitudinal, transverse and vertical components respectively.

Frequency content parameters. The response of soil deposits and soil structures during earthquakes is largely dependent on the frequency at which they are acted upon by the dynamic loads. The frequency content describes clearly how the amplitude of ground motion is distributed among different frequencies. The frequency content of a ground motion can be obtained by transforming the ground motion from time domain to a frequency domain through a Fourier transform. Figures 4 and 5 show Fourier amplitude spectrum for N 78 E and N 12 W components of acceleration time history of Bhuj earthquake.

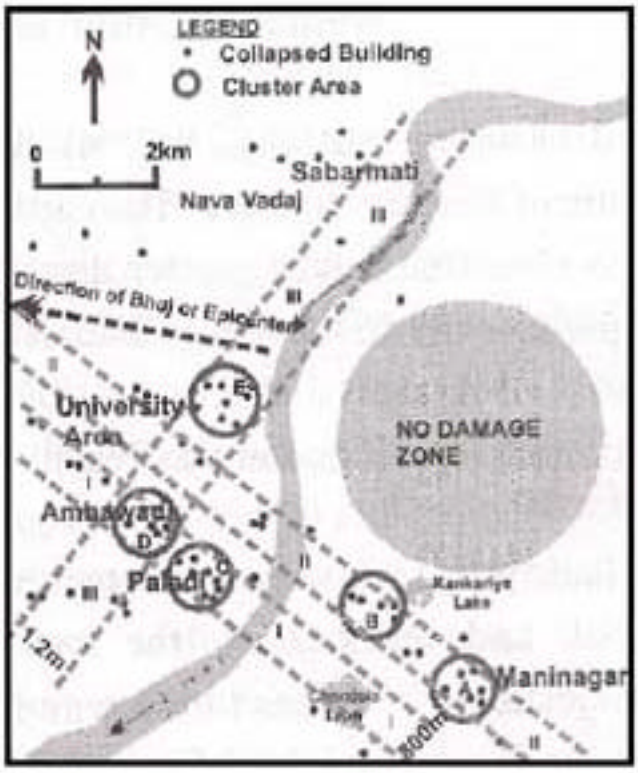

Figure 2. Pattern of collapsed buildings[2].

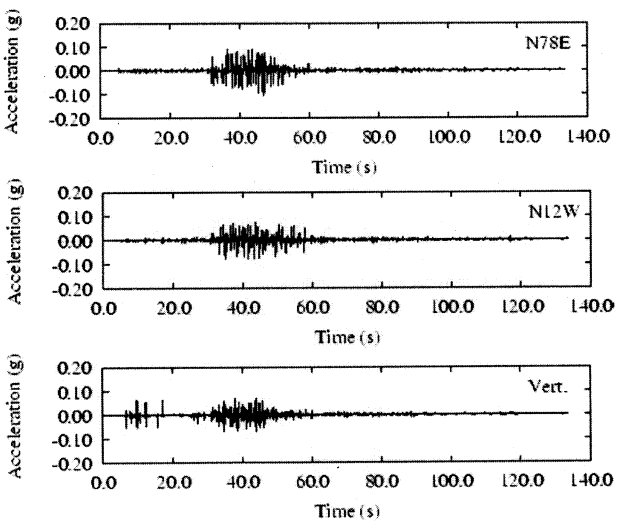

Figure 3. Strong motion accelerograms recorded on the ground floor of the Passport Office Building at Ahmedabad during Bhuj earthquake.

Duration of strong ground motion. The duration of strong earthquake ground motion is one of the main parameters characterizing this natural hazard and is important because the amount of cumulative damage incurred by the structures increases with number of cycles of loading. Duration of strong motion is usually defined in relation to the time required for the release of accumulated strain energy by rupture along the fault. Estimation of the duration of strong motion is quite difficult. The direct approach is to equate duration to the time between the first and last accelerations typically taken as 0.05 g (ref. 3). Trifunac and Brady[4] proposed a method based on the energy of ground motion. Accordingly, the duration is the time interval bet-

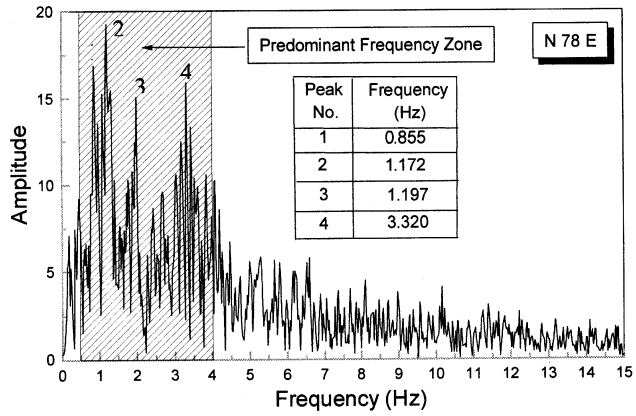

Figure 4. Fourier amplitude for N 78 E component.

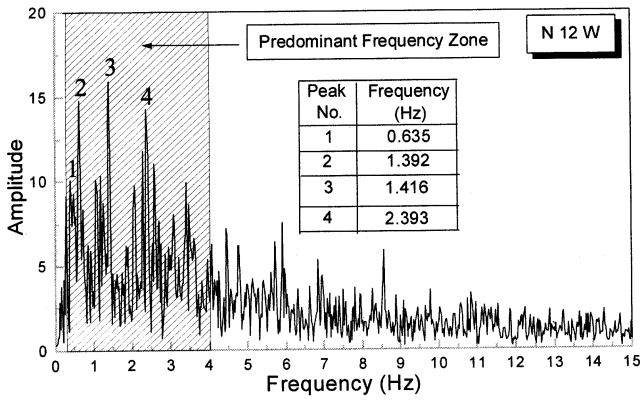

Figure 5. Fourier amplitude for N 12 W component.

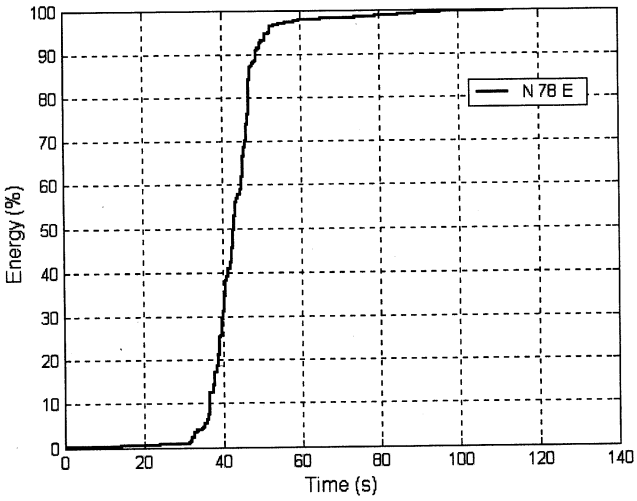

Figure 6. Release of strain energy with time for N 78 E component.

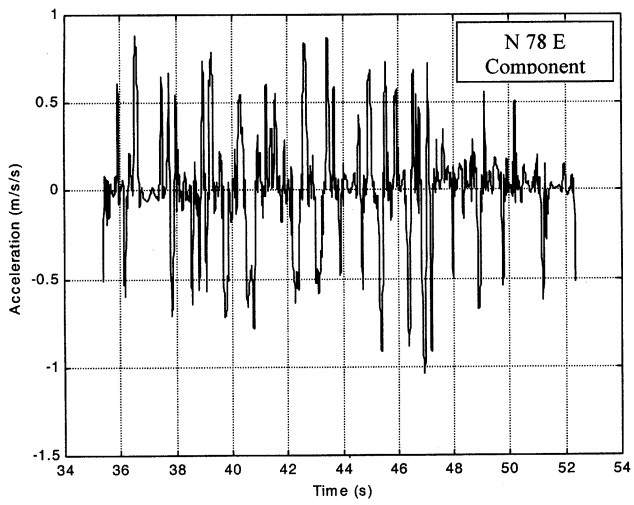

Figure 7. Strong ground motion for N 78 E component.

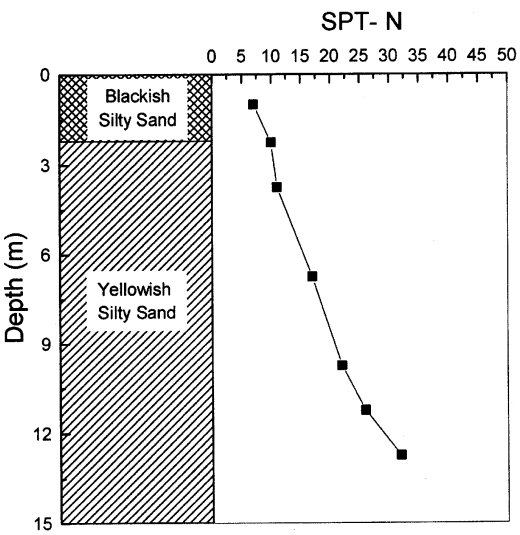

Figure 8. Soil profile.

ween the points at which 5% and 95% of the total energy has been recorded during an earthquake. Figure 6 shows the variation of energy released with time during Bhuj earthquake for N 78 E component.

Figure 7 shows the strong ground motion zone with time. The duration of the strong motion corresponding to 5% and 95% energy is between 35.37 s and 52.33 s respectively. Therefore the duration of strong ground motion during Bhuj earthquake can be estimated as 16.96 s.

Geotechnical investigation. Ahmedabad and surrounding areas are on Sabarmati alluvial belt. Soil exploration data of a site very close to Sabarmati river was collected from an agency in Ahmedabad. Figure 8 shows the typical soil profile of the location. It is evident from the site investigation that the soil is loose up to 3 m depth and exhibits relatively medium dense condition from 3 m to 15 m depths. The soil is silty sand throughout with a slight variation in the density from shallow to deeper depth. The natural moisture content varies from 8.51% at surface to 10.08% at 15 m depth with a degree of saturation ranging from 38.5% to 51.8%. Table 1 shows the characteristics of soil at the selected site.

Site effects. The effect of local soil conditions on ground response during earthquake has been evaluated using widely used computer program SHAKE 91 based on equivalent linear analysis[1]. In order to evaluate the dynamic properties of soil for the selected soil profile data, empiri-

Table 1. Soil characteristics

Depth (m)	Grain size analysis				Natural moisture content (%)	S.P.T. value (*N*)
	G (%)	S (%)	M (%)	C (%)		
0.00	0	86	11	3	8.51	–
1.00	1	84	13	2	8.56	7
1.50	0	86	10	4	8.62	–
2.00	1	86	10	3	8.74	–
2.25	0	87	11	2	8.87	10
3.00	0	87	10	3	8.88	–
3.75	0	87	10	3	8.95	11
4.00	3	80	13	4	8.95	–
4.50	0	83	13	4	9.01	–
5.00	0	86	11	3	9.04	–
6.00	0	85	13	2	9.08	–
6.75	0	86	10	4	9.18	17
7.50	0	87	10	3	9.27	–
9.00	3	84	11	2	9.32	–
9.75	0	87	10	3	9.35	22
10.00	4	83	10	3	9.44	–
10.50	0	83	13	4	9.53	–
11.25	0	83	13	4	9.65	26
12.00	0	86	11	3	9.68	–
12.75	2	83	13	2	9.70	32
13.50	0	86	10	4	9.83	–
14.00	0	87	10	3	9.95	–
15.00	2	85	11	2	10.08	–

cal equations (as recommended by Japan Road Association) for Standard Penetration Test *N* values and shear wave velocities 'Vs' are used[5]. The time history of acceleration of N 78 E horizontal component recorded at the ground floor of the Passport Office building in Ahmedabad was used as input motion. As this motion was not recorded directly on the ground surface, it may have some embedded dynamic response characteristics of the building also. The input motion was first used as object motion at the surface and the corresponding accelerations were obtained at 15 m depth of soil. The time history of acceleration thus obtained at 15 m depth of soil deposit was considered as new object motion at 15 m depth and the amplification of the site was estimated between ground surface and 15 m depth. Figure 9 shows the variation of maximum accelerations with the depth. There is a considerable modification in the acceleration values from 0.064 g to 0.106 g between 15 m depth and ground surface respectively (PGA amplification by a factor of 1.66).

Figure 10 shows the spectral accelerations at the ground surface for no damping and 5% damping. Figure 11 shows amplification between the surface motion and the base motion at varying frequencies for the N 78 E component. A Fourier amplification ratio of about 20 was observed at a frequency of 3.51 Hz. This clearly indicates that the ground response parameters are modified considerably at Ahmedabad area due to local site effects.

Earthquake-induced settlement. Earthquake-induced settlement of dry cohesionless soil depends on relative density of the soil, maximum shear strain induced by the design earthquake and the number of shear strain cycles. Ground surface settlement is evaluated by the method suggested[6]. The method requires the determination of earthquake-induced effective shear strain and hence the determination of volumetric strain from Figure 12 shown. The settlement for each layer is then the volumetric strain, expressed as decimal, times the thickness of the layer. The suggested method is most applicable for dry sands that have 5% or less fines. For silty soils, Tokimastu and Seed[6] suggest the appropriate adjustment to increase the SPT *N* values by adding the values of corrected SPT *N* values as indicated in the Table 2 (ref. 7).

Tokimastu and Seed[6] recommend the above method for damp and moist sands also due to the low capillary action between the cohesionless soil particles. Figures 13, 14 and 15 show the earthquake-induced cyclic shear stress, cyclic shear strain and the associated ground settlement respectively.

Analysis of framed structures. Reinforced Cement Concrete multistorey building plane frames for different configurations of number of bays (each of 4 m span) and storey height (3 m each) were analysed for their natural frequencies using Finite Element Analysis package (NISA)[8] without considering the infill effect. Figure 16 shows the variation of natural frequencies for different storey heights and bays corresponding to first and second modes respectively.

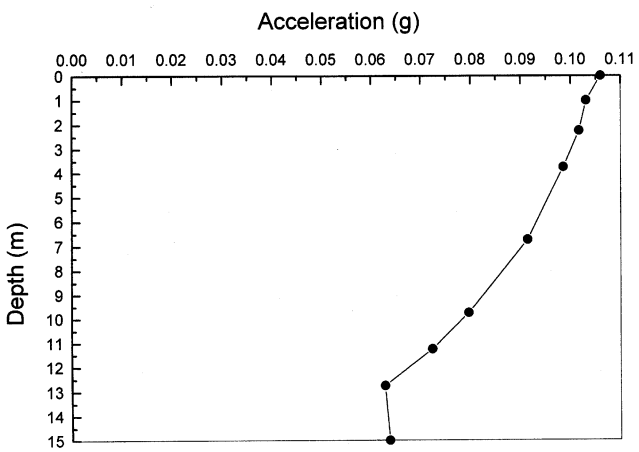

Figure 9. Variation of peak ground acceleration (PGA) with depth.

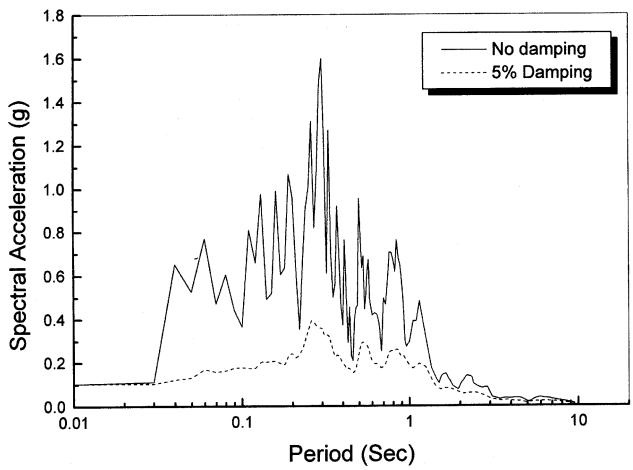

Figure 10. Response spectra of the ground acceleration time histories at Ahmedabad for N 78 E component.

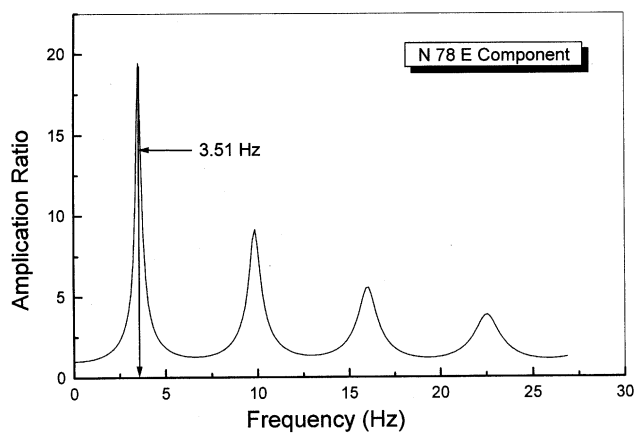

Figure 11. Amplification between surface and base motion.

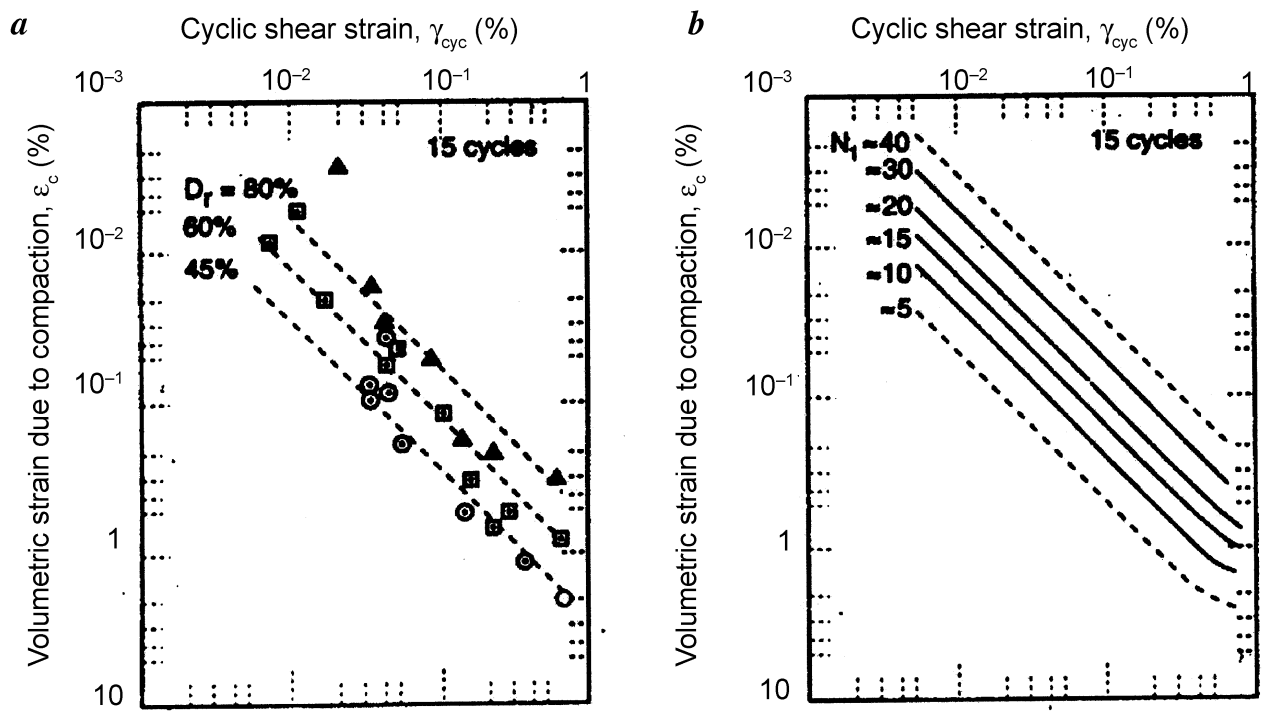

Figure 12. Plots to estimate volumetric strain based on cyclic shear strain from (***a***) relative density or (***b***) SPT *N* values.

Table 2. Corrections for SPT *N* values

Per cent fines	SPT *N* corrected
≤ 5	0
10	1
25	2
50	4
75	5

Discussions. From the index properties of soil (Table 1), it can be observed that the soil is partially saturated throughout with a degree of saturation varying from 38.5% at the surface to 51.8% at a depth of 15 m. This clearly indicates the remote chances of triggering of liquefaction and hence its associated damage to the constructed facilities at the location. From the ground surface settlement profile (Figure 15), a total surface settlement of 4.92 mm is estimated during the earthquake. The magnitude of this settlement is well within the permissible limit so as not to cause damage to the structures.

Figures 16 *a* and *b* represent the effect of number of bays and storey height on the natural frequency of the structure for first and second modes respectively. It is evident that the magnitude of natural frequencies is not much influenced by the number of bays. Further the natural frequency of the structure decreases as the number of storeys is increased. It can be noticed from the figures that for 4 to 10 storeyed buildings, the natural frequency ranges between 1.5 Hz to 3.0 Hz for the first mode and 2.5 Hz to 8 Hz for the second mode.

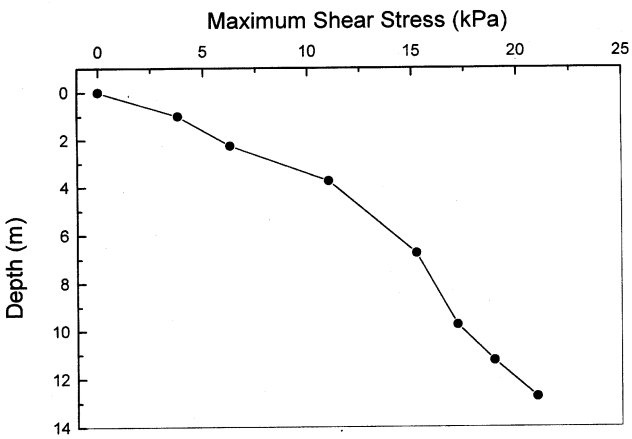

Figure 13. Variation of shear stress with depth.

It can be observed from Figure 11 that the value of natural frequency corresponding to the maximum amplification between the surface motion and the motion at the base is 3.51 Hz for 15 m deep soil. This clearly demonstrates that the high degree of damage is due to the consequence of large amplification of shear waves by the thick sandy soil deposit. This soil amplification has caused large accelerations to some buildings, in particular to the buildings above four storeys and up to ten storeys. Close matching of the resulting wave frequencies with resonant frequencies of the high-rise buildings is one of the factors responsible for their collapse (Table 3).

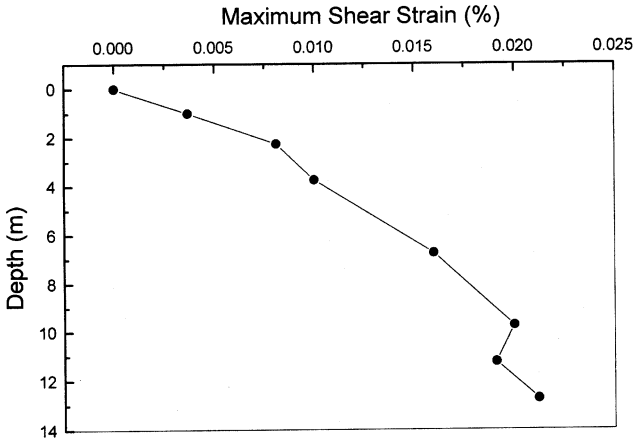

Figure 14. Variation of shear strain with depth.

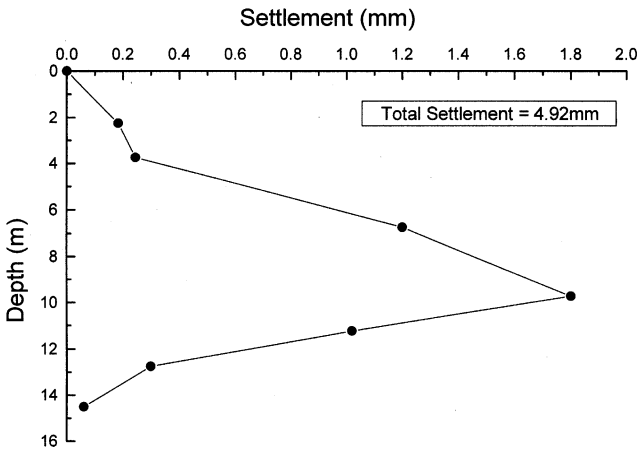

Figure 15. Variation of settlement with depth.

Table 3. Predominant frequency of earthquake and natural frequencies of ground and structure

Predominant frequency of earthquake recorded at Ahmedabad	0.86 Hz to 3.32 Hz (N 78 E)
Natural frequency of ground estimated from program SHAKE	3.51 Hz (N 78 E)
Natural frequency of four to ten storeyed building	1.5 Hz to 3.0 Hz (First mode) 2.5 Hz to 8 Hz (Second mode)

Current practice and recommendations

The foreword of the current Bureau of Indian Standards code IS 1893 (Part 1: 2002)[9] does mention about the inadequacy of the 'seismic coefficient' method and suggests that a detailed dynamic analysis be carried out in the case of important structures in earthquake-prone areas. The mentioned inadequacy of the 'seismic coefficient' method has a higher underlying significance in the case of geotechnical engineering dealing with design of flexible structures like slopes, dams, retaining walls, etc., because the effects of a complex, transient earthquake cannot be replaced by unidirectional pseudostatic force. A performance-based design has to be adopted, which requires ground motion characteristics as well as local site response. Other challenging problems to be addressed by the geotechnical engineers are settlement of dry and partially saturated soils and liquefaction. Even to use the 'simplified approach', one requires the magnitude of the earthquake, peak ground acceleration, shear stresses and shear strains at the site, which are end products of site-specific ground response analysis. There is a need to address these issues and include guidelines for conducting ground response analysis in the case of geotechnical structures in the relevant parts of the current IS code.

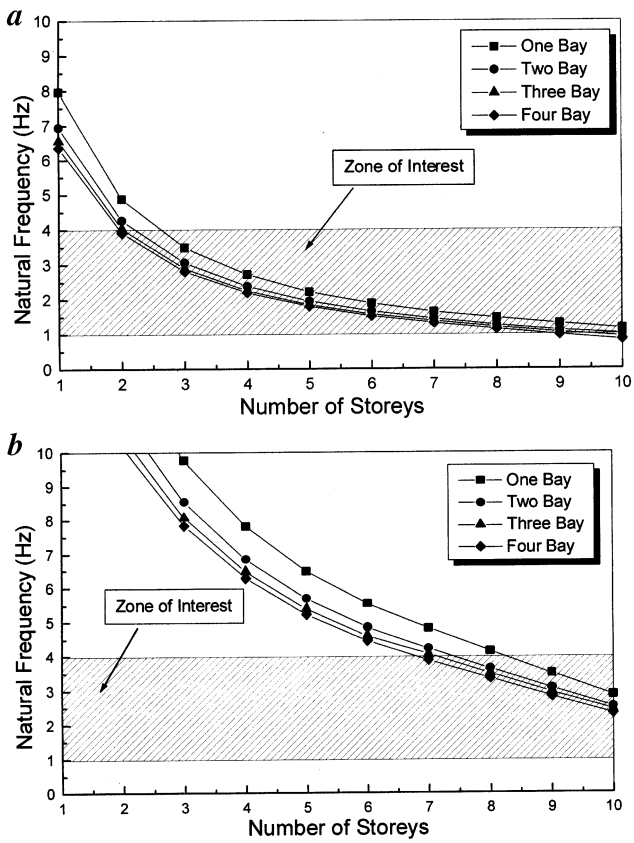

Figure 16. Variation of natural frequency with number of bays and storeys in (***a***) First mode (***b***) Second mode.

Concluding remarks

A review of various aspects of site-specific ground response analysis including its engineering importance, difficulties involved in conducting a complete ground response study and also the justification for widely used one-dimen-

sional ground response analysis is highlighted. A case study on the ground response analysis in terms of settlement of soil deposit and soil amplification of a site close to Sabarmati river belt in Ahmedabad City during the earthquake in Bhuj on 26 January 2001 is presented. The soil conditions at the selected location represent deep alluvial deposits with partially saturated condition indicating very remote chances for occurrence of liquefaction. The ground surface settlement associated with the ground shaking alone is insignificant. The high degree of damage to multistorey buildings is essentially due to the transfer of large accelerations to high rise buildings by soil amplification. Further, close matching of resulting wave frequencies with the resonant frequencies of the high rise structures is an added factor for their collapse. The 'seismic coefficient method' currently recommended by Bureau of Indian Standards Code is inadequate in dealing with the design of geotechnical structures and hence a performance-based design becomes imperative. There is a need to include guidelines on conducting ground response analysis in the case of geotechnical structures in the current IS Code.

1. Schnabel, P. B., Lysmer, J. and Seed, H. B., SHAKE – A computer program for earthquake response analysis of horizontally layered sites, *EERC Report 72-12*. Earthquake Engineering Research Center, Berkeley, California, 1972.
2. Bhandari, N. and Sharma, B. K., Damage pattern due to January, 2001 Bhuj earthquake, India: Importance of site amplification and interference of shear waves, Abstracts of International Conference on Seismic Hazard with particular reference to Bhuj Earthquake of 26 January 2001, New Delhi, 2001, pp. 19–21.
3. Dowrick, D. J., *Earthquake Resistant Design*, John Wiley, Singapore, 1987.
4. Trifunac, M. D. and Brady, A. G., A study on the duration of strong earthquake ground motion. *Bull. Seismol. Soc. Am.*, 1975, **65**, 581–626.
5. Shannon Hsien-Heng Lee, Analysis of the multicollinearity of regression equations of shear wave velocities. *Soil. Found.*, 1992, **32**, 205–214.
6. Tokimastu, K. and Seed, H. B., Evaluation of settlement in sand due to earthquake shaking. *J. Geotech. Eng.*, 1987, **113**, 861–878.
7. Day, R. W., *Geotechnical Earthquake Engineering Handbook*, McGraw-Hill, 2002.
8. NISA II/DISPLAY III, *Training Manual*, Engineering Mechanics Research Corporation, Michigan, USA, 1988, Version 8.0.
9. IS 1893, *Criteria for Earthquake Resistant Design of Structures*, Bureau of Indian Standards, New Delhi, 2002, Part 1.