7.1 INTRODUCTION

When a fault ruptures below the surface of the earth and thus triggers an earthquake, body waves travel away from the source in all directions. The waves, as they travel, undergo frequent reflection and refraction as a result of passage through the different geological medium. The velocity of wave propagation of shallower materials is generally lower than the deeper materials. The inclined rays that strike the horizontal layer boundaries are usually refracted in a vertical direction. Thus, a ray travelling through the soil media undergo multiple refractions, and by the time the ray reaches the ground surface, it tends to become vertical in its propagation direction. The ‘One Dimensional Site Response Analysis’ method assumes horizontal boundaries extended to infinity and that the response of a soil deposit to earthquake excitation is predominantly caused by shear (SH) - waves propagating vertically from the bedrock below. The bedrock is also assumed to extend to infinity horizontally. The method involves solving the wave equation (Kanai, 1951; Matthiesen et. al. 1964; Roesset and Whitman, 1969; Lysmer et. al. 1971) or on a lumped mass simulation (Idriss and Seed, 1968) for determination of the site response. Soil is known for its nonlinear behaviour. The site response is thus affected by the nonlinearity of the soil. Idriss and Seed (1967a) proposed the use of the one-dimensional equivalent-linear approach, which involved the approximation of the nonlinear response of soil by a
linear analysis with modified material properties. The approach employed the nonlinear properties of the soil through an equivalent linear approach. The method is generally known as One Dimensional Equivalent Linear Site Response Analysis.

7.2 TRANSFER FUNCTION

An important genre of techniques for ground response analysis is also based on the use of transfer functions. For the site response related problems, transfer functions are used to approximate various response parameters, such as displacement, velocity, acceleration, shear stress, and shear strain time history, to an input seismic excitation parameter such as bedrock acceleration time history. This approach is however limited to the analyses of linear systems. Approximation of nonlinear behaviour is possible with this approach provided an iterative procedure with equivalent linear soil properties is applied.

The transfer function approach as shown in Fig 7.1 involves manipulation of complex numbers. A known acceleration time history of bedrock motion (input motion) is represented as Fourier series, usually using the Fast Fourier Transform (FFT) algorithm as proposed by various researchers (e.g. Cooley and Tukey, 1965). Each term in the Fourier spectrum of the bedrock (input motion) is multiplied by the transfer function derived as given in section 7.2.2 to produce the Fourier spectrum of the ground surface (output) motion also known as free-field motion. The output motion thus obtained can then be expressed in the time domain using the inverse FFT. Thus, the transfer function determines how each frequency in the bedrock (input) motion is amplified, or de-amplified, by the soil deposit.
Figure 7.1: Schematic Representation of One-Dimensional Site Response Analysis.

The transfer function due to propagation of harmonic shear waves is generally determined for the following cases (Kramer, 1996):

1. For a uniform undamped soil on rigid rock.
2. For a uniform damped soil on rigid rock.
3. For a uniform damped soil on elastic rock.
4. For a layered, damped soil on elastic rock.

It can be seen that the first three cases are not practical (as uniform soil deposits are rarely encountered), and are of importance only to illustrate how the soil conditions influence the ground motion characteristics (Kramer, 1996). The transfer
function of the fourth case, which represents real field conditions involving soil deposits with different stiffness and damping characteristics, is discussed in the next section.

### 7.2.1 Kelvin-Voigt Soil

In real materials, part of the elastic energy of a travelling wave is always converted to heat. This conversion is accompanied by a decrease in the amplitude of the wave. Viscous damping, by virtue of its mathematical convenience, is often used to represent this dissipation of elastic energy. For viscoelastic wave propagation, soils are usually modelled as Kelvin-Voigt solids (i.e. those materials, for which, resistance to shearing deformation is the sum of an elastic part and a viscous part). A thin element of a Kelvin-Voigt solid is illustrated in Fig. 7.2.

**Figure 7.2: Thin Element of a Kelvin-Voigt Solid Subjected to Horizontal Shearing (Kramer, 1996)**

For a Kelvin-Voigt solid in shear, the stress-strain relationship is expressed as

\[
\tau = G\gamma + \eta \frac{\partial \gamma}{\partial t} \tag{7.1}
\]
Where $\tau = \sigma_{xz}$ is the shear stress, $\gamma = \frac{\partial u}{\partial z}$ is the shear strain, $G$ is the Shear Modulus, $t$ is the time and $\eta$ is the viscosity of the material. Thus, the shear stress is the sum of an elastic part (proportional to strain) and a viscous part (proportional to strain rate). For a harmonic shear strain of the form as in Eq. 7.2 the shear stress can be expressed in the form shown in the Eq. 7.3.

\[
\gamma = \gamma_0 \sin \omega t \tag{7.2}
\]

\[
\tau = G\gamma_0 \sin \omega t + \omega \eta \gamma_0 \cos \omega t \tag{7.3}
\]

Where, $\gamma_0$ is the initial shear strain and $\omega$ is the wave frequency.

Together, Eq. 7.2 & Eq. 7.3 shows that the stress-strain loop of a Kelvin-Voigt solid is elliptical. The elastic energy dissipated in a single cycle ($\Delta W$) is given by the area of the ellipse, or

\[
\Delta W = \int_{t_0}^{t_0+2\pi/\omega} \tau \frac{\partial \gamma}{\partial t} \, dt = \pi \eta \omega \gamma_0^2 \tag{7.4}
\]

This indicates that the dissipated energy is proportional to the frequency of loading. Real soils, however, dissipate elastic energy hysteretically, by the slippage of grains on each other. As a result, their energy dissipation characteristics are insensitive to frequency. For discrete Kelvin-Voigt systems, the damping ratio, $\xi$, can be related to the force-displacement (or, equivalently, the stress-strain) loop as shown in Fig. 7.3.

Since the peak energy stored in the cycle is

\[
W = \frac{1}{2} G\gamma_0^2 \tag{7.5}
\]
Then,

\[
\xi = \frac{1}{4\pi} \frac{\pi \omega \gamma_0}{\frac{1}{2} G \gamma_0^2} = \frac{\eta \omega}{2G} \tag{7.6}
\]

![Hysteresis Loop Diagram](image)

**Figure 7.3: Relationship Between Hysteresis Loop and Damping Ratio**

(Kramer, 1996)

To eliminate frequency dependence while maintaining the convenience of the viscoelastic formulation, Eq. 7.6 is often rearranged to produce an equivalent viscosity that is inversely proportional to frequency. The use of this equivalent viscosity ensures that the damping ratio is independent of frequency.

\[
\eta = \frac{2G}{\omega} \xi \tag{7.7}
\]

A Kelvin-Voigt solid for vertically propagating SH-waves may be represented by a stack of infinitesimal elements of the type shown schematically in Fig. 7.2.

### 7.2.2 Transfer Functions for a Layered Damped Soil on Elastic Rock due to the Propagation of Harmonic Shear Waves.

Actual cases of site response problems involve soil deposits with layers of varying stiffness and damping characteristics with elastic wave energy reflecting /
transmitting boundaries. Such conditions require the development of transfer functions for layered soil deposits.

### 7.2.2.1 Linear elastic wave propagation

For linear elastic one-dimensional wave propagation through a soil media the soil is assumed to behave as a Kelvin-Voigt solid. The one-dimensional equation of motion for vertically propagating SH-waves is written as shown in Eq. 7.8.

\[
\rho \frac{\partial^2 u}{\partial t^2} = \frac{\partial \sigma_{xz}}{\partial z} \tag{7.8}
\]

Where, \(\rho\) is the mass density, \(u\) is the displacement and \(\sigma_{xz}\) is the shear stress.

Substituting Eq. 7.1 into Eq. 7.8 with \(\tau = \sigma_{xz}\) and \(\gamma = \partial u / \partial z\), and differentiating the right side allows the wave equation to be expressed as in Eq. 7.9.

\[
\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial z^2} + \eta \frac{\partial^3 u}{\partial z^2 \partial t} \tag{7.9}
\]

For harmonic waves, the displacements is written as given in Eq. 7.10.

\[
u(z,t) = U(z) e^{i\omega t} \tag{7.10}
\]

When Eq. 7.10 is substituted into the wave equation, it yields the ordinary differential equation, as shown in Eq. 7.11 and 7.12.

\[
(G + i\omega \eta) \frac{d^2 U}{dz^2} = -\rho \omega^2 U \tag{7.11}
\]

or,

\[
G^* \frac{d^2 U}{dz^2} = -\rho \omega^2 U \tag{7.12}
\]

Where,
is the complex shear modulus. Using Eq. 7.7 to eliminate the frequency dependence, the complex shear modulus can also be expressed as

\[ G^* = G(1 + 2i\xi) \]  

(7.14)

This equation of motion thus has the solution,

\[ u(z, t) = Ce^{i(\omega t - k'z)} + De^{i(\omega t + k'z)} \]  

(7.15)

Where, C and D are wave amplitudes travelling in the upward and downward direction and depend upon the boundary conditions and \( k' = \omega \sqrt{\rho / G^*} \) is the complex wave number. Kolsky (1963) has shown that,

\[ k^* = k_1 + ik_2 \]  

(7.16)

Where,

\[ k_1^2 = \frac{\rho \omega^2}{2G(1 + 4\xi^2)} \left( \sqrt{1 + 4\xi^2} + 1 \right) \]  

(7.17)

\[ k_2^2 = \frac{\rho \omega^2}{2G(1 + 4\xi^2)} \left( \sqrt{1 + 4\xi^2} - 1 \right) \]  

(7.18)

And, only the positive root of \( k_1 \) and the negative root of \( k_2 \) have physical significance.

Now, assuming the soil to have the shearing characteristics of a Kelvin-Voigt solid, the wave equation can be written as shown in Eq. 7.9, which has a solution of the form

\[ u(z, t) = Ce^{i(\omega t + k'z)} + De^{i(\omega t - k'z)} \]  

(7.19)
Where $k^*$ is a complex wave number with real part $k_1$ and imaginary part $k_2$ as can be seen from Eq. 7.16.

Figure 7.4 represents a soil deposit consisting of $N$ number of soil layers extending horizontally to infinity. The $n^{th}$ layer is assumed to be the bedrock.

![Layered Soil Deposit Diagram](image)

**Figure 7.4: Nomenclature for Layered Soil Deposit on Elastic Bedrock (Kramer, 1996)**

Assuming that each layer of soil behaves as a Kelvin-Voigt solid, the wave equation is expressed as Eq. 7.9 which has the solution as expressed in the form as given in Eq. 7.19. Where $C$ and $D$ represent the amplitudes of waves travelling in the $-z$ (upward) and $+z$ (downward) directions, respectively. $\omega$, is the circular frequency of the applied loading, $\eta$ is the viscosity of the material, The shear stress, $\tau$, can now be expressed as the product of the complex shear modulus, $G^*$, and the shear strain, $\frac{\partial u}{\partial z}$, so

$$\tau(z,t) = G^* \frac{\partial u}{\partial z} = (G + i\omega\eta) \frac{\partial u}{\partial z} = G(1 + 2i\xi) \frac{\partial u}{\partial z} \quad \ldots \ldots \ldots \ldots (7.20)$$
Introduction of a local co-ordinate system, Z, for each layer, gives the displacement at the top and bottom of layer ‘m’ as

\[ u_m (Z_m = 0, t) = (C_m + D_m)e^{i\omega t} \] ............................(7.21)

\[ u_m (Z_m = h_m, t) = (C_m e^{ik_m^*h_m} + D_m e^{-ik_m^*h_m})e^{i\omega t} \] ..........................(7.22)

The displacements at layer boundaries must be compatible (i.e., the displacement at the top of a particular layer must be equal to the displacement at the bottom of the overlying layer). Applying this compatibility requirement to the boundary between layer ‘m’ and layer ‘m+1’, that is,

\[ u_m (Z_m = h_m, t) = u_{m+1} (Z_{m+1} = 0, t) \] ........................................(7.23)

yields

\[ C_{m+1} + D_{m+1} = C_m e^{ik_m^*h_m} + D_m e^{-ik_m^*h_m} \] ........................................(7.24)

The shear stresses at the top and bottom of layer ‘m’ are

\[ \tau_m (Z_m = 0, t) = ik_m^* G_m (C_m - D_m)e^{i\omega t} \] ........................................(7.25)

\[ \tau_m (Z_m = h_m, t) = ik_m^* G_m (C_m e^{ik_m^*h_m} - D_m e^{-ik_m^*h_m})e^{i\omega t} \] ........................................(7.26)

Since, stresses must be continuous at layer boundaries,

\[ \tau_m (Z_m = h_m, t) = \tau_{m+1} (Z_{m+1} = 0, t) \] ........................................(7.27)

So,

\[ C_{m+1} - D_{m+1} = \frac{k_m^* G_m}{k_{m+1}^* G_{m+1}} (C_m e^{ik_m^*h_m} - D_m e^{-ik_m^*h_m}) \] ........................................(7.28)

Solving Eq. 7.27 and Eq. 7.28 we get the recursion formulas
Where, $\alpha^*_m$ is the complex impedance ratio at the boundary between layers ‘m’ and ‘m+1’.

$$\alpha^*_m = \frac{k_m C_m^*}{k_{m+1} C_{m+1}} = \frac{\rho_m (\nu_{y,m}^*)}{\rho_{m+1} (\nu_{y,m+1}^*)} \tag{7.31}$$

At the ground surface, the shear stress must be equal to zero, which requires that $C_1 = D_1$. If the recursion formulas of Eq. 7.29 & Eq. 7.30 are repeatedly applied for all layers from ‘1’ to ‘m’, functions relating the amplitudes in layer ‘m’ to those in layer ‘1’ can be expressed by

$$C_m = c_m(\omega) C_1 \tag{7.32}$$

$$D_m = d_m(\omega) D_1 \tag{7.33}$$

The transfer function relating the displacement amplitude at layer ‘i’ to that at layer ‘j’ is given by

$$F_{ij}(\omega) = \frac{|u_j|}{|u_i|} = \frac{c_i(\omega) + d_i(\omega)}{c_j(\omega) + d(\omega)} \tag{7.34}$$

Because $|\ddot{u}| = \omega |\dot{u}| = w^2 |u|$ for harmonic motion, Eq. 7.34 describes the amplification of accelerations and velocities from layer i to layer j. Eq. 7.34 also represents the transfer function of one dimensional wave propagation for a linear system. Equation 7.34 indicates that the motion in any layer can be determined from the motion in any other layer. Hence, if the motion at any one point in the soil profile is known, the motion at any other layers can be contributed.
7.2.2.2 One Dimensional Equivalent Linear Site Response Analysis (1-DELSRA)

Since the nonlinearity of soil behaviour is well known, the linear approach is often modified to provide reasonable estimates of site response for actual field problems. Linear approach requires that Shear Modulus (G) and Damping Ratio (ξ) be constant for each soil layer irrespective of the strain level. However, during dynamic loading the strain levels constantly change at each time increment and hence the G and ξ values are also subjected to change to be compatible with the strain level. To address this problem the soil properties are assumed to be Equivalent Linear which provides a pseudo nonlinear behaviour of the soil properties. Equivalent linear soil properties provide a reasonable approximation of the actual nonlinear hysteretic stress-strain behaviour of soils subjected to a cyclic load. The equivalent linear shear modulus, G, is generally taken as secant modulus and the equivalent linear damping ratio, ξ, as the damping ratio that produces the same energy loss in a single cycle as the actual hysteresis loop. In equivalent linear site response analysis, the nonlinear response of the soil is approximated by modification of the linear elastic soil properties of the soil based on the induced shear strain level due to cyclic loading. This induced shear strains depend on the soil properties. The strain compatible shear modulus and damping ratio are calculated by subsequent iteration based on computed strain.

To have variable G and ξ, an objective definition of strain level in needed. The laboratory tests from which modulus reduction and damping ratio curves have been developed used simple harmonic loading and characterised the strain level by the peak shear strain amplitude. The time history of shear strain for a typical earthquake motion, however, is highly irregular with peak amplitude that may only be
approached by a few spikes in the record. Figure 7.5 shows both harmonic (as in a typical laboratory test) and transient (as in a typical earthquake) shear strain time histories that have the same peak cyclic shear strain. Clearly, the harmonic record represents a more severe loading condition than the transient record, although their peak values are identical. As a result, it is common to characterize the strain level of the transient record in terms of an effective shear strain which has been empirically found to vary between about 50% to 70% of the maximum shear strain. The computed response is not particularly sensitive to this percentage, however, and the effective shear strain is often taken as 65% of the peak strain (Kramer, 1996).

![Figure 7.5: Two Shear Strain Time Histories with Identical Peak Shear Strains. (Kramer, 1996)](image)

Since the computed strain level depends on the values of the equivalent linear properties, an iterative procedure is required to ensure that the properties used in the analysis are compatible with the computed strain levels is all layers. Equivalent-linear site response analysis requires that the strain–dependent nonlinear properties, G and ξ, be predefined. This is achieved through modulus reduction and damping curves. These curves describe the variation of the $G/G_{\text{max}}$ and $\xi$ with shear strain. Referring to Fig. 7.6, the iterative procedure operates as follows:
1. Initial estimates of $G_{\text{max}}$ using the Eq. 7.35 and $\xi$ as 5 % respectively are made for each layer. The initial estimated values usually correspond to the same strain level; the low-strain values are often used for the initial estimate.

$$G_{\text{max}} = \rho V_s^2 \quad \text{(7.35)}$$

Where, $\rho$ is the mass density of the site and $V_s$ is the measured shear wave velocity.

2. The estimated $G$ and $\xi$ values are used to compute the ground response, including time histories of shear strain for each layer.

3. The wave amplitudes $C$ and $D$ are computed for each of the layers.

4. The strain transfer function is calculated for each of the layers.

5. The maximum shear strain is computed for each layer by applying the strain transfer function to the input Fourier amplitude spectrum and thus the maximum response is obtained.

6. The effective shear strain in each layer is determined from the maximum shear strain in the computed shear strain time history. For layer $j$

$$\gamma_{\text{eff},j}^{(i)} = R_{\gamma}\gamma_{\text{max},j}^{(i)} \quad \text{(7.36)}$$

Where the superscript refers to the iteration number and $R_{\gamma}$ is the ratio of the effective shear strain to maximum shear strain. $R_{\gamma}$ depends on earthquake magnitude (Idriss and Sun, 1992) and can be estimated from

$$R_{\gamma} = \frac{M - 1}{10} \quad \text{(7.37)}$$

7. From this effective shear strain, the strain compatible shear modulus and damping ratio i.e. new equivalent linear values, $G^{(i+1)}$ and $\xi^{(i+1)}$ respectively
are recalculated for the next iteration using the discrete points from the curves as shown in Fig. 7.6.

8. Steps 2 to 7 is repeated until differences between the computed shear modulus and damping ratio values in two successive iterations fall below some predetermined value in all layers. Although convergence is not guaranteed, differences of less than 5 to 10% are usually achieved in three to five iterations (Schnabel et. al., 1972).

![Figure 7.6: Iteration towards Strain-Compatible Shear Modulus and Damping Ratio. (Kramer, 1996)](image)

Even though the process of iteration toward strain-compatible soil properties allows nonlinear soil behaviour to be approximated, it is important to note that the complex response method is still a linear method of analysis. The strain-compatible soil properties are constant throughout the duration of the earthquake, regardless of whether the strains at a particular time are small or large. The method is incapable of representing the changes in soil stiffness that actually occurs during the earthquake. The equivalent linear approach of one-dimensional ground response analysis of layered sites has been coded into a widely used computer program called SHAKE (Schnabel et. al., 1972). SHAKE91 (Idriss and Sun, 1992) and SHAKE2000
(Ordonez, 2006) are the updated versions of SHAKE (Schnabel et al. 1972). SHAKE2000 (Ordonez, 2006) has been used for site response analysis in this study.

7.3 **PRESENT STUDY**

In the present study, it is envisaged to develop a soil model for Western Guwahati to perform One-Dimensional Equivalent Linear Site Response Analysis by comparison of the frequency content of recorded ground motion and free-field ground motion generated as a result of the analysis. Within the geographical area of study, One-Dimensional Equivalent Linear Site Response Analysis has been carried out for three locations, Guwahati-Central, Boko-Palashbari and Goalpara. Data regarding soil profiles at the location of the earthquake recording stations has been collected by conducting and collecting Standard Penetration Tests (SPT) wherever possible, as well as collecting Litholog from Central Ground Water Board (CGWB), Govt. of India, Microzonation of Guwahati (DST, 2008) project and soil data from various agencies. The various Borelogs, Litho logs and Soil Properties and their sources used in this study have been provided in Appendix A, B and C respectively.

Three soil profiles representing the subsoil conditions of the sites under study is developed from the collected information and used in the analysis. The following sections give an insight as to how the soil profile has been developed for the study and what soil profiles have been used for the study.

7.3.1 **Methodology Adopted**

A simple methodology as shown in Fig. 7.7 is adopted in the present study.
First of all geotechnical characterization of Boko-Palashbari, Guwahati-Central and Goalpara sites has been carried out. During the process, Bore logs and Litholog data is combined to obtain a general pattern of horizontal soil layers in the study area. Standard modulus reduction and damping ratio curves available in the database of SHAKE2000, appropriate for the geotechnical units has been adopted to define the dynamic soil properties. The shear velocity profile is obtained from the field N-values and beyond the depth where N-values is not available, the shear wave velocity is assumed to vary linearly, up to the bedrock level. Recorded ground motions at Nongstoin have been used as the input motions.

Figure 7.7: Methodology of 1-DELSRA Adopted
1-DELSRA is carried out for the three locations namely, Guwahati-Central, Boko-Palashbari and Goalpara using the recorded ground motions at Nongstoin as the input motions. The frequency content in the form of Fourier Amplitude Spectrum (FAS) of the free field ground motions obtained from the 1-DELSRA analyses has been compared with that of the recorded motions to validate the geotechnical modelling methodology used in the study.

Finally, the methodology as described in the first step and the second step is used to define the problem statement on Guwahati Airport area where data of 10(ten) bore holes have been obtained. Using the same input motions, which have been scaled to 0.18g and 0.36g (for Guwahati Airport), 1-DELSRA is carried out.

7.3.2 Selection of Input Ground Motions and Justification for Selection.

It is generally recognized that the selection of input ground motion in one of the primary contributors to uncertainty in site response analysis (Matasovic and Hashash, 2012). Various codes and design guidance documents outline procedures for selection of design ground motions. ASCE(2006) (ASCE 7-05, Chapter 21, Section 21.1.1) requires that “at least five recorded or simulated rock outcrop horizontal ground motion acceleration time histories be selected from events having magnitudes and fault distances that are consistent with those that control the MCE (Maximum Considered Earthquake)” . To further minimize this uncertainty related to the selection of design ground motions, ASCE 7-05 also requires that the time histories be scaled such that the average acceleration response spectrum of each time history is approximately at the level of the MCE rock acceleration response spectrum over the period range of significance to structural response.
Five earthquakes events recorded at Nongstoin, obtained from PESMOS were used as input motion for the analysis. Nongstoin is located at a distance of approximately 100km from the study area and is classified as under site class A (Mittal et. al., 2012) having shear wave velocities within the range of 700m/sec to 1400m/sec. This classification satisfies the basic requirement of rock site motion required for site response analysis. The records were used as outcrop motions which were applied to the bedrock of the study area to obtain the free field ground motions. The basic properties of the 5 (five) input motions are provided in Table 7.1 & 7.2.

**Table 7.1: Input Motion Details**

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Epicenter</th>
<th>Magnitude (Mw)</th>
<th>Focal Depth (km)</th>
<th>Epicentral Distance (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11Aug 2009</td>
<td>24.4N 94.8E</td>
<td>5.6</td>
<td>22</td>
<td>378</td>
</tr>
<tr>
<td>03 Sep 2009</td>
<td>24.3N 94.6E</td>
<td>5.9</td>
<td>100</td>
<td>363</td>
</tr>
<tr>
<td>21 Sep 2009</td>
<td>27.3N 91.5E</td>
<td>6.2</td>
<td>8</td>
<td>197</td>
</tr>
<tr>
<td>29 Oct.2009</td>
<td>27.3N 91.4E</td>
<td>5.2</td>
<td>5</td>
<td>197</td>
</tr>
<tr>
<td>29 Oct.2009</td>
<td>26.6N 90.0E</td>
<td>4.2</td>
<td>10</td>
<td>174</td>
</tr>
</tbody>
</table>

**Table 7.2: Input Motion Properties**

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Maximum Acceleration (g)</th>
<th>Predominant Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11Aug 2009</td>
<td>0.015</td>
<td>0.20</td>
</tr>
<tr>
<td>03 Sep 2009</td>
<td>0.009</td>
<td>0.20</td>
</tr>
<tr>
<td>21 Sep 2009</td>
<td>0.028</td>
<td>0.22</td>
</tr>
<tr>
<td>29 Oct.2009</td>
<td>0.010</td>
<td>0.18</td>
</tr>
<tr>
<td>29 Oct.2009</td>
<td>0.006</td>
<td>0.18</td>
</tr>
</tbody>
</table>
Additionally, HVSR (Horizontal to Vertical Spectral Ratio) analysis indicates that the variation of horizontal spectral ratio as compared to vertical is almost equal to unity at low frequencies. The HVSR amplitude observed is quite low (below 5) at higher frequencies (around 5 Hz) which suggest negligible site effects for this station. This observation justifies the classification of Nongstoin motion as rock outcropping motion. Figure 7.8, depicts the HVSR curve of Nongstoin for all the five records considered, which shows an almost flat spectrum.

Considering the facts that Nongstoin falls under the Site Class A, has a HVSR amplitude almost equal to unity in the frequency range of interest (0.1 – 10 Hz) and the epicentral distance almost equal to the region under study, it can be concluded that the Nongstoin motion can be used as input motion for site response analysis.

![HVSR Plot (Station-Nongstoin)](image)

**Figure 7.8: HVSR Plot of Nongstoin Motions**

Considering the availability of existing ground motions, for Boko-Palashbari Site and Guwahati-Central Site the recorded motions of Nongstoin of 11-Aug-2009, 03-Sep-2009 and 21-Sep-2009 were used as input motions while for Goalpara Site all
the five Nongstoin motions were used as input motions. For Guwahati Airport, all the five Nongstoin motions were scaled to achieve a PGA of 0.18g and 0.36g and then used as the input motions. This is done to ensure that the worst case scenario was achieved as per IS:1893-2002 for seismic zone-V. Additional details of input motions are provided in Appendix D.

7.3.3 Shear Wave Velocity (Vs)

Shear wave velocity is obtained from the uncorrected field N value. Maheswari et. al., (2010), Mhaske and Choudhury (2011) has reported that the corrected and uncorrected N-Values predict Shear wave velocity (Vs) with reasonable good efficiency.

The relation proposed by Sharma et. al., (2013) and Sharma and Rahman (2016) as given by Eq. 7.38 applicable for the Guwahati region has been used to determine the shear wave velocity up to the depth to which N-Values were available.

\[ \text{Vs} = 74.639 \times N^{0.3876} \]

Beyond that, the shear wave velocity is assumed to be increasing linearly up to the engineering bedrock level. A shear wave velocity of 760 m/sec is assumed for the engineering bedrock level (Anbazhagan and Sitharam, 2008), up to which soil/rock profile is available. The competent bedrock is assigned a shear wave velocity of 1500 m/sec. Table 7.3, 7.4 & 7.5 shows the shear wave velocity, used in the analysis for Boko-Palashbari, Guwahati-Central and Goalpara sites respectively.
### Table 7.3: Shear Wave Velocity Profile for Boko-Palashbari Site

<table>
<thead>
<tr>
<th>Layer</th>
<th>N-Value</th>
<th>Shear Wave Velocity (Vs) (m/sec)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>7</td>
<td>158.68</td>
<td>Bore Log</td>
</tr>
<tr>
<td>II</td>
<td>27</td>
<td>267.77</td>
<td>Bore Log</td>
</tr>
<tr>
<td>III</td>
<td>**</td>
<td>449.52</td>
<td>Litholog</td>
</tr>
<tr>
<td>IV</td>
<td>**</td>
<td>465.86</td>
<td>Litholog</td>
</tr>
<tr>
<td>V</td>
<td>**</td>
<td>474.23</td>
<td>Litholog</td>
</tr>
<tr>
<td>VI</td>
<td>**</td>
<td>743.65</td>
<td>Litholog</td>
</tr>
<tr>
<td>VII</td>
<td>**</td>
<td>760.00</td>
<td>Litholog</td>
</tr>
<tr>
<td>VIII</td>
<td>**</td>
<td>1500.00</td>
<td>Litholog</td>
</tr>
</tbody>
</table>

### Table 7.4: Shear Wave Velocity Profile for Guwahati-Central Site

<table>
<thead>
<tr>
<th>Layer</th>
<th>N-Value</th>
<th>Shear Wave Velocity (Vs) (m/sec)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>16</td>
<td>218.62</td>
<td>Bore Log</td>
</tr>
<tr>
<td>II</td>
<td>31</td>
<td>282.50</td>
<td>Bore Log</td>
</tr>
<tr>
<td>III</td>
<td>**</td>
<td>436.53</td>
<td>Litholog</td>
</tr>
<tr>
<td>IV</td>
<td>**</td>
<td>760.00</td>
<td>Litholog</td>
</tr>
<tr>
<td>V</td>
<td></td>
<td>1500.00</td>
<td>Litholog</td>
</tr>
</tbody>
</table>

### Table 7.5: Shear Wave Velocity Profile for Goalpara Site

<table>
<thead>
<tr>
<th>Layer</th>
<th>N-Value</th>
<th>Shear Wave Velocity (Vs) (m/sec)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>6</td>
<td>149.48</td>
<td>Borelog</td>
</tr>
<tr>
<td>II</td>
<td>10</td>
<td>182.21</td>
<td>Borelog</td>
</tr>
<tr>
<td>III</td>
<td>19</td>
<td>233.68</td>
<td>Borelog</td>
</tr>
<tr>
<td>IV</td>
<td>**</td>
<td>277.87</td>
<td>Litholog</td>
</tr>
<tr>
<td>V</td>
<td>**</td>
<td>760.00</td>
<td>Litholog</td>
</tr>
<tr>
<td>VI</td>
<td>**</td>
<td>760.00</td>
<td>Litholog</td>
</tr>
<tr>
<td>VII</td>
<td>**</td>
<td>1500.00</td>
<td>Litholog</td>
</tr>
</tbody>
</table>
7.3.4 Soil Profiles used for Site Response Analysis

It is known that subsoil conditions are spatially varying in nature. A detailed subsoil investigation program involving collecting the layer details of subsoil of the study location and collection of litho logs showing soil stratification up to the bedrock layer has been carried out. This helped in the profiling of the soil model to be used in the one dimensional site response analysis. In the following section the generalised soil profile which has been used for analysis in this study has been discussed.

7.3.4.1 Boko-Palashbari site

The recording station of the Boko-Palashbari station has co-ordinates Latitude 25.976° N and Longitude 91.230° E and the site is classified as under Category ‘C’ (Mittal et. al., 2012). This category of site has a shear wave velocity of the range 200m/sec to 375m/sec for the upper 30 metre of soil cover.

A general assessment of the soil data show that the subsoil contains sandy / silty clay up to a depth of 15.00m from existing ground level, beyond that sandy stratum is encountered up to a depth of 89.55m. A clay layer of thickness 3.15m is shown on the Litholog beyond which gravelly strata has been encountered up to a depth of 200.25 m at which bedrock is indicated to be encountered. Table 7.6 shows the generalized soil profile considered for study in Boko-Palashbari.
Table 7.6: Generalized Soil Profile for Boko-Palashbari Site

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (m)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>9</td>
<td>Clay</td>
</tr>
<tr>
<td>II</td>
<td>6</td>
<td>Silt</td>
</tr>
<tr>
<td>III</td>
<td>68.4</td>
<td>Fine sand</td>
</tr>
<tr>
<td>IV</td>
<td>6.15</td>
<td>Fine / Medium sand</td>
</tr>
<tr>
<td>V</td>
<td>3.15</td>
<td>Clay</td>
</tr>
<tr>
<td>VI</td>
<td>101.40</td>
<td>Sand / Clay / Gravel</td>
</tr>
<tr>
<td>VII</td>
<td>6.15</td>
<td>Gravel</td>
</tr>
<tr>
<td>VIII</td>
<td></td>
<td>Bedrock</td>
</tr>
</tbody>
</table>

7.3.4.2 Guwahati-Central site

The recording station of the Guwahati station had co-ordinates Latitude 26.190° N and Longitude 91.746° E and the site is classified as under Category ‘C’ (Mittal et. al., 2012), which has a shear wave velocity of the range 200m/sec to 375m/sec for the upper 30 metre of soil cover. Soil data has been collected for sites within a radius of 500m around the recording station site except the North direction as the river Brahmaputra flows to the north of the recording station. The soil data has been obtained for the following locations KKH Hostel, Cotton College (26.186°N, 91.749°E), Assam Textile Institute (26.185°N, 91.754°E), Jorpukhuri Par (26.188°N, 91.737°E), Jahajghat Lower Primary School, Uzaan Bazar (26.198°N, 91.760°E), Dighalipukhuri Par, Near CCD (26.186°N, 91.752°E). Additionally, Litholog was collected for the following two locations Brahmaputra Ashoka Hotel (26.194°N, 91.751°E), Circuit House, Guwahati (26.193°N, 91.751°E). The litholog defined the bedrock to be at a depth of 42.50m and 68.75m at Brahmaputra Ashoka and Circuit...
House respectively, thus showing a falling gradient of component bedrock toward the west. This is in agreement with the information given in the basement map of Microzonation of Guwahati (DST, 2008) report for the region.

A general assessment of the soil data show that the subsoil contains sandy / silty clay up to a depth of 16.00m from existing ground level, beyond that sandy stratum is encountered up to a depth of 30m, followed by gravelly strata up to a depth of 42.50m. After 42.50m, a layer of gravel with pebbles is found up to a depth of 68.75m at which bedrock is established. Table 7.7 provides the layer details of the soil profile considered for study in Guwahati-Central.

**Table 7.7: Generalized Soil Profile for Guwahati-Central Site**

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness(m)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>16.00</td>
<td>Sandy/Silty Clay</td>
</tr>
<tr>
<td>II</td>
<td>14.00</td>
<td>Medium / Fine sand mixed with silt</td>
</tr>
<tr>
<td>III</td>
<td>12.50</td>
<td>Gravel</td>
</tr>
<tr>
<td>IV</td>
<td>26.25</td>
<td>Gravel with pebbles</td>
</tr>
<tr>
<td>V</td>
<td></td>
<td>Bedrock</td>
</tr>
</tbody>
</table>

**7.3.4.3 Goalpara site**

The recording station of the Goalpara station has geographical co-ordinates, Latitude 26.152° N and Longitude 90.627° E and the site is classified as under Category ‘C’ (Mittal et. al., 2012), which has a shear wave velocity of the range 200m/sec to 375m/sec for the upper 30 metre of soil cover.
Table 7.8: Generalized Soil Profile for Goalpara Site

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness (m)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>7.20</td>
<td>Silty Clay</td>
</tr>
<tr>
<td>II</td>
<td>1.40</td>
<td>Clayey Silt</td>
</tr>
<tr>
<td>III</td>
<td>21.40</td>
<td>Sand</td>
</tr>
<tr>
<td>IV</td>
<td>5.5</td>
<td>Gravel with fine sand</td>
</tr>
<tr>
<td>V</td>
<td>55.5</td>
<td>Gravel with fine/medium</td>
</tr>
<tr>
<td>VI</td>
<td>9.5</td>
<td>Gravel</td>
</tr>
<tr>
<td>VII</td>
<td></td>
<td>Bedrock</td>
</tr>
</tbody>
</table>

A general assessment of the soil data shows that the subsoil contains silty clay up to a depth of 7.20m from existing ground level, beyond that clayey / silty strata is encountered up to a depth of 8.60m, beyond that sandy stratum is observed up to a depth of about 30.00m. Beyond 30.00m gravelly strata were encountered up to a depth of 100.50 m at which bedrock is indicated to be encountered. Table 7.8 shows the typical soil profile considered for study in Boko-Palashbari.

7.3.4.4 Guwahati Airport

The soil profile of Guwahati Airport area is based on 10 (ten) no of borelogs and Litholog. Analyses have been conducted by using the recorded earthquake motion as mentioned in Table 7.1 as input motions. Considering the fact that the study area is classified to fall under seismic zone-V (IS:1893-2002) (having a zero period acceleration of 0.36g), the recorded motions were scaled to attain a PGA of 0.36g and 0.18g. Thus, a suite input motions containing 10 (ten) scaled motions of various earthquakes as in Table 7.1 has been used to excite the soil column. A total of 10 (ten) soil columns have been excited by the 10(ten) input motions. Thus, a total of 100 (one
hundred) analyses have been conducted for the Guwahati Airport area. Details of the soil profiles used for analysis in the case of Guwahati Airport is provided in Appendix-A.

7.3.5 Dynamic Material Properties

Site response analysis requires index properties such as density, Atterberg limits and relative density of the various layers. Strength properties such a friction angle and undrained shear strength are important input properties, especially, for soft soils and areas with high levels of shaking. In addition to these properties, dynamic soil properties of the soil layers need to be defined.

In a dynamic system, the properties that govern the response are the mass, stiffness, and damping (Kottke and Rathje, 2009). During seismic loading in soil, the mass of the soil system is characterized by the mass density $\rho$ and the layer height $h$, the stiffness is characterised by the shear modulus ($G$) and the damping is characterised but the viscous damping ($\zeta$).

Section 7.2.2.2 of has discussed about the equivalent linear site response analysis in which the nonlinear characteristics of the soil has been approximated to be represented by a linear system that use strain compatible dynamic soil properties ($G$ and $\xi$). The strain dependence of the soil system due to cyclic loading is represented in the form of modulus reduction curves ($G/G_{\text{max}}$ versus shear strain) and damping curves ($\xi$ versus shear strain).

Laboratory tests using cyclic triaxial, cyclic direct simple shear (DSS), and resonant column devices are by far the most common devices for defining the
dynamic behaviour of soils at a given site. These tests, however, depend on the availability of high-quality undisturbed samples, which might be available for cohesive soils, but are difficult to obtain for cohesionless soils. Cyclic laboratory tests require a high degree of sophistication and are often very expensive.

Many studies have been conducted to characterize the factors that affect \( G/G_{\text{max}} \) and D of soils (e.g., Seed and Idriss, 1970; Hardin and Drnevich, 1972; Iwasaki et. al., 1978; Ishibashi, 1981; Kokusho et. al., 1982; Imai and Tonouchi, 1982; Seed et. al., 1986; Sun et. al., 1988; Das, B.M., 1993; Vucatic and Dobry, 1991; Ishibashi and Zhang, 1993; EPRI, 1993; Rollins et. al., 1998; Darendeli, 2001; Stokoe et. al., 2004, Zhang, et. al., 2005; Okur and Ansal, 2007). Expressions suggested by them are intensively used in industry and research to compute the dynamic shear modulus of soil either by co-relating it to laboratory or insitu tests (Chowdhury et. al., 2015). The studies have developed standardized dynamic soil response curves as a function of shear strain that approximate the nonlinear hysteretic behaviour of soils under symmetrical cyclic loading by

1. Equivalent small strains shear modulus \((G_{\text{max}})\) that corresponds to the secant modulus through the endpoints of a hysteresis loop.

2. Equivalent viscous damping ratio \((\xi)\) which is proportional to the energy loss from a single cycle of shear deformation.

The small strain shear modulus \((G_{\text{max}})\) is best characterised by in-situ measurement of the shear-wave velocity as a function of depth. However, \(G_{\text{max}}\) can also be calculated, from shear wave velocity obtained from the empirical relations, with reasonable accuracy. An example of shear wave velocity profile is shown in Fig.
7.9. The profile depicts a generally increasing shear-wave velocity with increasing depth. Example of modulus reduction and damping curves for soil as represented in Fig 7.10. These curves idealize a decreasing value of soil stiffness and increasing value of damping ratio with the increase in shear strain.

Figure 7.9: Example of Shear Wave Velocity Profile (Kottke and Rathje, 2009)

Figure 7.10: Example of Standard Shear Modulus Reduction and Material Damping Curves (Kottke and Rathje, 2009)
Hardin and Drnevich (1970) had presented the first comprehensive study on the parameters affecting the shear modulus and damping factors of soils and thus the nonlinear behaviour of the soil. Empirical expressions for determination of the shear modulus and damping values have been postulated by them. Based on their study, the Table 7.9 shows the parameters and their relative importance in terms of their effect on shear modulus and material damping.

The empirical relationship for evaluating the maximum shear modulus as proposed by Hardin and Drnevich (1970) is given by the Eq. 7.39.

\[
G_{max} = 14760 \times \frac{(2.973 - e)^2}{1 + e} \times (OCR)^a (\sigma'_m)^{1/2} 
\] ..........................(7.39)

where, \(G_{max}\) = maximum shear modulus in psf

\(e\) = void ratio

\(a\) = a parameter that depends on the plasticity index of the soil

\(\sigma'_m\) = mean principal effective stress in psf.

The values of ‘a’ is presented in Table 7.10.
Table 7.9: Parameters the Control Nonlinear Behaviour and their Relative Importance in Terms of Affecting Shear Modulus and Material Damping as given by Hardin and Drnevich (1970) (after Darendeli, 2001)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Impact on Shear Modulus</th>
<th>Impact on Damping</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Clean sands</td>
<td>Cohesive soils</td>
</tr>
<tr>
<td>Strain Amplitude</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>Mean effective Confining Pressure</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>Number of Loading Cycles</td>
<td>+</td>
<td>*</td>
</tr>
<tr>
<td>Degree of saturation</td>
<td>*</td>
<td>***</td>
</tr>
<tr>
<td>Over-consolidation Ratio</td>
<td>*</td>
<td>**</td>
</tr>
<tr>
<td>Effective Strength Envelope</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Octahedral Shear Stress</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>Frequency of Loading (above 0.1 Hz)</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Other Time Effects</td>
<td>*</td>
<td>**</td>
</tr>
<tr>
<td>Grain size, Shape, Gradation, Mineralogy</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Soil Structure</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Volume Change due to Shearing Strain below 0.5 %</td>
<td>-</td>
<td>*</td>
</tr>
</tbody>
</table>

*** Very Important, **Less Important, * Relatively Important, + Relatively Unimportant except for saturated sand, - Unknown
Table 7.10: Value of ‘a’

<table>
<thead>
<tr>
<th>PI</th>
<th>a</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>20</td>
<td>0.18</td>
</tr>
<tr>
<td>40</td>
<td>0.30</td>
</tr>
<tr>
<td>60</td>
<td>0.41</td>
</tr>
<tr>
<td>80</td>
<td>0.48</td>
</tr>
<tr>
<td>≥100</td>
<td>0.50</td>
</tr>
</tbody>
</table>

The modulus value, $G$, corresponding to a strain level, $\gamma$, is then evaluated from the Eq. 7.40.

$$G = \frac{g_{max}}{1+\frac{\gamma}{\gamma_r}}$$

Where, $\gamma_r = \frac{\tau_{max}}{g_{max}}$

$$\tau_{max} = \left\{ \left( \frac{1+K_o}{2} \sigma'_v \sin \phi' + c' \cos \phi' \right)^2 - \left( \frac{1-K_o}{2} \sigma'_v \right)^2 \right\}^{\frac{1}{2}}$$

$K_o$ = coefficient of lateral stress at rest.

$\sigma'_v$ = vertical effective stress, and

$c', \phi'$ = static strength parameters in terms of effective stress.

The damping ratio value, $\xi$, at a strain level, $\gamma$, is given by the Eq. 7.42.

$$\xi = \frac{\lambda_{max} \frac{\gamma}{\gamma_r}}{1+\frac{\gamma}{\gamma_r}}$$
where, $\lambda_{\text{max}}$ = maximum damping ratio corresponding to very large strains.

For clean sands,

$$\lambda_{\text{max}} = D - 1.5 \log_{10} N$$

where, $D = 33\%$ for clean dry sands

$= 28\%$ for clean saturated sands.

$N =$ number of cycles.

For saturated cohesive soils,

$$\lambda_{\text{max}} = 31 - (3 + 0.03 f)(\sigma'_m)^{\frac{3}{2}} + 1.5 f^{\frac{1}{2}} - 1.5 \log N$$

where, $f =$ frequency of applied cyclic load in cycles per second

$\sigma'_m =$ mean principal effective stress in kg/cm$^2$

As reported by Darendeli (2001), the empirical relations in determining the shear modulus and damping due to cyclic loading, suggested by Hardin and Drnevich (1970) has remained essentially same. The exception has been that various other researchers have tried to refine, improve or generalize their results over that of Hardin and Drnevich (1970). This exception has given the shape of modulus reduction and material damping curves which are widely accepted and utilized in practice.

Equations 7.39, 7.40 and 7.42 thus form the basis on which various researchers like Seed and Idriss (1970), Schnabel (1973), Seed et. al. (1986), Vucetic and Dobry (1991), Darendeli (2001) worked upon in developing standard modulus reduction and damping ratio curves. The standard curve was classified and
generalized to be applicable to the various type of soils depending upon the Plasticity Index (PI), Over Consolidation Ratio (OCR), confining stress.

Vucetic and Dobry (1991) had concluded on the basis of the results of a large number of cyclic loading results that, “Plasticity Index (PI) is the main factor controlling the location of the modulus reduction curve \(G/G_{\text{max}}\) versus shear strain) and material damping curves \((\xi\ versus\ shear\ strain)\)”. This statement was applicable to a wide variety of saturated soils ranging from clays to sands. It was reported that as the PI goes up, \(G/G_{\text{max}}\) increases and shear strain decreases, i.e., higher plasticity soils generally exhibit a more linear cyclic stress-strain response which is applicable for both normally and overconsolidated soils.

Standard curves have been used in many studies in the past. Hashash et. al. (2011) had reported that in the absence of site-specific dynamic material properties, standard curves proposed by published literature can be considered as a better alternative for analysis. Kumar and Dey (2015), Kumar and Krishna (2013), DST (2008) has used standard modulus reduction curves and damping curves in site response analysis with reliable results for the Guwahati region.

7.3.5.1 Boko-Palashbari site

Standard curves proposed by Vucetic and Dobry (1991), Seed and Idriss (1970), Seed et.al. (1986) and Schnabel (1973) has been used for defining the dynamic material properties of Boko-Palashbari soil layers depending upon the geotechnical property and plasticity of the soil. Figure 7.11 & 7.12 shows the standard curves used for defining the material properties of the soil in Boko-Palashbari. It was found that the Plasticity Index (PI) varied from 15 – 25 for the region. From the
discussion in section 7.3.4.1 it can be observed that the soil profile generally consists of layers of clay, silt, sand and gravel. Standard curves of Vucetic and Dobry (1991) has been assigned to clay layers, while lower bound, average and upper bound curves of Seed and Idriss (1970) have been assigned to sandy layers. For layers containing gravel, curves of Seed and Idriss (1986) has been assigned. To define properties of the elastic bedrock, curves of Schnabel (1973) has been used.

![Shear Modulus Reduction Curves](image)

**Figure 7.11: Shear Modulus Reduction Curves (Boko-Palashbari Site)**
Figure 7.12: Damping Ratio Curves (Boko-Palashbari Site)

7.3.5.2 Guwahati-Central site

Standard curves proposed by Vucetic and Dobry (1991), Seed and Idriss (1970), Seed et.al. (1986) and Schnabel (1973) have been used for defining the dynamic material properties of Guwahati soil layers depending upon the geotechnical property and plasticity of the soil. Figure 7.13 & 7.14 shows the plots for modulus reduction \( \frac{G}{G_{\text{max}}} \) and damping ratio \( \xi \) versus shear strain \( \gamma \) curves. It can be observed that, for Guwahati-Central, the soil profile consisted of three main layers; silty clay, sand and gravel, just before encountering bedrock. For clayey soil, it was seen that the Plasticity Index (PI) ranged between 20 – 30. Based on the reported PI, standard curves for modulus reduction \( \frac{G}{G_{\text{max}}} \) and damping ratio \( \xi \) of Vucetic and Dobry (1991) has been assigned to the first layer. Beyond that sand/silt has been encountered for which standard curves of Seed and Idriss (1970) have been assigned.
Seed and Idriss (1970) curves are for cohesionless soils and are divided into three categories lower bound, average and upper bound. The lower bound refers to cohesionless soils of low stiffness while average and upper bound refers to average and high stiff cohesionless soils respectively. To define the gravels encountered, standard curves proposed by Seed et. al. (1986) has been used. To define the bedrock, standard curves as proposed by Schnabel (1973) have been assigned to the elastic bedrock.

![Image of Shear Modulus Reduction Curves](image)

**Figure 7.13: Shear Modulus Reduction Curves (Guwahati-Central Site)**
7.3.5.3 Goalpara Site

Standard curves proposed by Vucetic and Dobry (1991), Seed and Idriss (1970), Seed et.al. (1986) and Schnabel (1973) has been used for defining the dynamic material properties of Goalpara soil layers depending upon the geotechnical property and plasticity of the soil. It has been observed from the soil data collected that the PI values ranged from 20 – 30. Figure 7.15 & 7.16 shows the standard curves used for defining the material properties of the soil in Boko-Palashbari. From the discussion in section 7.3.2.3 it can be seen that the soil profile generally consisted of clay, sand and gravel. Standard curves of Vucetic and Dobry(1991) has been assigned to clay layers, while lower bound, bound curves of Seed and Idriss (1970) have been assigned to sandy layers. For layers containing gravel, curves of Seed and Idriss (1986) have been assigned. To define the elastic bedrock, curves of Schnabel (1973) have been used.
Figure 7.15 : Shear Modulus Reduction Curves (Goalpara Site)

Figure 7.16: Damping Ratio Curves (Goalpara Site)
7.3.5.4 Guwahati Airport

Standard curves proposed by Vucetic and Dobry (1991), Seed and Idriss (1970), Seed et.al. (1986) and Schnabel (1973) has been used for defining the dynamic material properties of Goalpara soil layers depending upon the geotechnical property and plasticity of the soil. It has been observed from the soil data collected that the PI values ranged from 20 – 25. Figure 7.17 & 7.18 shows the standard curves used for defining the material properties of the soil in Boko-Palashbari. From the data of 10(ten) soil profiles as provided in APPENDIX A it is observed that the soil profile generally consisted of clay underlain by sand of loose to dense consistency and gravel. Standard curves of Vucetic and Dobry(1991) has been assigned to clay layers, while lower bound, average and upper bound curves of Seed and Idriss (1970) have been assigned to sandy layers. For layers containing gravel, curves of Seed and Idriss (1986) have been assigned. To define the elastic bedrock, curves of Schnabel (1973) have been used.

Figure 7.17 : Shear Modulus Reduction Curves (Guwahati Airport Site)
7.3.6 Site Response Analyses

The numerical analysis for the three sites, viz, Guwahati Central, Boko-Palashbari and Goalpara are performed with the help of SHAKE2000 (Ordonez, 2006). Outcrop motions as defined in Section 7.3.2 were used as input motions and are applied at the bedrock of the soil profiles. An effective shear strain ratio as determined by Equation 7.36 is applied for all the analyses. The ground motions obtained from site response analysis were compared to that of the recorded ground motions of the same earthquake in the frequency domain.

7.3.6.1 Boko-Palashbari site

Site response analysis for Boko-Palashbari has been performed with the help of SHAKE2000 using the earthquake events recorded at Nongstoin (Site Class A) as input motion. The 08-Aug-2009, 03-Sep-2009 and 21-Sep-2009 earthquake events
were used as the input motions. Fig. 7.19 to 7.23 depicts the various results of the analyses. Fig. 7.19 shows the amplification plots of the site response analysis. Fig. 7.20 depicts the variation of the PGA from the bedrock to the top surface of the soil column. Fig. 7.21, 7.22 & 7.23 shows the comparison of the Fourier spectrum of the input, free surface analysed and free surface recorded motions for all the three earthquakes considered for the study.

Figure 7.19: Amplification Plot for Boko–Palashbari Site

Figure 7.20: Variation of Peak Acceleration at Layers (Boko-Palashbari Site)
Figure 7.21: Comparison of FAS of Input, Recorded and Ground Motion Obtained from Analysis for the 11-Aug-09 Earthquake (Boko-Palashbari Site)

Figure 7.22: Comparison of FAS of Input, Recorded and Ground Motion Obtained from Analysis for the 03-Sep-09 Earthquake (Boko-Palashbari Site).
Figure 7.23: Comparison of FAS of Input, Recorded and Ground Motion Obtained from Analysis for the 21-Sep-09 Earthquake (Boko-Palashbari Site)

7.3.6.2 Guwahati Central

Site response analysis for Guwahati Central site has been performed with the help of SHAKE2000 using the earthquake events recorded at Nongstoin (Site Class A) as input motion. The 08-Aug-2009, 03-Sep-2009 and 21-Sep-2009 earthquake events were used as the input motions. Fig. 7.24 shows the amplification plots of the site response analysis. Fig. 7.25 depicts the variation of the PGA from the bedrock to the top surface of the soil column. Fig. 7.26, 7.27 and 7.28 shows the comparison of the Fourier spectrum of the input, free surface analysed and free surface recorded motions.
Figure 7.24: Amplification Plot for Guwahati-Central Site

Figure 7.25: Variation of Peak acceleration at Layers (Guwahati Central Site)
Figure 7.26: Comparison of FAS of Input, Recorded and Ground Motion Obtained from Analysis for the 11-Aug-09 Earthquake (Guwahati Central Site)

Figure 7.27: Comparison of FAS of Input, Recorded and Ground Motion Obtained from Analysis for the 03-Sep-09 Earthquake (Guwahati Central Site)
Figure 7.28: Comparison of FAS of Input, Recorded and Ground Motion
Obtained from Analysis for the 21-Sep-09 Earthquake (Guwahati Central Site)

7.3.6.3 Goalpara

Site response analysis for Boko-Palashbari has been performed with the help of SHAKE2000 using the earthquake events recorded at Nongstoin (Site Class A) as input motion. The 08-Aug-2009, 03-Sep-2009, 21-Sep-2009 and two earthquake events that occurred on 29-Oct-2009 were used as the input motions. Fig. 7.29 shows the amplification plots of the site response analysis. Fig. 7.30 depicts the variation of the PGA from the bedrock to the top surface of the soil column. Fig. 7.31, 7.32 & 7.33 shows the comparison of the Fourier spectrum of the input; free surface analysed and free surface recorded motions.
Figure 7.29: Amplification Plot for Goalpara Site

Figure 7.30: Variation of Peak Acceleration at Layers (Goalpara Site)
Figure 7.31: Comparison of FAS of Input, Recorded and Ground Motion Obtained from Analysis for the 11-Aug-09 Earthquake (Goalpara Site)

Figure 7.32: Comparison of FAS of Input, Recorded and Ground Motion Obtained from Analysis for the 03-Sep-09 Earthquake (Goalpara Site)
Figure 7.33: Comparison of FAS of Input, Recorded and Ground Motion Obtained from Analysis for the 21-Sep-09 Earthquake (Goalpara Site)

Figure 7.34: 5 % Damped Spectral Acceleration Plots for 11-Aug-2009 Earthquake for Goalpara, Boko, Palashbari and Guwahati-Central Sites
Figure 7.35: 5% Damped Spectral Acceleration Plots for 03-Sep-2009 Earthquake for Goalpara, Boko, Palashbari and Guwahati-Central Sites

Figure 7.36: 5% Damped Spectral Acceleration Plots for 21-Sep-2009 Earthquake for Goalpara, Boko, Palashbari and Guwahati-Central Sites
Figure 7.37: 5% Damped Spectral Acceleration Plots for 10-Oct-2009 (4.2) Earthquake for Goalpara Site

Figure 7.38: 5% Damped Spectral Acceleration Plots for 10-Oct-2009 (5.2) Earthquake for Goalpara Site
7.3.6.4 Guwahati Airport

1-DELSRA is performed for the 10 (ten) soil profiles obtained in Guwahati Airport area. Table 7.11 provides the PGA amplification for the area.

Table 7.11: PGA Amplification in Guwahati Airport from 1-DELSRA

<table>
<thead>
<tr>
<th>Earthquake Event</th>
<th>Scaled to</th>
<th>BH #1</th>
<th>BH #2</th>
<th>BH #3</th>
<th>BH #4</th>
<th>BH #5</th>
<th>BH #6</th>
<th>BH #7</th>
<th>BH #8</th>
<th>BH #9</th>
<th>BH #10</th>
</tr>
</thead>
<tbody>
<tr>
<td>08-Aug-09</td>
<td>0.18 g</td>
<td>1.48</td>
<td>1.70</td>
<td>1.70</td>
<td>1.14</td>
<td>1.48</td>
<td>0.92</td>
<td>1.53</td>
<td>1.64</td>
<td>1.70</td>
<td>2.44</td>
</tr>
<tr>
<td></td>
<td>0.36 g</td>
<td>1.06</td>
<td>1.29</td>
<td>1.16</td>
<td>0.66</td>
<td>1.02</td>
<td>0.50</td>
<td>1.09</td>
<td>1.32</td>
<td>1.20</td>
<td>1.44</td>
</tr>
<tr>
<td>03-Sep-09</td>
<td>0.18 g</td>
<td>1.38</td>
<td>1.49</td>
<td>1.57</td>
<td>1.05</td>
<td>1.41</td>
<td>0.89</td>
<td>1.38</td>
<td>1.43</td>
<td>1.58</td>
<td>1.90</td>
</tr>
<tr>
<td></td>
<td>0.36 g</td>
<td>1.02</td>
<td>1.11</td>
<td>1.07</td>
<td>0.70</td>
<td>0.97</td>
<td>0.62</td>
<td>1.09</td>
<td>1.15</td>
<td>1.01</td>
<td>1.23</td>
</tr>
<tr>
<td>21-Sep-09</td>
<td>0.18 g</td>
<td>1.57</td>
<td>1.54</td>
<td>1.55</td>
<td>1.09</td>
<td>1.49</td>
<td>0.96</td>
<td>1.55</td>
<td>1.52</td>
<td>1.58</td>
<td>2.47</td>
</tr>
<tr>
<td></td>
<td>0.36 g</td>
<td>0.95</td>
<td>1.03</td>
<td>0.91</td>
<td>0.66</td>
<td>0.92</td>
<td>0.51</td>
<td>0.91</td>
<td>0.99</td>
<td>0.92</td>
<td>1.30</td>
</tr>
<tr>
<td>29-Oct-09(4.2)</td>
<td>0.18 g</td>
<td>1.76</td>
<td>2.26</td>
<td>2.00</td>
<td>1.23</td>
<td>1.77</td>
<td>1.10</td>
<td>2.12</td>
<td>2.53</td>
<td>1.96</td>
<td>2.29</td>
</tr>
<tr>
<td></td>
<td>0.36 g</td>
<td>1.16</td>
<td>2.00</td>
<td>1.93</td>
<td>0.76</td>
<td>1.15</td>
<td>0.66</td>
<td>1.92</td>
<td>1.88</td>
<td>1.49</td>
<td>1.36</td>
</tr>
<tr>
<td>29-Oct-09(5.2)</td>
<td>0.18 g</td>
<td>1.60</td>
<td>1.89</td>
<td>1.80</td>
<td>1.16</td>
<td>1.53</td>
<td>0.90</td>
<td>1.72</td>
<td>1.84</td>
<td>2.00</td>
<td>2.40</td>
</tr>
<tr>
<td></td>
<td>0.36 g</td>
<td>1.08</td>
<td>1.63</td>
<td>1.28</td>
<td>0.68</td>
<td>1.10</td>
<td>0.53</td>
<td>1.33</td>
<td>1.62</td>
<td>1.29</td>
<td>1.28</td>
</tr>
</tbody>
</table>

7.4 OBSERVATIONS & DISCUSSIONS

The comparison of Fourier Amplitude Spectrum (FAS), of the obtained surface motions (analysed) as obtained from 1-DELSRA, with that of the recorded motions at the same stations can be seen from figure 7.21, 7.22, 7.23, 7.26, 7.27, 7.28, 7.31, 7.32 and 7.33. It is observed that the FAS are comparable and that there is a reasonable agreement between the recorded and analysed FAS. Thus it can be deduced that the soil model is acceptable and can be justifiably applied for the western Guwahati region. However, at high frequencies, a certain amount of scattering is observed; this indicates the inability of the algorithm to provide accurate
site response information in the higher frequency domain as reported in Nath et. al. 2008. It is also observed that there is significant site amplification. The Soil profile of Guwahati Central had bedrock at a depth of about 68.75 m, for Boko-Palashbari the bedrock was established at a depth of 200.25 m and for that of Goalpara it was at a depth of 100.50 m. It is observed that the maximum acceleration of the free field ground increased for Boko-Palashbari from 0.012g to 0.046g for the 08-Aug-2009; 0.006g to 0.024g for the 03-Sep-2009 and 0.019g to 0.085g for the 21-Sep-2009 earthquakes; for Goalpara the amplification was 0.015g to 0.076g for the 08-Aug-2009; 0.009g to 0.045g for the 03-Sep-2009; 0.029g to 0.103g for the 21-Sep-2009; 0.006g to 0.033g for the 29-Oct-2009(4.2) and 0.010g to 0.053g for the 29-Oct-2009(5.2) earthquakes respectively. The increase in the maximum acceleration indicated amplification of earthquake motions. The increase in maximum acceleration can be visually interpreted for the three cases from Fig. 7.20, 7.25 and 7.30 respectively.

The site amplifications for the entire earthquake considered for analysis showed that for Boko-Palashbari the site amplification was maximum in the frequency range of 2.00 – 5.00 Hertz while that for Guwahati Central and Goalpara it was within the ranges of 1.00 – 3.00 Hertz and 1.00 - 5.00 Hertz respectively. The results indicate that the amplification is in a low-frequency range thus indicating the possibility of affecting a certain range of buildings. Additionally, amplification of ground motion can also be inspected from the Fig. 7.19, 7.24 and 7.29. Table 7.6 provides the peak amplification obtained from 1-DELSRA study of the region.

The site response at Guwahati Central, which is located near to Cotton College and having a sediment thickness of about 68.75 m provides an amplification factor of
6.78 from the geotechnical analysis (SHAKE analysis). Similar, amplification factor of 5.6 is reported for this site by Nath et. al. (2008). Peak amplification of the sites can be obtained from Table 7.12

Table 7.12: Peak Amplification from 1-DELSRA

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Site</th>
<th>Amplification Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Peak amplification ratio (FAS\textsubscript{surface} / FAS\textsubscript{input})</td>
</tr>
<tr>
<td>1</td>
<td>Boko-Palashbari</td>
<td>8.06</td>
</tr>
<tr>
<td>2</td>
<td>Guwahati-Central</td>
<td>6.78</td>
</tr>
<tr>
<td>3</td>
<td>Goalpara</td>
<td>8.24</td>
</tr>
<tr>
<td>4</td>
<td>Guwahati Airport</td>
<td>15.33</td>
</tr>
</tbody>
</table>

Spectral acceleration plot as shown in Fig. 7.34, 7.35, 7.36, 7.37 and 7.38 shows that there is a substantial change in zero period acceleration values from the range of 0.04 – 0.15g to 0.14 – 0.50g indicating PGA amplification in the predominant period range of 0.20 – 0.25 sec. This implies that there is an amplification of PGA in the order 3.33 – 3.50 for Boko-Palashbari, Guwahati-Central and Goalpara sites. In the Guwahati Airport area, the peak PGA amplification is obtained to be 2.53.

7.5 SUMMARY

In this chapter introduction to 1-dimensional equivalent linear analysis, which is performed for three idealised soil profiles viz., Boko-Palashbari and Goalpara (which falls in the Western Guwahati Region) and at Central Guwahati is presented. Additionally, site response analysis is also carried out for Guwahati Airport area,
adopting the soil modelling methodology applied for Boko-Palashbari, Guwahati-Central and Goalpara. It is seen from the results that significant amplification of the ground motions is present. From the results of 1-DELSRA, it is seen that the maximum site amplification ratio ranged from 8.06 to 15.33 in the Western Guwahati Region. Signification amplification of the PGA is also observed in the range of 2.53 to 5.58. Site response as obtained for the Guwahati Central region is found to be around 6.78 which are in good agreement with the value of 5.8 as obtained by Nath et. al. (2008). It can be said that 1-DELSRA is a very useful method for preliminary site response estimation using the available data. However, the reliability of the results can be enhanced with the use of actual site-specific dynamic soil properties, obtained by adopting suitable geotechnical and geophysical testing methods. Further, as can be seen from the comparison of analysed and recorded seismic records (Section 7.3.6), it can also asserted that the seismic hazard of an area, prone to high seismic activity, can be assessed in the absence of adequate number of seismic records, by obtaining simulated / convoluted/ de-convoluted time-history records for any MCE / DBE by the 1-DELSRA method.
REFERENCES:


Motions – A Synthesis of Highway Practice”, Transportation Research Board, Washington, D.C.


