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RESPONSE SPECTRA BASED ON SIMULATED EARTHQUAKES: ITS APPLICATION AND ASSESSMENT

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ABSTRACT: In this study efforts have been made to develop response spectra, which may be directly used in the dynamic analysis of a structure. To develop response spectra, a large number of earthquake ground motion recordings on different soil types are necessary. Since Bangladesh does not have any earthquake ground motion recordings, synthetic earthquakes are generated for different soil types. Response spectrum for a given earthquake record is quite irregular and has a number of peaks and valleys. Efforts have been made to construct smooth response spectra for various soil types from synthetic earthquakes. Statistical approach has been adopted to create a smoothed spectrum in order to make it suitable for design. In the second part of the study performance of the proposed spectra have been tested against Uniform Building Code (1994) spectra by analysing various moderately high moment resisting framed structures. The dynamic analysis procedure uses a response spectrum representation of the seismic input motions. In the present study, Complete Quadratic Combination (CQC) method with 5 percent of critical damping has been considered for modal combination. Two different spectra have been chosen for the purpose of response spectrum analysis. The chosen spectra were (a) UBC (1994) spectra, and (b) spectra developed using synthetic earthquakes. STRAND6 (1996) has been extensively used, to perform these computations. To calculate the base shear for different time periods, height of the moment resisting concrete frames have been varied from storey one to storey sixteen.

KEYWORDS: Synthetic earthquakes, response spectrum, base shear, application.

INTRODUCTION

Earthquake-resistant design is an evolutionary one and, although great progress has been made since seismic design was made mandatory by various building codes, it is still not completely understood. So, structural engineers have been giving more and more attention to the design of buildings for earthquake resistance. Among various unknowns or less known factors, the conversion of dynamic forces to static forces is, perhaps, one of the areas where additional works may be conducted. Notwithstanding the necessity of looking into the research area of arriving at equivalent static loads from their dynamic counterparts, it is believed that satisfactory design based on equivalent static forces can only be undertaken once all the factors, equations and curves that constitute such methods are realistically derived.

No ground motion record is available in Bangladesh. Thus, it is difficult to obtain a generalised shape of the average spectra. Keeping this in mind, a set of synthetic accelerograms has, therefore, been simulated for normalised peak ground acceleration of different soil

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conditions. The elastic spectra of each of the synthetic accelerograms has been obtained, and then averaged to get the shape of the smoothed spectra, considering various damping of structures. In this study efforts have been made to develop a computer program to generate simulated earthquakes. To generate the synthetic earthquake Kanai-Tajimi (K-T) power spectra and Shinozuka-Sato (1967) envelope has been chosen. Kanai-Tajimi power spectra can take the effect of soil type in the non-stationary time series. Efforts have also been made to construct the smooth response spectra for various soil types from synthetic earthquakes.

In the second part of the study these developed spectra have been compared with the corresponding spectra proposed by Uniform Building Code (UBC, 1994). Efforts have been geared towards arriving at an all encompassing response spectra which may either be readily used in dynamic analysis or may be conveniently adopted in static analysis by deriving a suitable numerical co-efficient from it. During the course of the study a wide range of synthetic earthquake data have been generated. The design aids have been compared with Uniform Building Code (1994) provisions by analysing various moderately high tall buildings.

METHODOLOGY

To generate synthetic earthquakes, a computer program has been developed in FORTRAN which accommodates K-T power spectra and Shinozuka Sato envelope. The detail theoretical background needed to develop the program has been mentioned in subsequent sections. The flow chart of the computer program has been given in Fig. 1. The input parameters that have been used to generate synthetic earthquakes for the present study are listed in Table 1. Figure 2 shows four typical generated earthquake for each of the four soil types. In total 160 earthquakes were generated by varying the parameters as stated in Table 1. Generation of such a huge amount of earthquakes was deemed essential for satisfactorily construction of response spectra via simulation. Real earthquake records contain both P (Compression-Expansion) wave and S (Shear) wave phases, in these figures only shear waves have been simulated. Simulation of P-waves, which is always present in real earthquake records due to the inherent fault-fracture mechanism of real seismic forces, the overall impact of such forces are minimal in structural dynamics. Again, in contrast to real earthquake records where site effect (that is, variation of soil type) could not be readily isolated in time domain, in the present case of simulated earthquakes, where soil effect could be explicitly identified by using suitable values of natural circular frequency (w), such site effects could be reflected in the simulated time history.

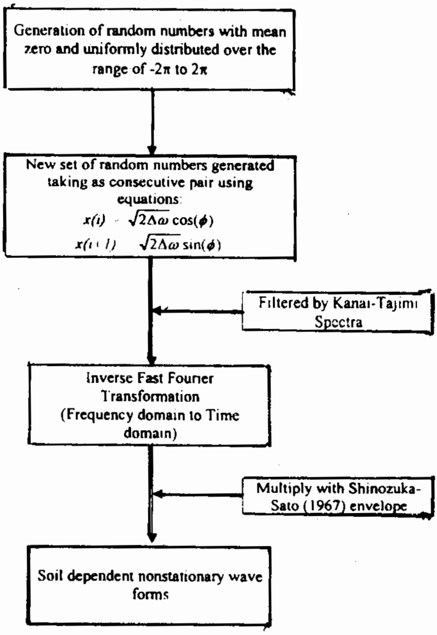

Fig 1. Schematic diagram for generating simulated earthquakes

Table 1. Input Parameters Used to Generate Synthetic Earthquakes

Parameters	Lower Limits	Upper Limits
G_0	50.0 cm/sec^2	500 cm/sec^2
wg	5.0 rad/sec	35.0 rad/sec
ζg	0.15	0.55
Total Time	20 .48 sec.	

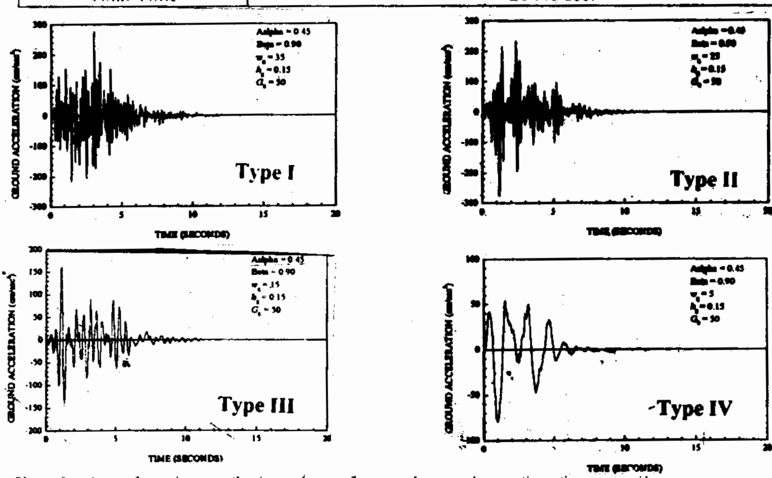

Fig 2. A selection of simulated earthquakes for four soil types

To construct simulated response spectra the amplitudes of the record and the soil type have been varied. Other variables, such as magnitude and duration of earthquakes, could not been considered for lack of sufficient information in Bangladesh. To construct the simulated spectral acceleration of individual earthquake data a computer program has been used. The elastic spectra of each of the accelerograms have been obtained, and averaged to get the smooth shape of the simulated spectra, considering the damping of various types of structures.

Artificially Generated Earthquake

In the case in which reliable data on the amplitude, frequency and duration of the seismic ground motion, and the mechanical characteristics of the soil layers are available, the seismic force can be defined, by means of artificial accelerograms. These are the accelerograms generated by using mathematical model based on the theory of the stochastic process (Ruiz and Penzien, 1969) and are used in structural analysis. Such procedures are recommended for seismic areas with no seismological data with a complete lack of seismic ground motion records. In the present study the seismic ground acceleration is modelled as a nonstationary random process. Some basic concepts of the stochastic process theory, used in mathematical modelling for generating artificial earthquake, have been given in the subsequent sections.

Nonstationary Model of Ground Acceleration

Many studies describe earlier modelled strong motion part of ground acceleration as stationary process. However, since recorded ground accelerations show the nonstationarity trend very clearly, it is necessary to use nonstationary model for representing the earthquake ground motion. Generally there are two ways to consider the nonstationarity. First, by using instantaneous power spectrum, which represents the power spectrum of ground acceleration as a function of time and frequency, the nonstationarity of earthquake ground motion can be expressed as

$$x(t) = \sqrt{2\Delta w G(t,w)}\cos\phi \qquad (1)$$

where $G(t,w)$ is the two-sided instantaneous power spectrum, Δw is the frequency interval and ϕ is the independent random phase angles distributed uniformly between 0 and 2π. The second way is obtained by assuming that the instantaneous power spectrum can be represented as a product of deterministic envelope function and stationary power spectrum. This assumption implies nonstationarity in intensity but stationarity or approximate stationarity in spectral characteristics, as expressed in mathematical form as

$$x(t) = \psi(t)n(t) \qquad (2)$$

$$n(t) = \sqrt{2\Delta w G(w)} \cos\phi \tag{3}$$

where $\psi(t)$ is the deterministic envelope function and $n(t)$ is the random process with stationary mean and variance of zero and unity, respectively.

Use of the second way has an advantage since the parameters which are controlling the shape of envelope function are independent of frequency and realisation of the nonstationary process. Therefore, in this study the second method will be employed for modelling the nonstationary ground acceleration.

Deterministic Envelope Function

From Eqn. 2, the variance of the process can be expressed as

$$Var[\{x(t)\}] = Var[\psi(t)\{n(t)\}] = \psi^2(t) Var[\{n(t)\}] = \psi^2(t) \tag{4}$$

Thus, the envelope function can be obtained. The variance can be estimated for single record using short time-averaged (Bendat and Piersol, 1986) as

$$Var[x(t)] = \frac{1}{\theta} \int_{t-\theta/2}^{t+\theta/2} x^2(t) dt \tag{5}$$

Since the estimated variance still fluctuates, it should necessarily be fitted by a smooth function. Many function have been suggested to describe the smoothed time dependent variance. In this study, the function proposed by Shinozuka and Sato (1967) will be adopted which takes the form of

$$\psi(t) = e^{-\alpha t} - e^{-\beta t} \tag{6}$$

For generation of simulated earthquake, in this study, α has been varied from 0.25 to 0.45 and β has been varied between 0.50 to 0.90. For all the cases $\alpha<\beta$.

FREQUENCY CONTENTS OF GROUND ACCELERATION

Frequency contents of recorded ground acceleration are generally expressed by the power spectral density function proposed by Kanai (1957) and Tajimi (1960), and it is expressed as

$$G(w) = \frac{1 + 4\xi_g^2 (w/w_g)^2}{\left[1 - (w/w_g)^2\right]^2 + (2\xi_g w/w_g)^2} G_0 \tag{7}$$

Where ξ_g, w_g and G_0 represent the damping ratio, predominant frequency and spectral density at 0 Hz, respectively. For generation of earthquake predominant frequency has been divided into four values to represent four different soil types. The natural time period, which has been used to divided the soil, are listed in the Table 2.

Table 2. Property of Kanai-Tajimi Power Spectra

Site Category	No of earthquake record generated	w_g	ζ_g	
			Maximum	Minimum
Soil Type I	40	35	0.15	0.55
Soil Type II	40	25	0.15	0.55
Soil Type III	40	15	0.15	0.55
Soil Type IV	40	5	0.15	0.55

Generation of Synthetic Accelerograms

The following procedure was adopted to develop the synthetic accelerograms. A set of random numbers, denoted by ϕ_1, ϕ_2, ϕ_3... were generated with mean zero and uniform distribution over the range of -2π to 2π. These random numbers then taken as consecutive pairs were filtered, by K-T power spectra, to correspond consecutive pair of new random numbers using Eqn. 8 and Eqn. 9 having zero mean and unit variance.

$$x(i) = \sqrt{2\Delta w G(w)}\cos(\phi) \tag{8}$$

$$x(i+1) = \sqrt{2\Delta w G(w)}\sin(\phi) \tag{9}$$

Graphical representation of this K-T power spectra filter for various soil types is shown in Fig. 3. Inverse Fast Fourier Transformation (FFT) was used to convert these numbers from frequency domain to time domain. This stationary type of waveform $x(t)$ then multiplied by Shinozuka and Sato (1967) deterministic envelope function $\psi(t)$ to convert these into nonstationary form $y(t)$. The function used for this purpose has been given in Eqn. 6 and shown in Fig. 4. This nonstationary waveform $y(t)$, thus, gives the soil dependent synthetic accelerograms.

CONSTRUCTION OF SMOOTH RESPONSE SPECTRA

Response spectrum for a given earthquake record is quite irregular and has a number of peaks and valleys. The design response spectra for a particular site should not be developed from a single acceleration time history, but rather should be obtained from the ensemble of possible earthquake motions that could be experienced at the site. To plot the response spectra several generated earthquake records have been selected for the present study. For the present study 2 percent, 5 percent and 10 percent damping were selected. Figure 5 shows normalised simulated spectra plotted against the time period of the structure for soil type I and 5% damping. A single simulated earthquake record has a particular frequency content which gives rise to the jagged, saw tooth appearance of peaks and valleys as shown in Fig. 5. This feature is not suitable for design, since for a given period, the structure may fall in a valley of the response spectrum resulting in an unconservative design for an earthquake with slightly different response characteristics. Conversely, for a small change in period, the structural response might fall on a peak, resulting in a very conservative design. To alleviate this problem the concept of the smoothed response spectrum has been introduced for design.

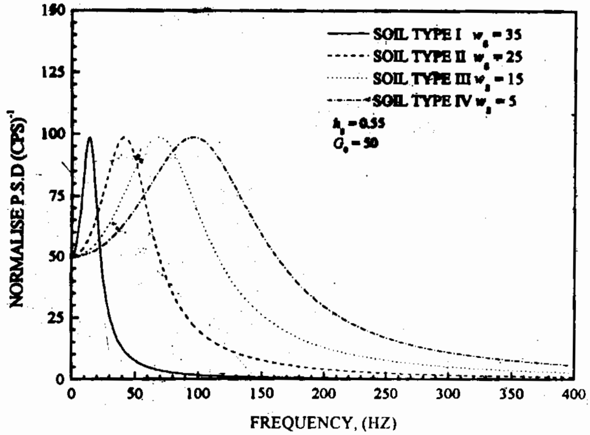

Fig 3. Normalised power spectral densities for horizontal motions for various soil types

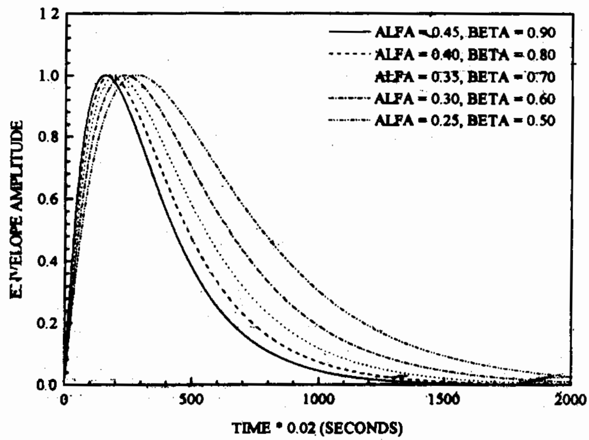

Fig 4. Shinozuka-Sato (1967) envelope for various alpha and beta

Design spectra are presented in combination of smooth curves and straight lines. Then mean-minus-one-standard-deviation, mean, and mean-plus-one-standard-deviation have been plotted in Fig. 6. This figure has been plotted in logarithmic scale. Statistical approach has been adopted to create a smoothed spectrum in order to make it suitable for design. The mean value or median spectrum can generally be used for earthquake-resistant design of normal building structures. Use of this spectrum implies that there is a 50 percent probability that the design level will be exceeded. Structures that are generally sensitive to earthquakes or that have a high risk may be designed to higher level, such as the mean-plus-one-standard deviation, which implies that the probability of exceedance is only 15.9 percent. Structures having a very high risk are often designed for an enveloping spectrum, which envelops the spectra of the entire ensemble of possible site motions. It can be observed from Fig. 6 that, the attenuation of spectral ordinate with time period plotted in logarithmic scale does not show clearly the real nature of the curve. It is clear from Fig. 6 that, all sharp irregularities which was present in Fig. 5 are absent in this figure due to the adoption of statistical procedure for the sake of clarity. The mean-plus-one-standard deviation spectral shapes determined by the present study based on 160 synthetic earthquakes are shown in Fig. 7. This curve has been prepared by combining the curves of individual soil types. The curves in this figure, thus, apply to the four soil conditions considered. These spectra may be modified to use it in the seismic codal provisions. In order to achieve this modification a minimum period has been chosen below which the spectral ordinates were kept constant to be in the conservative side. A computer program has been developed to fit the spectral curve and several computer runs were undertaken to perform this operation. These values of time period for different soil conditions have been listed in Table 3, it is apparent from the table that the natural time period of soil under considerations matched satisfactorily with the chosen time period for maximum ordinates. It can be said that resonance can occur when structural time period coincides with the natural time period of the soil beneath.

Table 3 Classification of Ground Conditions for Earthquake Stations

Site Category	Time period of maximum ordinate	Definition by Natural period
Soil Type I (Rock)	0.16 sec.	$T < 0.2$ sec.
Soil Type II (Hard Soil)	0.21 sec.	$0.2 \leq T < 0.4$ sec.
Soil Type III (Medium Soil)	0.35 sec.	$0.4 \leq T < 0.6$ sec.
Soil Type IV (Soft Soil)	1.10 sec.	$T \geq 0.6$ sec.

The formula $S = aT^b$ has been used to modify the simulated response spectra. S bears a constant value for $T \leq T_{sm}$. When $T > T_{sm}$, the value of S can be derived by using the formula $S = aT^b$. Here S is the spectral ordinate, a and b are constants, T is the time period of the structure, and T_{sm} is the time period corresponding to the maximum response. Figure 8 shows the value of a and b and error in their calculation. Chi square test has been conducted during modifying the data, to ascertain the correctness of the modification. Figure 9 has been plotted to get an overall understanding of the spectral ordinates of various soil types. Modified simulated accleration spectra for various soil types for 2 percent and 10 percent of critical damping is available elsewhere in Noor (1997).

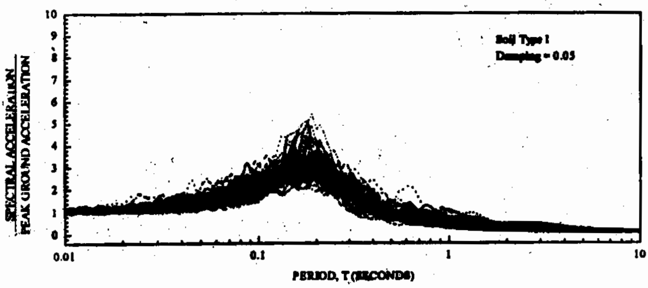

Fig 5. Normalised simulated response spectral shape of soil type I for 5 percent of critical damping

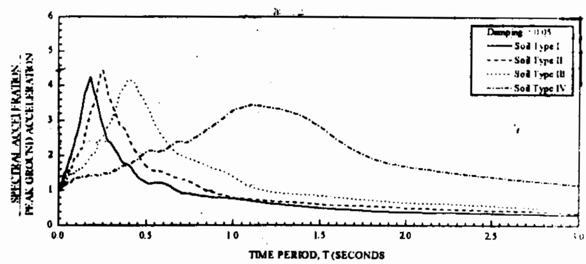

Fig 6. Simulated acceleration spectra of soil type I for 5 percent of critical damping, considering, mean-minus-one standard deviation, mean, and mean-plus-one standard deviation

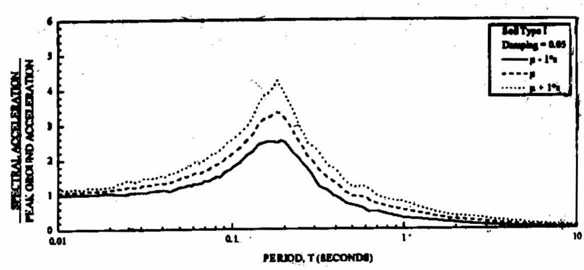

Fig 7. Simulated acceleration spectra of various soil types for 5 percent of critical damping, considering, mean-plus-one standard deviation

BUILDING ANALYSIS USING PROPOSED AND UBC SPECTRA

In order to evaluate the performance of the developed spectra using simulated earthquakes a typical beam column frame has been selected. The typical floor plan of the building that was selected for this study has been shown in Fig. 10. Description of the model building has been presented in a previous paper (Munaz et al., 1997). The developed spectra have been compared with the existing Uniform Building Code (1994) spectra. Base shear of the building which has been calculated for different time periods using the software STRAND6 (1996) has been selected as the criterion for comparison. For the purpose of spectral analysis, base acceleration has been applied to the direction parallel to the short planar dimension of the building. Natural frequencies of buildings have been calculated considering short direction, long direction and the building as a whole. When short or long direction analyses were performed, degrees of freedom of other directions were kept restrained. This has been done to minimise the computer running time. Short direction of the building has been used for further analyses. The importance factor (I) and the zone factor (Z) have been taken equal to 1.0 and 4.0, respectively, for the purpose of response spectrum analysis. The value of R_W = 12 has been used in the analysis. In the present study Complete Quadratic Combination (CQC) method with 5

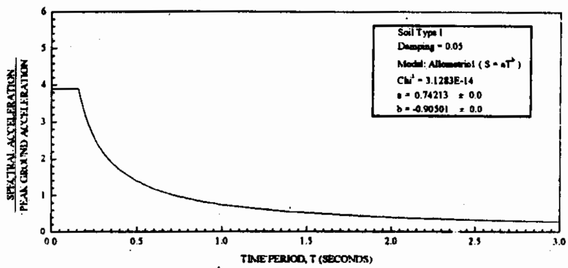

Fig 8. Modified simulated acceleration spectra of soil type I for 5 percent of critical damping

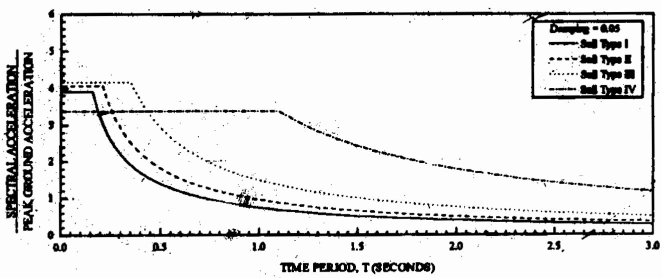

Fig 9. Modified simulated acceleration spectra of various soil types for 5 percent of critical damping

percent of critical damping has been considered for modal combination. Two different spectra have been chosen for the purpose of response spectrum analysis. The chosen spectra were (a) UBC (1994) spectra, and (b) Spectra developed using simulated earthquakes.

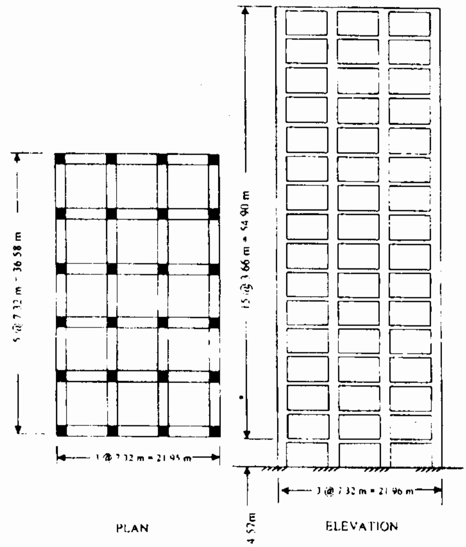

Fig 10. Typical plan and elevation of moment resisting concrete building

Response Spectrum Analysis

The dynamic analysis procedure described in this section uses a response spectrum representation of the seismic input motions. The procedure is applicable to linear elastic building models developed in accordance with the requirements of Section 106.3 of Uniform Building Code (1994). It consists of the following steps (Clough and Penzien, 1975):

a) Principles of mechanics have been used to compute the natural period and mode shape for the first N normal modes of the building model, where N is the significant number of modes.

b) The response spectrum at the natural period of the n^{th} normal mode, $^{n}(T)$, have been entered to obtain the corresponding spectral acceleration in the k^{th} direction, $^{n}(S_{ak})$. Here k may be the x or y horizontal direction or the z vertical direction. Table 4 lists the spectral ordinates for 10 and 16 story buildings.

c) This spectral acceleration has been used together with the mode shapes and the model's consistant/lumped mass values to compute the building's modal participation factor $^{n}(P_k)$ for the n^{th} mode and the k^{th}

direction corresponding to that of ${}^{n}(S_{ak})$ as shown in Eqn. 10. Participation factor for the 10 and 16 storey buildings are listed in Table 5.

$$ {}^{n}(P_k) = \sum_{i=1}^{NP} \frac{{}^{n}(\phi_{ik})(M_{ik})}{{}^{n}(M)} \qquad (10) $$

Where:

${}^{n}(\phi_{ik})$ = Mode shape amplitude for the n^{th} mode, i^{th} nodal point and the k^{th} direction.

M_{ik} = Component of mass for the i^{th} node point and the k^{th} direction.

${}^{n}(M)$ = Modal mass for n^{th} mode

$$ = \sum_{i=1}^{NP} \sum_{k=1}^{NK} {}^{n}(\phi_{ik})^2 (M_{ik}) \qquad (11) $$

NP = Total number of node points.

NK = Total number of directions of motion at a node point.

Table 4. Natural Frequency and Corresponding Spectral Value for 10 and 16 Storey Building. (Soil Type II, UBC, 1994, Short Direction.)

Mode	10 storey building		16 storey building		Damping Ratio
	Frequency	Spectral Value	Frequency	Spectral Value	
1	0.657191	1.13	0.406120	0.82	0.05
2	2.007950	2.39	1.234592	1.73	0.05
3	3.500128	2.50	2.153795	2.5	0.05
4	5.103582	2.50	3.081435	2.5	0.05
5	6.445510	2.50	4.070213	2.5	0.05
6	6.871075	2.50	4.236972	2.5	0.05

Table 5. Seismic Mass Participation Factors for 10 and 16 Storey Building (Soil Type II, UBC, 1994, Short Direction and Whole Building)

Mode	10 storey building		16 storey building	
	Short direction	Whole building	Short direction	Whole building
1	84.43	0.00	82.08	0.00
2	9.73	84.44	10.60	82.08
3	3.05	0.00	3.30	0.00
4	1.37	0.00	1.59	0.00
5	0.00	9.73	0.88	10.60
6	0.69	0.00	0.00	0.00
7	-	0.00	-	0.00
8	-	3.05	-	3.30
9	-	0.00	-	0.00
Total	99.28	97.22	98.48	95.98

d) The above parameters have been used to compute the peak value of any building response quantity when the building is vibrating in its n^{th} normal mode. For example peak acceleration at i^{th} node point and in k^{th} direction is given in Eqn. 12

$$^{n}(\ddot{u}_{ik}) = {}^{n}(P_{k}) \times {}^{n}(\phi_{ik}) \times {}^{n}(S_{ak}) \tag{12}$$

Also, for response in any other direction j is given in Eqn. 13.

$$^{n}(\ddot{u}_{ij}) = {}^{n}(P_{k}) \times {}^{n}(\phi_{ij}) \times {}^{n}(S_{ak}) \tag{13}$$

Lateral force at i^{th} node point in k^{th} direction has been calculated using Eqn. 14.

$$^{n}(F_{ik}) = (M_{ik}) \times {}^{n}(\phi_{ik}) \times {}^{n}(P_{k}) \times {}^{n}(S_{ak}) \tag{14}$$

Base shear force in k^{th} direction has been calculated using Eqn. 15.

$$^{n}(V_{k}) = {}^{n}(P_{k})^{2} \times {}^{n}(M) \times {}^{n}(S_{ak}) \tag{15}$$

Overturning moment due to lateral forces in the k^{th} direction has been calculated using Eqn. 16.

$$^{n}(OM_{k}) = \sum_{i=1}^{NP} M_{ik} \times {}^{n}(\phi_{ik}) \times {}^{n}(P_{k}) \times {}^{n}(S_{ak})(h_{i}) \tag{16}$$

Where h_i is the vertical distance form the i^{th} node point to the base of the building.

e) Steps *b* through *d* have been repeated for each of the *N* normal modes. Then, the resulting peak modal response quantities have been combined for each mode using the procedure described in the next section, in order to estimate the composite peak response value.

Combining Modes

The response spectrum analysis procedure described in previous section provides the maximum responses of the structure when it is vibrating in each of its significant normal nodes. However, because these maximum modal responses will not occur at the same time during the earthquake ground motion, it is necessary to use approximate procedures to estimate the maximum composite response of the structure. Such procedures are typically based on an appropriate combination of the maximum individual modal responses, and should account for possible interaction between any closely spaced modal responses that may exist.

A simple and accurate modal combination approach that satisfies this requirement is the Complete Quadratic Combination (CQC) method (Wilson et al., 1981, Der Kiureghian, 1981, and Wilson and Bolton, 1982). This approach is based on random vibration concepts and assumes that:

The duration of the earthquake shaking is long when compared to the fundamental period of the structure and the design response spectrum exhibits slowly varying amplitudes over a wide range of periods that include the dominant modes of the structure. On this basis, the CQC method leads to the following expression for the structure's maximum composite response, u_k, at its k_{th} degree of freedom:

$$u_k = \left[\sum_{i=1}^{N} \sum_{j=1}^{N} u_{ki} \rho_{ij} u_{kj} \right]^{1/2} \tag{17}$$

Where u_{ki} and u_{kj} correspond to the structure's maximum modal response in its k^{th} degree of freedom when it is vibrating it in its i^{th} and j^{th} mode respectively, and ρ_{ij} is the cross-modal coefficient. It is noted that here, u_k, u_{ki}, u_{kj} are general symbols and may correspond to total acceleration, relative (to base) displacement, inter story drift, base shear, overturning moment, or any other structural response quantity. Furthermore, when computing u_k in accordance with the above expression, the signs of u_{ki} and u_{kj} should be preserved.

The cross-modal coefficient ρ_{ij} as denoted above is dependent on the damping ratios and the natural periods of the i^{th} and j^{th} mode. When the modes have identical damping ratios ξ, ρ_{ij} is expressed as:

$$\rho_{ij} = \frac{8\xi^2(1+r)r^{3/2}}{(1-r^2)^2+4\xi^2 r(1+r)^2} = \rho_{ji} \tag{18}$$

where r is the ratio of the natural period of the j^{th} mode, T_j, to the natural period of the i^{th} mode, T_i (that is, $r = T_j/T_i$).

From Eqn. 18, it can be shown that: a) ρ_{ij} = 1 when r = 1; and b) ρ_{ij} decreases with decreasing r in a manner that is dependant on the modal damping ratio ξ. Furthermore, when the modal periods are well spaced such that:

$$r = \frac{T_j}{T_i} \leq \frac{0.1}{0.1+\xi} (T_i > T_j) \tag{19}$$

then: $\rho_{ij} \approx 0$ (i = j) and the CQC expression for computing the maximum composite response given in Eqn. 17 becomes:

$$u_k = \left[\sum_{i=1}^{N} u_{ki}^2 \right]^{1/2} \tag{20}$$

That corresponds to the square-root-sum-of-the-squares (SRSS) modal combination approach. This shows that the SRSS approach is a special case of the more general CQC method, and can be applied when the modal periods are sufficiently well spaced in accordance with Eqn.

19. Furthermore, the quantities ρ_{ij} for $i \neq j$ can be visualized as corrections to the SRSS approach in order to incorporate effects of coupling between closely spaced modes. These coupling effects become more important as the modal damping ratio increases. Also, these effects are typically important for three-dimensional structural systems, which often have closely spaced frequencies.

In Section 106.2.2 of Uniform Building Code (1994), the largest damping ratio that can be considered when developing site-specific spectra is specified to be 0.05. Furthermore, ρ_{ij} can be assumed to be negligible when

$$r = \frac{T_j}{T_i} \leq 0.67 \qquad (21)$$

Analysis Scheme

For spectral analysis purposes, using different acceleration spectra, only one direction, that is, short direction of the building have been selected. This directional analysis have been performed by locking the global degrees of freedom of other direction and discussed in detail in Munaz (1997). As spectral acceleration was applied towards the short dimension of the building, the mode shapes transverse to the direction and torsional mode shapes, should have no mass participation factor to the final analysis. To validate this, two analyses have been performed taking 16 and 10 storey special moment resisting concrete frames keeping. all the degrees of freedoms unlocked. The mass participation factors for both the analyses have been listed in Table 5. It is clearly observed from Table 5 that modes perpendicular to the direction of analysis and torsional modes have no participation to final result. Taking this fact into account and to minimise the time of computer run, short direction analysis scheme have been adopted for further analyses, which is expected to produce equally good accuracy of the analytical analysis.

COMPARISON OF BASE SHEAR

Efforts have been made to compare the base shear calculated using the Code specified response spectra and response spectra developed in this study using synthetic earthquakes. Response spectrum analysis which has been discussed in detail in this study has been used to calculate the base shear of the moment resisting concrete frame. STRAND6 (1996) has been extensively used, to perform these computations.

To calculate the base shear for different time periods, height of the moment resisting concrete frames have been varied from storey one to storey sixteen. Spectral analysis has been done for soil type II and shown in Fig. 11. Base shear for a particular building has been found by summing all the horizontal forces of each of the column base for a particular direction. It is observed from Fig. 11 that base shear coefficient for developed spectra is greater than the base shear coefficient produced by Uniform Building Code (1994) at lower time period and decreased faster in higher time periods. It can be concluded by observing Fig. 11 that serious thought should be given during future updating of the spectral shapes. Whereas the higher values of spectral ordinates at lower time periods might be left unchanged (as they would lead to more conservative (that is, safer)) design, faster attenuation at higher time periods, as observed here, may be modified to keep in tune with the conservative nature of existing spectra.

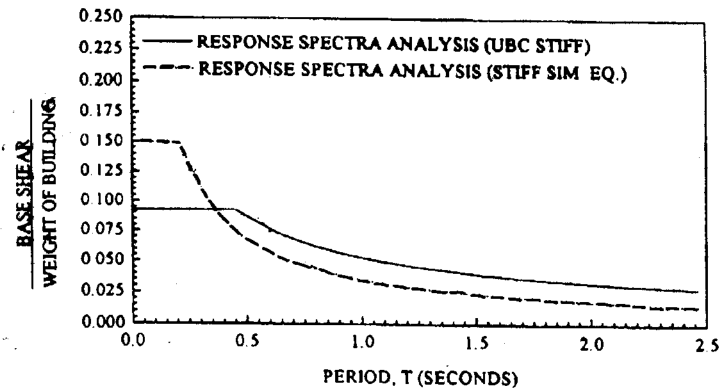

Fig 11. Comparison of base shear coefficient of developed spectra with UBC (1994) for soil type II

COMPARISON OF BASE SHEAR DISTRIBUTION

Base shear distribution has been compared with the corresponding shear distribution proposed by Uniform Building Code (1994). For the purpose of comparison, 16 and 10 storey special moment resisting concrete frames have been selected. In Fig. 12 normalised storey shear has been plotted against percent height using UBC (1994) acceleration spectra, proposed simulated acceleration spectra for soil type II. Normalised storey shear distribution proposed by UBC (1994) has also been in these figures for comparison purpose. It is observed from the these figures that storey shear distribution for different spectral analysis have very little difference. The static storey shear distribution of UBC (1994) adopt linear formula which is absent in spectral solutions. It can be said that, further improvement of storey shear distribution can be done for future adoption in seismic Codes.

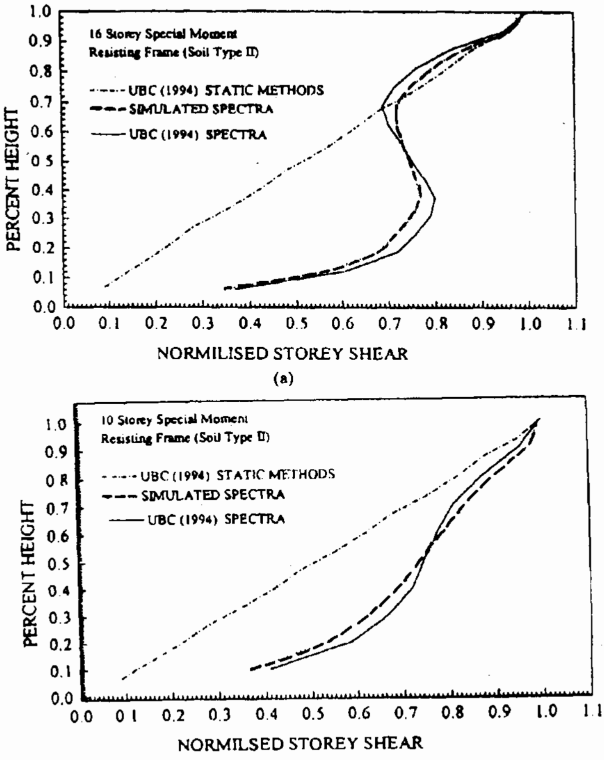

Fig 12. Base shear distribution of 10 and 16 storey moment resisting concrete frame for soil type II

PROPOSED DESIGN SPECTRA

In this study response spectra for real and simulated earthquakes have been developed. Further these spectra have been modified and compared with UBC (1994) and has been found to be quite consistent. Efforts have been here to propose a design response spectra based present study, for 5 percent damping.

Table 6 list the time period for maximum spectra, maximum spectral ordinate and coefficients *a, b* of the equation $S = aT^b$ which has been used to modify response spectra for simulated earthquake. It is observed from the Table 6 that the coefficient *b* which represents the rate of attenuation is always near or greater than 1, as such the value of *b* has been fixed to 1. This has been done to remain conservative in higher time period. Higher values at lower time period has been retained as it has found. The final design spectra after this modification for simulated earthquakes is shown in Fig. 13. The static equivalent equations have been listed in Table 7 for different soil types derived using simulated earthquakes.

Table 6. Value of Maximum Spectral Ordinate and Coefficient a, b for Modified Simulated Response Spectra for Different Site Category

Site Category	Time Period (s)	Maximum Value of Spectral Ordinate	Coefficient *a*	Coefficient *b*
I	0.16	3.18	0.74	-0.90
II	0.21	3.30	0.95	-0.93
III	0.35	3.37	1.48	-0.98
IV	1.10	2.83	3.74	-1.07

Table 7. Site coefficient derived from simulated earthquake for different site category

Site Category	Site Coefficient
I	$S = 0.51/T$
II	$S = 0.69/T$
III	$S = 1.18/T$
IV	$S = 3.11/T$

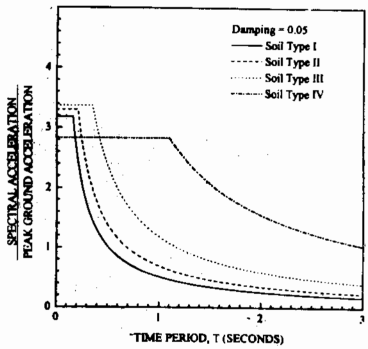

Fig 13. Design spectra based on simulated earthquakes for different soil types

It is clear from the above discussion that maximum amplitude of the acceleration spectra decreased as the soil type changed from soft to rock. For larger periods, it is evident that soft soil spectral acceleration is greater than rock spectral acceleration. It is also observed that largest amplification occur near the natural time period of the soil. It has also been understand that attenuation of rock was faster than stiff soil and so was the case as the soil became softer. Rate of attenuation is faster for the earthquakes taken in the present study than the present-day codes. It has been observed that an increase in damping results in a corresponding decrease in the spectral amplification.

CONCLUDING REMARKS

Due to the fact that Bangladesh lacks heavily on seismic instruments, simulated earthquakes had to be generated to arrive at site specific response spectra suitable for dynamic analysis. Whereas the spectra generated here may be used in dynamic analysis of structures, it is imperative to install suitable number of seismic stations so that in future, spectra based on site specific real earthquake records can developed.

It has been observed from above discussion that, for directional analysis in three-dimensional analysis any mode shapes transverse to the direction and torsional modes shapes have no participation to the final result of the structure.

It is also observed that ordinate of response spectra, is greater that UBC (1994) spectra at lower time period and decrease faster in higher time periods. It has been proposed in this study to keep unchanged the higher spectral value at lower time period. Faster attenuation has been modified and attenuation rate has been suggested which is inversely proportional to time period.

REFERENCES

Bendat, J. S. and Piersol, A. G. (1986), Random Data: Analysis and Measurement Procedures. Willey-Interscience, Division of John Wiley and Sons, Inc, New York, USA.

Clough, R. W. and Penzien, J. (1975), Dynamics of Structures, McGraw-Hill, New York, USA.

Der Kiureghian, A. (1981), A Response Spectrum Method for Random Vibration Analysis of MDF Systems, Earthquake Engineering and Structural Dynamics, Vol. 9, John Wiley and Sons.

Kanai, K. (1957), Semi-Imperical Formula for the Seismic Characteristics of the Ground, Bull. Earthquake Research Inst., Vol. 35, University of Tokyo.

Noor, M. A., Ansary, M. A. and Seraj, S. M. (1997), Critical Evaluation and Comparison of Different Seismic Code Provisions, Jour. of Civil Engg., IEB, Vol. CE 25 (1), pp. 1-20.

Noor, M. A. (1997), Reappraisal of Code Provisions for Structures Subjected to Seismic Loading, M.Sc. Engg. Thesis, BUET, Dkaka.

Ruiz, P. and Penzien, J. (1969), Probabilistic Study of Behaviour of Structures During Earthquake, Report of the Earthquake Engineering Research Centre, 69-3, University of California, Berkeley, California.

Shinozuka, M. and Sato, Y. (1967), Simulation of Nonstationary Random Process, Jour. Engg. Mech. Div., ASCE, Vol. 93, No. EM1, 11-40.

STRAND6 (1996), G+D Computing Pty Ltd, Sydney Australia.

Tajimi, H. (1960), A Statistical Method of Determining the Maximum Response of a Building Structure During an Earthquake, Proc. 2nd world Conf. Earthquake Eng., Vol. II, Tokyo,781-797.

Uniform Building Code (1994), Chapter 23, Part III, Earthquake Design, U. S. A.

Wilson, E. L., et al. (1981), A Replacement for the SRSS Method in Seismic Analysis, Earthquake Engineering and Structural Dynamics, Vol. 9, John Wiley and Sons.

Wilson, E. L. and Button, M. R. (1982), Three-Dimensional Dynamic Analysis for Multi-Component Earthquake Spectra, Earthquake Engineering and Structural Dynamics, Vol. 10, John Wiley and Sons.

NOTATIONS

ζ_g	Damping ratio
w_g	Predominant frequency
G_0	Spectral density at zero frequency
ϕ	Random phase angle distributed uniformly between 0 and 2π
$x(t)$	Stationary waveform
$\psi(t)$	Deterministic envelope function
$y(t)$	Nonstationary waveform
$G(t,w)$	Two-sided instantaneous power spectrum
$n(t)$	Random process with zero mean and variance of unity
α, β	Parameters for envelope function
ϕ_1, ϕ_2, ϕ_3	A set of random numbers
S	Spectral ordinate
a, b	Constants
T	Time period of structure
T_{sm}	Time period corresponding to maximum response
I	Importance factor
Z	Zone factor
R_w	Numerical coefficient depend on basic structural system
N	Significant number of modes
S_{ak}	Spectral acceleration in the k^{th} direction
P_k	Building's modal participation factor
${}^{n}(\phi_{ik})$	Mode shape amplitude for n^{th} mode, i^{th} nodal point and k^{th} direction
M_{ik}	Component of mass for i^{th} node point and k^{th} direction
${}^{n}M$	Modal mass for n^{th} mode
NP	Total number of node points
NK	Total number of directions of motion at a node point
OM_k	Overturning moment due to lateral forces in k^{th} direction
F_{ik}	Lateral force at i^{th} node point in k^{th} direction
V_k	Base shear force in k^{th} direction
h_i	Vertical distance from i^{th} node point to base of building
u_k	Structure's maximum composite response
ρ_{ij}	Cross-modal coefficient
r	Ratio of natural period of j^{th} mode, T_j, to natural period of i^{th} mode, T_i
T_i	Natural period of i^{th} mode
T_j	Natural period of j^{th} mode