NONLINEAR SEISMIC GROUND RESPONSE BY CONSIDERING PORE PRESSURES AND DRAINAGE

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With Specialization in Soil Dynamics)

By

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JUNE, 2004

CANDIDATE'S DECLARATION

I hereby declare that the work, which is being presented in this dissertation entitled **"NONLINEAR SEISMIC GROUND RESPONSE BY CONSIDERING PORE PRESSURES AND DRAINAGE"** in partial fulfilment of the requirements for the award of the degree of **Master of Technology** in **Earthquake Engineering** with specialization in **Soil Dynamics**, submitted in the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, is an authentic record of my own work carried out during the period from July, 2003 to June, 2004 under the guidance of **Dr. V. H. Joshi**, Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Date: 30 June, 2004

Place: Roorkee

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CERTIFICATE

This is to certify that the above declaration made by the candidate is true to the best of my knowledge.

Date: 30 June, 2004

30/06/04

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ACKNOWLEDGEMENTS

I have great pleasure in expressing my deepest sense of gratitude to my supervisor **Dr. V. H. Joshi**, Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, India, for his invaluable guidance, utmost help and constant encouragement throughout the course of this work.

I also take this opportunity to express my gratitude towards **Dr. D. Kashyap**, Department of Civil Engineering, Indian Institute of Technology, Roorkee, for his valuable guidance and solving my difficulties from time to time. I am also thankful to my friends from the Department of Earthquake Engineering and Civil Engineering for their valuable help extended during my dissertation work.

Last but not the least, I find myself fortunate enough to express my immense love and gratitude towards my parents and didi who have always been a source of constant inspiration and strength to me.

Date: 30 June, 2004

Place: Roorkee

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ABSTRACT

One part of the dynamic analysis is determination of spatial/temporal distribution of seismic ground response along the boundaries chosen for the system that include the structure and the surrounding medium. Seismic ground motion records indicate that spatial variations in ground motions can be significant, especially when the structure has a considerable length. Some structure like dam, bridge, nuclear power plant and under ground structures is critical from an overall viewpoint and the provision of relief services after earthquakes. Thus it is very important to determine as precise as possible the spatial/ temporal distribution of the seismic ground response.

The work described here in may be divided into two major parts the determination of spatial/temporal distribution of seismic ground response and the parametric studies. In this investigation, the seismic ground analysis is being carried out with due regard to pore pressure generation and dissipation due to drainage during the seismic motion. Computer program has been made for the analysis.

The analysis described here has two part, in first part pore pressure generation has been carried out by making some modification in the Seed and Idriss method and the second part is pore pressure dissipation due to drainage by considering a system of vertical drains. Temporal distribution of seismic ground response is calculated for the above two cases and the comparisons have been made. Limited parametric studies have been carried out. Expected behaviour has been observed from the results obtained, which shows the credibility of the analysis.

LIST OF SYMBOLS

C_r	--	Correction factor
D_r	--	Relative density of the soil
D	--	Downward propagation of shear wave
$Db_i(t)$	--	Component response at the bottom of i_{th} layer due to downward propagating wave at time t
$Dt_i(t)$	--	Component response at the top of i_{th} layer due to downward propagating wave at time t
g	--	Coefficient of gravitational force
G	--	Shear Modulus
G_i	--	Shear Modulus for i_{th} layer
H	--	Thickness of surface layer
H_i	--	Thickness of i_{th} sub-surface layer
H_{eq}	--	Equivalent thickness of surface layer
K_i	--	Impedence ratio of i_{th} layer
N_{eq}	--	Number of niform stress cycles equivalent to earthquake loading.
N_l	--	Number of cycles to cause liquefaction
r_u	--	Pore pressure ratio
r_d	--	Stress reduction coefficient at depth, h
R_{di}	--	Coefficient of downward reflection at i_{th} interface
R_{ui}	--	Coefficient of upward reflection at i_{th} interface
t	--	Time station
T	--	period of component vibration of shear wave
Td_i	--	Coefficient of downward transmission
Tg	--	Predominant period of surface layer
Tg_{eq}	--	Equivalent predominant period of surface layer
Tg_{equi}	--	Period of vibration of equivalent layer system
Tu_i	--	Coefficient of upward transmission
u	--	Excess pore water pressure

LIST OF FIGURES

FIGURE NO. **DESCRIPTION**

CONTENTS

CHAPTER-1

INTRODUCTION

1.1 PREAMBLE

For any earthquake resistance design of structures, design seismic parameters are essential. For this purpose computation of seismic ground response is required. The soil exhibits nonlinear behaviour, which has to be accounted for in such analysis. Besides, the liquefaction of soil causes changes in pore water pressure resulting into loss of shear strength for fine grained cohesionless soils like fine sands, silts and silty sands. This also affects material properties of the layered media. As such, seismic response analysis of layered media is a very important complicated and challenging problem.

The nonlinearity of the soil is best represented by hyperbolic stress-strain curve as per the state of the art. The slope of the stress- strain curve, G_{γ}, at any shear strain, γ, and the corresponding shear wave velocity, V_s (given by $\sqrt{G_{\gamma}/\rho}$), may be obtained by using the hyperbolic stress-strain curve at any time t, and for layer under consideration.

The stress-strain behaviour of soils under dynamic loads may be represented by hysteresis loop. However for computing seismic ground response, the radiational damping is more important than material damping. Hence, material damping may be neglected, i.e., the hysteresis loop may be replaced by a mean non-linear stress strain curve, to represent elastic nonlinear stress-strain behaviour of the material. This has been assumed in the proposed analysis.

It is possible to represent any earthquake acclerogram by any equivalent sinusoidal loading pattern as proposed by Seed (1969). From the pore pressure-time relationships obtained experimentally for liquefying soils, an appropriate relationship may be developed for predicting pore pressure as a function of time for a given sinusoidal loading under consideration. This is useful in predicting pore pressures as a function of time. This pore pressures has to be accounted for in the computation of the effective normal stress and the corresponding shear modulus G, from the present day state of art. This will be accounted for in the proposed nonlinear response analysis.

1.2 OBJECTIVE AND SCOPE OF PROPOSED INVESTIGATION

The above procedure does not account for the decrease in pore pressures due to drainage under field conditions. This results into overestimation of damage due to liquefaction. However, it is possible to account for the reduction in the pore pressures due to drainage by using the principles of flow of water through soils. Incorporating this in above analysis improves prediction of pore pressures at any time during the analysis and hence improves the predicted seismic ground response. This can also be used to examine the efficacy of a given outlay of vertical drains system in restricting the generated pore pressures to a pre determined design value.

It is proposed to develop a suitable computer program for this investigation incorporating above cited improvements. In addition to the development of computer program, it is aimed to compare the results of nonlinear analysis with and without consideration of generation of pore water pressure due to seismic ground vibvrations and the associated drainage. Since the development of computer program is a difficult and complicated task for this problem, limited parametric studies will be carried out for highlighting the advantages of proposed analysis.

1.3 ORGANIZATION OF DISSERTATION

The present dissertation is divided into five chapters. Chapter 2 describes the basic concept of seismic wave propagation method and response of layered media. Attempt is also been made to present the review of research on the topic in this chapter. The third chapter discusses proposed method of analysis for computation of seismic ground response considering generation of pore pressures due to earthquake and drainage of generated pore pressures due to provision of vertical drains. Results followed by discussion are discussed in chapter 4. Chapter 5 concludes the dissertation and makes suggestions for the future research.

CHAPTER-2

LITERATURE REVIEW

2.1 PROPAGATION OF SEISMIC WAVES

The continuous nature of geological formation causes soil dynamics and geotechnical earthquake engineering to diverge from their structural counterparts. While most structures can readily be idealized as assemblage of discrete masses with discrete sources of stiffness, geological materials cannot. They must be treated as continua, and their response to seismic disturbances should be described in the context of seismic wave propagation.

Basic concepts of wave propagation are presented in this chapter. An earthquake generates two types of body waves. Primary waves or P- waves have particle movements in the direction of propagation of wave. Secondary wave or S-waves have particle motions transverse to direction of propagation. P-wave attenuate very fast in a short travel length from source as they spend more energy in propagating. Shear waves travel relatively a longer distance with lesser amplitude attenuation. So, for most sites where seismic ground motions are strong enough to cause structural damage, potential cause to damage is mostly due to shear waves. Body waves incident at the interface of two layers generate surface waves, Raleigh waves being the most important of them and are also know to cause significant damage to engineering structures.

Study of S- waves is very important. These are the waves, which cause much damage to structures. When incidence at the interface of layered media and especially at ground level, these waves generate surface waves. R-waves are the most important waves in this category. When shear wave are incident at interface of two media, it is partially reflected and partially transmitted. When seismic wave propagate from the source, it originate from a considerable depth (many kilometers) below the ground level. In this process, it passes through base rock layer and progressively encounters soil near the ground level. An incidence wave in transmitted from a denser medium to rare medium is more close to the normal to the interface of incidence. When such a wave arrives at the ground level, it propagates nearly in the vertical direction making very small angle with the vertical. Hence, vertical propagation of shear wave is convenient and useful assumption in computation of seismic ground response.

2.2 PROPAGATION OF PLANE WAVES

Shear waves generated due to seismic disturbance move in outward direction from source in form of spherical wave front. In general, waves will not approach interfaces at 90o angles. Orientation of an inclined body wave can strongly influence manner in which energy is reflected and transmitted across as interface. Fermat's principle defines the propagation time of a seismic pulse between two arbitrary points A and B as the minimum travel time along any continuous path connecting A and B. The path producing the minimum travel time is called a ray path, and the vector "ray" represents its direction. Seismic waves traveling larger distance from source have flatter wave front and smaller curvature. A wave front is a surface of equal travel time. Consequently, a ray path is perpendicular to wave front (Fig. 2.1). Snell considered the change of direction of ray paths at interfaces between materials with different wave propagation velocities. Using Fermat's principle, Snell showed that:

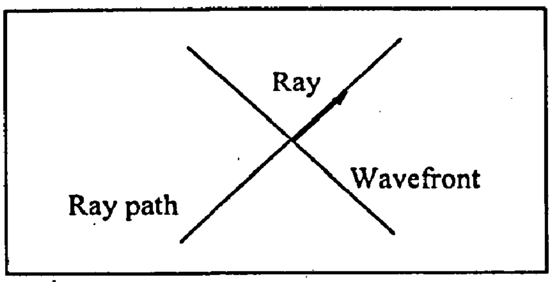
$$\frac{\sin\ i}{v} = \text{cons tan t} \tag{2.2.1}$$

where i is the angle between the ray path and the normal to the interface and v velocity of the wave of intrest. This relationship holds good for both reflected and transmitted waves. It indicates that the transmitted wave will be refracted (except when i = 0) when the wave propagation velocities are different on each side of the interface.

For a sufficiently small site, neglecting the small curvature of wave and assuming plane wave propagation may be justified. Advantage of this assumption is that response of wave in x-z vertical plane is independent of that in its orthogonal y-z vertical plane.

2.3 DIRECTION OF PROPAGATION

Amount of energy reaching ground surface depends upon angle of incidence of waves at different layer interfaces. Zoeppritz (1919) determined the nature of reflected and transmitted waves and distribution of energy between these layers. He concluded that for two layered media



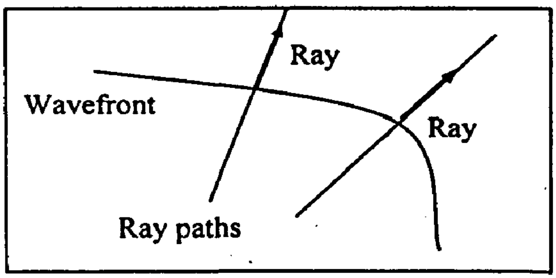


Fig. 2.1 Ray path, ray, and wavefront for plane and curved wave fronts

layered media amplitude of resultant waves is a function of incident angle only. Using Snell's law, equations for incident SV- waves are solved for ratio of resultant wave amplitude in terms of incident angle. From relationship between amplitude ratio and incident angle for $\rho_1 > \rho_2$ and $Vs_1 > Vs_2$ (Fig. 2.2) where ρ_1 and ρ_2 are mass densities and Vs_1 and Vs_2 are shear wave velocities for the first and second layers respectively, it may be concluded that when angle of incidence is zero, i.e., when shear wave falls normally over the interface, P-wave are not generated. Reflected and refracted S-waves are prominently generated. For SV-waves, reflected and refracted amplitude of P-waves are insignificant for small angles of incidence less than 30°. For same range, SV-waves are almost completely refracted. For incident angles larger than 30°, most of incident wave is reflected downwards. So at distant sites, most incident SV-wave energy is directed away from site to reduce amplitudes of seismic ground motion in formation above.

When velocity of reflected or refracted wave is greater than that of incident wave, there will be critical angle of incidence for which the angle of reflection or refraction will be 90°. For angle of incidence greater than critical angle, a disturbance which decays rapidly with distance from interface is created, which does not transmit energy away from the interface. In this case, complex function must be introduced in equations. This doubles the number of equations that must be solved. However, the imaginary amplitude ratios for resultant waves found from these equations and represented by dashed curved in Fig. (2.2) have no physical significance.

2.4 ANGLE OF PROPAGATION OF SEISMIC WAVE WITH VERTICAL

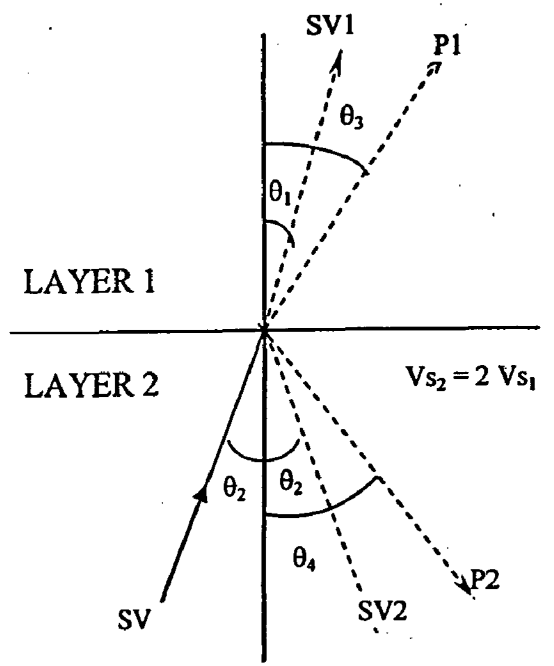
Amount of energy reaching ground surface, depends upon angle of incidence of wave at different interfaces of layers. For epicentral distances close to zero, seismic waves propagate almost vertically. Angle of incidence rapidly increases with increasing epicentral distances. Largest angle of incidence for such a set up (focal depth of 8 m and epicentral distance of 200 km) will be 88°. Angle of incidence of shear waves at ground level depends upon epicentral distance, shear wave velocity in surface layer and focal depth. Nair (1974) suggested that angle of incidence in any layer is proportional to the shear wave velocity of the layer. Chandra (1972) gave the relation between angle of incidence and velocity of shear wave, for nearby site as:

$$\theta \text{ (in degree)} = \left(\frac{\text{Velocity of shear wave in the layer(m/s)}}{73.15} \right) \tag{2.4.1}$$

For distant site θ is equal to zero. For most of engineering problems, seismologists refer angle of incidence at the ground level, as the angle of incidence at base rock level. Snell's law holds good for the incidence of shear wave on interface of two layers, i.e.,

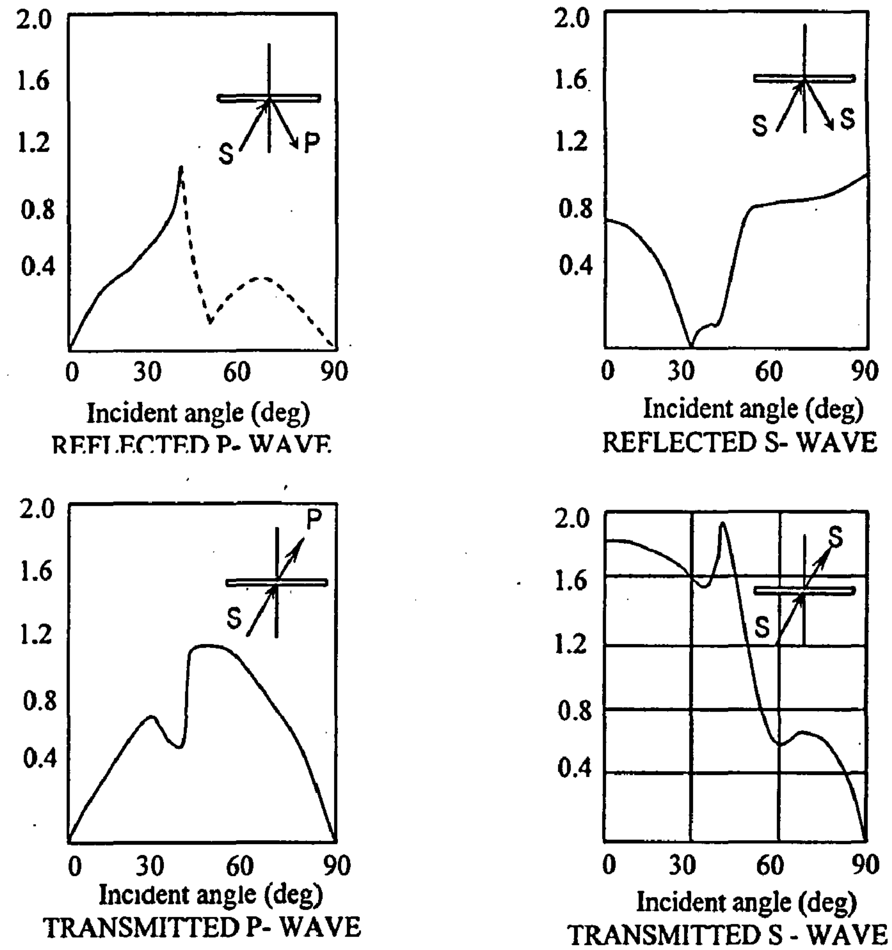
$$\text{Sin(r)} = \text{Sin (i)} \frac{V_{Sr}}{V_{Si}} \tag{2.4.2}$$

where V_{Sr} is shear wave velocity in the layer into which wave is refracted and V_{Si} is shear wave velocity in the layer through which it is incident.



$$\sin\theta_1 = \sin\theta_2 = \sin\theta_3 = \sin\theta_4$$

a) Decomposition of shear wave at interface between two elastic media



b) Relation between amplitude ratio of emitted waves and incident angle

Fig. 2.2 Decomposition of Shear Waves with Incident Angle [McCamy, K., Meyer, R.P., and Smith, T.J (1962)]

2.5 INCLINED PROPAGATION OF SHEAR WAVE

Ground response due to vertically propagating shear wave through layered media has been discussed in earlier articles. However, shear waves may often travel at angle to vertical, particularly in epicentral regions and nearby sites. Considering inclined propagation of shear waves in a layer media is very important for structures with larger horizontal dimensions, because, it includes a relatively large time lag between responses of two points in the same horizontal plane but separated by a large distance in the plane of propagation of wave. In such case, it is necessary to obtain seismic ground motions with due consideration to angle of incidence of shear waves. Following article deals with response and behavior of media with respect to inclined propagation of shear waves.

Waves produced by incident P, SV and SH waves are shown in Fig. 2.2. Since incident P and SV waves involve particle motion transverse to plane of interface; they each produce reflected and refracted P and SV waves. Incident SH waves do not involve particle motion perpendicular to interface; consequently, only SH wave are reflected and refracted. Direction and relative amplitudes of waves produced at interface depend on both direction and amplitude of incident wave. Using Snell's law and conditions of equilibrium and compatibility, these directions and amplitude can be obtained.

Angles of refraction and incidence are uniquely inter-related by the ratio of wave velocity of the material on each side of interface. From Snell's law, wave traveling from higher velocity medium into lower velocity medium will be refracted closer to normal to interface. In other words, waves propagating upward through horizontal layers of successively lower velocity (as common near earth's surface) will be refracted closer and closer to a vertical path (Fig. 2.3). This phenomenon is employed in proposed seismic response analysis.

2.5.1 Propagation of Shear Wave at an angle with the Vertical

In previous article (angle of incidence), the method for obtaining the angle of incidence and its limiting values has been discussed. It is clear from that discussion that the angle of incidence, θ, in surface layer is indeed very small such that $\cos\theta \approx 1$ and $\sin\theta \approx \theta \approx \tan\theta$. In such a situation, the estimation of horizontal seismic response may be obtained

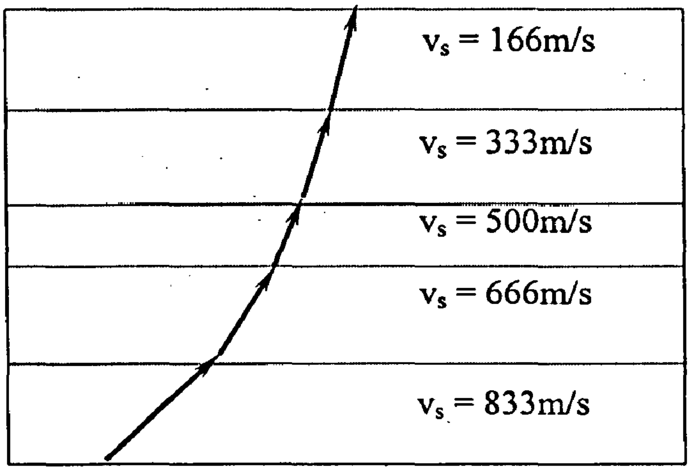


Figure 2.3 Refraction of SH-wave ray path through series of successively softer (lower v_s) layers. Note that orientation of ray path becomes closer to vertical as ground surface is approached. Reflected rays are not shown.

by method for vertical propagation of shear waves explained earlier. However, in view of θ being not equal to zero, there will be a vertical component of response generated and obtained as product of horizontal response and θ in radians.

When shear wave front AB propagating at an angle, θ, with vertical is incident (Fig. 2.4) on interface AB′ at A, the response due to this wave along the wave front AB is same at all points. Response at B will travel with a velocity equal to Vs_2 in lower layer to reach the interface at B′ after a time interval equal to Δt; which is given by:

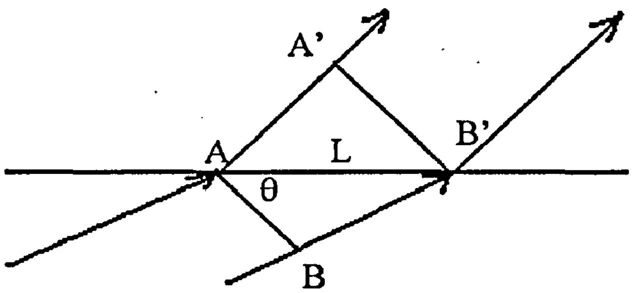


Fig. 2.4 Inclined Propagation of Shear Wave

$$\Delta t = \frac{BB'}{Vs_2} = \frac{LSin\theta}{Vs_2} \tag{2.4.1}$$

In other words response at B′ at time station, t, is the same as the response at time station, $(t - \Delta t)$ at B, which in turn is same as response at A at time $(t - \Delta t)$. Using this methodology, it is possible to obtain spatial/temporal variation of ground response of a layered system for a known seismic response history at any point within the system.

From above discussion, it may be summarized that when site under consideration is at a large distance from focus of the earthquake, assumption of propagation of plane wave is reasonable, particularly when dimensions of the site are small. Assumption of plane wave also simplifies the analysis. Ground motion due to propagation in vertical X-Z plane has no influence on that in orthogonal Y-Z plane and vice versa. Hence, ground motion in these planes may be computed independently, which greatly simplifies analysis. Since formation close to ground level are generally weaker than those deeper below, multiple reflection and refraction of waves through many layers of Earth's crust make seismic waves propagate nearly vertically. For such a reasonable assumption, vertical response is product of horizontal response and $\mathrm{Sin}\theta$, θ being angle of propagation of shear wave.

Though earthquake generates primary and secondary waves, P-waves attenuate fast as they dissipate a lot of energy in propagation. A substantial portion of their energy gets converted into S-wave or R-wave. Comparatively much less energy is spent in S-wave propagation. Hence, most energy of S-waves tends to remain in the same form.

Part of energy of incident body waves at interfaces and ground level gets converted in to surface waves, which propagates along interfaces. Their strength is largest at ground level and reduces with depth below. R-waves are most important surface waves propagating with velocity, V_R=0.93 V_s, they almost arrive simultaneously at intermediate sites with overlapping durations of dominance in seismogram. It is difficult to segregate contributions of S and R-waves. For distant sites, R-waves may arrive after, S-waves exit from site. In this case segregation of R and S waves is explicit. For such sites, considering entire seismic ground response to consist of S-wave only is reasonable, simplifies computation of ground motions.

2.6 WAVE PROPAGATION IN A SINGLE LAYER

Soil is nonlinear and it has a very low proportional limit. Earthquake induced strains may easily exceed this limit and dynamic properties do not remain constant. Therefore, assuming constant material properties leads to conservative estimation of response and conservative design of structures. For precise determination of ground response, nonlinear analysis should be carried out.

Although many studies have been reported on linear analysis of seismic response, nonlinear analysis started getting attention only recently as it requires huge amount of computational effort in considering time dependent material properties. Recently, many computer programs for nonlinear analysis for seismic response have been reported (give references in this regard). The proposed investigation is one such attempt. A brief review of the state of art for linear and nonlinear analysis is presented here.

2.6.1 Predominant Period and Amplification of Ground Response

The *predominant period,* T_g, provides a useful, although somewhat crude representation of frequency content of ground motion. Ground amplification factor, μ, is ratio of maximum response at any point to the maximum response at base of a layered system. Factor μ for ground level is generally of great interest. In alluvial area, velocity of seismic waves is usually slower near ground surface than in formations further below. As a result, the ground vibration will be amplified with dominance vibrations of certain specific periods. This is predominant period, T_g, determined by using profile of layered system and material properties of layers. Greatest amplification factor is expected and greatly influenced by *fundamental frequency* of layered system. Period of vibration corresponding to the fundamental frequency is also called the characteristic site period or predominant period. For single layer (Fig. 2.5) of thickness, H, and velocity of shear wave, V_s, predominant period, T_g, is given by:

$$T_g = \frac{4H}{V_s} \qquad (2.5.1.1)$$

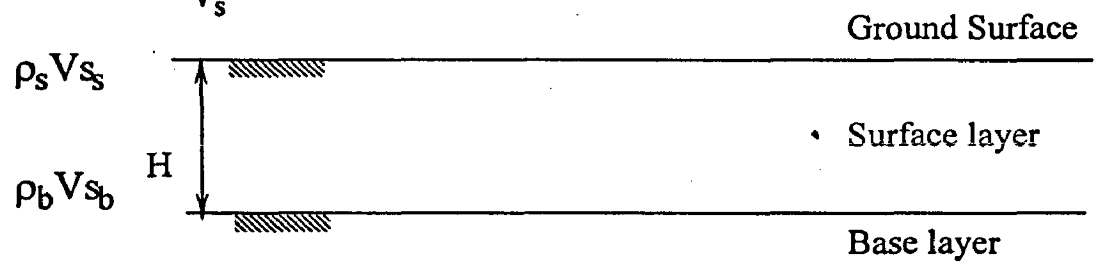


Fig. 2.5 Single Layer System over Base Layer System

For a single layer (Fig2.5), amplification factor, μ, given by empirical formula of Kanai(1951) by combining field measurements with theoretical derivation is by:

$$\mu = 1 + \frac{1}{\sqrt{\left[\frac{0.3}{\sqrt{T_g}}\frac{T}{T_g}\right]^2 + \left[\frac{1+k}{1-k}\left\{1-\left\{\frac{T}{T_g}\right\}^2\right\}\right]^2}} \qquad (2.5.1.2.)$$

where $$k = \frac{\rho_s Vs_s}{\rho_b Vs_b} \text{ is impedance ratio;} \qquad (2.5.1.3)$$

where T_g is predominant period of surface layer, T period of component vibration of seismic wave, γ_s density of surface layer, γ_b density of base layer; V_{Ss} velocity of shear wave in surface layer and V_{Sb} velocity of shear wave in base ground. However, for multi layer system (Fig 2.6), Kanai gave depth of equivalent single layer, H_{eq}, as:

$$H_{eq} = \sum_{i=1}^{n} H_i \qquad (2.5.1.4)$$

where H_i , thickness of i^{th} layer. The use of depth of layer as the weighting function is reasonable, because, a thin layer has less influence on computed layer response compared to thick and stiff layer in a given layered system of ground. By using depth of each layer as a weighting function, he proposed expression for equivalent shear wave velocity, Vs_{eq}, and equivalent unit weight, ρ_{eq}, for an equivalent single layer system given as:

$$Vs_{eq} = \frac{\left[\sum_{i=1}^{n} Vs_{si} H_i\right]}{\sum_{i=1}^{n} H_i} \qquad (2.5.1.5)$$

$$\rho_{eq} = \frac{\left[\sum_{i=1}^{n} \rho_1 H_1\right]}{\sum_{i=1}^{n} H_1} \qquad (2.5.1.6)$$

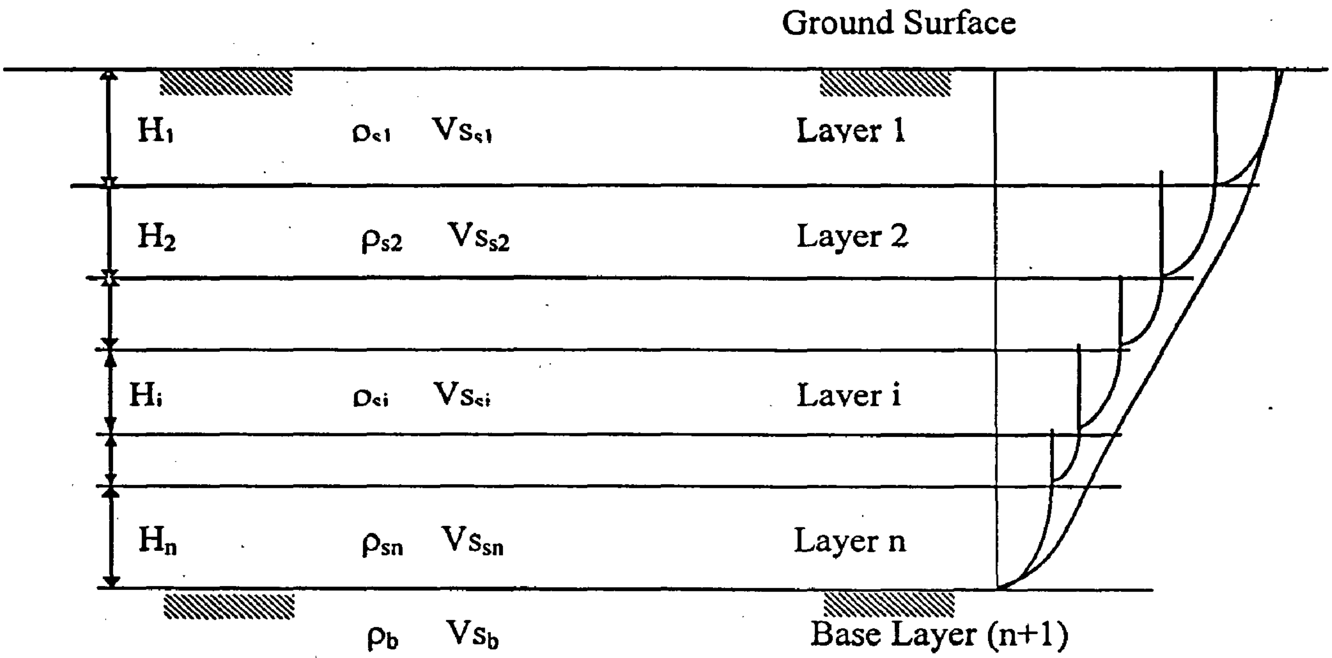


Fig. 2.6 Multi Layer Systems over Base Layer System

Kanai in his expression does not propose to obtain T_g given by:

$$T_g = 4\frac{H_{eq}}{Vs_{eq}} \tag{2.2.1.7}$$

This computation is hypothetical, because, there is no layer existing with property of V_{Seq}, which is a purely an imaginary mathematical quantity. On the other hand, T_{gi} given by ($4H_i/V_{Si}$) for the i^{th} layer is a real quantity because, H_i and V_{Si} exist and hence, it is a more realistic entity. It also represents period of fundamental vibration for i^{th} layer only. Instead, he proposed to compute T_g given by:

$$T_g = \sum_{i=1}^{n} T_{gi} = \sum \frac{4H_i}{Vs_i} \tag{2.5.1.7}$$

By application of the theory of multi-reflection of waves in layered system, it is possible to obtain the predominant period of an alluvial layer by calculation. When the surface layer is comprised of single layer, the predominant period is given by Eq.(2.1.5.8) According to this relationship, predominant period is longer for the thicker layer. This is in agreement with observed facts. But in reality, ground surface seldom consists of single layer. In such a case a detailed calculation as described above must be carried out to obtain ground vibration characteristics. However, when there is not much difference between properties of various layers, the longest predominant period may be determined by using Eq. (2.5.1.8).

It has been reported (Okamoto 1973) that for Mexico city, the fundamental period of vibration of layer system at point, for two different base layer stratum considered, the T_g obtained by calculations by Eq.(2.5.1.1) and Eq.(2.5.1.8) was 2.12 sec and 2.63 sec respectively. On the other hand, predominant periods obtained by actual observation were 2.63 sec. at one site and 2.15 sec at other site, which were roughly in agreement with calculated values. This indicates that if evaluation of effective velocity of seismic waves is correct, the through multi-reflection theory calculations should give reliable results. However, as it is difficult to obtain undisturbed samples of soil from great depths this method of calculation has not yet reached the stage of full utility.

The period of component vibration as seismic wave is best obtained by studying seismic ground vibration record at or near firm ground. It depends upon material properties and the depth of firm ground level as well as magnitude, focal depth and epicentral distance of earthquake. If focal distance is large, most of the high frequency vibrations will be absent as they tend to die down very rapidly within the epicentral region. If the magnitude of earthquake is large, it has enough energy to excite deeper layer below the ground level, which results into predominance of long period vibrations. Therefore, for same site, the period of component seismic wave vibration may vary depending upon the magnitude epicentral distance, and focal depth of earthquake. All these factors need to be considered in estimating the predominant period of shear wave motion to the top of base layer.

2.6.2 Spatial/Temporal Distribution of Response

Methods of Analysis

Nair (1974), Kobayashi (1972) and Joshi (1980) gave methods for determination of component response in multi layer system if response at base rock level is known.

Nair's Method (1974)

Nair assumed that the total response at the base rock level is equal to component response due to the upward propagating wave in base rock layer and in layer immediately above base rock and that there is no downward reflection, downward transmission and upward reflection at the base rock interface.

Kobayashi's Method (1972)

Kobayashi made the same assumption but considered upward reflection and downward transmission at the base rock level, which makes it better than Nair's method. However, the assumptions made in both of these methods violate principle of shear wave propagating when applied to the interface at base rock level. These two methods, result in over-estimation of response of surface layer, which may be of the order of about to 50 percent in some cases, which is not desirable.

Joshi's Method (1980)

Joshi proposed a method in which he considered the reflection and refraction in the upward as well as in the downward directions at the base rock level in accordance with the principles of shear wave propagation. He assumed the base rock formation extend to infinite depth below the base rock level, which is reasonable for all practical purposes. As such, a downward propagating wave has no chance of being reflected upwards within the duration of the earthquake under consideration.

Okamoto (1973) observed from field records that the seismic ground response decreases with increasing depth below the ground level. This is clear from the recorded ground motions at various elevations along a deep shaft at Kinugawa Power Station in Japan and excited by Niigata earthquake in 1964. He concluded that decrease in the intensity of ground motion with depth below is expected for two reasons. Firstly, the layers at greater depth below usually have larger elastic modulus by virtue of higher confining pressures and overburden pressure. A higher modulus results into smaller response for a given level of force. Secondly, the material at greater depth has to carry a substantial overburden pressure when it vibrates under earthquake-induced forces. These two factor lead to smaller seismic response.

He recommended that these factors should be considered while obtaining seismic ground motion distribution in a layered system, which is done by assigning an appropriate value of shear modulus and other material properties for different layers. He also observed that buried structures like tunnels, underground power houses, basements of tall buildings etc experience much smaller seismic ground vibrations compared to those near the ground level.

2.7 RESPONSE OF A LAYERED MEDIUM

One of the most important and most commonly encountered problems in geotechnical earthquake engineering is the evaluation of ground response. Ground response analyses are used to predict ground surface motions for development of design response spectra, to evaluate dynamic stresses and strains for evaluation of liquefaction hazards, and to determine the earthquake-induced forces that can lead to instability of earth and earth-retaining structures.

Under ideal conditions, a complete ground response analysis would model the rupture mechanism at the source of an earthquake, the propagation of stress waves through the earth to the top of bedrock beneath a particular site, and would then determine how the ground surface motion is influenced by the soils that lie above the bedrock. In reality, the mechanism of fault rupture is so complicated and the nature of energy transmission between the source and the site so uncertain that this approach is not practical for common engineering applications. In practice, empirical methods based on the characteristics of recorded earthquakes are used to develop predictive relationships. These predictive relationships are often used in conjunction with a seismic hazard analysis to predict bedrock motion characteristics at the site. The problem of ground response analysis then becomes one of determining the response of the soil deposit to the motion of the bedrock immediately beneath it.

2.7.1 Equation of Motion For Shear Wave Propagation

The following assumptions are made in developing the equation of motion for shear wave propagation:

- Linear elastic medium.

- Layer and interfaces are horizontal, and extend to infinity.

Vertical propagation of plane shear waves

Seismic waves transmitted through ground are approximately elastic waves of which shear waves are the most important from an engineering standpoint. With shear waves the particles within an elastic body move in a direction orthogonal to the direction of advance of the wave itself. Here the direction of advance of the wave is taken as *z* and the direction of displacement of particles is taken as, x. If u is the response at any general point at a distance of ,z, below the ground level and at any time station, t, the shear stress differential, dτ, is obtained as:

$$d\tau = \frac{\partial \tau}{\partial z}.dz = \frac{\partial^2 u}{\partial t^2}.\rho.dz \qquad (2.6.1.1)$$

where τ and $\frac{\partial u}{\partial z}$ are shear stress and shear strain respectively.

$$\tau = G\frac{\partial u}{\partial z} \qquad (2.6.1.2)$$

From these relationships, we have

$$\frac{\partial^2 u}{\partial t^2} - V_s{}^2\frac{\partial^2 u}{\partial z^2} = 0 \qquad (2.6.1.3)$$

where $$V_s{}^2 = \frac{G}{\rho}$$

Equation 2.6.1.3 is differential equation of shear vibrations for propagation in vertical directions. This equation has two solution, U(z, t), and ,D(z, t),. In any layer, there are two waves; one is rising upward and second one going downward. Both these propagation may access simultaneously in the layer.

- Net response, u (z, t), at any time, t and at depth, z, is given by:

$$u(z,t) = U(z,t) + D(z,t) \qquad (2.6.1.4)$$

- If shear wave velocity in the medium is V_s, then U(z, t) & D(z, t) at any depth, z,

and any time, t are give by $U\left(t+\frac{z}{V_s}\right)$ and $D\left(t-\frac{z}{V_s}\right)$ respectively. From this, we get:

$$u(z,t) = U\left(t + \frac{z}{V_s}\right) + D\left(t - \frac{z}{V_s}\right) \quad (2.6.1.5)$$

which is the solution of equation 2.6.1.3

2.7.2 Reflection and Transmission of Wave Motion at the Ground Surface

The wave motion (shear wave) rising up through the ground on reaching the surface is reflected and propagated downward. It is not possible for them to get transmitted into the medium of air above the ground level, because, the velocity of shear wave in the air is zero. The shear modulus for air is also zero for all geotechnical engineering purposes. As such, all the energy associated with the upward traveling shear waves is totally reflected downward. In other words, the component response of upward traveling incident shear wave is equal to that of the downward traveling reflected shear wave at the ground level i.e.,

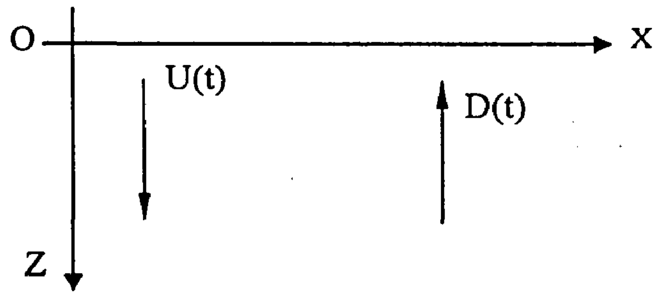


Figure 2.7 Reflection And Transmission At Ground Surface

$$-U(t) + D(t) = 0$$

i.e., $$U(t) = D(t)$$

i.e., $$u = 2U(t) \quad (2.6.2.1)$$

When the incident SV-wave is not exactly vertical, it gives rise to reflected P-waves in addition to reflected SV-waves. However, for small angle of incidence, the reflected P-wave ma be neglected for all practical purposes. When SH-waves are incident, they generate no P-waves.

The explicit knowledge of component responses of upward and downward traveling waves at the ground level is useful in obtaining the component responses of upward and downward traveling waves in all the layer of the system. From this, it is possible to obtain the spatial/temporal variations of seismic ground motions for a known response at the ground level for the entire time period, using principles of wave propagation.

2.7.3 Reflection and Transmission at Interfaces

As shown in previous chapter the layer system of soil medium of different properties, consider the i^{th} interface shown in Fig. (2.8) for the i^{th} layer with impedance, $(\rho_i V_{s_i})$ and $(i+1)^{th}$, layer with impedance, $(\rho_{(i+1)} V s_{(i+1)})$. The response in i^{th} layer at a distance, z, from the interface in terms of the component responses at the bottom of, i^{th} layer at any time, t, is give below :

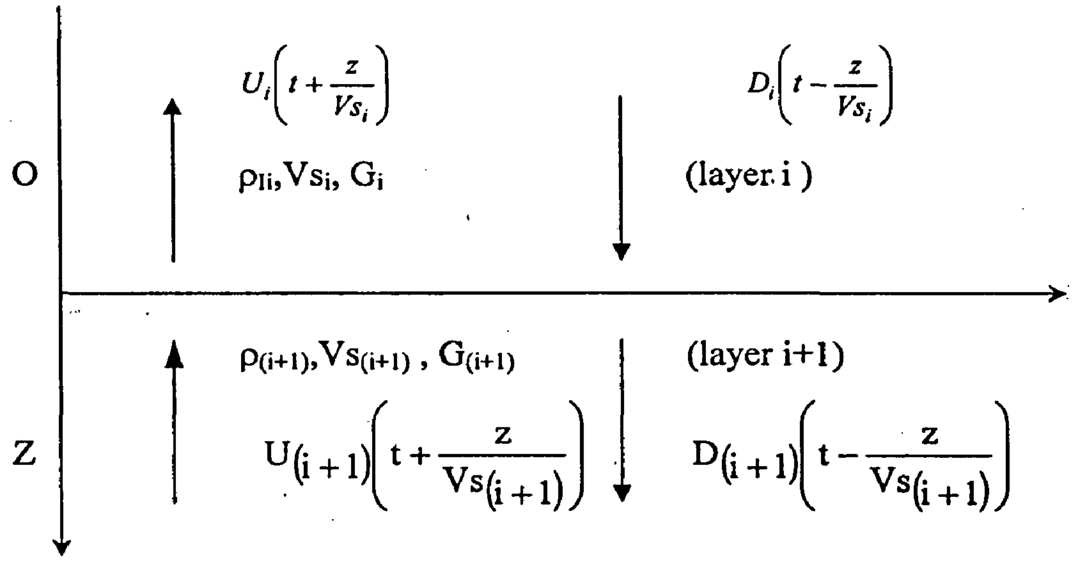


Fig. 2.8 Reflection and Transmission at Interface

$$u_i = Ub_i\left(t + \frac{z}{V_{s_i}}\right) + Db_i\left(t - \frac{z}{V_{s_i}}\right) \qquad (2.6.3.1)$$

Similarly, the response at any point in layer i+1, distant z from the interface may also be expressed in terms of component responses at the top of same layer as :

$$u_{(i+1)} = Ut_{i+1}\left(t + \frac{z}{V_{s\,(i+1)}}\right) + Dt_{(i+1)}\left(t - \frac{z}{V_{s\,(i+1)}}\right)$$

(2.6.3.2)

These two responses should be identical to satisfy compatibility at the interface, i.e.,

$$(u_i)_{z=0} = (u_{(i+1)})_{z=0} \tag{2.6.3.3}$$

$$\left(G_i \frac{\partial u_i}{\partial z}\right)_{z=0} = \left(G_{(i+1)} \frac{\partial u_{(i+1)}}{\partial z}\right)_{z=0} \tag{2.6.3.4}$$

The component response at any time, t, due to upward traveling wave at the top end of (i+1)th layer may be denoted by $ut_{(i+1)}(t)$ and that at any bottom of the ith layer may be denoted by ub_i (t). Similarly, the corresponding component responses due to downward traveling wave at the interface may be denoted by $Dt_{(i+1)}(t)$ and $Db_i(t)$. Therefore,

$$Ub_i(t) + Db_i(t) = Ut_{(i+1)}(t) + Dt_{(i+1)}(t) \tag{2.6.3.5}$$

$$\frac{G_i}{V_{s_i}}[Ub_i(t) - Db_i(t)] = \frac{G_{(i+1)}}{Vs_{(i+1)}}[Ut_{(i+1)}(t) + Dt_{(i+1)}(t)] \tag{2.6.3.6}$$

Integrating above equation, we get :

$$Ub_i(t) - Db_i(t) = \frac{1}{k_i}[Ut_{(i+1)}(t) + Dt_{(i+1)}(t)] \tag{2.6.3.7}$$

where, $$k_i = \frac{G_i V_{s(i+1)}}{G_{(i+1)} V_s} = \frac{\rho_i V_{s_i}}{\rho_{(i+1)} V_{s(i+1)}}$$

Solving equation (2.6.3.5) and (2.6.3.7), simultaneously, we get :

$$Ub_i(t) = Ut_{(i+1)}(t) \,.\, Tu_i + Db_i(t) \,.\, Ru_i \tag{2.6.3.8}$$

$$Dt_{(i+1)}(t) = Ut_{(i+1)}(t) \,.\, Rd_i + Db_i(t) \,.\, Td_i \tag{2.6.3.9}$$

where

$Rd_i = \dfrac{1-k_i}{1+k_i}$ → coefficient of downward reflection

$Tu_i = 1 + Rd_i$ → coefficient of upward transmission

$Ru_i = -Rd_i$ → coefficient of upward reflection

$Td_i = 1 - Rd_i$ → coefficient of downward transmission

From the above discussion it is clear that if we known the component responses incident at any interface, the component responses generate by them in the upward and downward direction can be evaluated using the coefficients of reflection and transmission in upward and downward directions.

Soil properties are not uniform both in vertical and horizontal extents. It consists of different layers of different properties. Dynamic response depends upon material properties. Therefore seismic response computation of layered system is necessary. Using wave propagation theory, it may be possible to compute the upward and downward responses separately and which are called as component responses.

2.7.4 Component Responses of a Multi-Layer System

The expression for component responses Fig. (2.9)are:

$$Ub_i(t) = Ut_{(i+1)}(t) \,.\, Tu_i + Db_i(t) \,.\, Ru_i \tag{2.6.4.1}$$

$$Dt_i(t) = Db_{(i-1)}(t) \,.\, Td_{(i-1)} + Ut_i(t) \,.\, Rd_{(i-1)} \tag{2.6.4.2}$$

$$Ut_i(t) = Ub_i(t-t_i) \tag{2.6.4.3}$$

$$Db_i(t) = Dt_i(t-t_i) \tag{2.6.4.4}$$

where t_i, is time of travel for seismic wave to cover the depth of i^{th} layer. Similar expressions may be obtained for other layers. Component Response for Topmost Layer are given as :

$$Ut_1(t) = Dt_1(t) = X_1(t)/2 \tag{2.6.4.5}$$

where $X_1(t)$ is response of ground level at time t.

Component Response at Base Rock Level

Joshi (1980), assumed that the base rock formation extend to infinite depth below the base rock level, which is reasonable for all practical purposes. As such, a downward propagating wave has no chance of being reflected upward within the total duration of the earthquake under consideration. Denoting the base rock as $(n+1)^{th}$ layer, the total response of the base rock, Xt_n (t), at any time, t, is :

$$Xt_n(t) = Ut_{(n+1)}(t) + Dt_{(n\_1)}(t)$$

i.e.,

$$Dt_{(n+1)}(t) = Xt_n(t) - Ut_{(n+1)}(t). \tag{2.6.4.6}$$

$$Dt_{(n+1)}(t) = Ut_{(n+1)}(t) \,.\, Rd_n + Db_n(t) \,.\, Td_n$$

i.e.,

$$Ut_{(n+1)}(t) \,.\, Rd_n + Db_n(t) \,.\, Td_n = Xt_n(t) - Ut_{(n+1)}(t)$$

i.e., $$Ut_{(n+1)}(t) = \frac{Xt_n(t) - Db_n(t) \cdot Td_n}{1 + Rd_n} \qquad (2.6.4.7)$$

The quantity $Db_n(t)$ does not exit till the downward reflected wave inside the n^{th} layer reaches the base rock level which needs a time interval equal to $2t_n$. For any other time stations, $Db_n(t)$ is also a known quantity. Using this information and equations 2.6.4.1 to 2.6.4.7 the component responses of the level at which response and net response is known may be obtained from the known response. Using these component responses and net response at any other interface below can also be compared.

2.7.5 Computation of Properties of Main-layer and Sublayer

The program computes bulk unit weight and saturated unit weight of each main layer Fig. (2.9) using the input data. The ultimate shear strength of each main layer is also computed by using coulomb equation:

$$\tau_u = c + \bar{\sigma} \tan \Phi \qquad (2.6.5.1)$$

where c, cohesion, $\bar{\sigma}$, effective stress which is effective vertical stress (σ_1), ϕ angle of shearing resistance of the main layer under consideration. Properties of main layer are worked out on the bases of stresses computed at the mid depth of that layer. The effective stress at the mid depth of each main layer is obtained and the lateral stresses σ_2 and σ_3 are computed by assuming at rest earth pressure condition by assuming K_0 given by;

$$K_0 = (1 - \sin\phi) \qquad (2.6.5.2)$$

and octahedral stress or mean effective principle stress $\sigma o'$ is obtained as

$\sigma o' = (\sigma_1 + \sigma_2 + \sigma_3)/3.0$.

Shear modulus at any strain level is obtained as tangent modulus defined as shear stress at that instant divided by shear strain at that instant. Parameters used to define a hyperbolic stress-strain relation are shown in Fig. (2.10). The hyperbolic curve is asymptotic to the horizontal line defined by $\tau = \tau_{max}$, in which τ is the shear stress and τ_{max} is the shear stress at failure. The initial slope of the hyperbolic curve (at the origin) is G_{max} and is equal to the maximum value of G. If the line through the origin with slope G_{max} is extended to intersect the line $\tau = \tau_{max}$, the intersection defines a strain as given below which is used as the reference strain.

$$\gamma_r = \frac{\tau_{max}}{G_{max}} \qquad (2.6.5.3)$$

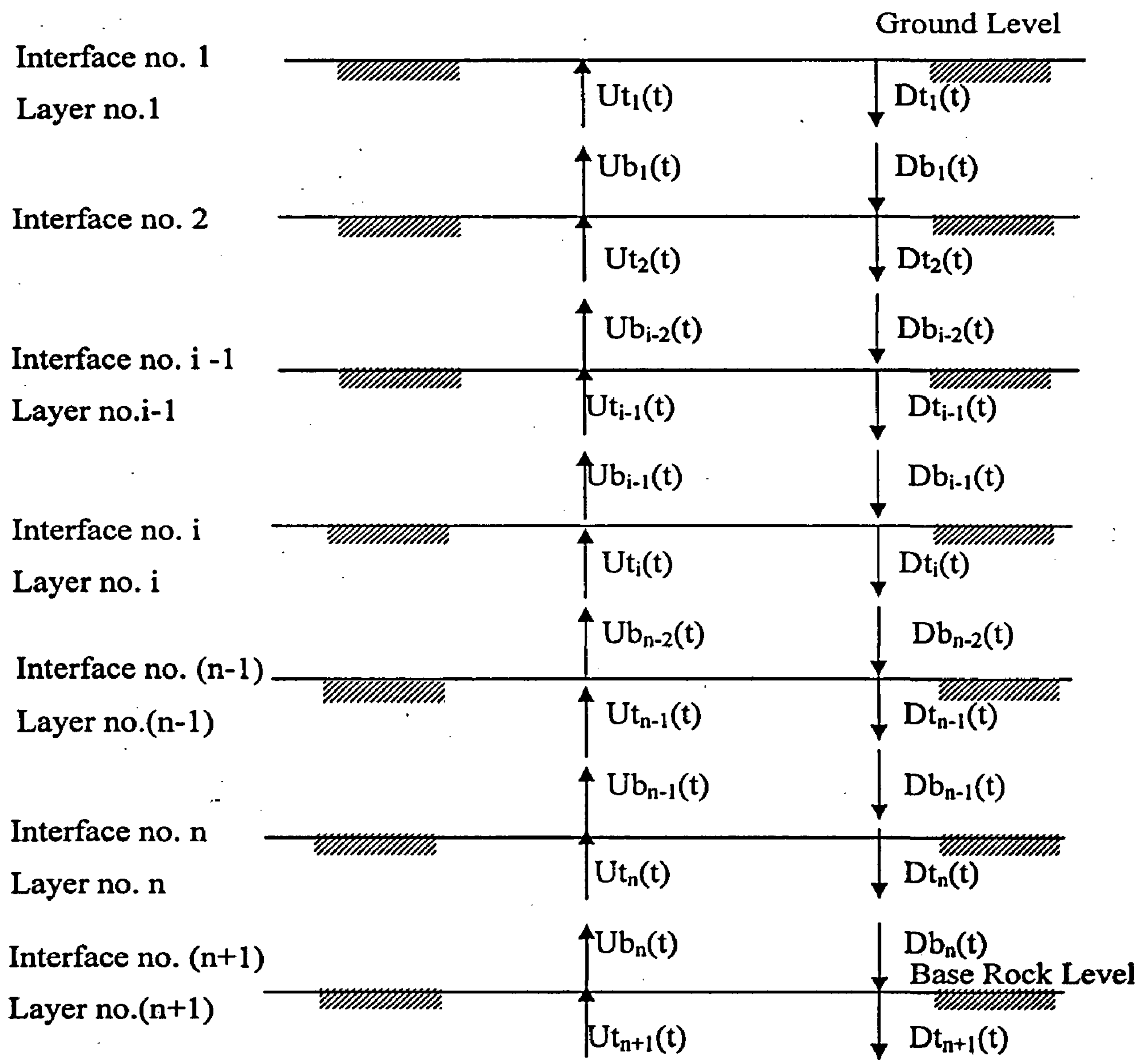


Figure 2.9 Multi Layer System

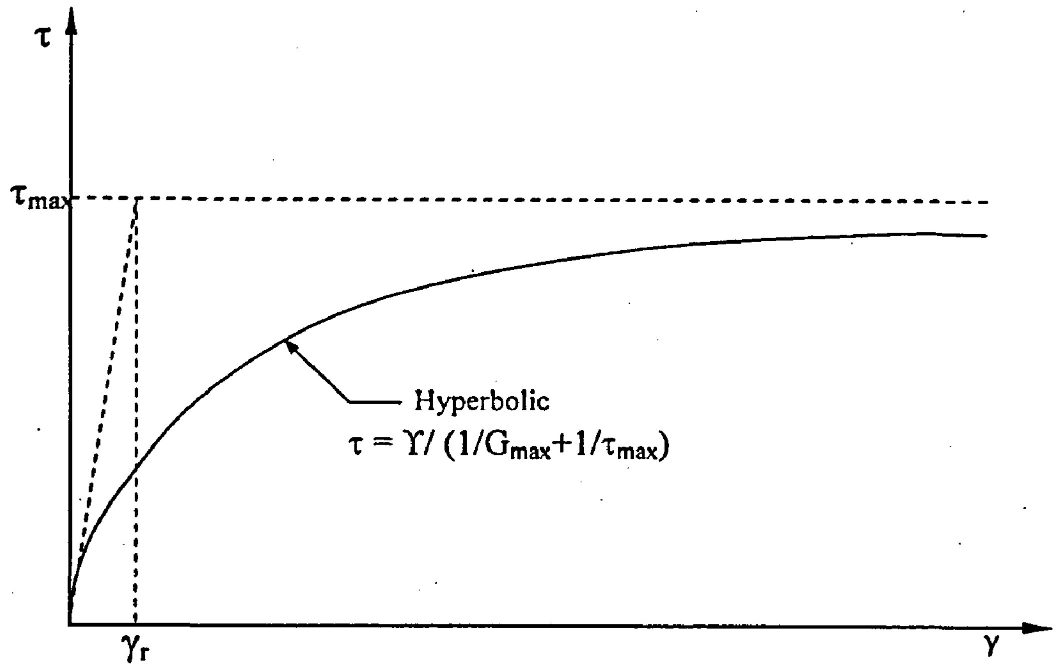


Fig. 2.10 Hyperbolic Stress-Strain Relationship (Hardin and Drenvich, 1972)

Hardin and Black (1972) have shown, (and the study confirms) that for many undisturbed cohesive soils, as well as sands, G_{max} can be calculated from

$$G_{max} = 1230 \frac{(2.973 - e)^2}{(1+e)} (OCR)^k \bar{\sigma}_0^{1/2} \tag{2.6.5.4}$$

where e, void ratio, OCR, overconsolidation ratio, σ_0', mean principal effective stress and both σ_0' and G_{max} are in ponds per square inch. Value of K depends on plasticity index, PI, of the soil and can be obtained by interpolation from the values given in Table 2.6.5.1.

Value of τ_{max} depends on the initial state of stress in the soil and the way in which shear stress is applied. Hardin and Drenvich(1972) have recommended following expression for computation of τ_{max} corresponding to G_{max} for initial at rest conditions.

$$\tau = \left\{ \left[\frac{1+ko}{2} \sigma' \sin\phi' + c' \cos\phi' \right]^2 - \left[\frac{1-ko}{2} \sigma' \right]^2 \right\}^{1/2} \tag{2.6.5.5}$$

in which k_o is coefficient of lateral stress at rest, σ_0' vertical effective stress and c' and ϕ' are the static strength parameters in terms of effective stress. Effective stresses are used in Eq. (2.6.5.5) even though undrained conditions will likely exist for dynamic loading. This is valid for cohesive as well as cohesionless soils.

Table 2.6.5. 1 Value of K

PI	K
0	0
20	0.18
40	0.30
60	0.41
80	0.48
>100	0.50

By substituting c'=0 for cohesive soils equation (2.6.5.5) gets modified to sandy soils and is given by:

$$\tau = \left\{ \left[\frac{1+ko}{2} \sigma' \sin\phi' \right]^2 - \left[\frac{1-ko}{2} \sigma' \right]^2 \right\}^{1/2}$$

$$\tau = \sigma' \left\{ \left[\frac{1+ko}{2} \sin\phi' \right]^2 - \left[\frac{1-ko}{2} \right]^2 \right\}^{1/2} \qquad (2.6.5.6)$$

Equation (2.6.5.6) will be used for the computing G_{max} and τ_{max}.

2.7.6 Nonlinear Analysis

Soil is nonlinear material and it has very low proportionality limit. Therefore, earthquake induced strains may easily exceed proportional limit of such materials. In such cases, dynamic properties do not remain constant. So, assumption of constant material properties always leads, to less precise response and extra safe design of structures. Thus for precise determination of ground response, nonlinear analysis should be carried out.

All the assumptions, criteria and procedures adopted for linear analysis has been used for nonlinear analysis except consideration of strain level independent material property.

2.7.6.1 Studies of Nonlinear Analysis

Kanai (1953) studied the relationship between nature of the surface layer and amplitude of displacements considering the problem of the oscillations of doubly stratified visco-elastic layer excited by seismic waves. He found that amplitude at ground surface in general becomes maximum when the period of exciting wave synchronizes with the fundamental period of first layer. But actually damping in the first layer is not zero, because, damping increases with strain levels. If the first layer is rather thin, the amplitude, influenced by damping, cannot become very large even if the period of seismic excitation synchronizes with that of the first layer.

Okamoto (1973) considered an elasto-plastic surface layer resting over an elastic base layer and took 3 different levels of strain at elastic limit of surface layer. He noticed that when the elastic limit is lower, the vibration amplification is no more prominent and the period of component vibrations having largest amplitude becomes longer.

Procedure for Non Linear Analysis

i. Each layer of the layered system under consideration is divided into adequate number of thin sub-layers,
ii. Assume some initial material properties of each sub-layer (Vs_i,G_i,etc.),
iii. For each time station:
   a. Compute response (in particular displacement) at interfaces of sub-layers considering linear material properties,
   b. Strain induced in each sub-layer is computed,
   c. If the difference of strain induced at current time station and at that previous time station is less than a predefined value of small strain tolerance, then, don't revise the shear modulus and the step (f) may be followed. Otherwise follow step (d) and (e) before going to step (f),
   d. Compute shear modulus of material of sub-layer, corresponding to strain induced in that sub-layer.
   e. Using the modified shear modulus, the velocity of propagation of shear wave in the sub-layer may be computed which may be used for computation of response.

f. Compute the response of the sub-layers at the next time station with revised sub-layer properties if applicable.

g. The above procedure is repeated for the entire duration of the earthquake.

2.8 MISCELLANEOUS

Biot (1956) concluded that presence of water table in the soil mass changes the wave propagation characteristics of the soil medium (Sherman, 1945). The soils above and below the water table tent to behave as of they are separate layers. The upper layer transmits energy through soil structure while the lower layer, which is saturated, transmits energy through both soil and fluid. When shear wave are incident at saturated layer, it is divided into S-waves and P-waves. S-waves propagate through soil structure and P-wave through the fluid. In saturated layer, since fluid has no shearing stiffness, there is no structural, coupling between the elastic structure and the fluid. He proposes a formula for computation of shear wave velocity in saturated soil layer. It may be noted that due to presence of pore fluid, the unit weight of the soil medium changes due to buoyancy. Besides, its inertia forces also changes due to presence of pore fluid.

From the above discussion, it may be summarized that, when the response is known at the ground level, it is possible to obtain the component response of upward and downward propagating shear wave. However, such a determination of these component responses is not possible when response is not known at base rock level. Joshi (1980), proposed a method for obtaining component responses at top of base rock level. Also as compared with other researchers such as Nair and Kobayashi's method, Joshi's method is more realistic and close to actual values. He assumed the base rock formation extended upto infinite depth below so that the reflected wave propagating in the downward direction has no chance of returning back to the base rock level considering within the time duration of the proposed analysis.

Using this proposition Srivastava (1995), proposed method of analysis considering nonlinear stress strain characteristics of the soil. However, he did not consider the presence of ground water table and the initial range of elastic behaviour of soil over range of strain level. Besides the nonlinear property of any soil layer of the layered system considered to be the same for the entire thin sub layer within that soil layer. Jain (2001) tried to improve upon the shortcomings the above proposed method by considering the pore pressures. However, he did not consider the drainage and dissipation of pore water pressures in his analysis. Hence, it is desirable to consider a dissipation of pore water pressures into account while doing the analysis.

From the above discussion, it is clear that there is a need for developing a new method of analysis, which is free from the above sighted shortcomings.

CHAPTER-3

PROPOSED METHOD OF ANALYSIS

3.1 INTRODUCTION

Recent developments in studies of soil liquefaction have made it possible to further extend studies to include consideration of pore water pressure generation and dissipation which takes place during the period of earthquake shaking as well as the period following the earthquake. Accordingly it is the purpose of the present study to present a means of analyzing the development and dissipation of pore water pressures in a horizontally stratified deposit of sand, both during and after the earthquake vibrations and to illustrate the significance of temporal changes of pore pressure.

3.2 PROCEDURE FOR EVALUATING STRESSES INDUCED BY EARTHQUAKE:

In a sand deposit consider a column of soil of depth, h, and unit cross section subjected to maximum ground acceleration a_{max} (Fig.3.1). Assuming the soil column to behave as s rigid body, the maximum shear stress τ_{max} at a depth, h, is given by

$$(\tau_{max}) = \frac{\gamma h}{g}.a_{max} \tag{3.1}$$

where, γ is the unit weight of soil and g is acceleration due to gravity. In reality the soil column behave as a deformable body.

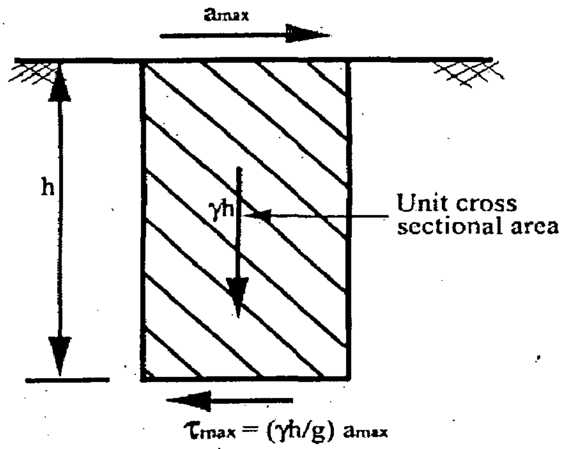


Fig. 3. 1 Maximum shear stress at a depth for a rigid soil column

At the depth h, the actual shear stress, $(\tau_{max})_{act}$, is taken as

$$(\tau_{max})_{act} = r_d \cdot \tau_{max} = r_d \cdot \frac{\gamma h}{g} \cdot a_{max} \qquad (3.2)$$

where, r_d is the stress reduction coefficient at depth, h. Its value is less than unity.

Variation of r_d for a wide variety of earthquake motions and soil conditions having sand in the upper 16-17m (about 50 ft) is shown in Fig. 3.2, which was proposed by Seed and Idriss (1971). There is a wide band of r_d values at depths greater than 14 m due to scatter in results. Thus for depths up to 14 m (40 ft), a reasonably accurate assessment of r_d may be made.

Seed and Idriss (1971) recommended that average equivalent uniform shear stress, τ_{av}, is given by:

$$\tau_{av} = 0.65 \cdot \frac{\gamma h}{g} \cdot a_{max} \cdot r_d \qquad (3.3)$$

Stress Causing Liquefaction

Determination of the cyclic shear stress causing liquefaction of a given soil in a given number of stress cycles may be obtained by a laboratory testing using cyclic triaxial test apparatus. The stress ratio causing liquefaction in the field may be estimated from the relationship

$$\left(\frac{\tau}{\sigma_o'}\right)_{field,D_r} = \left(\frac{\sigma_d}{2\sigma_3}\right)_{triax,50} \cdot C_r \cdot \left(\frac{D_r}{50}\right) \qquad (3.4)$$

where τ is the shear stress developed, σ_0' is the initial effective overburden pressure, D_r is relative density, σ_d is the cyclic deviator stress, σ_3 is the initial ambient pressure under which the sample was consolidated and C_r is a correction factor to be applied to laboratory triaxial test data to obtain the stress condition causing liquefaction in the field. From Fig.(3.3) value of C_r may be obtained. The number 50 in the Eq. (3.4) stands for 50% relative density. $(\tau / \sigma'_o)_{field,Dr}$ is stress ratio causing liquefaction at a relative density D_r under field condition and $(\sigma_d / 2\sigma_3)_{50}$ is stress ratio obtained by laboratory tests. With the help of above information, the stress required for causing liquefaction at the depth, h may be obtained. Figures 3.1, 3.2 and 3.3 have been digitized. The computer program developed in this M.Tech. dissertation makes use of the same to evaluate C_r and stress ratio causing liquefaction.

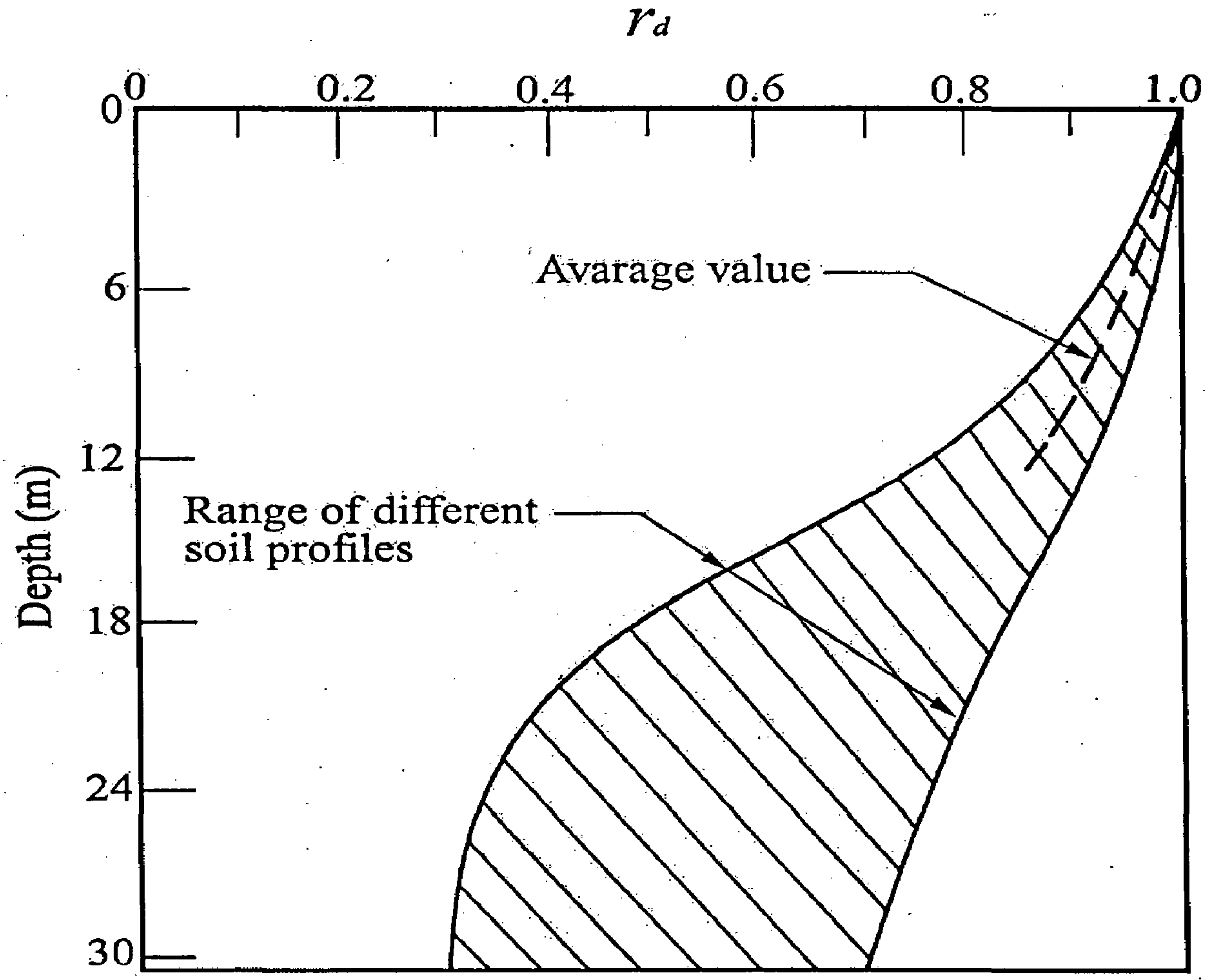


Fig. 3. 2 Reduction factor r_d versus depths (Seed and Idriss, 1971)

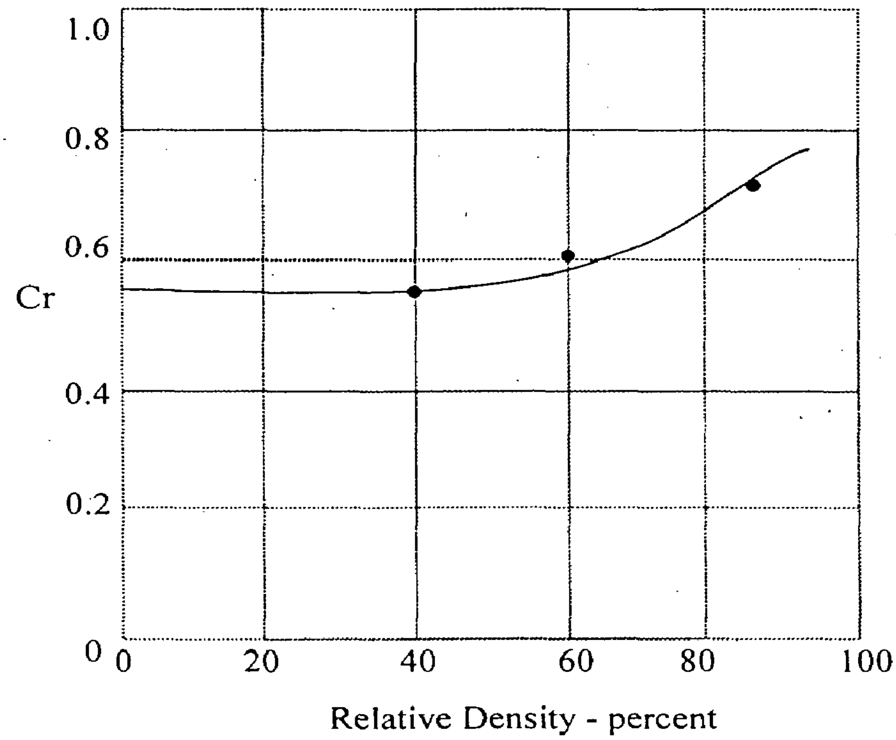


Fig. 3. 4 Relation between C_r and Relative Density

3.3 PROPOSED METHOD FOR PREDICTING TEMPORAL VARIATION OF PORE PRESSURES DURING EARTHQUAKE VIBRATIONS

The method of Seed and Idriss (1971) is used to evaluate the possibility of liquefaction at the point at the depth, h below the ground level. For this analysis, prior evaluation of a_{max} at the point under consideration is necessary and without which the analysis cannot be performed. The value of a_{max} at any point with depth h, below the ground level is not known before performing the analysis for computation of ground response. Therefore, it is necessary to make a reasonable estimation of a_{max} to begin the analysis. If maximum response, $(a_{max})_{BRL}$, at base rock level is known as input data, the maximum acceleration, $(a_{max})_{GL}$, at ground level may be obtained as :

$$(a_{max})_{GL} = (a_{max})_{BRL}\, \mu_f \tag{3.5}$$

where μ_f is Kanai magnification factor. Maximum acceleration at different depths may be obtained by assuming linear variation of (a_{max}) from base rock level to ground level. Therefore, maximum response, $(a_{max})_h$, at the depth, h below the ground level may be obtained as :

$$(a_{max})_h = (a_{max})_{GL} + ((a_{max})_{GL} - (a_{max})_{BRL}) * (h/H) \tag{3.6}$$

where H, is the distance between the ground level and the base rock level.

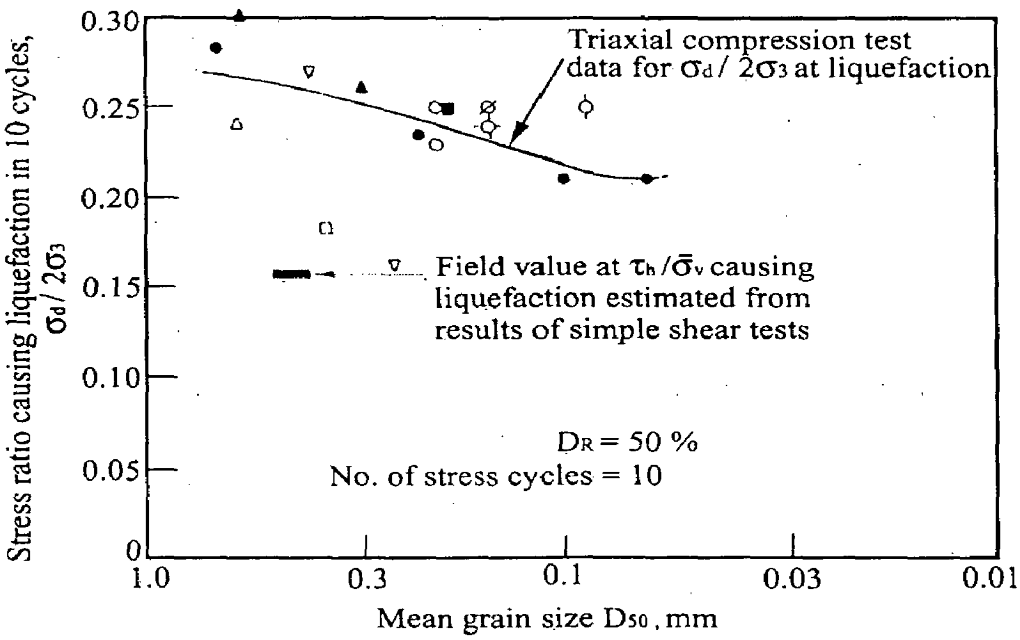
Using Eq. 3.3, the shear stress, τ_{av}, induced by the earthquake at the depth, h can be evaluated. At this depth, correction factor C_r, grain size corresponding to 50% finer fraction, D_{50}, and relative density of the soil, D_r, are also known. The value of stress ratio, $(\sigma_d/2\sigma_3)_h$, corresponding to τ_{av} may be evaluated by using Eq. 3.4 and given by:

$$(\sigma_d/2\sigma_3)_h = (\tau/\sigma_o')/\ [C_r\ (D_r/50)] \qquad (3.7)$$

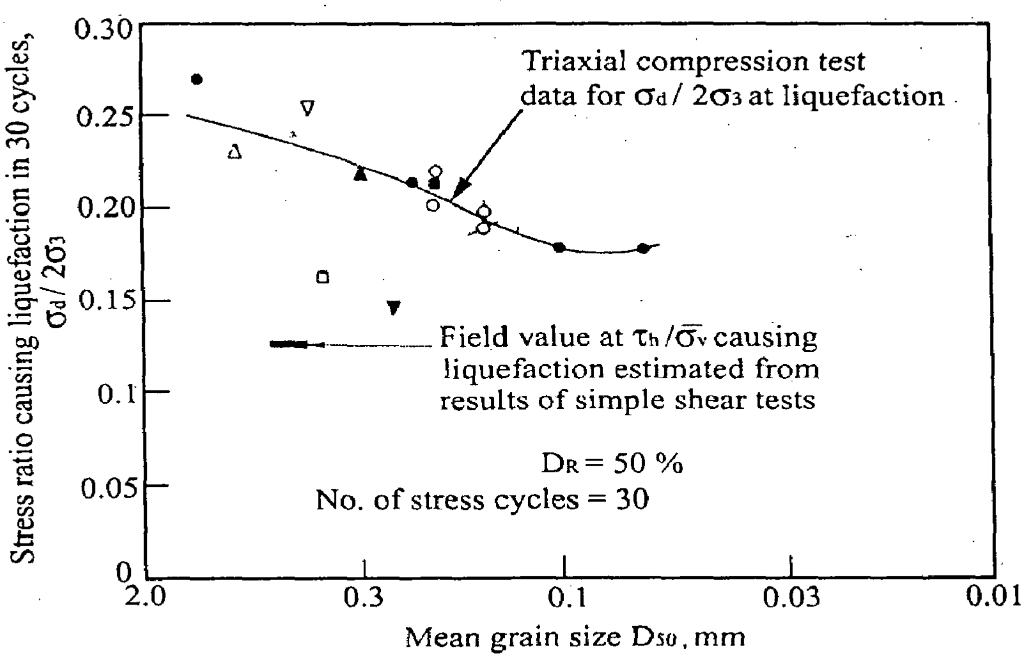
Using D_{50}, the stress ratio, $(\sigma_d/2\sigma_3)_{10}$, required to cause liquefaction in 10 cycles and stress ratio, $(\sigma_d/2\sigma_3)_{30}$, required to cause liquefaction in 30 cycles may be obtained from Fig. 3.2 and Fig. 3.3 respectively. By using linear interpolation, the number of cycles, N_l, required to cause complete liquefaction of soil corresponding to stress ratio $(\sigma_d/2\sigma_3)_h$ may be evaluated using $(\sigma_d/2\sigma_3)_{10}$, $(\sigma_d/2\sigma_3)_{30}$, 10 and 30.

At complete liquefaction, the pore pressure ratio, r_u, defined as the ratio of dynamic excess pore pressure, u, to initial effective vertical normal stress, σ_o', is unity. At the beginning of the earthquake the value of r_u is zero. The value of r_u is assumed to vary from zero to unity linearly. This is reasonably supported by experimental results reported by many investigators (Lee and Albaisa, 1974; De Alba et al., 1975; De Alba et al., 1976; Tanaka et al., 1983, 1984; Xu, 1991). The number of cycles in such a relationship may be converted into time by dividing it by frequency, $f_k = 1/\ T_k$, where T_k is fundamental period of ground vibration given by Kanai. Similarly, the pore pressure at any time, t, may be obtained by multiplying by corresponding value of r_u from above relationship with σ_o'.

The seismic response of layered media using nonlinear shear wave propagation may begin in usual way by using a_{max} obtained by liner interpolation of a_{max}, at base rock and at ground level. After completing the wave propagation analysis for the first time station, accelerations at all interfaces of sublayer of the layered system are obtained, which on double integration will yield displacement response at interfaces. These are used to compute shear strains in the sublayer. Using the hyperbolic stress-strain relationship, the shear modulus at the mid point of the sub layer corresponding to the shear strain in that sublayer may be obtained. The effective stress at the middle of the sublayer under consideration may be now corrected by adding the dynamic pore pressures at that instant of time and by subtracting the reduction in the pore pressure due to assumed type of drainage.



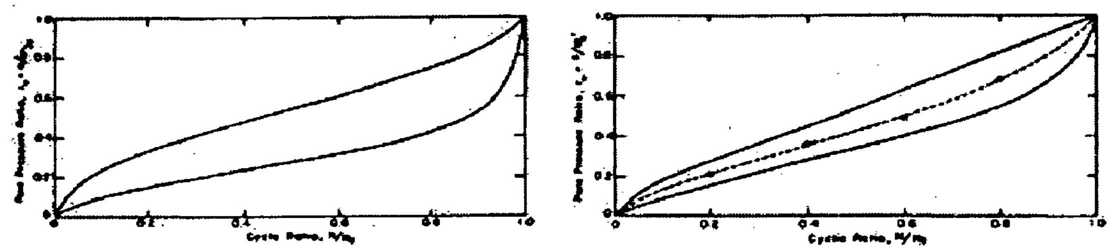
a) In 10 cycles



b) In 30 cycles

Fig. 3. 5 Stress conditions causing liquefaction of sands

(Seed and Idriss, 1971)



(a) cyclic triaxial tests (Lee & Albaisa, 1974) (b) cyclic simple shear tests (DeAlba et al., 1975)

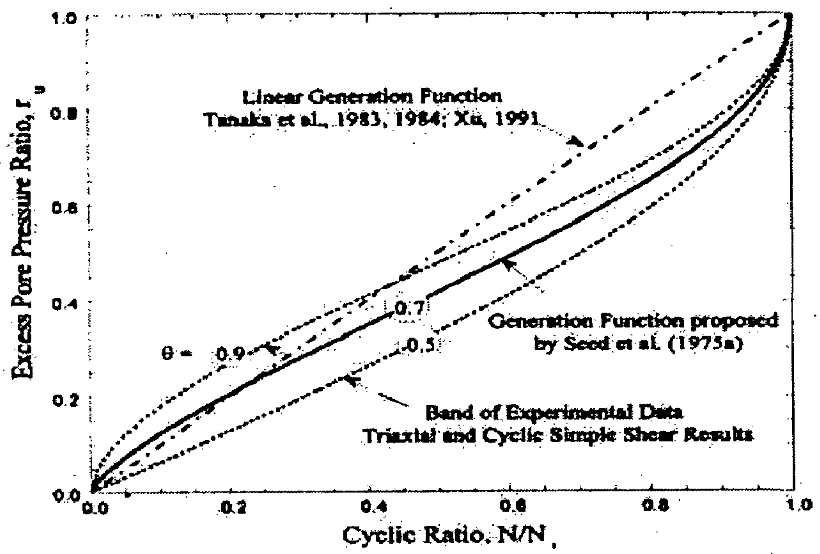


FIG. 3. 6 Excess pore Pressure Generation

The shear modulus G may be further modified in the light of the dynamic pore water pressure and its dissipation due to drainage by using square root of effective octahedral normal stress as the weighting factor. The modified, $G_{modified}$, may be expressed as :

$$G_{modified} = (1/G_{max}) / [(1/G_{max}) + \gamma/\tau_{max}] \qquad (3.8)$$

In the liquefaction analysis the Seeds approach is semi empirical and many constants are empirical in nature. As such the use of initial effective vertical stress σ_v', and mean initial effective confining triaxial pressure, σ_0' are to be used as recommended by them without tempering for calculation of N_l. On the other hand in the wave propagation analysis, the definition of G(Shear modulus) is dependent on the actual effective octahedral normal stress at the time junction under consideration. This two approaches are different and do not affect each other as well as independent of each other.

3.4 ANALYSIS OF VERTICAL DRAIN SYSTEM

Drainage is one of the attractive options for liquefaction remediation, especially when used in conjunction with densification techniques. The fundamental principle of vertical drains is to allow for fast pore pressure dissipation during earthquake loading, thus preventing the development of large excess pore pressures leading to liquefaction (Seed et al., 1975a). they can also be used to reduce the liquefaction potential of surface layers due to upward seepage resulting from dissipation of excess pore pressures in deep soil deposits after the main event (e.g., Seed and Lee, 1966; Ambraseys and Sarma, 1969).

Saturated granular material subjected to cyclic loading which involves shear stress reversal will exhibit a tendency to contract, and thus will generate excess pore pressure if it is not allowed to drain. Depending on its initial formation density and its cyclic stress history, the soil may develop pore pressures high enough to cause a complete loss if shear strength at essentially zero effective stress (liquefaction) or cause excessive deformation (i.e., liquefaction with limited strain potential, Seed et al., 1975a). These phenomena are particularly severe for loose, relatively uniform, cohesionless soil deposits such as those developed during conventional reclamation work (i.e., hydraulically placed fills).

Structural damage as a result of liquefaction is induced by one of the two types of soil response. In the first case, increase in pore water pressure result in a complete loss of shear strength of the soil such that external forces can not be sustained and structural stability is compromised (e.g., bearing capacity failure, buoyancy of underground structures). In the second case, high pore water pressures cause a significant (but not complete) reduction of shear strength and large permanent ground displacements are observed (e.g., lateral spreading). In both cases, a significant amount of settlement can develop as a result of pore pressure dissipation and subsequent compression of the soil layers (e.g., Matsui et al., 1996). Any of these soil responses can serve damage to structures and their foundations, or to earth systems constructed with liquefiable material.

Analytical framework

The problem of pore pressure generation and dissipation during earthquake loading has been analyzed for conditions of purely vertical drainage (Seed et al., 1975a), for purely radial drainage (Seed and Booker, 1977) and combined radial-vertical drainage conditions (Booker et al., 1976). These analyses assumed soil profiles with initial water table at the surface and no drainage resistance.

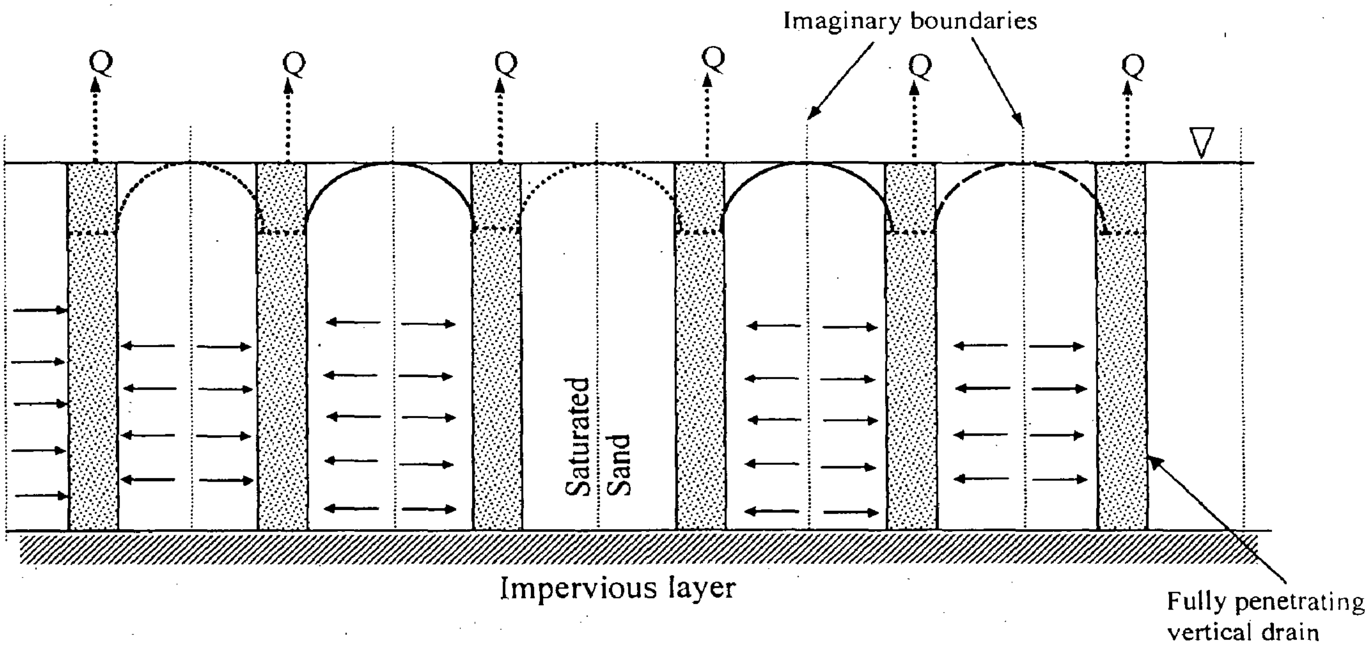


Figure 3. 7 Schematic arrangement of vertical drain

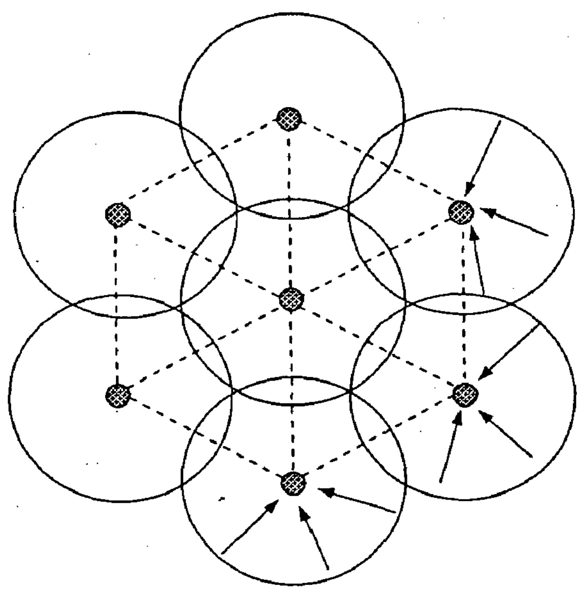


Figure 3.8 Effective circular area for triangular drain installation

One-dimensional drainage of a deep layer is time consuming as the shortest possible distance for drainage is the height of the layer. It is not possible to increase the drainage of natural layer. If hexagonal arrangement of vertical drain is used (Fig.

3.8), the shortest distance required for drainage is the radial distance. Hence, pore water generated is dissipated in lesser time.

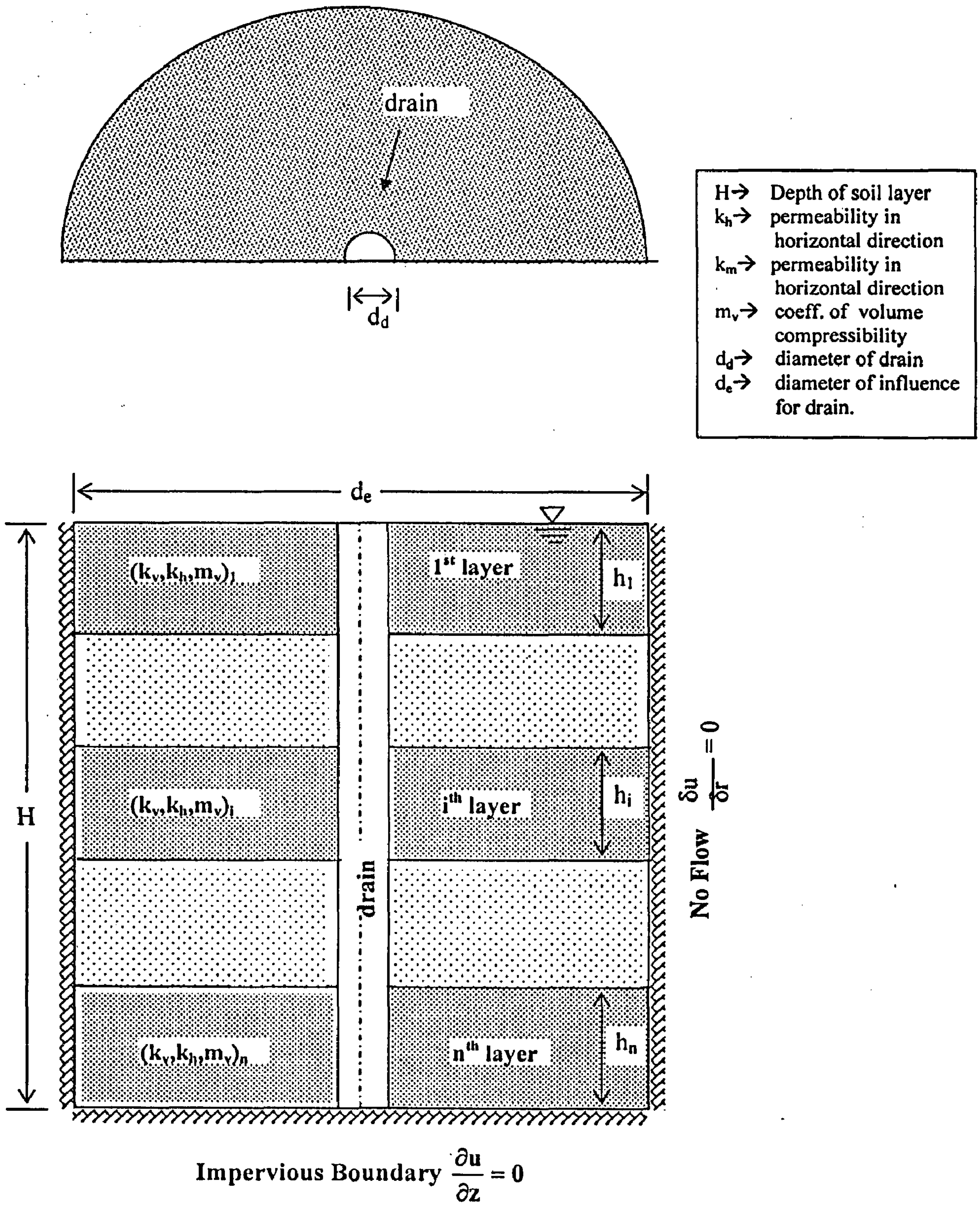


Figure 3. 9 Boundary conditions for analysis of vertical drain systems

The water flowing into the drain will follow the curvilinear path. At the periphery of drain, this curve will have tangent at angle *i*. But, in this analysis, the curvilinear path is assumed to be linear at angle *i'*. Here, $i > i'$ which gives lesser value of the discharge through the drain (by Darcy's law). This is a simplified assumption which gives conservative resuts.

The proposed model considers radial drainage condition. In this model, vertical drains with radius r_1 are installed. Fig. 3.9 explains the proposed model. Discharge in to the drain at any time at any instant is given by Darcy's equation which is valid for laminar flow and is given by,

$$Q = k\, i\, A \tag{3.9}$$

where Q is discharge, k is permeability, i is hydraulic gradient and A is cross sectional area normal to the flow.

Hydarulic gradient i is given by,

$$i = \Delta H / r_2 - r_1 \tag{3.10}$$

where ΔH is head loss, r_2 is radius of influence of drain and r_1 is radius of drain.

This discharge is for a given instant of time and is given by,

$$u_d = Q / A \tag{3.11}$$

This volume of water dissipated is volume of drained out through the drain. Hence, total volume of water drained out is obtained by integrating u over total time t.

The current formulation assumes full saturation within the soil profile both above and below the water table (thus neglecting limited storage capacity on soils above water table). Modifications to the formulation will be required to address the issue of saturation, but are not considered to be vital in most practical scenarios.

Having established the relationship for pore pressures generation and dissipation in a soil deposit, the following procedure now may be reasonably followed.

1. For a soil layer at any depth in a deposit, the time-history of horizontal shear stresses developed by the earthquake is evaluated by any appropriate procedure; this may be done by ground response analysis as proposed.
2. By means of as appropriate weighting procedure, the effects of the actual stress history can be represented by an equivalent number, N_{eq}, of uniform

stress cycle having a peak stress amplitude τ_{eq}, developed over a selected duration of shaking; that is, at a particular period, T_{eq}, of cyclic stress application.

3. For the known condition of overburden pressure and density of the sand, and the equivalent uniform cyclic stress determined in step (2) above, determine the number of cycles N required to produce a condition of liquefaction in the sand layer.
   (a) If $N_l < N_{eq}$, the layer will liquefy before the shaking is completed and after a period of time equal to $N_l * T_{eq}$. Thus the pore pressures build-up, will follow the curve as shown in Fig 3.5., with pore pressure ratio becoming unity after $N_l * T_{eq}$ seconds of shaking;
   (b) If $N_l > N_{eq}$, the pore pressure will follow the curve shown in Fig. With the pore pressure being equal to that given by the curve for the appropriate value of N_{eq}/N_l at the end of the period of shaking being considered.

In the procedure suggested by Seed et al. (1975b), the effect of actual stress history is converted to an equivalent number of uniform shear stress cycles, N_{eq}, with magnitude $\tau_{eq} = 0.65\ \tau_{max}$, where τ_{max} is the maximum shear developed at a given elevation. Table 3.1. gives the value of Equivalent number of cycles due to earthquake loading (Seed and Idriss, 1982).

Table 3.1 Equivalent number of cycles due to earthquake loading (Seed and Idriss, 1982)

Magnitude	N_{eq}
5 ¼	2-3
6	5-6
6 ¾	10
7½	15
8½	26

CHAPTER-4

RESULTS AND DISCUSSIONS

4.1 PREAMBLE

Results of investigations obtained by using the computer program for different layered system for a variety of excitation at the firm ground/base rock level are presented in this chapter. The parametric studies vary various parameters to study their influence on seismic ground response and other factors based on the ground response. The analysis is carried out in linear as well as nonlinear domains and the results obtained are compared. The results obtained are also used for the demonstrating the capability of the program for the study of ground response for a layered system as well as for preparation of data for developing seismic microzonation maps for areas/regions under consideration.

4.2 INPUT DATA FOR INVESTIGATION

For this investigation, a layered system consisting of 'n' main layers resting on base rock/firm ground is employed. Fig 4.1 shows details of the system with five main layers (n=5) such as thickness, cohesion, failure strain, angle of shearing resistance, specific gravity, void ration and water content of each layer as well as properties of base layer which is np^{111} layer(n+l). Table no.4.2.1 and Table 4.2.2 gives various details of the layer which include input data computed data respectively. The layers are numbered serially with top most layer named as layer no.l. The interfaces are also numbered serially with ground level denoted as interface no. 1.

The ground water table is always considered as an interface. In case the input data dose not complies with this requirement, the program automatically renumbers the layer and interfaces show that ground water table becomes an interface.

As explained in the earlier chapters the main layers of the system are subdivided into thinner layers or sublayers so that each sublayer is reasonably thin to the desire extend. Sublayers of each main layer have the same thickness. For obvious region, the ground water table becomes an interface between two sublayers. The interfaces of the sublayer are also numbered serially from one to nsubp where nsubp tends for largest rank of interface I number. Interface no 1 remains to be at the ground level. Sublayer no 1 is the top most sublayer, nsub^{111} layer is the lower most sublayer and baserock or firm ground layer is denoted by the number nsubp^{111} (nsub+1). Figure 4.2 shows details of the system with sublayer. Other input data employed for parametric studies are the base excitation (amplitude of acceleration aeq, feq and duration of excitation teq), depth of ground water table below the ground level, duration of time interval ti and minimum thickness of the sublayer.

Even though the computer program is capable of reading digitized acceleration time history of actual earthquakes, in this investigation only sinusoidal base excitation is employed. Sinusoidal excitation is useful in carrying out parametric studies to study the 1 influence of amplitude and frequency of base excitation. It is also useful in the study of resonance and quais-resonance condition on the response of layered system.

In this investigation, unless otherwise stated, the value of input data will be the first value of the parameter under consideration. The second and subsequent values of any parameters are considered only when the influence of that particular parameter is being considered.

Ground level	GWT	Interface no. 1
Layer No. 1	H1 = 9m	Interface no.2
Layer No.2	H2= 7m	
Layer No.3	H3= 9m	
Layer No.4	H4= 11m	Interface no.'n'
Layer No.5 (n^{th} layer)	H5= 14m	Interface no. 'n+1'
BASE LAYER/FIRM GROUND LAYER		

Fig. 4.1 Five layer system

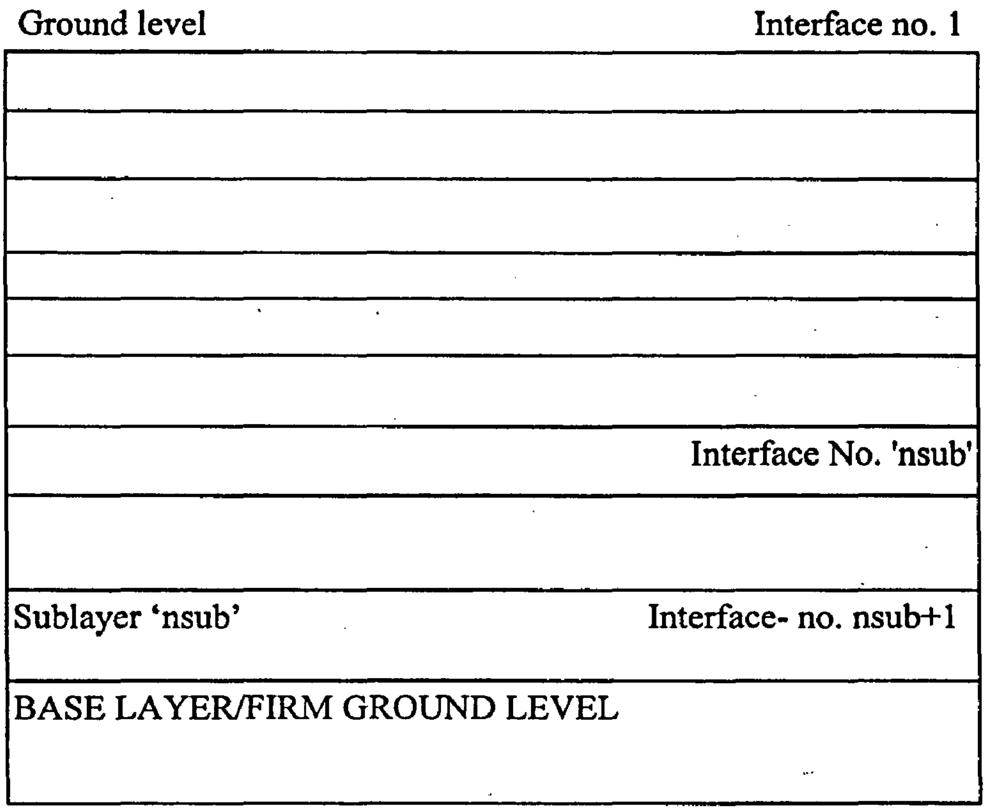


Figure 4.2 Division of layers

Table No.4.2.1 Details of soil properties as input data used for parametric studies

Layer No.	Thickness (H)m	Cohesion (c)kN	Failure strain(%)	Angle of shearing resistance (degree)	Void ratio (e)	Specific gravity	Water content
1	9	0.981	12.5	10	0.9	2.47	0.2
2	7	1.4715	10.0	15	0.8	2.57	1.0
3	9	0.0	9.0	20	0.7	2.67	1.0
4	11	0.0	8.0	25	0.6	2.67	1.0
5	14	0.0	7.0	30	0.5	2.67	1.0

Table 4.2.2 Details of computed data

Layer No	Bulk ut.wt (kN/m^3)	Saturated ut.wt (kN/m^3)	Shear wave velocity (m/sec)	Limiting shear stress(kN/m^2)
1	15.30	17.30	184.56	22.056
2	18.37	18.37	230.9591	30.207
3	19.45	19.45	259.780	60.77
4	20.05	20.05	294.4882	85.556
5	20.73	20.73	232.6374	100.393

4.3 RESULTS

- **Temporal variation of shear modulus with time.**

Fig.(4.3) shows variation of shear modulus with time . It may be observed that the initial modulus of the soil is the highest value of modulus for the entire duration of the earthquake. As nonlinearity of the soil behaviour increases the value of the shear modulus keeps on reducing. When pore pressure is considered it is found that the shear modulus reduces, because the material properties of the soil changes due to pore pressure and soil becomes weak. When drainage is considered the material property of the soil again changes and the soil gains strength and hence improves the shear modulus.

- **Temporal variation of mobilized shear stress.**

Fig.(4.4) shows the variation of mobilized shear stress as a function of time. It varies in a nonlinerly with the time. Sometimes it crosses the time axis and sometimes it does not. As the soil strain increases the mobilized shear stress also increases and vice-versa. Shear stress improves with when drainage is considered and almost approaches the value as for the without pore pressure case.

- **Temporal variation of mobilized shear strain.**

Fig(4.5) shows variation of shear strain. For pore pressure case the strain goes on increasing which is expected as the soil tends to become loose and its displacement increases. For case when drainage is considered the strains are not much large. This is due to the fact that excess pore water pressure is dissipated and material properties of the soil are improved.

- **Variation of pore pressure with time.**

The variation of pore pressure with time is given in Fig.(4.6) which is based on the method of analysis proposed by Seed and Idriss(1970). The pore water pressure varies linearly with time as proposed in the method of analysis.

- **Temporal variation of ground acceleration.**

Variation of ground acceleration with time is given in Fig.(4.7). It is observed that variation of ground acceleration with pore pressure is greater than the ground acceleration without pore pressure. With pore pressure the ground becomes soft and the material properties changes and consequently the amplitude increases which is expected. Result indicates that the amplitude increases with pore pressure and decreases when pore pressure is allowed to dissipate. This is again on the expected lines.

- **Temporal variation of shear wave velocity.**

Variation of shear wave velocity is shown in Fig. (4.8). Shear wave velocity variation is in line with variation of shear modulus which is on the expected lines. Due to pore pressure the soil becomes soft and the shear wave velocity decreases and with drainage it tries to improve.

- **Temporal variation of fundamental period of ground vibration**

The fundamental period of ground vibration, T_g may be computed by using Kanai's method given by, $T_g = 4H/V_s$, where, H is the depth of the soil layer and V_s is the velocity of shear wave. For a layer system, T_g is expressed summation as of $4H/V_s$. Fig.(4.7) shows the temporal variation of T_g with pore pressure, without pore pressure and for drainage condition. T_g increases as the pore pressure increases which is expected and decreases when dissipation of pore water pressure is accounted for. This also indicate that the consideration of pore water pressure generation and drainage in the computation of response, alter the dynamic characteristic of the layer system which is an important finding for use in the earthquake resistant design.

- **Variation of Shear wave velocity with pressure.**

  Shear wave velocity increases with the increase in pressure as shown in Fig.(4.10). This is in line with the variation obtained by Hardin and Richart (1963). It has higher value for dry case and lower value for saturated case.

- **Variation of shear wave velocity with depth for initial condition**

  Shear wave velocity increases with the depth which is as shown in Fig.(4.11).

- **Variation of shear stress with depth for initial condition.**

  Shear stress increases with depth as shown in Fig(4.12). With depth soil properties improves and hence the shearing stress also improves.

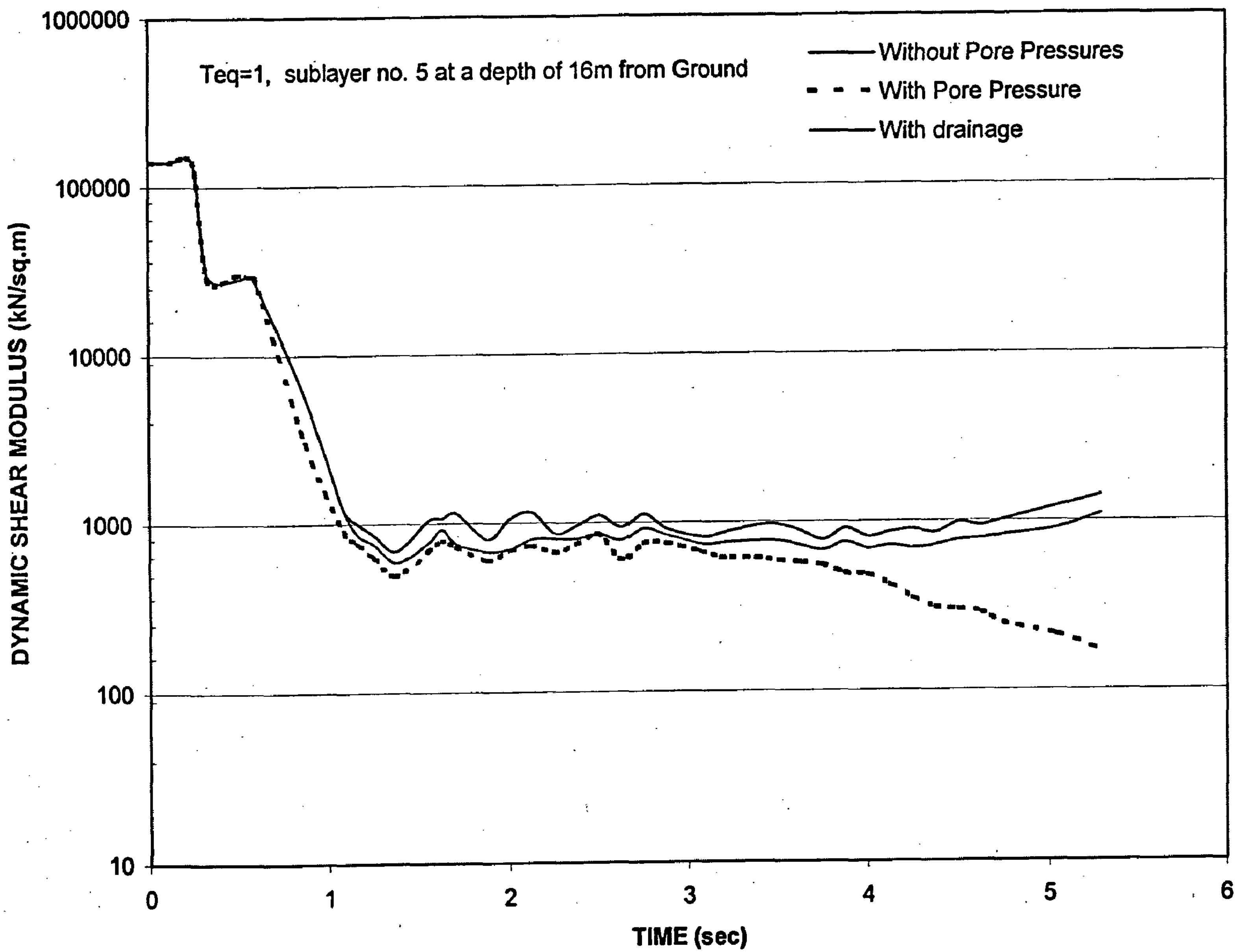


Fig. 4.3 Temporal Variation of Shear Modulus with Time

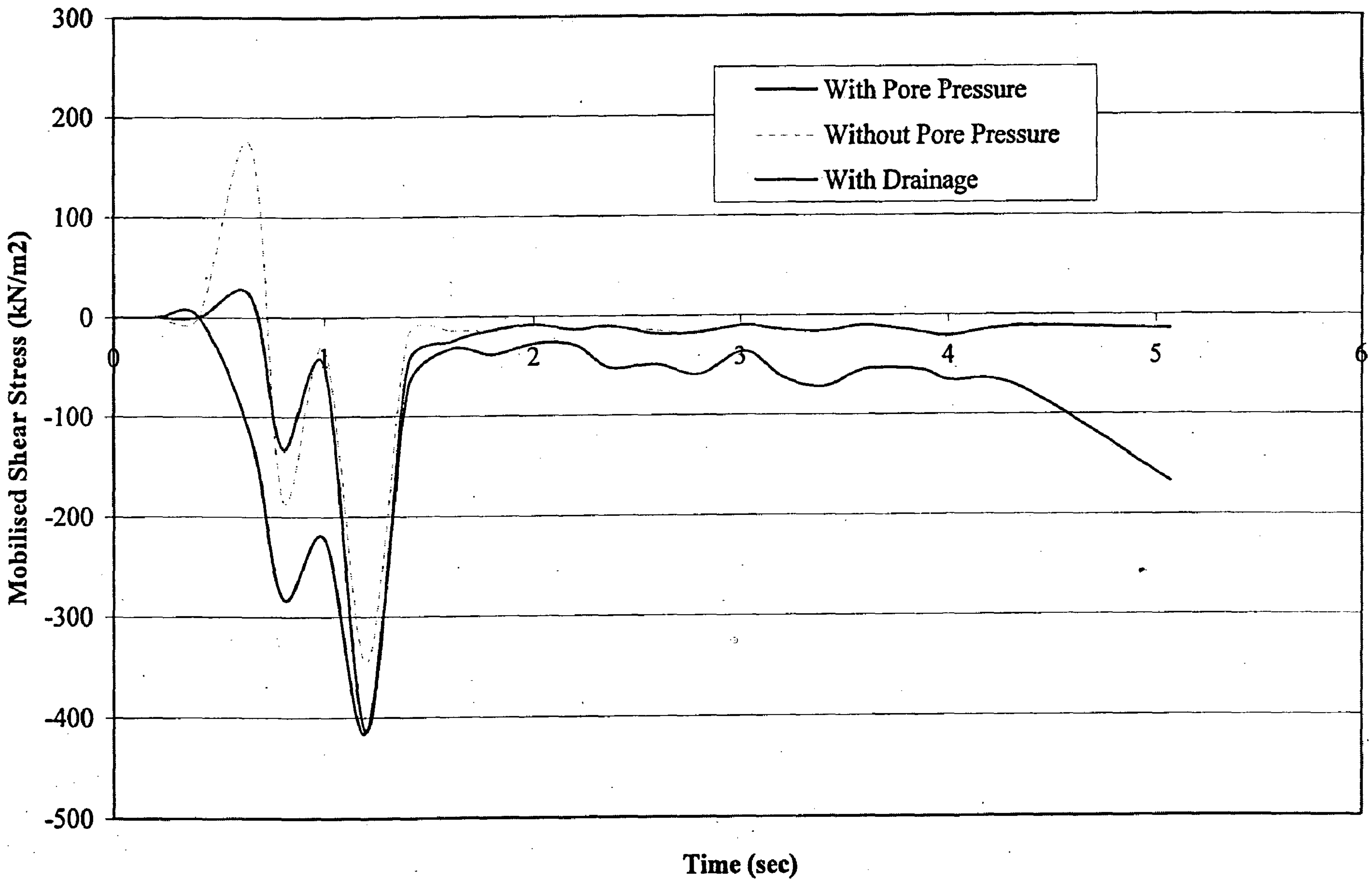


Fig. 4.4 Temporal variation of Mobilised Shear Stress

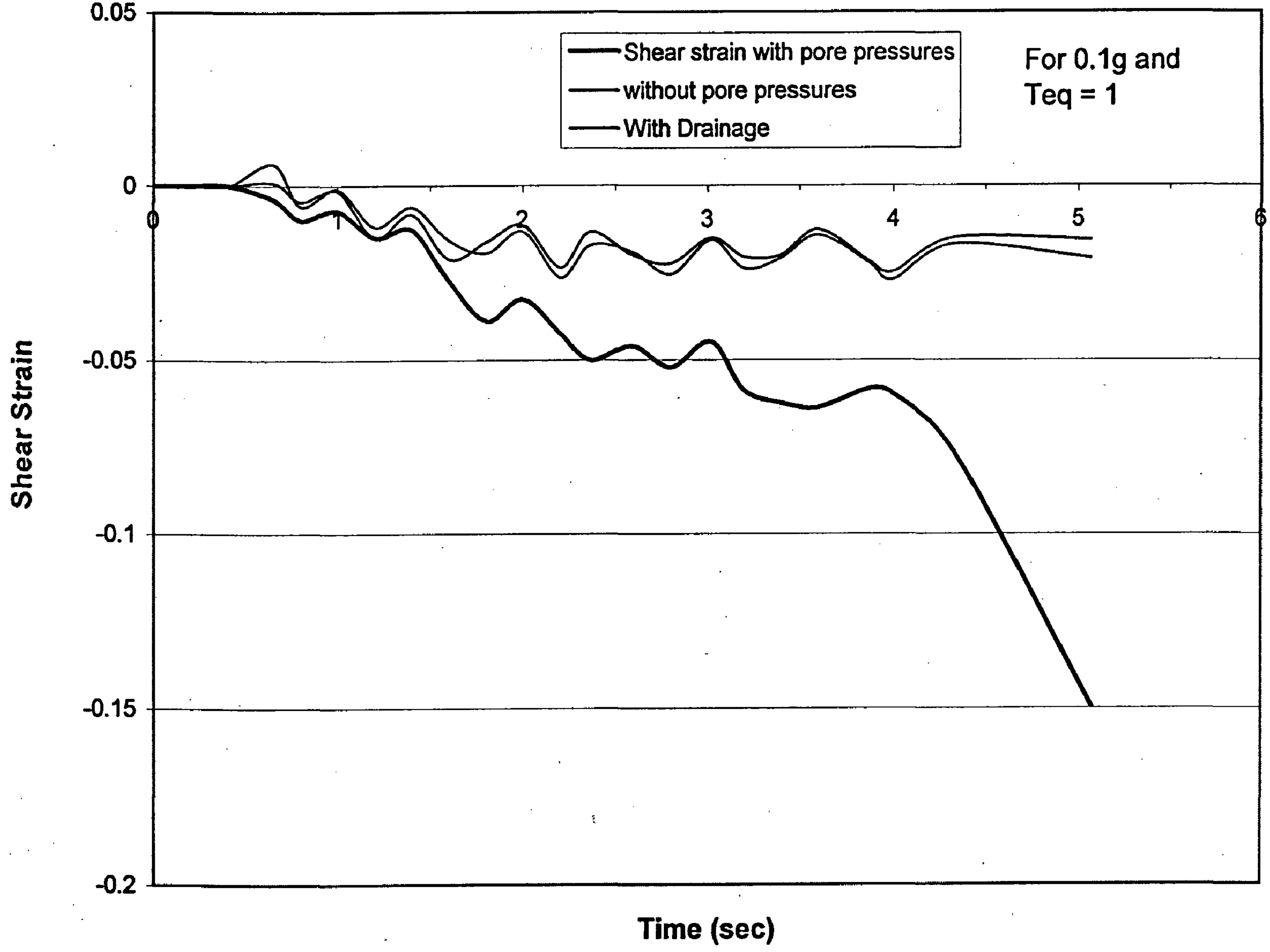


Fig. 4.5 Temporal variation of Shear Strain

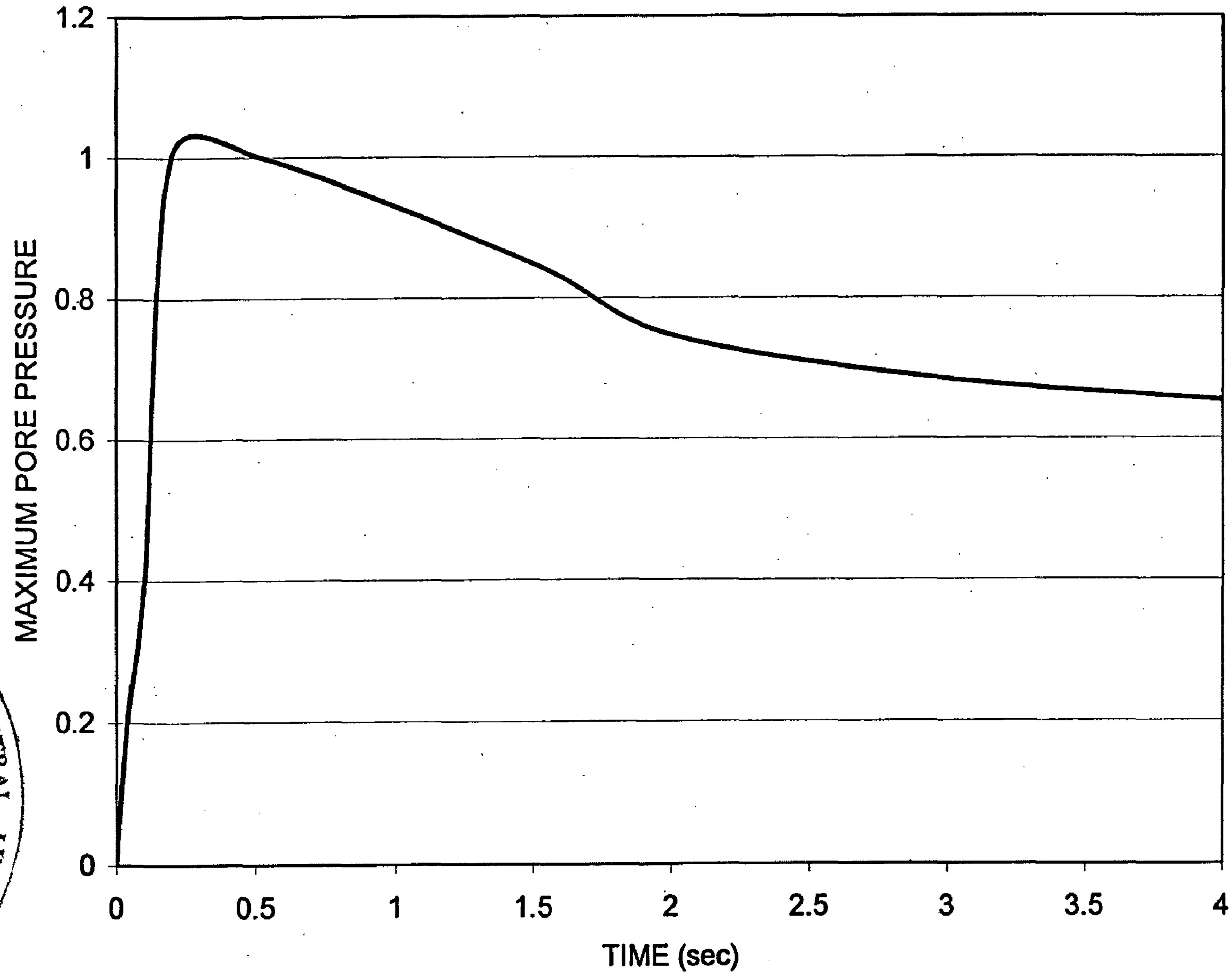


Fig. 4.6 Variation of Pore Pressure with Time

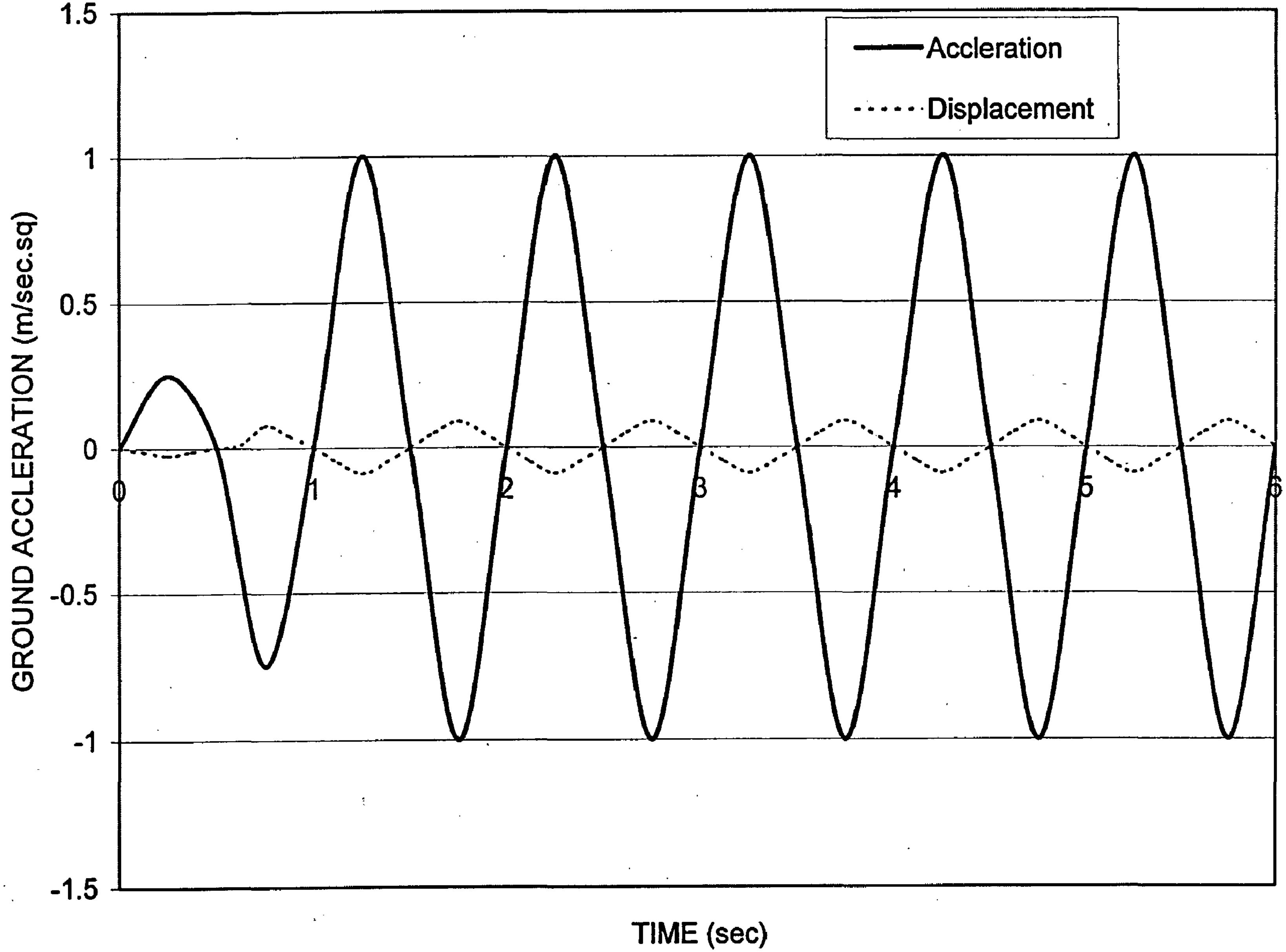


Fig. 4.7-a Base Excitation

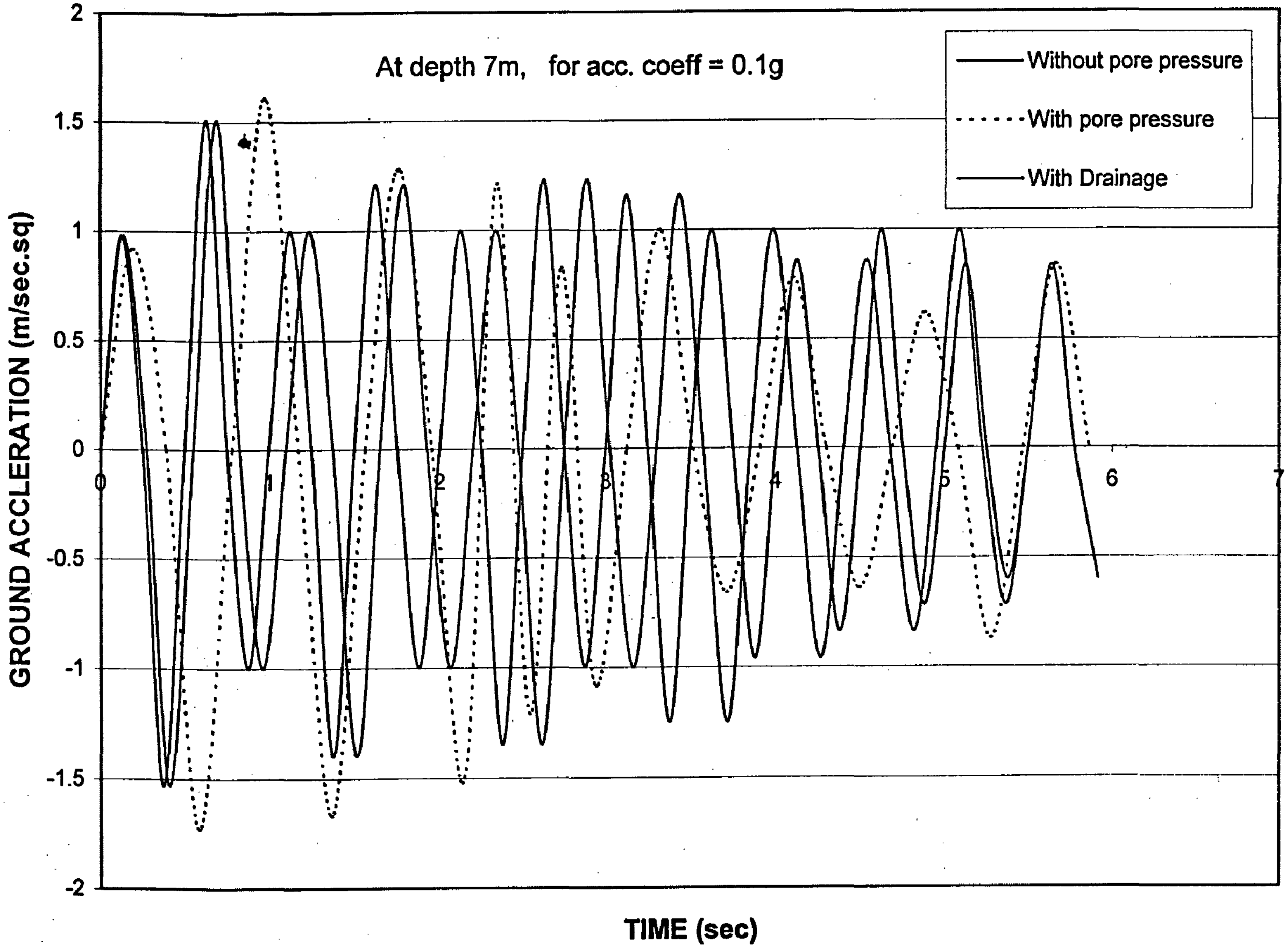


Fig. 4.7-b Temporal variation of Ground Acceleration

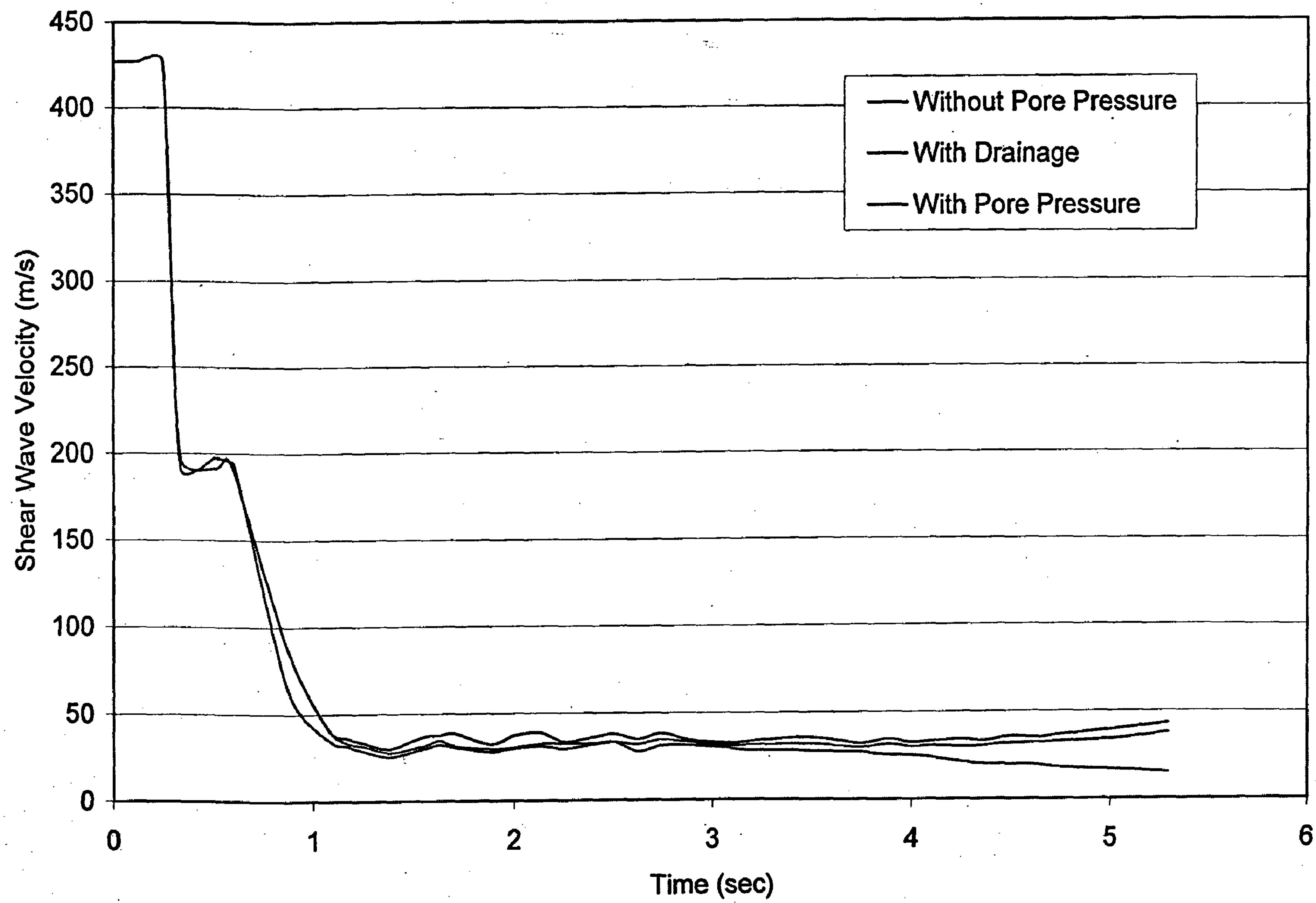


Fig. 4.8 Temporal variation of Shear Wave Velocity

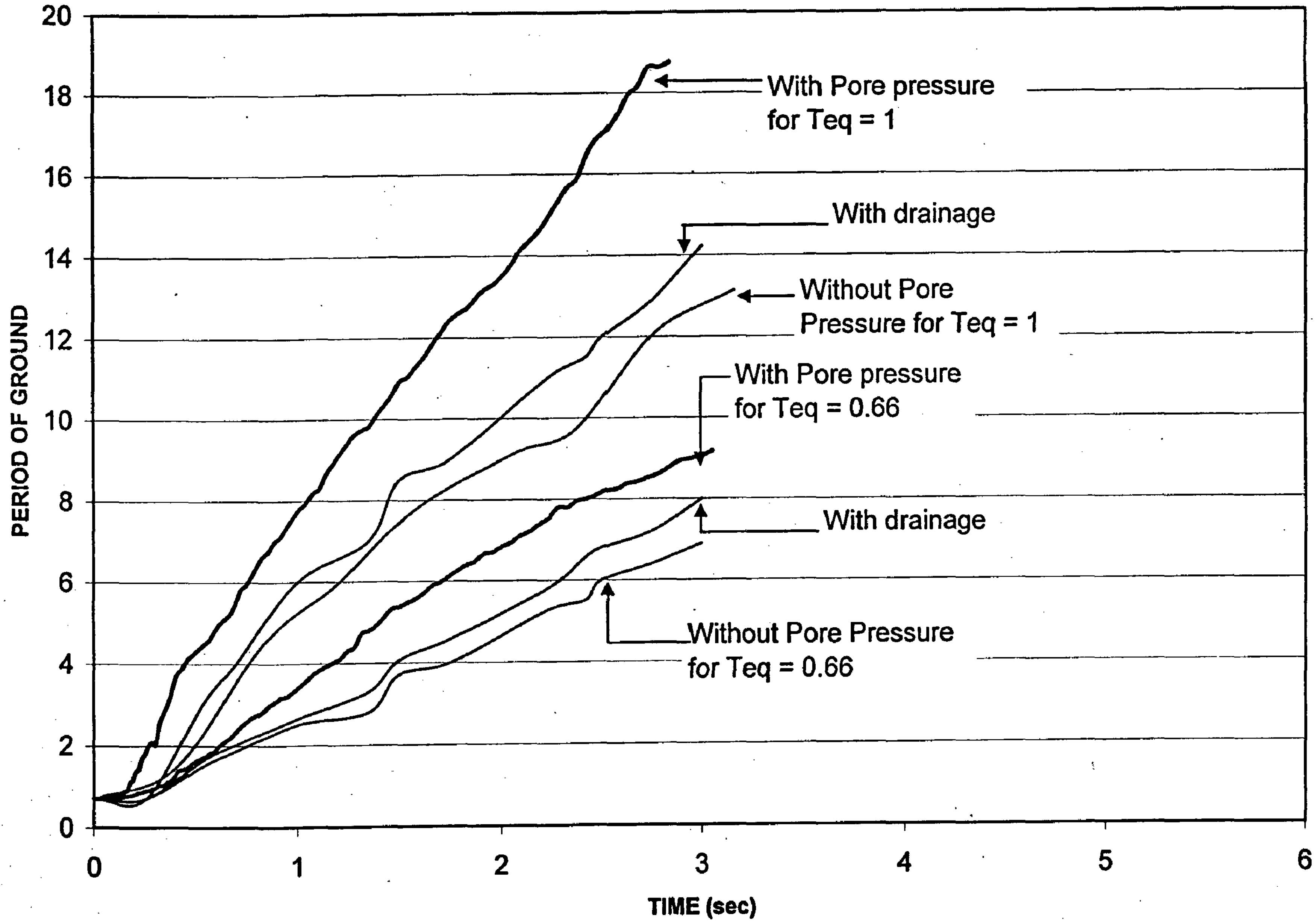


Fig. 4.9 Temporal variation of Fundamental Period of Ground Vibration

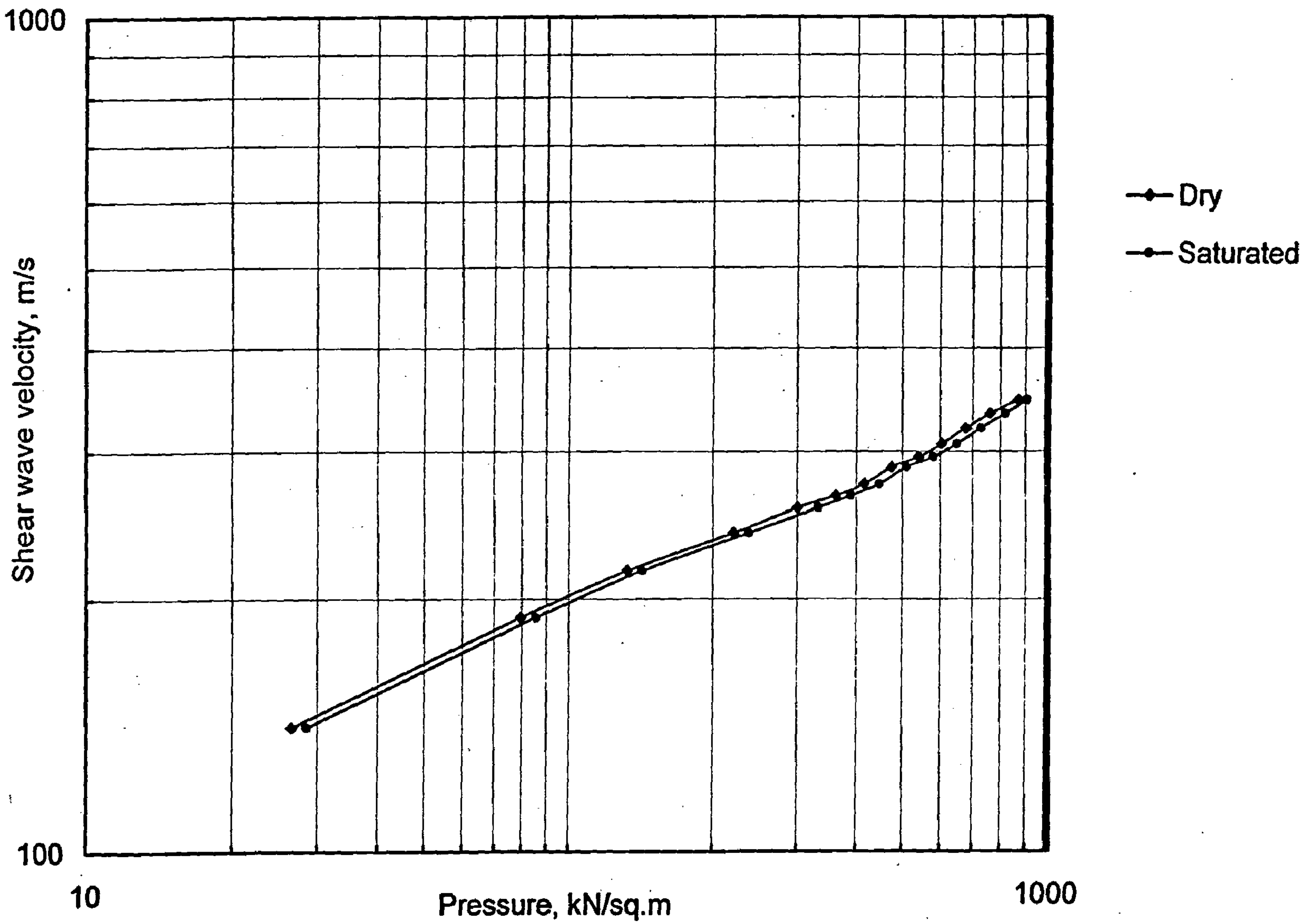


Fig. 4.10 Variation of Shear Wave Velocity with Pressure

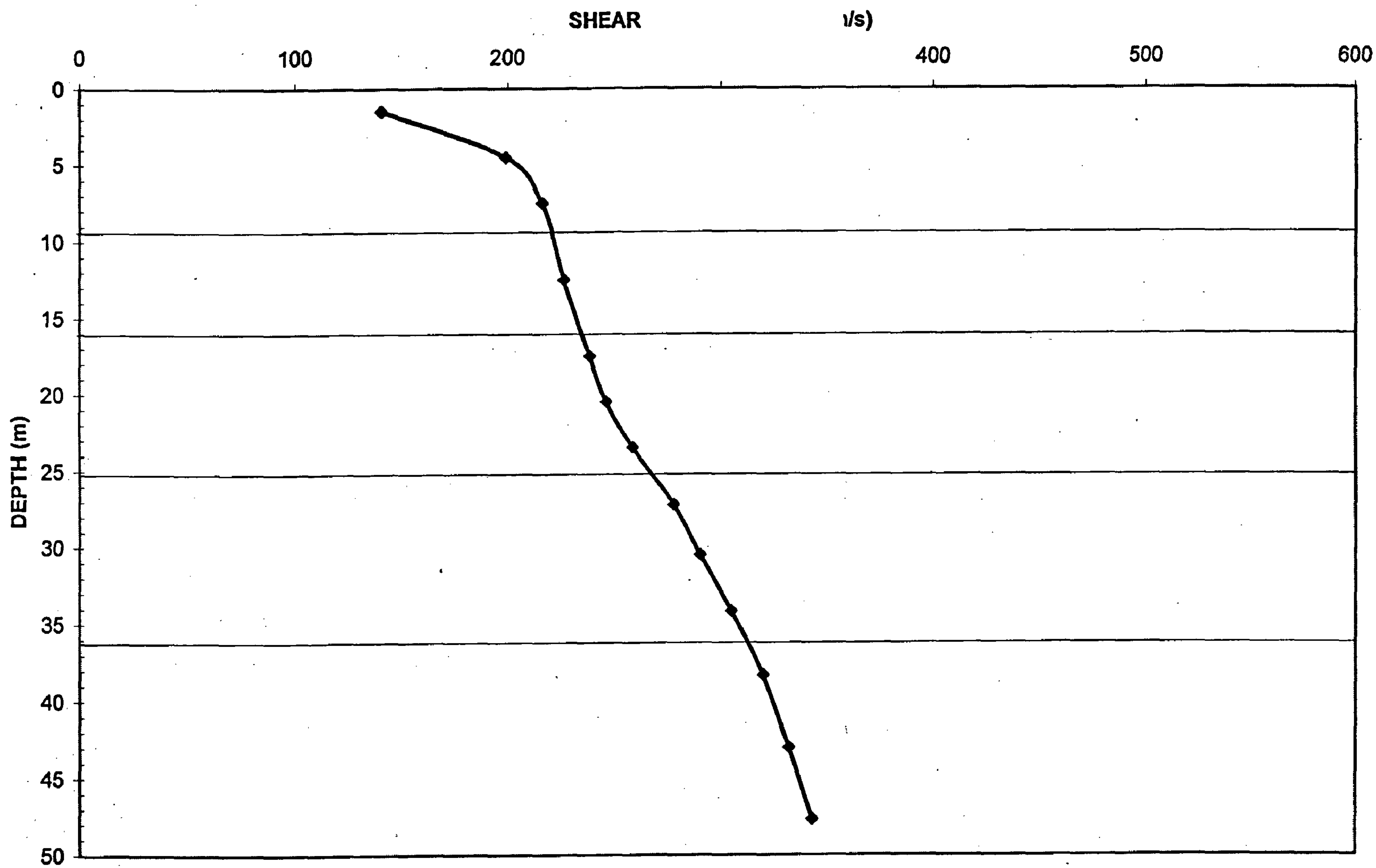


Fig. 4.11 Variation of Shear Wave Velocity with Depth for Initial Condition

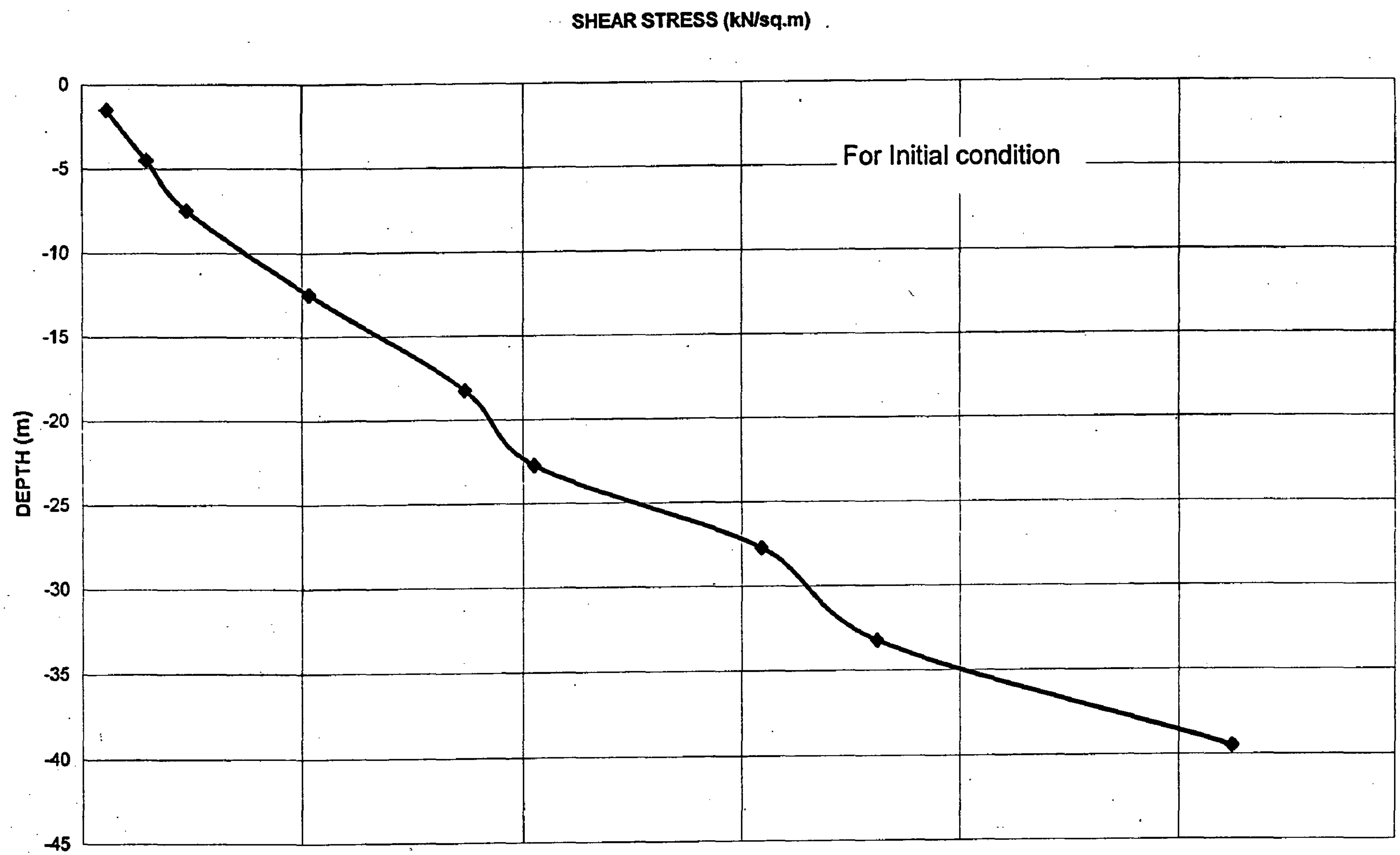


Fig. 4.12 Variation of Shear Stress with Depth

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The following are the conclusions drawn from the study:

a. Nonlinear analysis is required for soft layer seismic response.

b. Even through base response may be small the ground response may be high resulting into nonlinearity due to amplification.

c. Plane wave propagation method is ideal for seismic response analysis as it accounts for radiational damping completely.

d. The method of computation of component response due to upward and downward propagating shear wave for a given seismic response history at the base rock level recommended by the Joshi (1980) with due consideration to compatibility conditions at the base rock level is useful in analysis of seismic ground response of layered system.

e. Stress-Strain properties of the soil to obtained from experimental investigation. If this is not available, artificial stress-strain relationship may be considered in the form of elliptical, parabola or composite curve with flat slope at the failure strain.

f. The computer program developed for obtaining spatial/temporal distribution of seismic ground response. This program is easy to use and efficient to handle the response of any system with horizontal layers with due regard to nonlinear behaviour. The program may be run on commonly available personal computers. As such it can be used for obtaining the design seismic data for analysis and earthquake resistant design seismic design of structure by various design office/research organizations.

g. The computer program the following quantities for linear as well as nonlinear analysis.

- History of acceleration, velocity & displacement at selected or overall points within the system at the discretion of the user.
- The history of strain, mobilized shear stress, mobilized shear modulus of each

sublayer of the system.

- History of the fundamental period of equivalent single layer system.
- History of average strain in any main layer.
- +ve and -ve maximum value of strain of sublayer anywhere with in the system and the instant of time of their occurrence as well as the variation of strain in the entire layered system at that instant of time.

h. For a given sinusoidal base excitation, the acceleration increases with increasing base excitation amplitude as long as nonlinearity does not come into play. Once the nonlinearity begins to appear, the rate of increase of the amplitude of seismic vibration will increase at a much slower rate with the increase of base excitation. In contrast, the amplitude of ground acceleration predicted by linear analysis increases considerably with increasing base excitation amplitude to unrealistically high values.

i. The strain level obtained by nonlinear analysis is much larger than those obtained by linear analysis. This is reasonable, because, with increasing nonlinearity the material becomes softer and gets easily deformed resulting into large deformation and strain.

j. The period of vibration of the equivalent layer remains unchanged as long as the system ' vibrates in the elastic domain. When nonlinear behaviour is manifested, the material becomes softer and the value of shear modulus decreases with increasing strains. This results into the fluctuation in the value of the shear modulus with time. The larger the degree of nonlinearity, the greater the period of vibration of the system.

k. The maximum strain amplitude occurs in the softer layers near the ground level which is expected. With increasing depth below the ground level where velocity of shear wave is larger, the strain levels decreases with depth. Besides, the strain levels generally increases with increasing amplitude of acceleration of base excitation. This effect is much more pronounced in case of nonlinear analysis.

l. With decrease in the time interval between two consecutive time stations, the accuracy of the determination of the response reasonably increased.

m. When period of sinusoidal base excitation is varied from 2.05 sec to 0.5 sec the period of single layer system changes considerably due to non linear behaviour. Importantly the computed ground period appreciably increased, when the excitation period is very close to fundamental period of system. This is basically due to occurrence of resonance/quasi-resonance condition prevailing. For such a case the resultant strains are very large and the mobilized values of the shear modulus are very low.

n. Using drains the more precise seismic response can be accurately calculated.

5.2 RECOMMENDATIONS

1. Further studies can be carried out with radial as well as vertical drainage.
2. Permeability is assumed to be constant for a whole depth. But during seismic vibration soil properties changes and hence the permeability. So permeability variation must be considered.
3. Effect of spacing of drains is not studied. It is recommended that spacing of drains must be studied.
4. Only gravel drains are considered. Study can be carried out with different drain material

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