SOILS LIQUEFATION POSSIBILITY IN BANGLADESH

A DISSERTATION SUBMITTED BY

MD. SARAFAT HOSSAIN KHAN

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MD. SARAFAT HOSSAIN KHAN

Approved as to style and content by:

A. M. M. Safiullah 8/1/89

Dr. A.M.M. Safiullah — Chairman
Professor,
Dept. of Civil Engineering,
BUET, Dhaka.

A Murtuza 8/1/89

Dr. Md. Alee Murtuza — Member
Professor and Head,
Dept. of Civil Engineering,
BUET, Dhaka.

M. Hossain Ali 8/1/89

Dr. M. Hossain Ali — Member
Professor,
Dept. of Civil Engineering,
BUET, Dhaka.

April, 1988

Dedicated To My Loving

Wife and Daughter

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ABSTRACT

The major cause of damages to structure during an earthquake has been the development of liquefaction in saturated sand deposits. Liquefaction phenomena have been recorded and have developed in many parts of the world where ground shaking is frequent and soils consists of loose fine sands under water table. Liquefaction criteria has been established by many workers based on cyclic stress ratio and the modified penetration resistance. These inturn are related to the induced earthquake history and the geological formation of soil. From historical evidence, it can be interpreted that the north eastern region of Bangladesh are tectonically active region. Geologically, the whole country is divided into two major parts one the pleistocene and the other the recent flood plains. The recent flood plains are basically loosely compacted soils with high water content. These may undergo liquefaction during a reasonable intensity of earthquake. Analysis was made on the historical earthquake records and the available soil data. From the analysis, it can be interpreted that a part of the north eastern region of Bangladesh may undergo liquefaction within small depths under water-table due to an earthquake of significant magnitude.

CHAPTER 1

INTRODUCTION

1.1 General

One of the most dramatic causes of damages to structure during an earthquake has been the development of liquefaction in saturated sand deposit. Liquefaction may be defined as a phenomena whereby there is at least a temporary loss of strength and stiffness as a result of excess pore water pressure developed due to monotonic or cyclic loads. During the earthquake the normal stress on the plane remains constant but cyclic shear stresses are induced by the ground shaking. As a results, the pore water pressure may develop progressively in sand deposits. Liquefaction phenomena have been recorded and have developed in may parts of the world where ground shaking is frequent and soils consists of loose fine sand under water table. Mention may be made of Niigata in Japan where liquefaction occurred due to earthquake of 1964. This is one of the most well documented earthquake and a lot have been known about the effects of these earthquake from the studies of Seed and Idriss (1967).

Bangladesh is largely an alluvial plain consisting of fine sand and silt deposits. The ground water table is low in most places. Some low elongated hillocks of soft shale and sand stone of tertiary age exists in the eastern and extreme north-eastern border belt. These rocks however have undergone a fair degree of folding with increase in intensity towards the east. During the recorded history of last one hundred years widespread damages were caused by only the Great Assam earthquake of 1897 which had its epicentral tract in the Shillong Plateau. Two other major earthquakes, the Bengal Earthquake of 1885 and Srimongal earthquake of 1918 caused severe damages only in the limited areas surrounding their epicenters (stuart 1920). Although these earthquakes have well documented records, very limited study exists regarding liquefaction possibilities.

1.2 Seismicity condition of Bangladesh

According to the report of the National Expert Committee on Earthquake (NECE 1979), it can be interpreted that geologically Bangladesh can be divided into two principal tectonic units, one the precambrian platform covering north-western Bangladesh and the other the Bengal Foredeep covering central, Southern and the eastern parts. No earthquake has so far been related to this structural element through the junction between the platform and the foredeep. The Foredeep is terminated in the north-east by the Dauki fault at the southern margin of the Shillong Plateau. Movement along this fault has perhaps caused several major earthquakes in this region. However, the north-eastern parts of Bangladesh are tectonically very active region. According to the suggestion of the NECE (1979), Bangladesh has been divided into three seismic zones as shown in Fig.1.1. The North eastern part that includes the towns of Sylhet, Mymensingh and Rangpur are in zone-I, the town of Dinajpur, Bogra, Dhaka and Chittagong are in zone-II and the

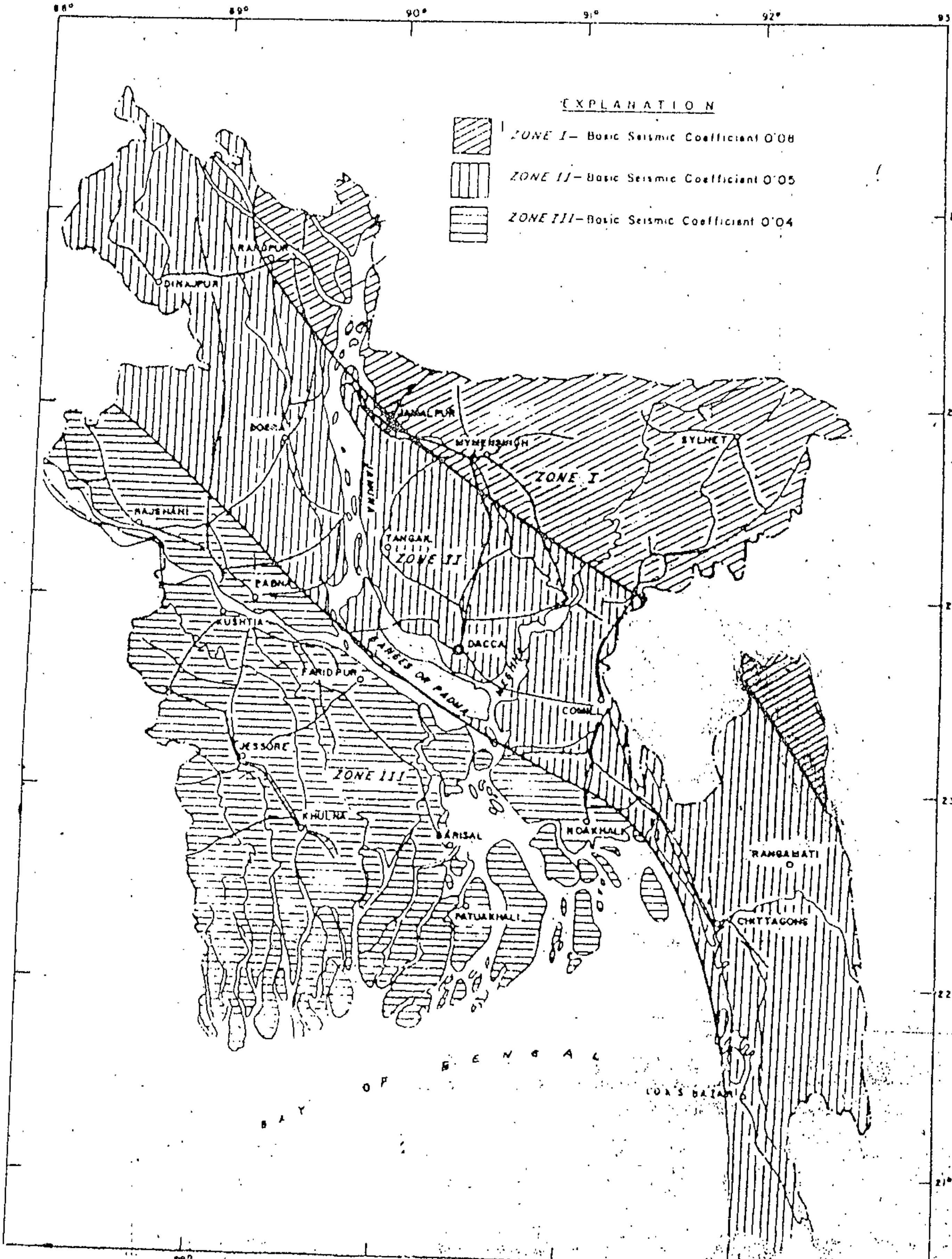


Fig. 1.1 : Seismic Zoning Map of Bangladesh

(After The National Expert Committee on Earthquake, 1979.)

south western part is in zone-III. The extent of damage due to earthquake can be expressed by Modified Mercalli Scale. According to this Scale, the maximum possible earthquake intensity of these zone-I, zone-II, and zone-III are IX, VIII and VII respectively.

1.3 Necessity for the Study of Liquefaction Phenomena

Bangladesh is a developing country. For its economic development and to enhance the living standards of its people a number of projects in which large structures such as Teesta Barrage will have to be built up. These projects require huge investment in money. Therefore, all large structure for such projects should be provided with adequate safety. Such safety measures should include safety against possible liquefaction. This consideration particularly for the case of foundation design of large structures such as earthdams, regulators and barrages, is very important. Any structure, if not properly designed for liquefaction, may lead to its failure or incurr excessive cost through conservative design. A little study has so far been undertaken to establish liquefaction possibilities at local levels in various locations in Bangladesh. In the absence of a definite knowledge of liquefaction possibilities at various localities, difficulties arise in finalising design of many important structures. This adds to the construction time and cost. It was, therefore, felt necessary to undertake a study on the liquefaction phenomena that may develop in various locations in Bangladesh.

1.4 Subject Matter of this Study

Earthequake is a global phenomena. So, it depends on specific geological situation. When an earthquake occurs in the earthcrust, elastic vibrations are setup and transmitted through the earth in all directions. The nature of these vibration can be interpreted by compressional wave and shear wave. In the compressional wave, the particles vibrate back and forth in the direction of wave progression that is alternating pulses of compression and rarefaction through the rock. The wave is called P-wave (Primary wave). In shear wave, the particles vibrate at right angles to the direction of wave . This is called the S-wave (Secondary or shear wave). The third type of wave is a surface wave called L-wave, which is similar to the S- wave without any vertical displacement. The epicenter is the point on the earth's surface directly above the spot where the earthquake originates, faulting and energy release initiates. Epicenters for various earthquakes in the Bengal Basin and Assam region are shown in Fig. 1.2. Most of the north-eastern part of Bangladesh are in the very close to these points. Moreover, the type of soils of these locations are sandy or silty. These north-eastern part of Bangladesh is tectonically very active region. It would be interesting to know how various earthquake wave transmissions such as P, S and L wave effect the soil formation. This inturn would clarify the ideas on possibilities of liquefaction in different soil formations. It was therefore, necessary for this study to review available ideas and explanations. These in subsequently

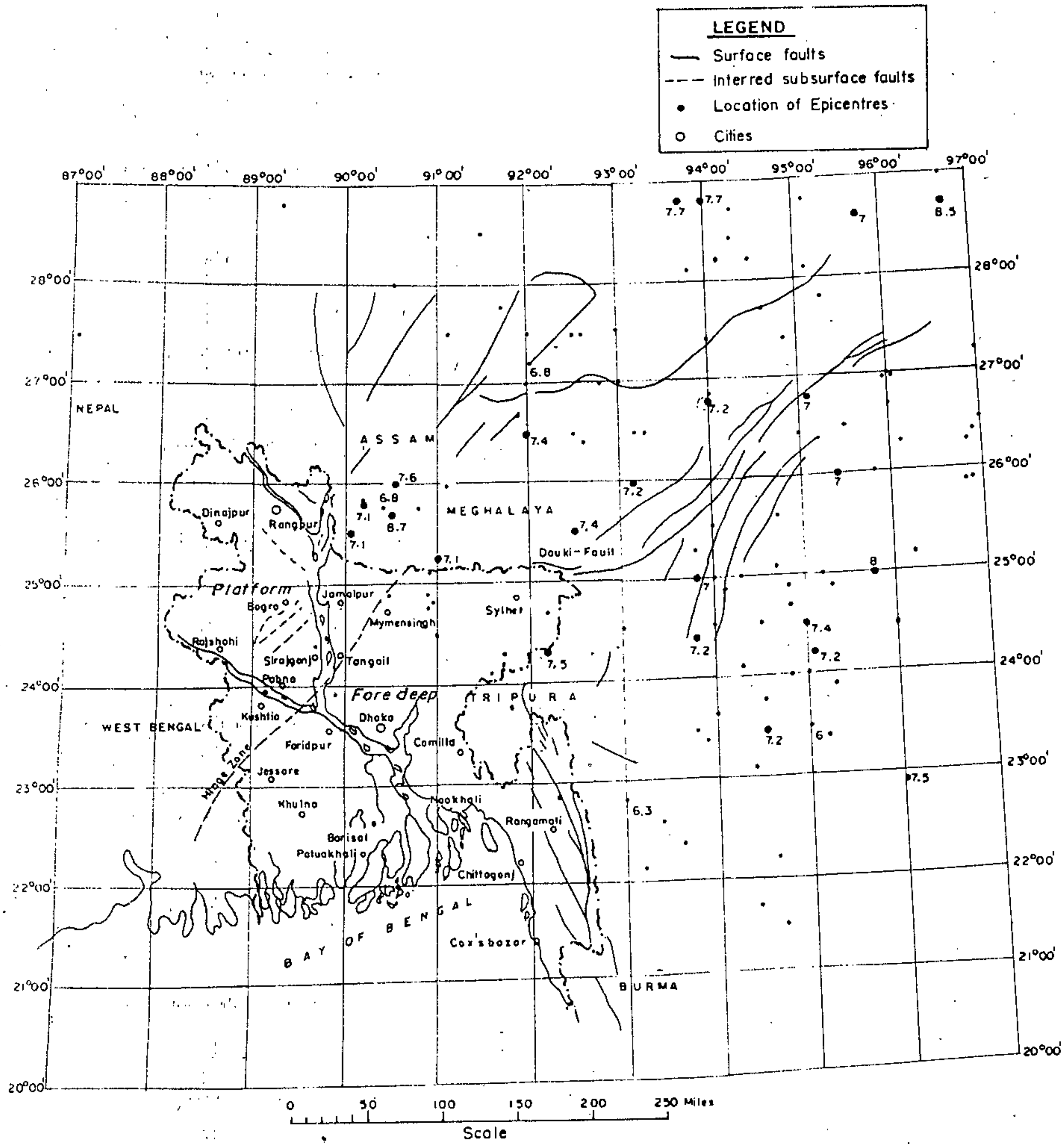


Fig.1.2 : Location of Earthquake Epicentres & Tectonic Features in and Around Bangladesh.

are used to measure liquefaction potential around the north-eastern region of Bangladesh. A number of areas were selected within Bangladesh which were studied for liquefaction probability considering their soil formation, geological environment and earthquake history.

CHAPTER-2

LITERATURE REVIEW

2.1 Introduction

In 1967 Seed and Idriss Studied Niigata earthquake in Japan of 1964 and showed that the soil liquefaction was induced only by cyclic loading during earthquake. During such loading a soil would undergo continued deformation at a constant low residual stress due to build up of high pore water pressure. This pore water pressure built up is the result of cyclic loading which inturn reduces the effective confining pressure to a very low value. The amount of pore water pressure built up will depend on the confining pressure, the type of soil, the formations of the soil properties such as void ratio or relative density. Liquefaction study, therefore, needs the knowledge of geological formations and characteristics of the cyclic stress applications. In this chapter, the geological formations of Bangladesh , the earthquake ground motions and the methods for evaluating the liquefaction potential have been reviewed.

2.2 Geological formation of Bangladesh

The estimation of earthquake hazards have not been attempted seriously in Bangladesh. However, from the report prepared by NECE (1979), it can be said that Bangladesh is underlain by the precambrian basement complex whose surface dips southwards. The basement complex has been recorded at 450 feet depth in Dinajpur district and at 9000 feet at Pabna. Further south it underlies more recent formations to a great depth. The basement complex is underlain by a discontinuous sequence of strata. The near-surface formations belong to the quarternary alluvial complex, which covers most of the country. Only small tertiary inliears in the Quarternary alluvium are found in Sylhet and Comilla districts and bigger tertiary outcrops in the Chittagong regions.

On the basis of plate tectonic theory many geologists assume that the junction between the Indian plate and the Burmese plate runs north-south east of Arakan Ranges in Burma and the eastern Himalayan region as shown in Fig.2.1 (Anwar and Hossain 1980). In the north eastern part of sub-continent, there are two trends one approximately parallel to the Himalayas in Assam and another parallel to the Arakan Yoma ranges in Burma. North eastern Assam where the two trends converage is the area of maximum seismicity. Many large earthquakes have originated along the southern front of the Himalayas in the Shillong plateau region which is an uplifted block of the Indian shield. The Arakan Yoma ranges have also been the location for many shallow focus earthquake. As these regions are adjacent to Bangladesh, these earthquakes have also affected Bangladesh on occasions seriously.

2.3 Physiographic Sub-region of Bangladesh

Bangladesh consists primarily of a large alluvial basin floored with quarternary sediment deposits by the Ganges and

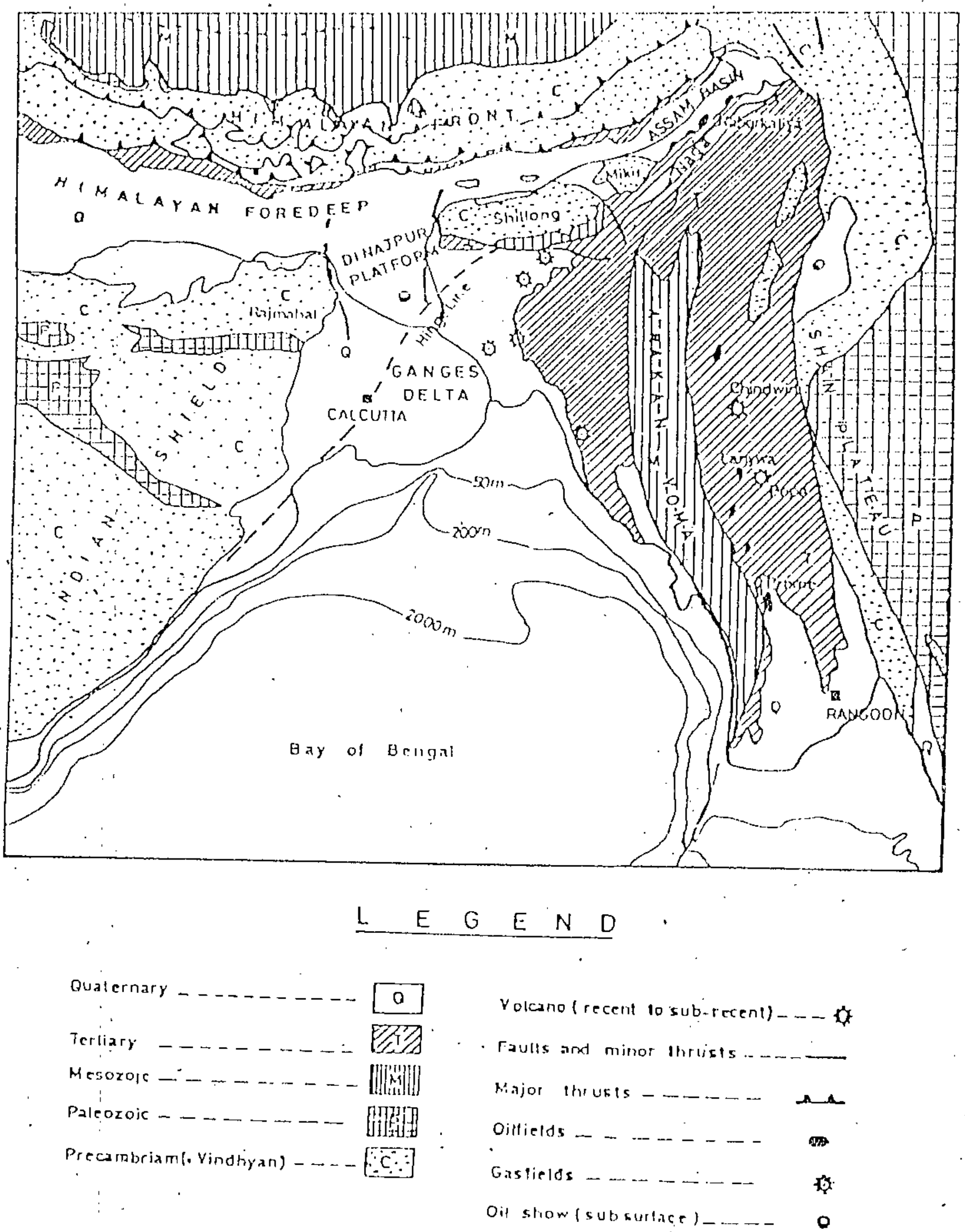


Fig.2.1 : Tectonic Sketch Map of Bangladesh and Surrounding Areas (After, Anwar and Hussain,1980)

Brahmaputra rivers and their numerious associated streams and distributaries. This country is bounded on the north east by the Shillong Plateau and on the eastern units are the Tripura Hills to the north and Chittagong Hills to the south. The Geological Map of India (West,1949) indicates they are composed of sediments of Paleocene through Pliocene age and include sediments representing of Siwalic system. Morgan and Mclntire (1959) studied the physical characteristics of the Bengal basin. This basin covers most of the areas of Bangladesh. According to their study the Bengal basin can be divided into areas of Pleistocene and Recent Flood Plain deposits. Fig.2.2 shows the distribution of above deposits. The four main areas of pleistocene sediments and several smaller outliers stand topographically above the active flood plains. Of the four areas one is east of the Rajmahal Hill system, and the second is west of the folded Tripura Hills. The other two major areas of pleistocene sediments lie within the Bengal Basin and are known as the Barind and the Madhupur Jungle (some times called Terrace deposits). Most of the areas of this country except these four major areas, are recent flood plains. Sediments examined in the pleistocene areas were deposited as flood plains of earlier Ganges Brahmaputra river systems. As far as grain size and mineral content are concerned, pleistocene sediments are atmost identical with those of the recent flood plain. Pleistocene and Recent samples do not differ in maximum grain size, but an excess of very fine-grained material is found in the pleistocene samples. Despite the similarity in grain size between Pleistocene and

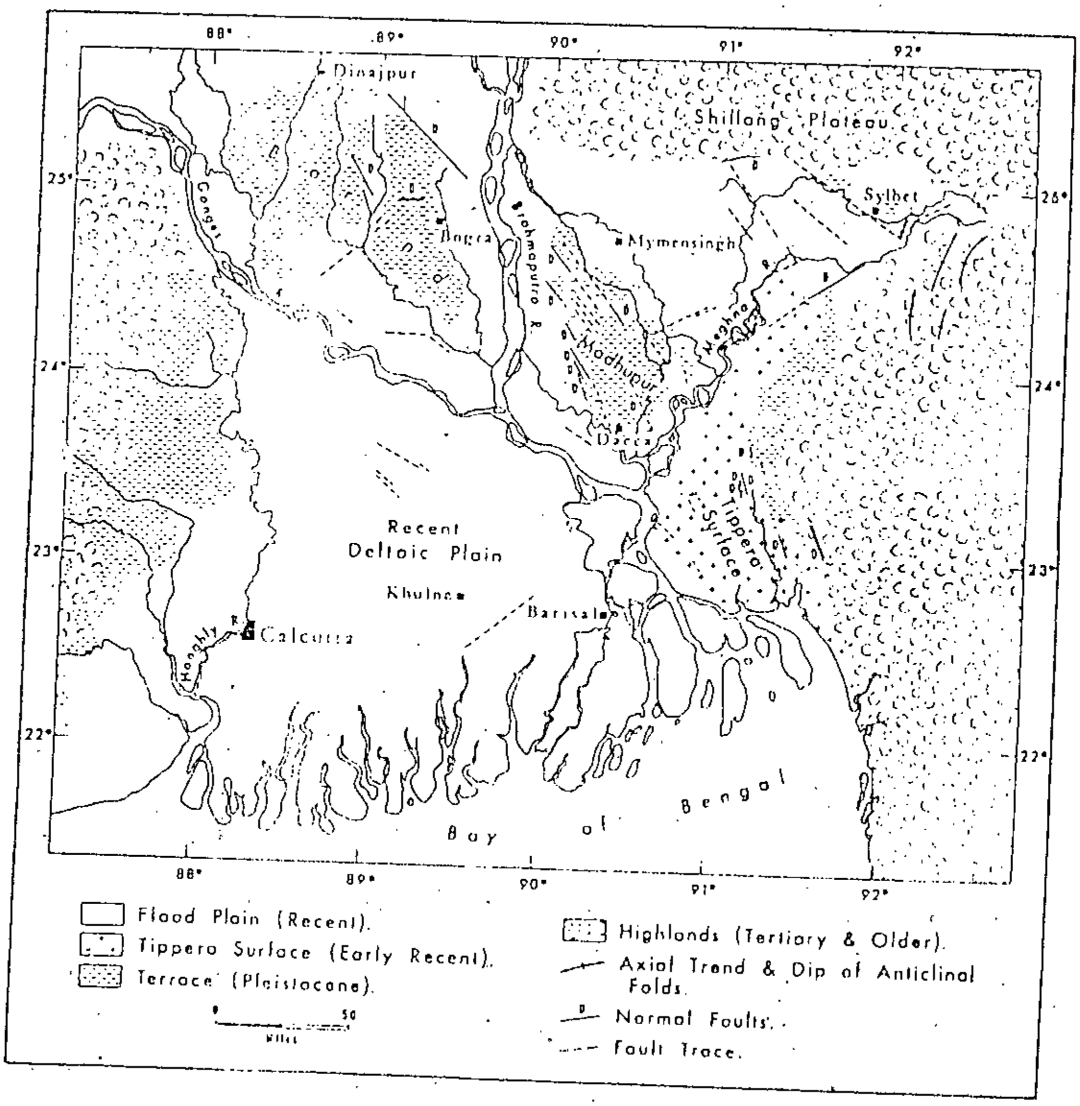


Fig. 2.2 : Quarternary Geology of the Bangal Basin.

(After Morgan McIntire, 1959).

Recent sediments, field differentiation is simple. Recent sediment are typically dark, loosely compacted, and have a high water content and variable, but appreciable quantities of organic material.. Pleistocene sediments, on the other hand, are well oxidized and typically of reddish, brown or tan in colour and are mottled. They commonly contain ferruginous or calcareous nodules. Water content is lower, resulting in firmer, more compacted materials. Organic material in pleistocene sediments is commonly continued to the surface soil. The pleistocene sediments are flood plain deposit of earlier Ganges and Brahmaputra rivers. That they occur in several plains indicates that there has been differential movement between pleistocene and recent time. Except those four pleistocene region, shown in Fig.2.2, the almost all the rest are Recent deposits which comprise the combined deltaic masses and flood plains of the Ganges, Brahmaputra, and Meghna rivers. This Recent deposits are divisible into four categories like alluvial fans, Tippera surface, Sylhet basin and Bengal deltaic plain. The Teesta, Atrai, Mahananda, Purnabhaba, Old Jamuna, and Karatoya rivers which flank and subdivide the Barind pleistocene surface rise as a part of the drainage system of the Himalaya foot hills. These and many other streams of similar nature have developed an extensive piedmont alluvial plain.

In the eastern part of the country flanking the Tripura Hills is a distinctive alluvial physiographic unit (Fig.2.2). This unit 3000 square miles in area, has been named the Tippera

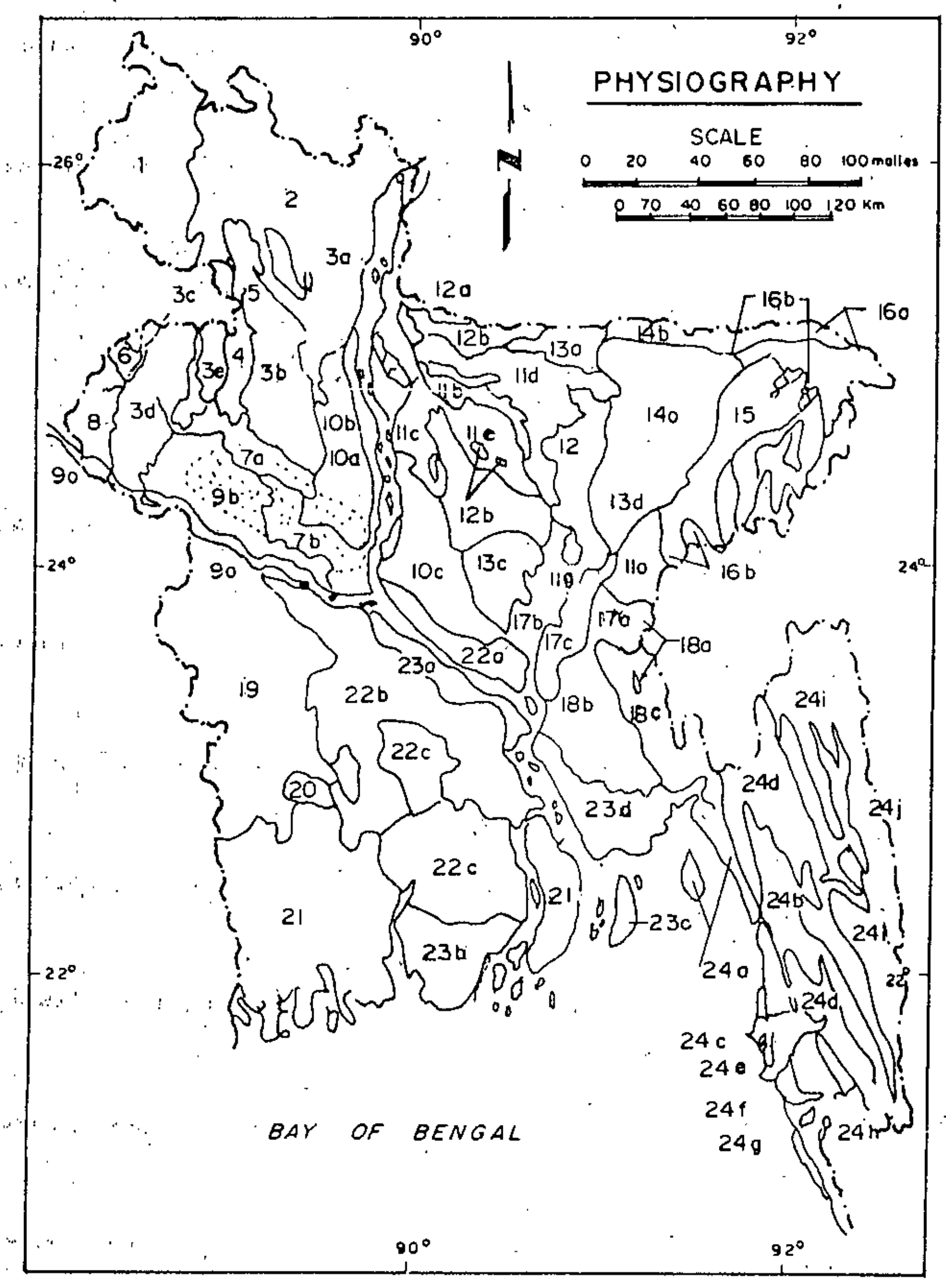


Fig.2.3 : Physiographic Subregions of Bangladesh.

surface. This surface is delineated on the north by an important northeast - southwest trending fault (Fig.2.2). It is uplifted to the southwest a few feet higher than the adjacent flood plain of the Meghna river. A branch of the Meghna impinging against the Tippera surface along the fault scarp has caused appreciable local erosion. Most of the western limit of the Tippera is being eroded by the Meghna or its branches. Along the southwest and southern margin the Tippera is flanked by a narrow band of recent river deposits. The eastern limit of the Tippera surface is the abrupt boundary with pleistocene hills. According to Morgan and McIntire(1959), the sediments of the Tippera surface correspond physically with the Recent flood-plain sediments except that they are more compacted and are slightly oxidized. The sediments display higher oxidation than Recent flood plain deposits but are not comparable with the reddish-brown pleistocene terrace materials.

The Sylhet Basin has been bounded on the north by the Shillong plateau and on the east and south by the plunging, anticlinically folded Tripura Hills. The south flank of the Shillong plateau is delineated in part by a major fault scarp. According to Morgan and McIntire (1959), it has been suggested that the Sylhet Basin has subsided about 30-40 feet within the last several hundred years. Although sediments are probably undergoing compaction, the subsidence of the Sylhet Basin is primarily tectonic and is related to the major fault systems bounding the northern side of the basin. The many lakes immediately adjacent to the fault scarp suggest that the downthrown block has been rotated, carrying the Sylhet Basin down more rapidly than it is being alluviated. The remaining region of Bangladesh, bounded by the pleistocene terrace on the west, the Barind and Madhupur Jungle on the north, and the Tippera surface on the east, consists of the Recent deltaic plain. It is complex in that it has been the site of sedimentary deposition by two of the world's major rivers. It is composed of a number of overlapping sub-deltas. The deltaic region is constantly subsiding, owing mainly to compaction of Recent sediments and possibly to structural downwarping.

Rashid (1977) mentioned five physiographic regions of the Bengal Basin. Of these three only fall within Bangladesh. Bangladesh can again be divided into twenty four sub-regions and fifty four units on the basis of physical features and drainage pattern. Fig. 2.3 shows region and sub-regions with units that fall within Banglades. The characteristics of these sub-regions are described in Table 2.1.

Table 2.1

Physiographic sub-regions and it's characteristioes of deposits

Sl. No.	Sub-regions	Units	Characteristics
1.	Himalyan Piedmont	Himalyan Piedmont Plain	Piedmont deposits
2.	Tista Flood Plain	Tista Flood Flood	Inerstream alluvial deposits
3.	Barind Traet	a. North-eastern oulier	Older alluvial
		b. Eastern Barind	Older alluvial
		c. West central barind	Older alluvial
		d. Western barind	Older alluvial
4.	Little Jamuna flood plain	Little Jamuna flood plain	Inter-stream alluvial deposits
5.	Middle Atrai flood plain	Middle Atri flood plain	Inter-stream alluvial deposits
6.	Lower purnabhaba Valley	Lower Purnab-Valley	Inter-stream alluvial deposits
7.	Bhar Basin	Western	Swamp deposit
8.	Lower Mahananda flood plain	Lower Mahananda flood	Terrace and meander deposits
9.	Ganges flood plain	a. Diaras and chars	Terrace meander deposits
		b. North Ganges old flood plain	Inter-steam alluvial deposits
10.	Brahmaputra Jamuna flood plain	a. Bangali-Koratatoa flood plain	Inter-steam alluvial deposits
		b. Diaram and chars	Inter-steam alluvial deposit

Table 2.1

Physiographic sub-regions and it's characteristioes of deposits

Sl. No.	Sub-regions	Units	Characteristics
		C. Jamuna -Ka-igonj flood plain	Inter-steam alluvial deposits
11.	Old Brahmaputra flood plain	a. High ridges	Older alluvial
		b. Flood plain complex	Swamp and deltaic deposit
		C. Western plain	Terrace and meander deposits
		d. Northern plain	Inter-steam alluvial deposits
		e. Southern plain	Inter-stream alluvial deposits
		e. Eastern plain	Terrace and meander deposits
		f. South-eastern plain	Terrace meander deposits
12.	Susang Hills and Piedmont	a. Susang Hills	Sylhet limestone
		b. Piedmont plains	Piedmont deposits
13.	Madhupur Tract	a. Nothern Tract	Older alluvial
		b. Central Tract	Older alluvial
		c. Southern Tract	Older alluvial
14.	Haor Basins	a. Central Basin	Recent deposits
		b. Susang piedmont basin	Piedmont deposits

Table 2.1

Physiographic sub-regions and it's characteristioes of deposits

Sl. No.	Sub-regions	Units	Characteristics
		c. Meghalaya Piedmont depoession	Piedmont deposits
		d. Central Sylhet lowland	Recent deposits
15.	Sylhet High & Plain	Sylhet High Plain	Piedmont deposits
16.	Sylhet Hills	a. Meghalaya foot hills	Piedmont deposits
		b. Tile ranges	Dining and Dupitilc formations
		c. Meghalaya Piedmont depoession	Piedmont deposits
17.	Meghna flood plain	a. Titas Basin low plain	Inter-stream alluvial deposite
		b. Meghna Lak-kha Doab	Terrace and meander deposits
		c. Middle Meghna flood plain	Inter-stream alluvial deposits
18.	Tippera surface	a. Eastern piedmont strip and Lalmai range	Older alluvial
		b. Low flood plain	Inter-stream alluvial deposits
		c. High flood plain	Inter-stream deposits
19.	Moribund Delta	Moribund Delta	Terrace and Meaunder deposits

Table 2.1

Physiographic sub-regions and it's characteristioes of deposits

Sl. No.	Sub-regions	Units	Characteristics
20.	Central Delta Basins	Central Delta Basins	Inter-stream alluvial deposit
21.	Immature Delta	Immature Delta	Swamp and Deltaic deposits
22.	Mature Delta	a. Old Ganga flood plain	Deltaic deposits
		b. Padma-Madh-umati flood plain	Deltaic deposits
		c. Saline tidal flood plain	Tidal flat deposits
23.	Active Delta	a. Active Padma flood plain	Deltaic deposits
		b. Mehendiganj Islands	Tidal flat deposits
		c. Meghna estuary islands and chars	Swamp and deltaic deposits
		d. Meghna esturine flood plain	Deltaic deposits
24.	Chittagong Region	a. Northern coastal plain	Beach deposits
		b. Central valley	Piedmont deposits
		c. Matamori delta and coastal islands	Swamp and deltaic deposits
		d. Western Hills	Dihing and Dupitilo formation

Table 2.1

Physiographic sub-regions and it's characteristioes of deposits

Sl. No.	Sub-regions	Units	Characteristics
		e. Middle Karnafuli System Valleys	Surma Formations
		f. Bakkhali river valleys	Tipam formations Beach deposits
		g. Nhila-Teknaf plain	Beach deposits
		h. Southern beach plain	Surma formations
		i. Jinjira islets and reefs	Surma formations
		j. Mountain ranges and eastern hills	Dihing and dupitilo formations

2.4 Earthquake Ground Motions

Information of earthquake ground motion characteristics is obtained from accelerograph records of past earthquakes. Generally, any typical accelerograph records shows the acceleration time history of the ground in two horizontal directions (at right angles to each other) and the time history of vertical accelerations. Each component of horizontal motion can readily be integrated with respect to time as to obtain first the time history of ground velocity at the recording station and then the time history of displacements. Therefore, from accelerograph records the main characteristics of the ground motions, such as the maximum ground acceleration, ground velocity, ground displacement and duration of significant ground shaking can readily be read. These characteristics of earthquake ground motions at any site are influenced by a number of factors including magnitude of the earthquake, distance of the site from the source of energy release, geologic characteristics of the rocks along the wave transmission path, source mechanism of the earthquake, wave interference effects related to the direction and speed of fault rupturing and soil conditions at the site (Seed and Idriss, 1982). The variation of intensity of ground motions with distance from the source of energy release called attenuation curve have been studied for many years. Plots of peak acceleration as a function of distance for different earthquake are called acceleration attenuation curves and have been studied by numerous workers such as Gutenberg and Richter (1954), Housner(1952), Seed et al (1969), Schnabel and Seed(1972). While there has been reasonable agreement about the form of such attenuation curves proposed by different workers for earthquakes with magnitude of 6 to 7 and for distance of 25 to 60 kms, there have often been marked differences of opinion concerning accelerations in the nearfield and those developed by higher magnitude events. The main reason for these differences was the lack of data for large magnitude at field recording stations.

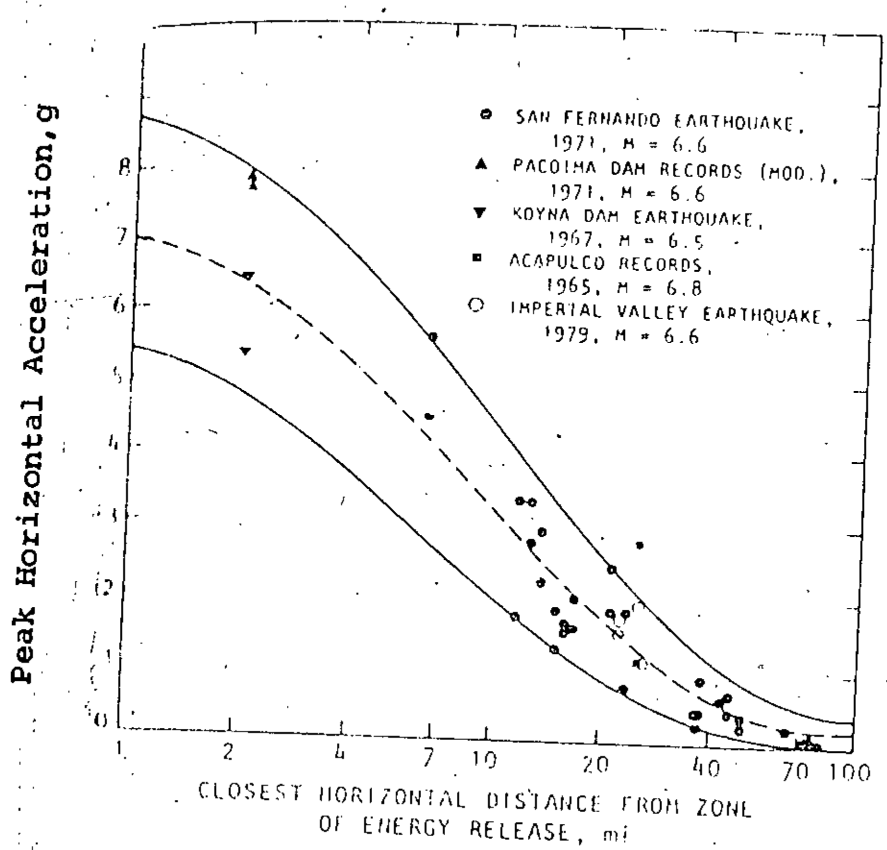


Fig. 2.4 Maximum accelerations in rock for earthquake with magnitude

(After Seed & Idriss, 1982)

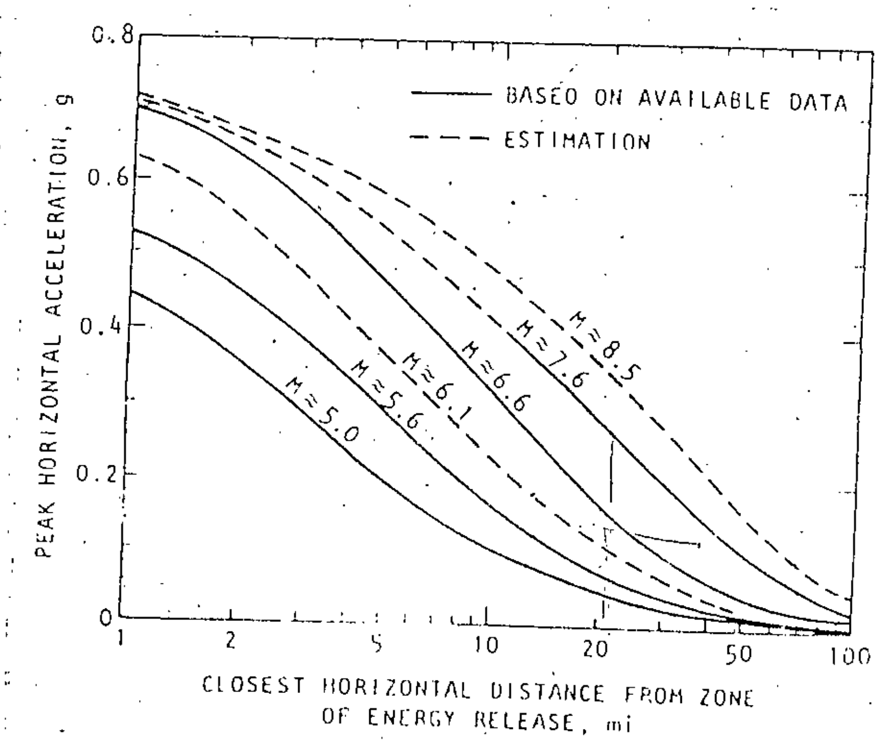


Fig. 2.5 : Average value of maximum accelerations in rock.

(After Seed & Idriss,1982)

2.4.1 Acceleration Attenuation Curves in Rocks

An Attenuation relationship based on records of peak acceleration recorded only on rocks for an earthquake of magnitude 6.6 Richter scale is shown in Fig.2.4. Although the near field data for this figure are not so abundant, it nevertheless defines an attenuation curve of similar shape. The dotted curve in Fig. 2.4 may be considered a general attenuation curve. Attenuation curves for motion on this type of rocks for different earthquake magnitudes have been developed by Seed and Schnabel (1972) and the resulting plots are shown in Fig.2.5. Families of curves of this type can be used to estimate peak accelerations at different distances from different magnitude earthquakes.

2.4.2 Acceleration Attenuation Curves in Soils

The influence of soil conditions on peak ground accelerations can be determined by comparing the mean peak acceleration attenuation curve for deep soil deposite with that for acceleration recorded on rock sites. This comparison is shown in Fig.2.6. It can be seen that at comparable distances from the source, peak acceleration recorded on rock are somewhat higher than those recorded on deep alluvium. At lower acceleration levels, acceleration on deep soil deposits seem to be higher than those on rock. The results of a detailed study of the relative values of peak accelerations developed on four different types deposits: Rock; stiff soils deposits involving cohesionless soils or stiff clays to about 200 ft in depth;

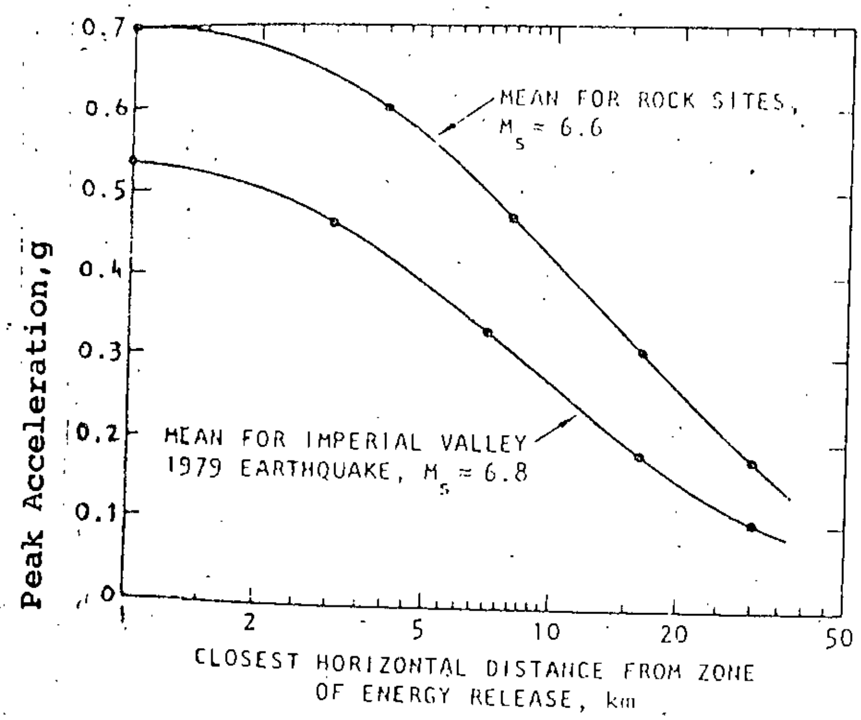


Fig.2.6 Comparison of attenuation curves for rocksites and deep soil deposits (Imperial Earthquate).

(After Seed & Idriss, 1982.)

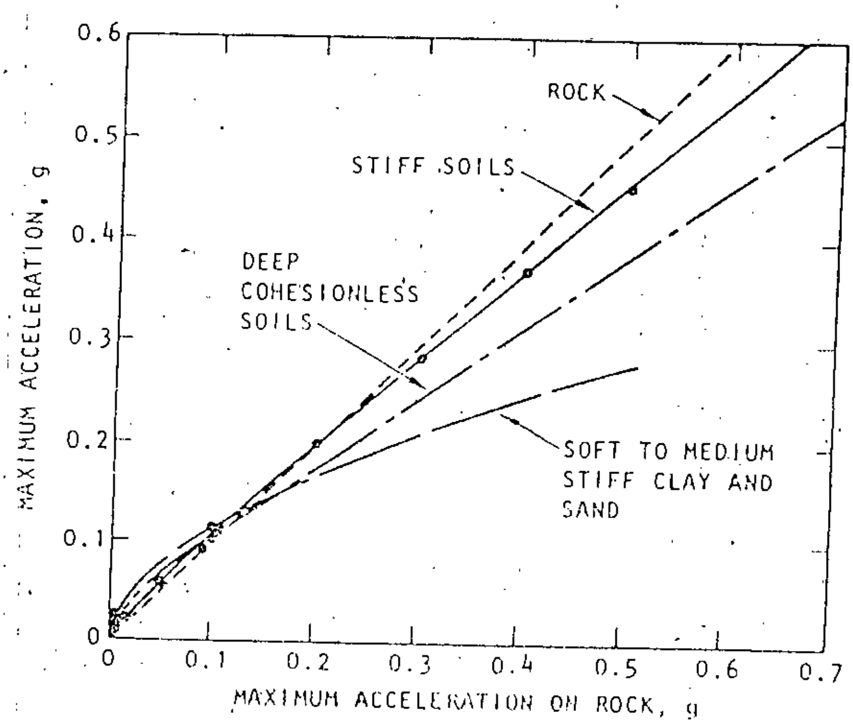


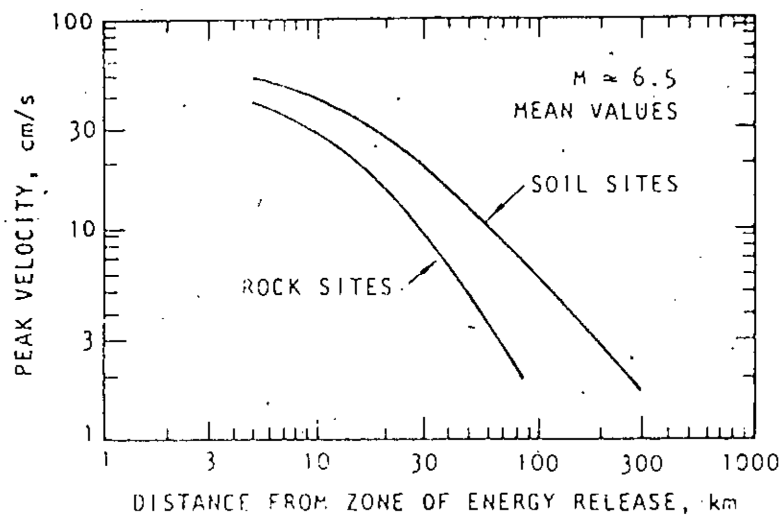
Fig. 2.7 Approximate relationships between maximum accelerations on rock and other local site conditions.

(After Seed & Idriss, 1969)

deep cohesionless soil deposits with depths greater than about 250 ft; deposits of soft to medium stiff clays and sands are shown in Fig. 2.7 taken from Seed and Idriss(1969). It may be seen that apart from deposits involving soft to medium stiff clay, values of peak acceleration developed on different types of soil do not differ appreciably, particularly at acceleration levels less than about 0.3g to 0.4g. Even at high acceleration levels on rock of the order of 0.7g, acceleration on deposits of any depth not involving soft to medium stiff clays are likely to be only about 25% less than those on rock. Variations of this magnitude may not be significant in engineering practice and for most practical purposes it may well be considered that peak acceleration values on rock and stiff soils of any depth are about the same.

2.4.3 Velocity Attenuation Curves

In the same way that peak acceleration data can be plotted to determine acceleration attenuation curves, values of peak velocity determined from recorded accelerograms can be plotted to determine velocity attenuation curves. A detailed study of such curves by Sadigh et al (1979) has led to the results shown in Fig.2.8 for Magnitude M =6.5 Richter scale earthquakes and velocities recorded on rock and soil conditions. In this case it may be seen that the local soil conditions can have a pronounced effect on the peak velocity developed, with velocity values on soil deposits typically being about twice those recorded on rock sites.



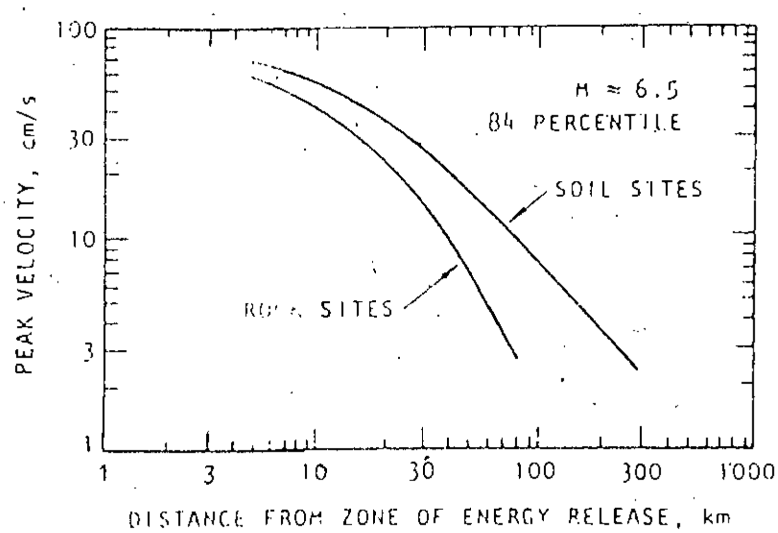


Fig.2.8 Variation of maximum ground velocity with distance for different site conditions.

(After Sadigh et al, 1979)

2.5 Methods for Evaluating Liquefaction Potential of Sand Deposits

There are mainly two methods available for evaluating the liquefaction potential of a deposit of saturated sand subjected to earthquake shaking. One method relies on performance of sand deposits in previous earthquakes, another method is based on evaluation of stress conditions in the field from a comparison of stresses causing liquefaction of soils in the laboratory. These methods are described below.

2.6 Methods Based on Observation of Sand Deposits Previously Subjected to earthquake

It is evident that geotechnical engineers took serious interest in the general phenomenon of earthquake induced liquefaction after Alaska and Niigata earthquakes of 1964. In Niigata Earthquake, a number of Japanese engineers studied the liquefaction phenomenon by correlating primarily with field standard penetration resistance of the sand deposits. Subsequently a more comprehensive study, of actual site conditions that lead to liquefaction or no liquefaction was made by Seed and Peacock(1971). Based on their findings criteria for liquefaction potential of a soil deposit were developed and these provided a basis for the relationship between field values of cyclic stress ratio τ_h/σ_o' and relative density of sands. The cyclic stress ratio may be defined as the ratio of shear stress (τ_h) developed due to horizontal ground shaking during earthquake to the initial effective over burden pressure (σ_o'). The relative density of sand deposits are to be evaluated from the standard penetration values correlated for depths as proposed by Gibbs and Holtz(1957). Later on, Castro(1975), Christian and Swiger(1975) also collected the field data to provide a basis for evaluating liquefaction potential by providing other correlation between liquefaction producing parameter and penetration resistance. In 1977, Seed, et al collected the values of stress ratio known to be associated with some evidence of liquefaction or no liquefaction in the field and established a relationship between these values and the corrected average Standard Penetration Resistance N_1 of the sand deposits. This relationship is shown in Fig.2.9. In this figure the horizontal axis represents the Standard Penetration Values corrected for field overburden pressure as proposed by Mercuson and Bieganousky(1977) for the soil strata in question. The details of the method of N-value correction is presented in Article 4.5. The vertical axis represents the cyclic stress ratio at the point where liquefaction is considered. The farm line in Fig.2.9 provides a boundary seperating liquefaction and no liquefaction zones. The upper portion of this line provides a condition of liquefaction and lower portion of no liquefaction. Based on this figure liquefaction phenomenon can be assessed by determining the probable value of cyclic stress ratio that would develop during earthquake and the corresponding corrected penetration resistance of the sand deposits.

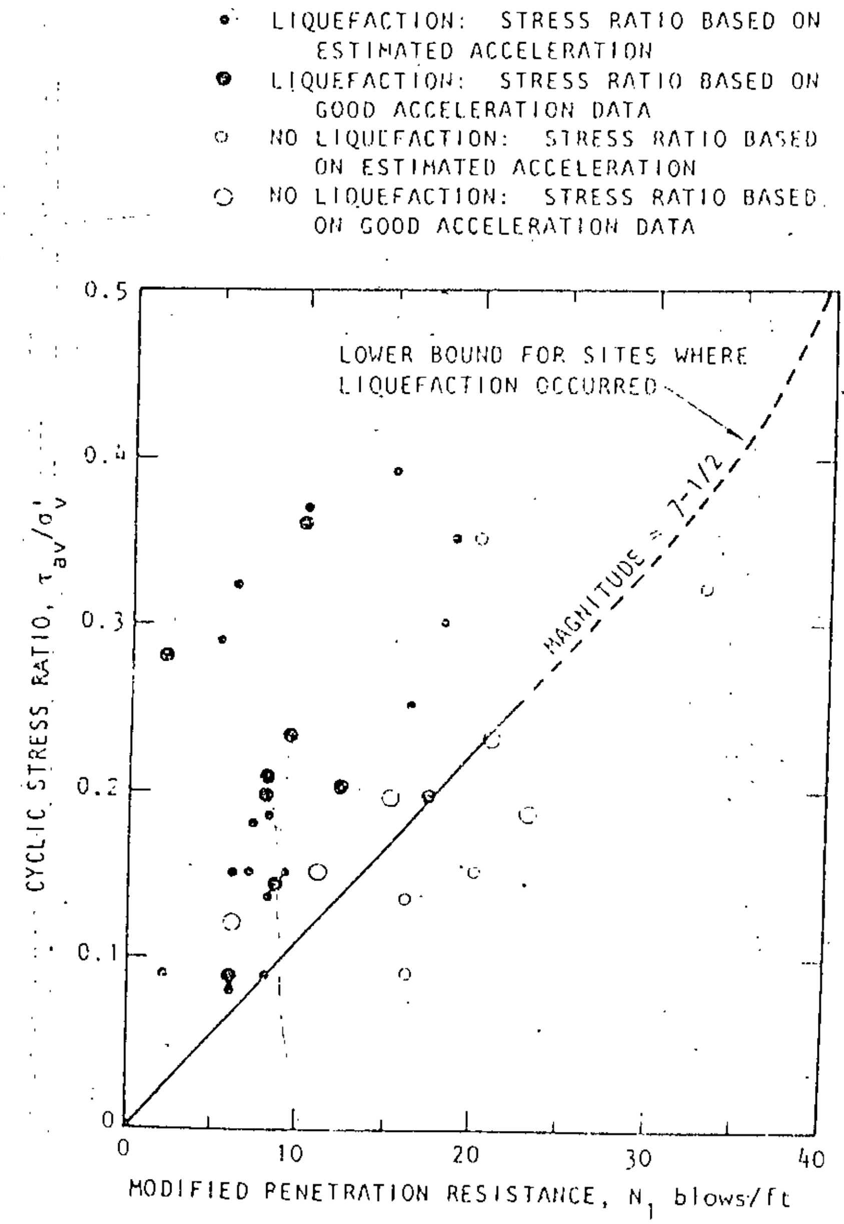


Fig. 2.9 Correlation between stress ratio causing liquefaction in the field and penetration resistance of sand.

(After Seed, et al, 1977)

The cyclic stress ratio at any depth in the ground causing liquefaction can be determined with acceptable accuracy by the method proposed by Seed and Idriss(1971). According to their method the shear stresses developed at any point in a soil deposit during an earthquake appear to be due primarily to the vertical propagation of shear waves in the deposit. If the soil element at depth h shown in Fig.2.10(a), behaved as a rigid body, the maximum shear stress on it would be

$$(\tau_{max})_r = \frac{\gamma h}{g} a_{max} \qquad (2.1)$$

$(\tau_{max})_r$ = maximum shear stress when soil element behaved as a rigid body.

a_{max} = maximum ground surface acceleration, as shown

γ = unit wt. of the soil.

Because the soil column behaves as a deformable body, the actual shear stress at depth h is $(\tau_{max})_d$ and will be less than $(\tau_{max})_r$ as determined by a ground response analysis might be expressed by

$$(\tau_{max})_d = r_d . (\tau_{max})_r \qquad (2.2)$$

where, r_d = a stress reduction coefficient with a value less than 1.

Seed and Idriss (1971) suggested a typical variation of $(\tau_{max})_r$ and $(\tau_{max})_d$ in the form shown in Fig.2.10(b). The value

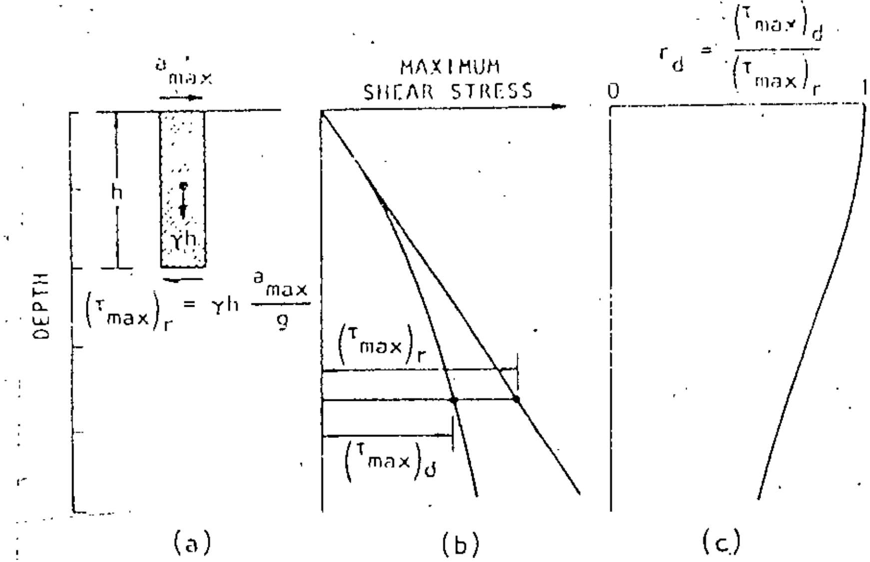


Fig.2.10 Procedure for determining maximum shear stress

(After Seed & Idriss, 1982)

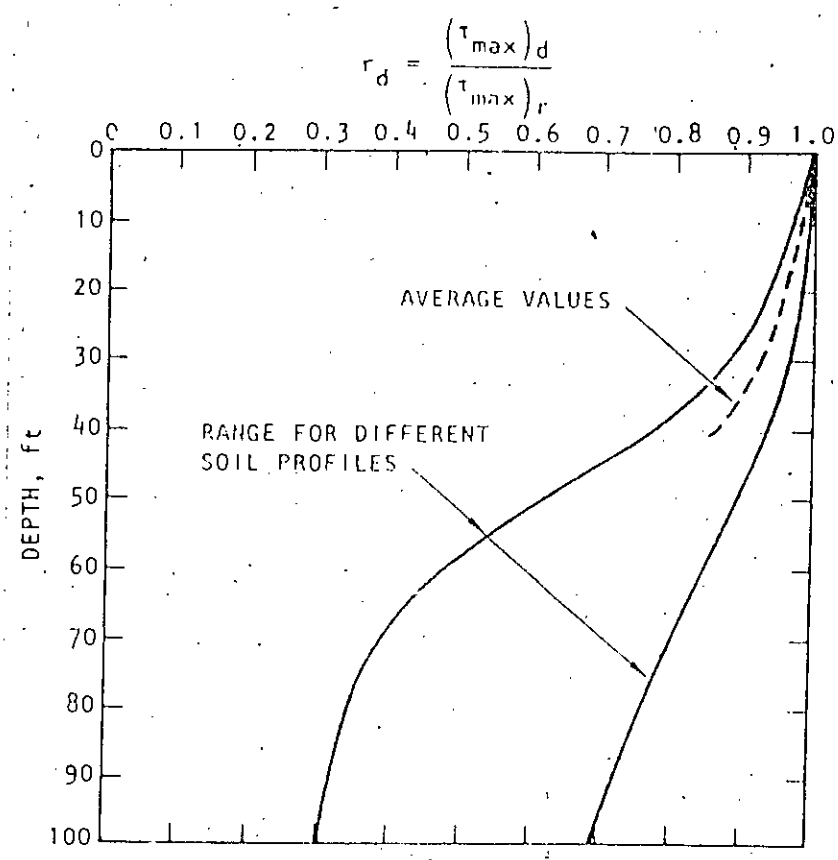


Fig.2.11 Range of values of r_d for different soil profiles.

(After Seed & Idriss, 1971)

of r_d will decrease from a value of 1 at the ground surface to much lower values at large depths as shown in Fig.2.10(c). Computations of the value of r_d for a wide variety of earthquake motions and soil conditions having sand in the upper 50 ft. have shown that r_d generally falls within the range of values shown in Fig. 2.11 proposed by Seed and Idriss(1971). It may be seen that in the upper 30 ft. or 40 ft. the scatter of the results is not great (ranges between 0.85 to 0.95), the error involved in using the average values shown by the dashed line would be less. On this basis Seed and Idriss(1971) suggested that for a depths of about 40 ft. the maximum shear stress developed during an earthquake can be made from the relationship

$$\tau_{max} = \frac{\gamma h}{g} \cdot a_{max} \; r_d \qquad (2.3)$$

where values of r_d are taken from the dashed line and ranges between (0.85 to 1). In order to testify the liquefaction criteria, the equivalent average shear stress is to be obtained from the value of τ_{max}. The actual time history of shear stress at any point in a soil deposit during an earthquake will have an irregular form such as that shown in Fig.2.12. From such relationships it is necessary to determine the equivalent uniform average shear stress. By appropriate weighting of the individual stress cycles, this determinations for a number of different cases Seed and Idriss(1971) found that with a reasonable degree of accuracy, the average equivalent uniform shear stress, τ_{av}, is about 65% of the

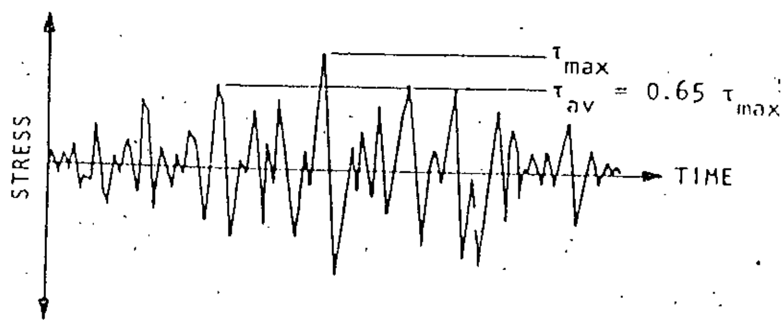


Fig.2.12 Time history of shear stresses during earthquake.

(After Seed & **Idriss**, 1982)

maximum shear stress τ_{max} Thus

$$\tau_{av.} = .65 \frac{\gamma h}{g} a_{max} \, r_d \qquad (2.4)$$

Therefore, the cyclic shear stress developed on a soil element during an eartthquake shaking can be expressed by

$$\frac{\tau_{av}}{\sigma'_o} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma'_o} r_d \qquad (2.5)$$

where σ_o = Total overburden pressure on sand layer under consideration;

σ'_o = effective overburden pressure on sand layer under consideration.

Thus for any given site and a given value of maximum ground surface acceleration, the possibility of liquefaction can readily be checked by the criteria shown in Fig. 2.9.

2.7 Methods Based on Field Soil Stress Condition and Laboratory Comparison

Analytical procedures for evaluating the liquefaction potential of soil deposits were first proposed by Seed and Idriss(1971) and involved two independent determinations. An evaluation of the cyclic stresses induced at different levels in the soil deposit by the earthquake shaking and a laboratory investigation to determine the cyclic stresses which, at given confining pressure, will cause the soil to develope a peak cyclic pore pressure ratio of 100%. The term "peak cyclic pore pressure ratio of 100%" denotes a condition where, during the course of cyclic stress applications, the residual pore water pressure on completion of any full stress cycle becomes equal to the applied confining pressure. The evaluation of liquefaction potential was then based on a comparison of the cyclic stresses induced in the field with the stress required to cause a peak cyclic pore pressure ratio of 100% in representative samples in the laboratory. This type of stress analysis-property determination approach requires a five steps procedures :

(i) Development of a suitable analytical procedure for evaluating the stresses developed in a potentially liquefable layer in the ground during a given earthquake;

(ii) Development of a suitable procedure for representing the irregular stress history produced by the earthquakes by an equivalent uniform cyclic stress series;

(iii) Development of a suitable test procedure for measuring the cyclic stress conditions causing a peak pore pressure ratio of 100% in representative samples of soil;

(iv) Development of an understanding of all the factors having a significant influence on the liquefaction characteristics of soil; and

(v) Development of an understanding of the effects of sample disturbance on the in-situ properties of natural deposits. Clearly this approach is more appropriate to the soil engineer than the use of the empirical approach described previously . In the following pages, procedures developed for accomplishing the five basic steps in the above procedures will be briefly reviewed.

2.7.1 Evaluation of Stresses Induced in Ground due to Earthquake Shaking

The cyclic shear stresses induced in the ground by an earthquake may readily be computed by several methods. In 1967 Seed and Idriss computed cyclic shear stresses by a ground response analysis that neglects the pore pressure build up as the earthquake progresses. Seed and Idriss (1971) also developed a method for calculating the cyclic shear stresses by a simplified procedure based on a knowledge of the maximum ground surface acceleration. Later, Finn, et al. (1977) and Liou, et al. (1977) introduced a procedure that takes into account the pore pressures generated in the soil as the earthquake progresses. In general it does not appear that the effects of the pore pressure build up on the computed stresses in a soil deposit, up to the onset of liquefaction if it occurs, are particularly significant and thus reasonable evaluations can usually be made by methods that do not take these pore pressures into account. However, in some cases this may be expected to lead to somewhat conservative results. The use of methods that do consider these pore pressures leads to a direct evaluation of the liquefaction potential of the deposit without further studies; as such they represent the most sophisticated approaches currently available. They do require the determination of more material properties in order to make the analyses and some of them are vulnerable to testing errors or require further study before they are fully understood. For this reason Seed (1979) showed that the simplified procedure is more practical and less vulnerable alternative at the present time. According to the simplified procedure proposed by Seed and Idriss(1971) the maximum shear stress developed in a soil deposit during an earthquake shaking can be given by an equation (2.3).

2.7.2 Converting Irregular Stress Histories into Equivalent Uniform Cyclic Stress Series

In the more common method of approach, where the stresses induced in the ground are compared directly with those determined to cause liquefaction of representative soil samples, it is usually necessary to convert the irregular stress history into an equivalent uniform cyclic stress series. This is so because it is usually more convenient to perform laboratory tests using uniform cyclic stress applications than to attempt to reproduce the actual field stress history. The general validity of this approach has been demonastrated by detailed studies by Annaki and Lee (1977). There are three basic methods by which it can be accomplished:

a) By estimation from a visual inspection of the irregular time history involved ; surprisingly this can be accomplished quite accurately with little experience.

b) By weighting procedure for individual stress cycles developed by Seed et al. (1969) and described by Lee and Chan (1972), Seed at al. (1969) and Annaki and Lee (1977). The basic procedure involves the use of an experimentally determined pore pressure response liquefaction curve to establish the relative effects of different numbers and levels of cyclic stress applications and the final determination of an equivalent number of uniform stress cycles (i.e., constant amplitude and frequancy) representative of any given irregular sequence.

c) A cumulative damage approach developed by Donovan (1971) based on Miner's law, and involving the natural period of the deposit and the duration of the earthquake shaking. The method is described in detail by Valera & Donovan (1977).

Valera and Donovan (1977) represented a detailed comparison of the effects of the different methods of evaluating the effects of an irregular stress history using data obtained from cyclic load tests conducted with uniform stress applications. Later, Seed,(1979) shows that different procedures that may be used in this step of the analysis have little effect in the final result.

By adopting a representative average weighting curve, Seed, et al (1975) have made a statistical study of the representative numbers of cycles for a number of different earthquake motions. These typical results are shown in Table

2.2 and provide a convenient basis for selecting an equivalent uniform cyclic stress series for earthquake of different magnitudes.

Table 2.2

Equivalent uniform stress cycles for different magnitudes of Earthquakes.

Earthquake magnitude	No. of repesentative cycles at 0.65 T_{max}.
5 $\frac{1}{4}$	2-3
6	5
6 $\frac{1}{4}$	10
7 $\frac{1}{2}$	15
8 $\frac{1}{2}$	26

2.7.3 Development of Suitable Test Procedures for reproducing earthquake effect.

The most desirable laboratory test procedures for reproducing the effects of an earthquake on soil test samples is a cyclic loading simple shear test. This procedure provides a reasonably close simulation of the stresses induced on a soil element by one component of earthquake motion in the field. In the early stages of cyclic stress applications, pore water pressures build up in the sample but there is no significant deformation. After a number of application, the pore pressure suddenly jumps to a value equal to the vertical confining pressure, reducing the effective pressure to zero and at the same time the sample begins to undergo large cyclic deformation. These denotes the onset of liquefaction. The number of stress cycles required to cause the sample to liquefy depends on the magnitude of the total applied shear stress and the initial vertical effective pressure under which the sample is consolidated. Typical results of a series of tests on identical samples of a sand are shown in Fig.2.13. From the plot it is easy to read off the cyclic shear stress ratio, causing liquefaction in the number of stress cycles representative design earthquakes. In laboratory tests, the samples is in unidirectional stress condition; but the soil in the field are in multi-directional stress condition. Under multi-directional stress condition, pore pressures build up faster than under unidirectional stress condition. Seed has shown that the stresses required to cause liquefaction in unidirectional cyclic simple shear tests should be reduced by about 10% to provide results representative of multi-directional shear condition.

Since equipment for conducting any type of simple shear test is somewhat complicated and not readily available, the cyclic loading triaxial compression test was developed by Seed and Lee (1966) to provide a practical and convenient alternative. In the performance of these tests to represent level ground conditions in the field, samples are first

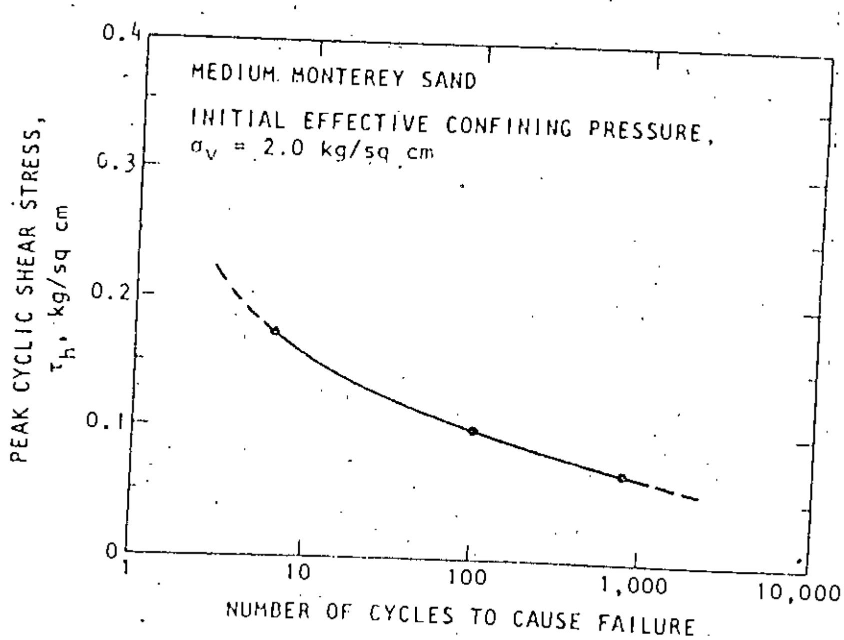


Fig. 2.13 Typical form of the relationship between cyclic shear stress and the number of cycles to cause failure - simple shear condition.

(After Seed & Idriss , 1982)

consolidated by an ambient confining pressure, σ_a (the lateral earthpressure coefficient, $K_o = 1$) and then subjected to cyclic deviator stress applications, σ_{dc}. Test data can be interpreted to determine the cyclic stress ratio, $\sigma_{dc}/2\sigma_a$ which causes liquefaction in the desired number of cycles. Tests of this type do not represent field conditions nearly as well as do cyclic simple shear tests but it has been found that the cyclic stress ratio, τ_h/σ'_v, causing liquefaction under multidimensional shaking conditions in the field can be related to the cyclic stress ratio causing liquefaction of a triaxial test sample in the laboratory by the expression (Seed, 1979).

$$\left(\frac{\tau_h}{\sigma'_v}\right)_{t\ \text{field}} = C_r \left(\frac{\sigma_{dc}}{2\sigma_a}\right)_{t\text{-traxial}} \qquad (2.6)$$

Where values of C_r are

$$C_r = \begin{cases} 0.57 \text{ for } k_0 = 0.4 \text{ and} \\ 0.9 \text{ to } 1 \text{ for } k_0 = 1 \end{cases}$$

Experience indicates that carefully conducted cyclic triaxial tests performed and interpreted in this way can provide valid data on the cyclic loading characteristics of sands up to the development of pore pressure ratios of 100% for the samples. Reliable data connot be obtained if nonuniform conditions exist in the initial sample placement in the triaxial cell. However, meaningful results can be obtained only if tests are performed on undisturbed samples representative of the in-situ deposit.

2.7.4 Factors influencing liquefaction characteristics of soils

Liquefaction potential of cohesionless soil are influenced not only by the density of the deposit but also by such factors as the characteristics of the grain composing the deposit, the coefficient of lateral earth pressure, and the age of the deposit. Accordingly, importance has correctly been given to the necessity for obtaining and testing truely, undisturbed and representative samples if meaningful evaluation of liquefaction potential are to be made.

2.7.5 Effects of Sample Disturbance of in-situ Deposits

When a sampling tube is pushed into the sand, some disturbance occurs. For loose to medium dense sands, the movement of grains causes an increase in density (which tends to increase the cyclic loading resistance) but it also breaks down some of the cementation at grain contacts (which tends to reduce the cyclic loading resistance). The combined effect is apparently to cause little change in the cyclic loading resistance of the sand so that test data on good quality "undisturbed samples" give a good basis for evaluating the liquefaction potential of the deposit (Seed and Idriss 1982).

2.8 Factor of Safety in Evaluating Liquefaction Potential

In evaluating the liquefaction potential of a saturated earthquake condition, it is customary to express the result in terms of a factor of safety expressed as

$$\text{Factor of safety} = \frac{\tau_l}{\tau_d}$$

Where, τ_l = average cyclic stress required to cause liquefaction in N-cycles

and τ_d = average cyclic stress induced by Earthquake for N-cycles.

This requires determinations of the stresses induced by the earthquake and the stresses which must be applied to the sand to cause liquefaction. As a guideline, acceptable factors of safety range from about 1.25 to 1.5.

2.9 Summary

The soil liquefaction is induced only by cyclic loading during earthquake shaking. Liquefaction phenomena depends on geological formations, soil characteristics and earthquake history. In the previous pages an attempt has been made to summarize the essential elements of the current state of the art for evaluating the liquefaction potential of soil deposits due to earthquake shaking, clearly it has been seen that the north-eastern region of Bangladesh is tectonically very active zone. There are mainly two methods available for evaluating liquefaction potential. In first method, the liquefaction potential can be evaluated by determining the soil condition such as penetration resistance and computing the cyclic stress ratio. Liquefaction probability can be judged by using the design chart such as shown in Fig. 2.9. In the second method, the liquefaction potential can be determined by calculating the stresses induced in the ground by the design earthquake using simplified procedure and to compare with those required to cause liquefaction of representative samples in the laboratory. In doing this it will often be necessary to convert the irregular stress sequence to an equivalent uniform cyclic stress series.

CHAPTER - 3

RESEARCH SCHEMES

3.1 Introduction :

Liquefaction generally occurs in loose sand deposits during earthquakes. This depends on soil characteristics and mode of occurence of an earthquake. Methods for evaluation of liquefaction potential have been discussed in previous chapter. There are mainly two methods available for determination of liquefaction potential. It has been observed in many earthquakes that a few types of geological deposits, have shown a high degree of susceptibility to liquefaction leading to ground failures. In Bangladesh most of the areas are aluvial-plain deposits. Moreover, zone-I in Fig. 1.2 is seismically active area. Stuart (1920) reported the presence of sporadic to abundant lateral spreading, bearing capacity failures, ground settlement, filling of wells and borrow pits, fissures and sand boils in a 108 miles long zone in and around the epicentral region after the occurrance of Srimangol earthquake in 1918. Brahmaputra and its tributary river valleys have Holocene fluvial and alluvial-plain deposits. Therefore, Zone-I may have been susceptible to liquefaction. On that regards, an investigation has been made to find out the liquefaction potential of selected earthquake prone areas within Bangladesh.

3.2 Objectives of the Research

Liquefaction potential can be assessed by determining the cyclic stress ratio and the modified penetration resistance. In determination of cyclic stress ratio, the horizontal ground acceleration is to be known. The ground accelaration can be found out from the acceleration attenuation curves. The value of ground acceleration depends on the magnitude of earthquake & the distance between the epicenter and the site under consideration . Soil properties like relative density depends on the geological formations. In order to study liquefaction potential of the north-eastern zone of Bangladesh the places, Muktagacha, Sylhet, Maulvibazar, Hobigonj, Sherpur, Sarail, Comilla, Bhairabbazar, Narsingdi & Lakshmipur are selected. The selection of these area were based on the consideration of geological formations and earthquake intensity. The following aspects were studied for these locations :

i) The general characteristics of the soil.

ii) The ground acceleration of these areas are determined from the acceleration attenuation curves shown in Fig. 2.5 and Fig. 2.7.

iii) Liquefaction potential of these areas have been evaluated by method outlined in Article 2.6.

CHAPTER - 4

EVALUATION OF LIQUEFACTION POTENTIAL

4.1 Introduction :

Methods for evaluation of liquefaction potential have been discussed in chapter-2. It has been show that in order to predict liquefaction potential of a region we need to know the characteristics of the geological formations, soil properties and the earthquake history for the region. A list of geological formations including physiographic sub-regions of Bangladesh have also been provided earlier and their salient characteristics discussed. Tectonically active regions are mapped and shown in Fig.1.1. The necessary soil characteristics required to study soil liquefaction include soil properties such as void-ratio or relative density. The later inturn can be correlated from the Standard Penetration values. In this chapter the soil characteristics of physiographic sub-regions likely to be subjected to earthquake hazards have been studied and analysed from available soil boring reports. Probable earthquake transmission characteristics in these soils are also discussed. On the basis of above studies liquefaction potential of some areas of Bangladesh have been predicted.

4.2 Structural forces affecting Bengal Basin :

Figure 2.2 shows that the eastern and northern parts of Bangladesh have been subjected to more structural modification than the western and southern parts. Nearly all faults and

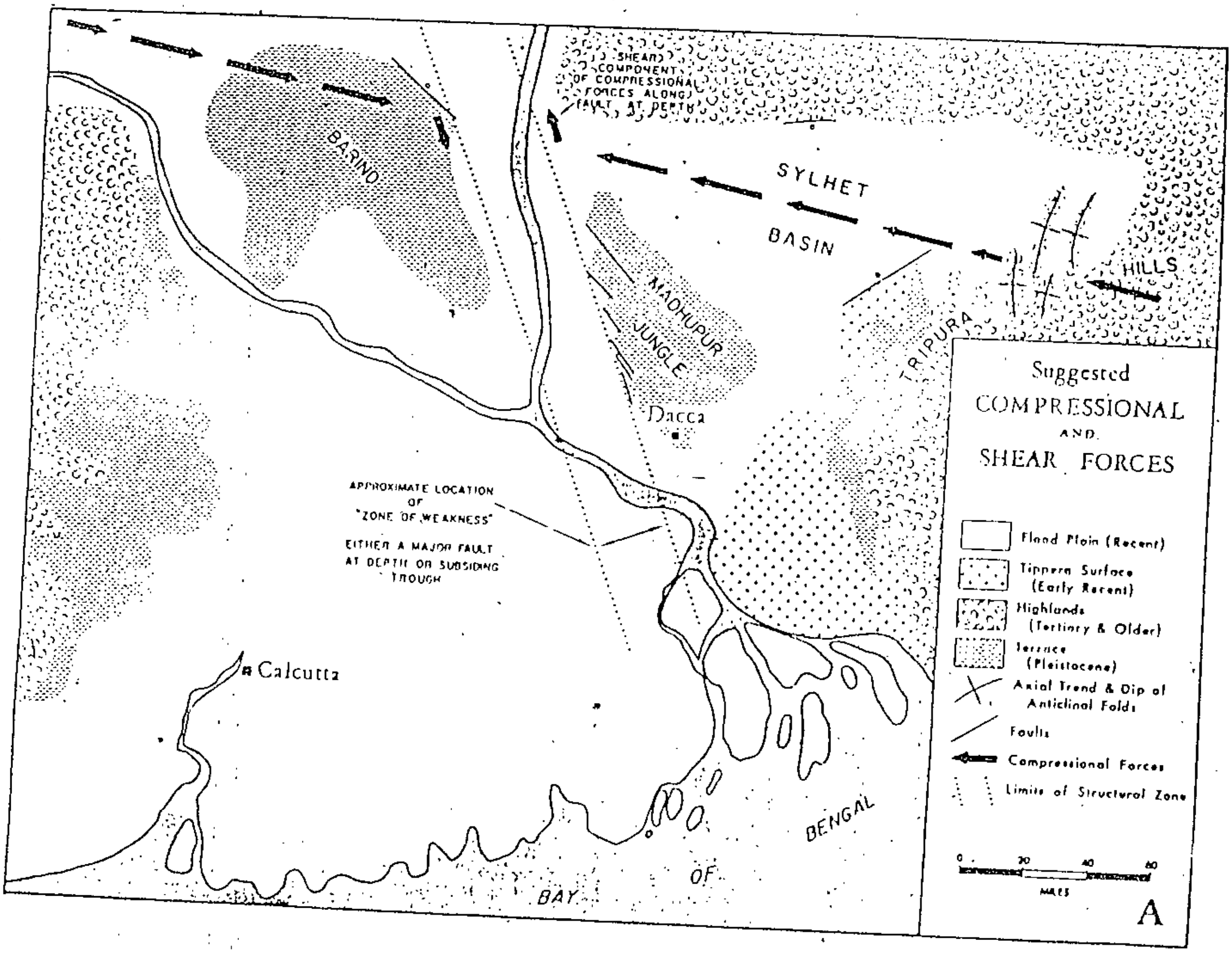


Fig.4.1 Structural Forces Affecting the Bengal Basin

(After Morgan & McIntire,1959)

folds mapped are located north of the Ganges River. The most impressive structure noted is the series of echelon faults flanking the west side of the tilted Madhupur Jungle. In addition there is notable subsidence in the Sylhet Basin, faulting, tilting and uplift of the eastern Barind, relative uplift of the Tippera surface and recent folding of the Tripura Hills. These structural phenomena and a general subsidence of the deltaic plain are to be interrelated. Morgan and McIntire (1959) proposed two theories to explain echelon faulting. The first assumes a major fault at depth, along which a couple, is acting, with the result that a number of secondary echelon faults developed at the surface. This theory is represented in Fig-4.1. The effect of such fault at depth is reflected by a zone of weakness at the surface shown by the pair of dotted lines. Considerable evidence suggests the presence of such a 'zone of weakness', passing between the Barind and Madhupur pleistocene inliers. The second theory used to explain echelon faulting demands torsion or warping of a surface to produce tear faults. In Bangladesh a major deltaic sedimentary mass is subsiding. This has been discussed in article 2.3. The Sylhet Basin is also subsiding. The two subsiding areas are marked in Fig.-4.2. These two areas however are counter balanced by the Barind to the northwest and recent Tippera surface to the southeast, which are not subsiding and may be rising slightly. Such warping could result in a large area subjected to torsion, which would tend to be relieved through echelon faulting. Krishnan (1953) suggests that the western side of the Shillong Plateau may be marked by fracturing which would agree with

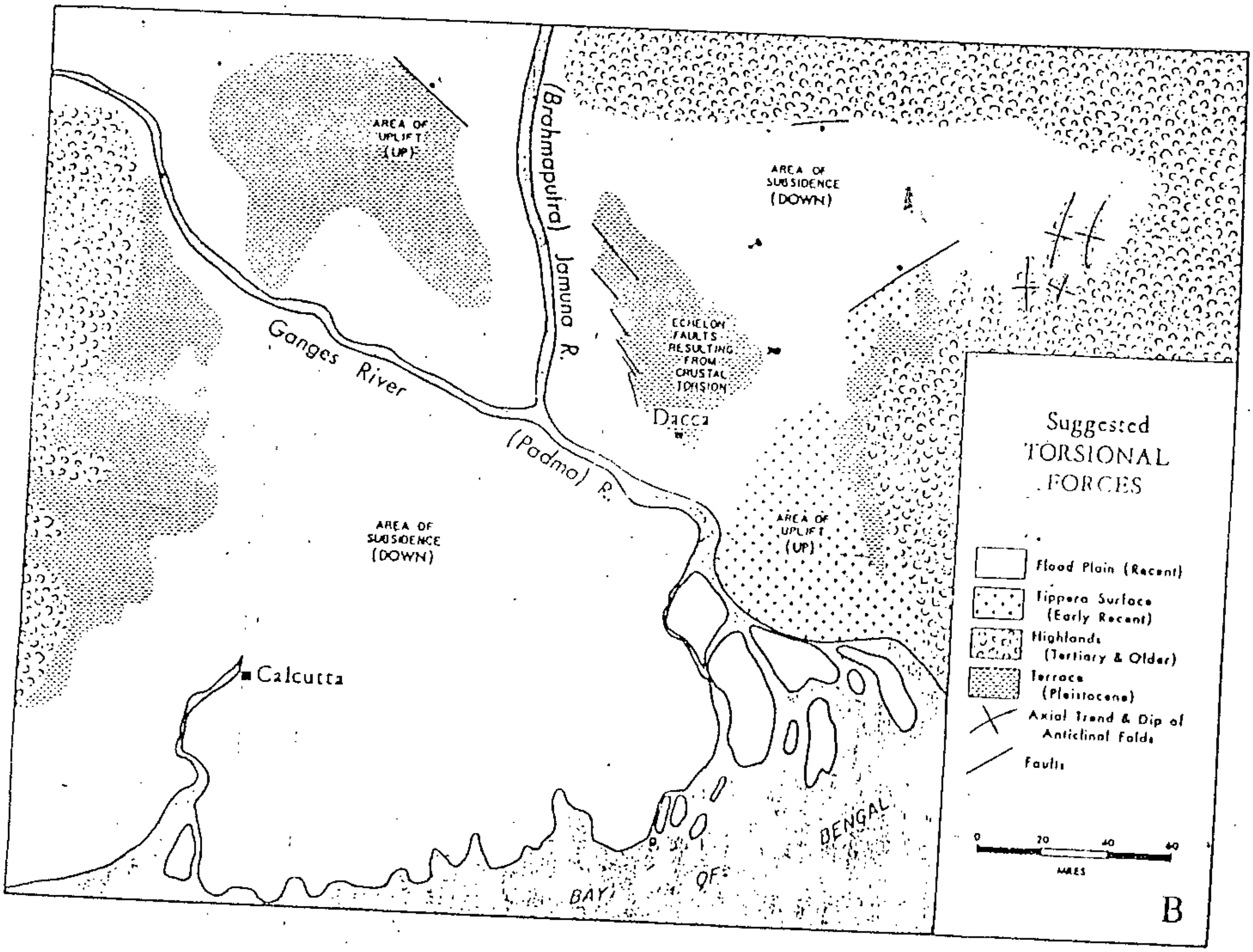


Fig.4.2 Structural Forces Affecting the Bengal Basin (After Morgan & McIntire, 1959)

other evidence for the "zone of weakness" which follows approximately the trend of the Jamuna Padma Meghna river system. According to Morgan and McIntire (1959), it can be inferred that west-northwest compression across the Sylhet Basin could have a component along the "zone of weakness". This "zone of weakness" points either to a subsiding trough or to a single major fault at depth. The north-eastern region of Bangladesh has an earthquake effect. But no earthquake waves are likely to be transmitted from this north-eastern region of Bangladesh to the south-western region through this "zone of weakness". The general interference from the historical records is that only the norrow belt facing the Shillong Plateau is liable to severe earthquake damage and the south-western part of Bangladesh is fairly safe from earthquake damages and therefore have been excluded in this study.

4.3 Areas Selected for Analysis :

Study of liquefaction potential are necessary in earthquake prone areas. Previously, it has been said that the north-eastern region of Bangladesh are Tectonically active region. Sylhet is a important place on a district head quater. The Monno-barrage projects are lying in Maulvibazar. The Upazilla named Sherpur, Hobiganj, Sarail, Bhairabbazar, Narsingdi and Muktagacha are also important places in present political concepts. All the above places are situated in north-eastern region of Bangladesh. Therefore, analysis of soil liquefaction potential of the said areas are attempted.

Fig.4.3 : Locations considered for Liquefaction Potential Analysis.

The places that are taken into consideration for liquefaction potential analysis are shown in Fig-4.3.

4.4 Soil Characteristics of earthquake Prone region of Bangladesh

It has been already discussed that the soil formation of Bengal Basin consists of two principal deposits the pleistocene and recent flood plains. The physiographic distribution of these deposits are shown in Fig-2.3. It is very difficult to seperate these formation on the basis of grain size analysis. Yet it has been seen that the pleistocene sediment contains very fine grained material than recent flood plains. Moreover, Pleistocene sediments are well oxidized and more compacted. Since the south-western region of Bangladesh is fairly safe from earthquake hazards, no investigation about soil characteristics of that region have been reported in this dissertation. The soil characteristics of Sylhet basin, Madhupur Jungle and Tippera Surface of North-Eastern Zone have been generalized on the basis of available boring logs. These boring logs of the said areas are collected from various organizations and these are shown in Appendix-A. Based on the analysis of boring logs, the soil characteristics of these selected zones can be summarized as follows.

4.4.1 Sylhet Basin :

Figure-4.4 shows the typical boring logs of this region. In this figure bore-log A is representative of the boundary between the Madhupur pleistocene and the Sylhet basin, bore-log B is that for the central portion of this basin and bore-log C

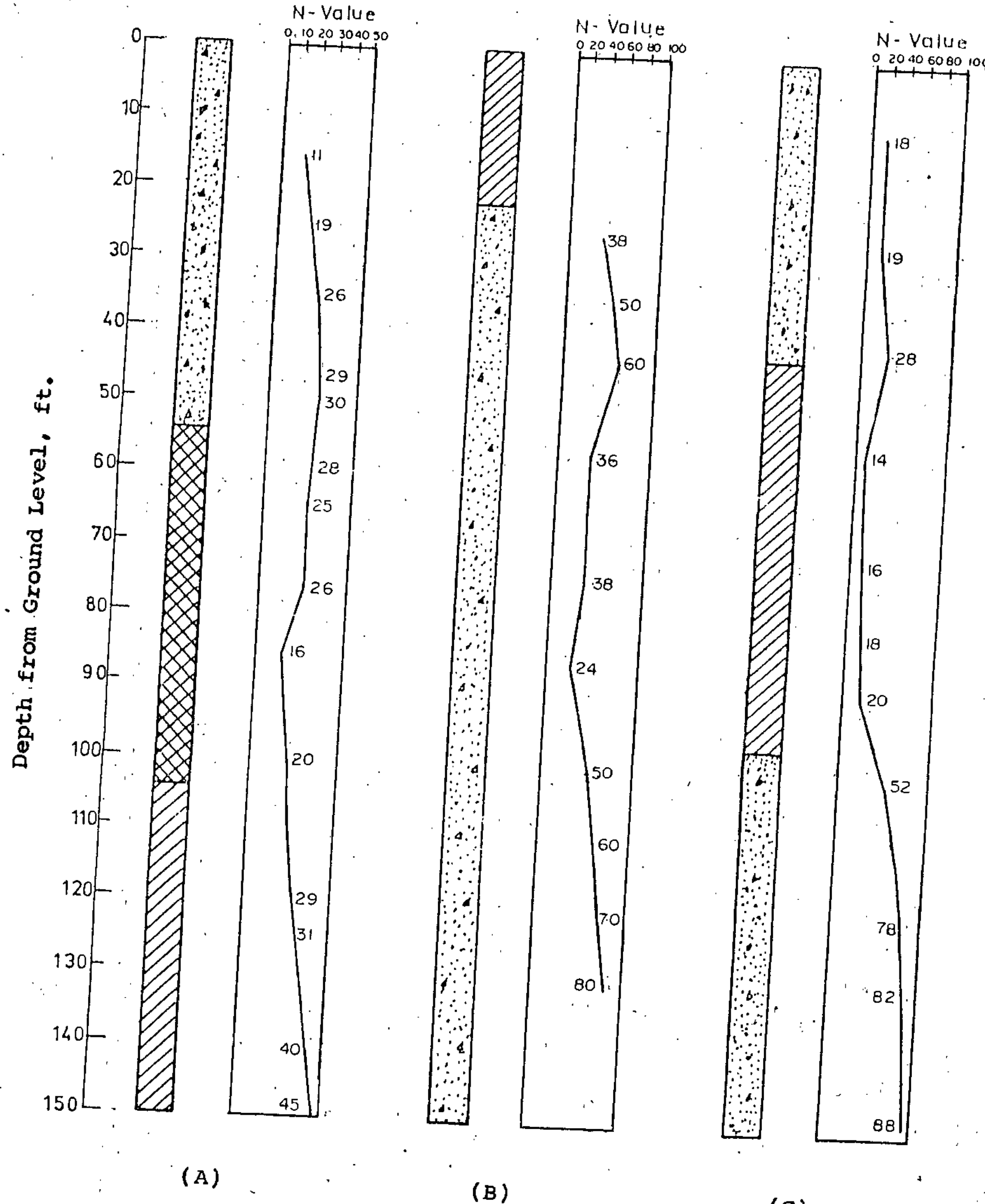
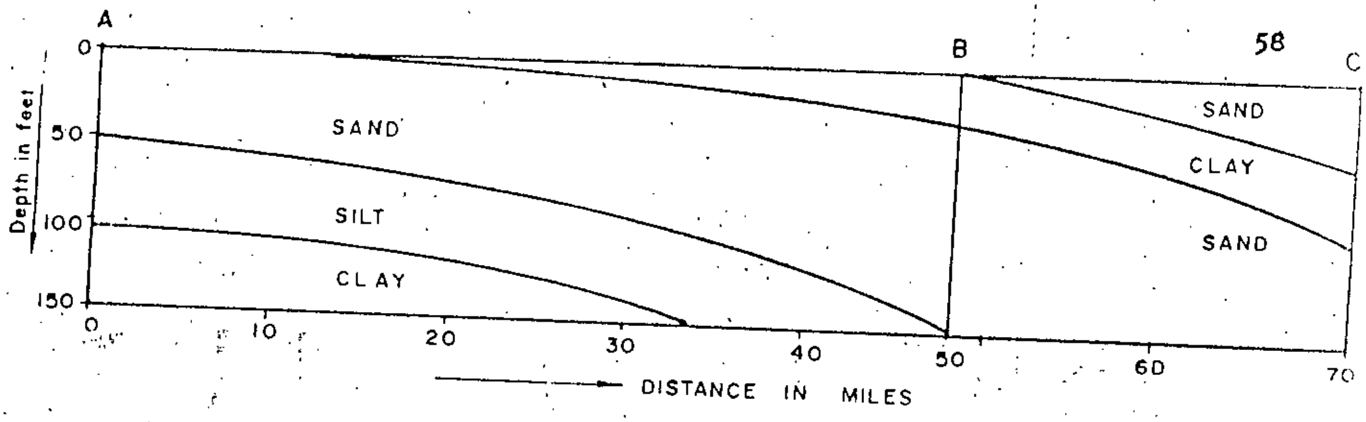


Fig.4.4 Typical Boring Logs within Sylhet Basin

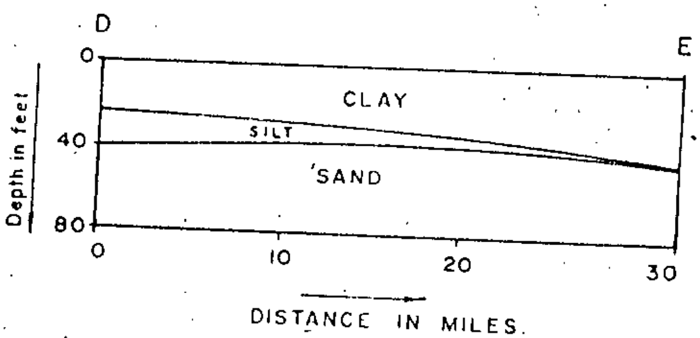
is for the boundary between this basin and the Tippera surface. The soil profile of this basin in the east west direction is shown in Fig-4.5. (a).

The reason for these three types of formation can be interpreted in the following way. Generally the Sylhet basin was covered by piedmont alluvial fans, but it was subsidded 30-40 ft. during the last several hundred years (Morgan and McIntire, 1959). Due to this subsidence, the central portion of the basin transformed into a depression called beel or Haor, these depressed areas were innundated by stagnant water collected during heavy rainfall. These water contains fine particles, sedimentation of these fine particles resulted in clay formations at the top 22 ft and is shown in fig-4.5. The recent depositions of clay are soft. The sand layer just below the clay layer (in fig-4.4, bore-log B) were probably deposited before subsidence, and then later compacted The increase in N-values with depth and high N-values of this deposits indicate their dense condition.

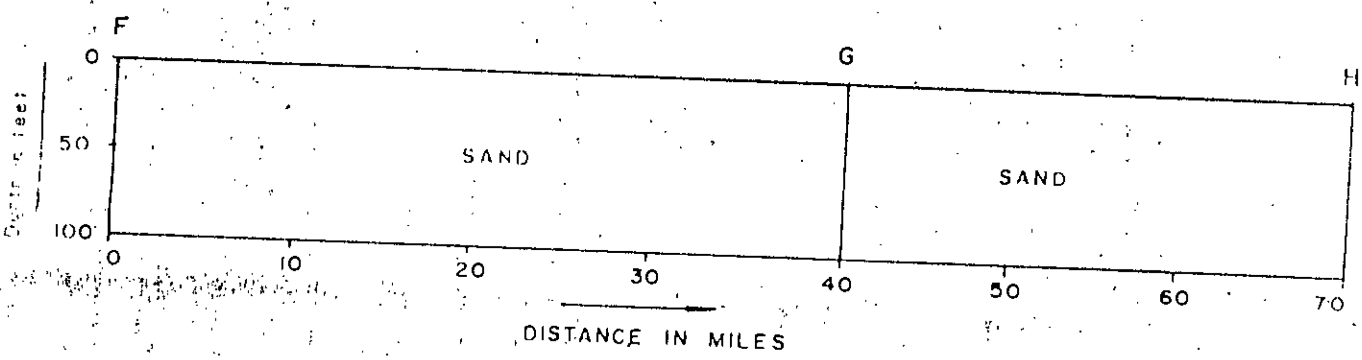
According to Morgan and McIntire (1959), the Madhupur pleistocene tilted towards the east. Due to the tilt, a clay formation shown in Fig. 4.5 dips towards the Sylhet basin. Above this clay formation, a silt and sand layer are found and this layer are of recent deposits. The N-values of this sand layer are found to be less than the sand layer represented in fig-4.4, log B.



(a) Soil Profile Along Section A-B-C.



(b) Soil Profile Along Section D-E



(c) Soil Profile Along Section F-G-H.

FIG-4.5 Soil Profile across Different Physiographic Unit(Fig.4.2)

The first 42 feet sand layer shown in fig-4.4 in bore log C is of recent origin and resides within the flood plain. This is followed by 55 feet of clay layer which has consolidated and subsided with time. The sand layer underneath this clay layer was alluvial deposited and is well compacted. The N-values of this layer are very high. But the N-values of the upper 42 feet of sand layer are very low indicating loose condition. The deposits may undergo liquefaction and therefore have been investigated for liquefaction possibility.

4.4.2 Madhupur Pleistocene :

The extent of Madhupur Pleistocene is shown in Fig-2.2. The pleistocene formed terraces and inliers in the districts of Rajshahi, Bogra, Dinajpur, Mymensingh, Dhaka and Tangail and also formed marginal low hills in the district of Sylhet, Comilla and Chittagong. The pleistocene sediments are flood-plain deposits of earlier Ganges and Brahmaputra rivers. That they occur in several extensive areas above the level of present flood plains indicated that there has been differential movement between pleistocene and recent time. Differential relief between pleistocene and recent flood plains is extremely variable. Where fault scarps occur, as along the west flank of the Madhupur Jungle and around the Lalmai Hills, the relief is obvious. Where the Ganges River flows close to the southern end of the Barind, the relief is considerable. In other places, notably along the pleistocene areas which flank the Bengal basin on the east and west along the northeast and west sides of the Barind, and along the eastern and southern side of

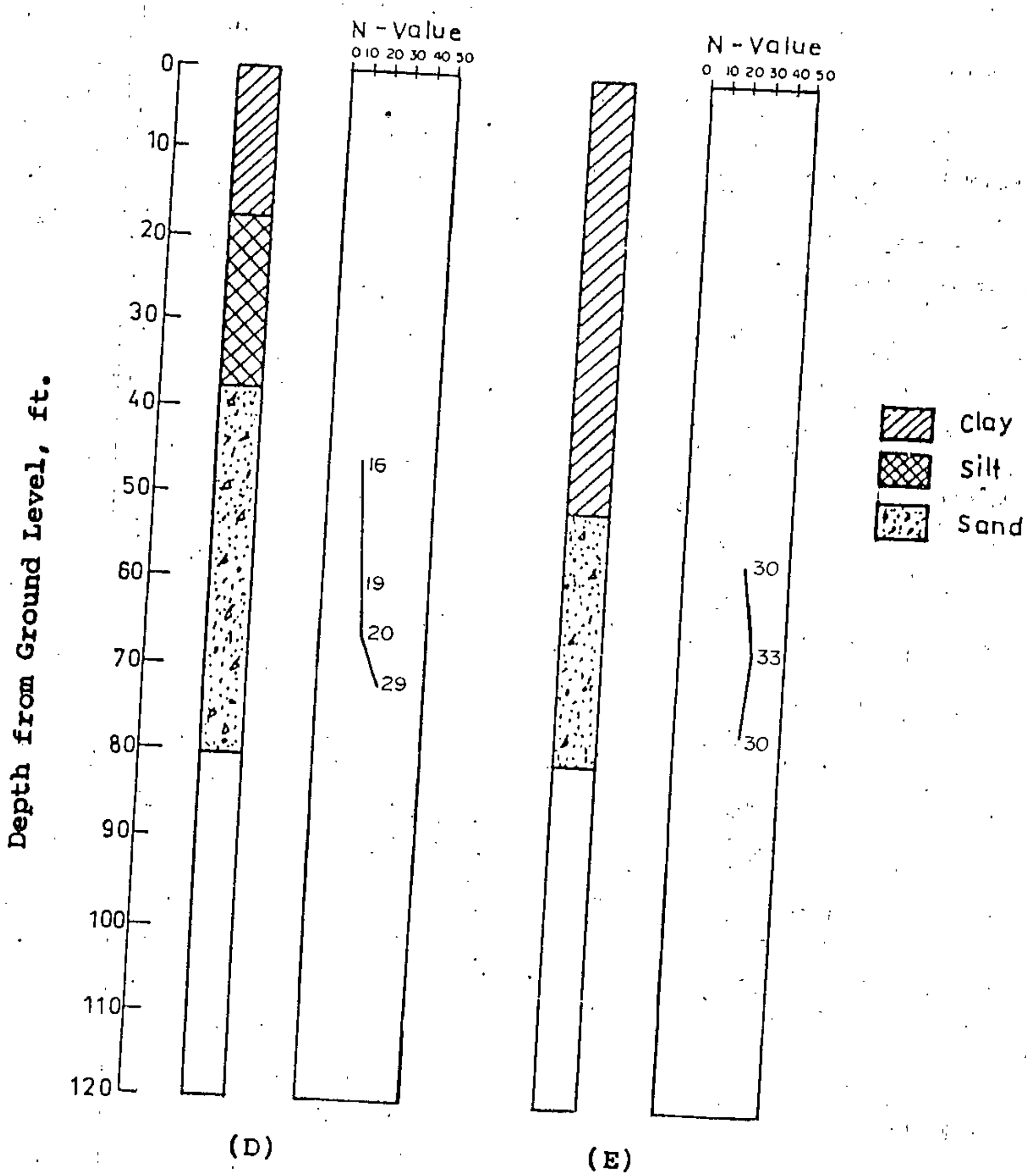


Fig.4.6 Typical Boring Logs within Madhupur Pleistocene

Madhupur, the pleistocene slopes gradually beneath the overlapping recent sediments. Since, Madhupur pleistocene lies at a slightly higher elevation than the recent flood plains, pleistocene age-sediments are generally more weathered and Fig. 4.6 in bore log D is representative of west side of the Madhupur jungle. Thickness of Madhupur clay is variable and it ranges from a few feet to more than 106 feet. The formation is 106 feet in Dinajpur, 54 feet in Paharpur (Rajshahi) and 100 feet in Madhupur area (Morgan & McIntire, 1959). The lower part of the deposit is mottled while the upper part is red and yellow. Fig-4.6 show the typical boring logs within the Madhupur pleistocene. In Fig.-4.6 bore log E is that for the central portion. The Madhupur Pleistocene in west slopes gradually beneath the overlapping recent sediments. The 18 feet clay layer shown in bore log D in Fig. 4.6 Madhupur clay is reddish or mottled. The next silt and sand layer are of recent deposits. The N-value of the sand layer are less indicating loose condition. But the clay layer in bore log E in Fig. 4.6 is the Madhupur clay which is stiff. This clay may be either mottled or red clay. Mottled clay is earthy grey with patches of red, brown and yellow-colours. This clay is stiff, compact at places, vesicular and contains oolites. It contains intercalated white clay and white sand. The contact of the sediment with the overlying red clay is gradational. The colour of red clay varies from red, orange to brownish red to brick red. It is mainly stiff and plastic but occasionally vesicular containing nodules. The clay is intercalated with fine sands and silts. This clay layer lies on the sand layer which is reddish in colour. The N-value of the sand layer is high containing dense condition. The soil profile of the Madhupur pleistocene in east-west direction is shown in fig-4.5 (b).

4.4.3 Tippera surface :

The Tippera surface is the recent flood-plain formation. All evidence suggests slight uplift of the entire Tippera surface. Cultural modifications of the natural drainage system, displacement along the bordering fault to the north, and oxidation of sediments all indicate that the Tippera surface has been uplifted during recent time (Morgan and McIntire, 1959). Amount and nature of uplift are not precisely known. Figure 4.7 shows a typical boring log in Tippera surface. Since it is of recent origin, the surface is almost covered by a sand deposit, but in some where this sand deposits are mixed with silt and clay. The sands are of very fine variety. On aerial photographs or detailed maps, the drainage system of the Tippera surface displays a well-developed rectangular pattern in contrast to the braiding and meandering pattern of the recent flood plain. In addition to the rectangular system are a few small meandering streams which form the drainage system for the adjacent hills. In contrast to the meandering streams of the Barind Pleistocene surface, these exhibit numerous cutoff and oxbow lakes, indicating that the deposits are less consolidated (Morgan & McIntire, 1959). The N-values obtained for this surface are low indicating their

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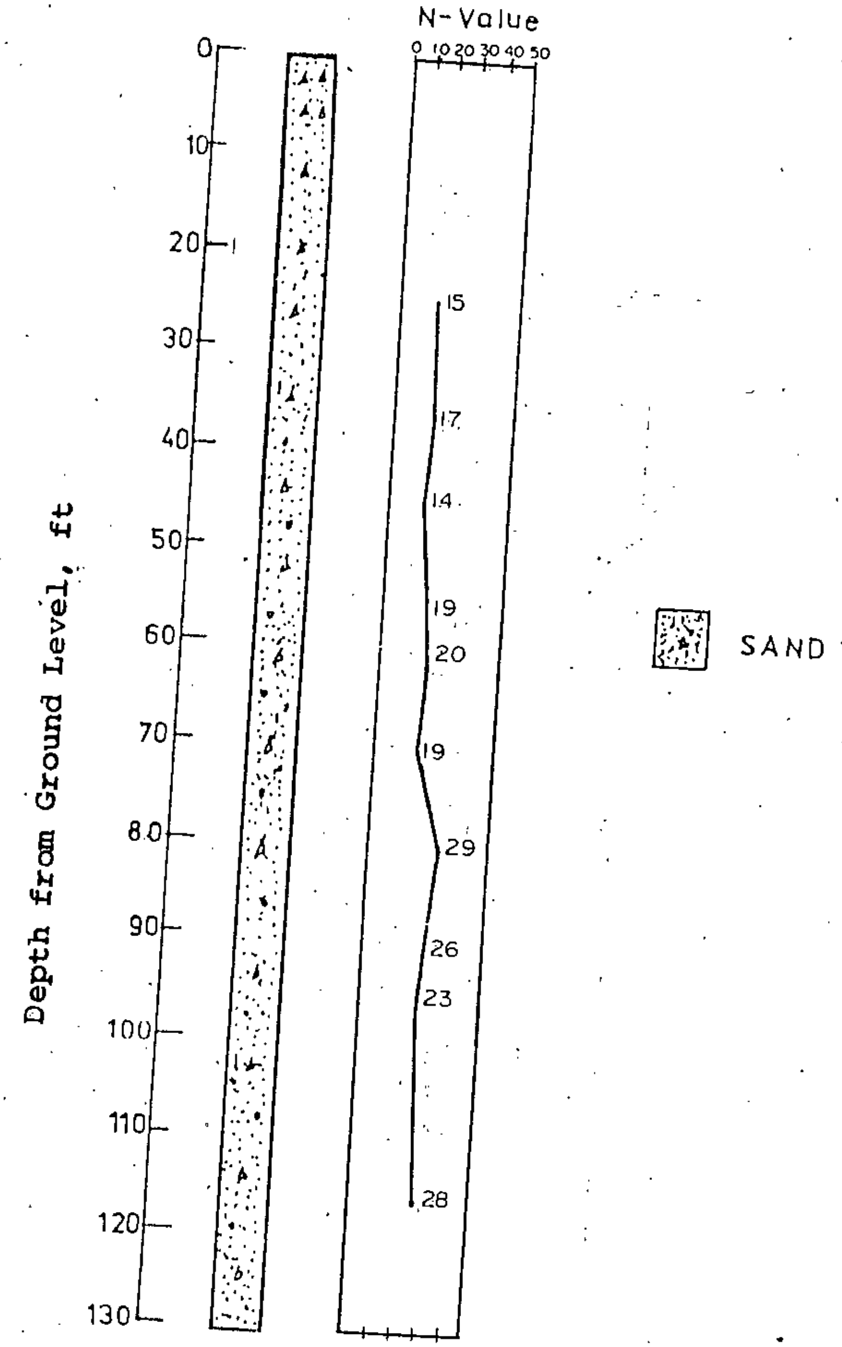


Fig.4.7 Typical Boring Log in Tippera Surface

loose condition. The soil profile of Tippera surface in the North-South direction is shown in fig-4.5 (c).

4.5 Modification of Standard Penetration Values due to Overburden :

Figure 2.9 shows the relationship between cyclic stress ratio and the modified penetration resistance N_1 which relates to condition for liquefaction for a soil. This reflects the influence of soil properties and the effective overburden pressure that may create liquefaction. Hence the modified penetration resistance N_1 is the measured penetration resistance corrected to an effective overburden pressure of 1 ton per sq. ft. The value of N_1 for any sand can be determined from the measured value N from the relationship suggested by Gibbs & Holtz (1957).

$$N_1 = C_N N$$

where C_N is a function of the effective overburden pressure at the depth where the penetration test was conducted. The value of C_N can be determined either by the results presented by Gibbs and Holtz (1957) or Marcuson and Bieganousky (1977). The chart for the value of C_N is shown in Fig-4.8 which is based on studies conducted at the waterways Experiment Station (Mercuson and Bieganousky, 1977) and collected by Seed (1980).

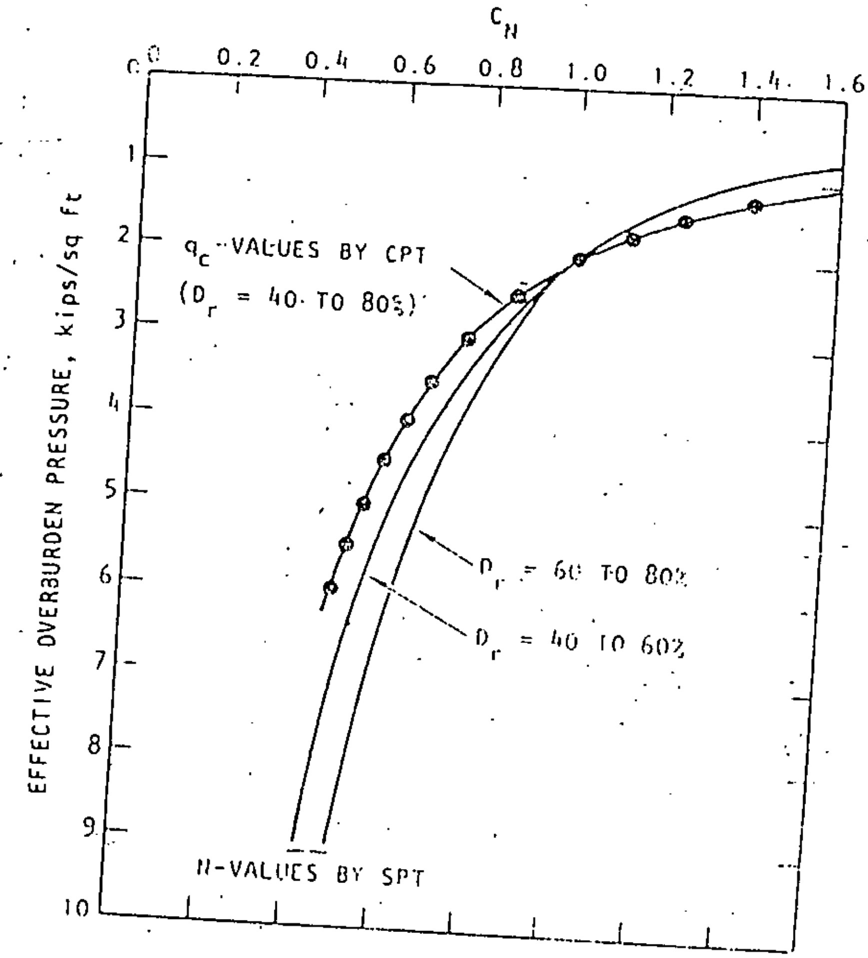


Fig.4.8 : Chart for values of C_N

(After Seed H.B. 1980).

4.6 Determination of Liquefaction Potential :

Liquefaction Potential of a soil layer can be assessed from the modified penetration resistance and cyclic stress ratio for that soil formations, Methods of determining the modified penetration resistance has been disscussed in art 4.4. The cyclic stress ratio of any location can be determined by using the equation (2.5) :

$$\frac{\tau_{av}}{\sigma'_o} = 0.65 \; \frac{a_{max}}{g} \; \frac{\sigma_o}{\sigma'_o} \; r_d$$

This equation requires the evaluation of a_{max}, σ'_o, σ_o and r_d. The maximum ground surface acceleration, a_{max}, can be evaluated by using different attenuation curves such as shown in Fig.-2.5. First of all, it is necessary to determine the distance between the location, under consideration, and the nearest epicentres of known magnitudes. The epcentres of different earthquakes that occured around Bengal basin are shown in fig-1.2. After tracing the distance, the maximum ground acceleration on rock can be determined by using fig-2.5. For example, the boring log in Sylhet is 28 miles apart from the epicentre of magnitudes 6.0. Now from figure-2.5, the average value of maximum acceleration in cohesionless soil is 0,18g. The value of σ_o and σ'_o, at depth under consideration, can be determined after knowing the unit weight of soil and depth of water. The term r_d, using in equation 2.5, is the depth factor. The average value of r_d can be evaluated upto 40 ft. from the dashed line shown in figure 2.11 (Seed and Idriss,

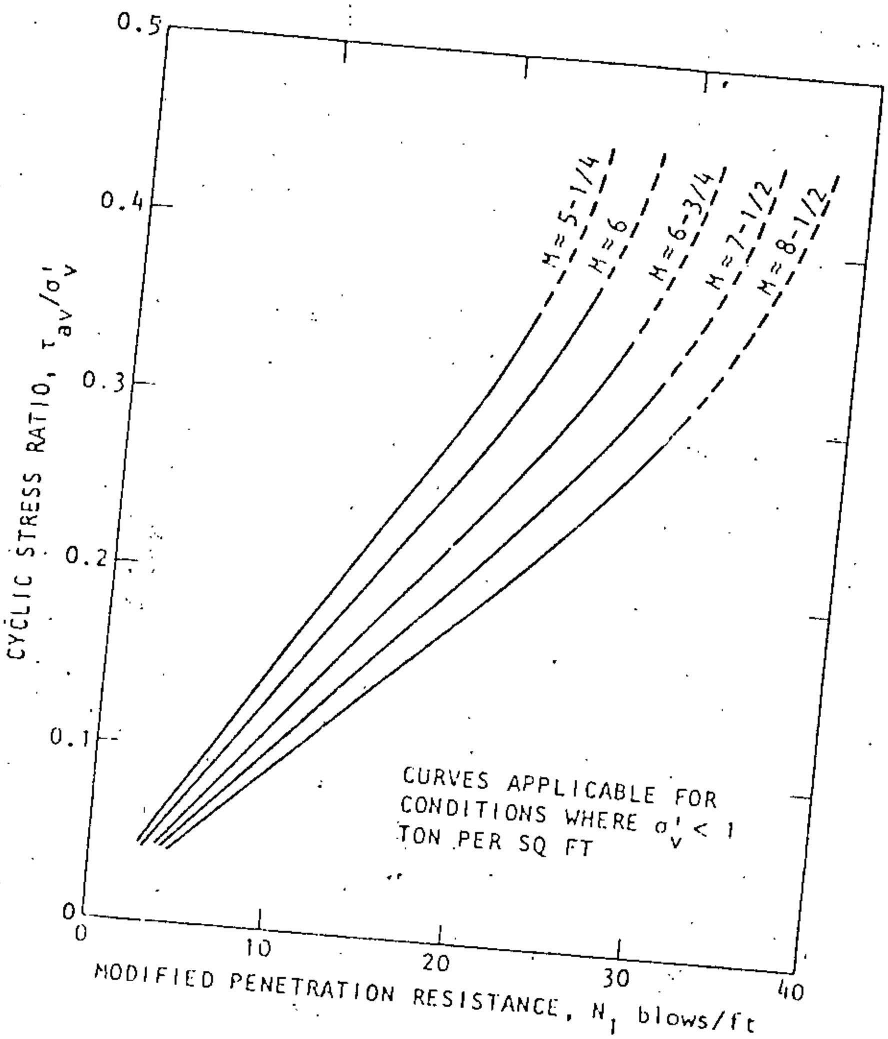


Fig.4.9 Chart for evaluation of liquefaction potential for sands for different magnitude earthquakes.

(After Seed H.B., 1980)

1982). After determination of cyclic stress ratio and modified penetration resistance, for a soil formation, these values may be ploted in figure 4.9. The location of such plot will indicate qualitatively whether the formation is likely to liquefy or not. For example, if the point is located above the corresponding line for a earthquake of given magnitudes, the soil is to liquefy where in reverse case, liquefaction is unlikely to occur.

4.7 Results of Liquefaction Potential Analysis

Typical bore logs from seven locations within the north-eastern region of Bangladesh are used to analyse whether liquefaetion will occur or not at several depths. This analyses are summarized in Table 4.1, 4.2, 4.3, 4.4 & 4.5 Table 4.1 shows the liquefaction potential at depth 26 feet on existing N-value. From this table, it can be interpreted that the places of Maulvibazar and Sarail are susceptible to liquefaction if an earthquake of magnitudes 7.6 (Richter scale) occurs at the nearest epicentres. The N-value upper than 26 feet was not available. Therefore, the maximum limit of corrected N-value i.e N_1 causing liquefaction at different depths are shown in Table 4.2, 4.3, 4.4 & 4.5. In Table 4.2, the depth taken into consideration is 5 feet and the water table is also 5 feet. If the N-value of Sylhet at depth 5 feet is 7 or less, the soil will undergo liquefaction during an earthquake of magnitude 6.0 under water table 5 feet. Liquefaction may also be occurred in sherpur, Maulvibazar, at depth 5 feet in loose fine sand if the corrected N-value i.e.

N_1 is 7 & 13 respectively during an earthquake of magnitude 6.3. In table 4.3 the depth taken into consideration is 10 feet but the water table is 5 feet. The maximum limit of corrected N-value of the places Sylhet, Sherpur, Maulvibazar and Sarail are 10, 10, 15 and 8 respectively. Lesser than these respective corrected N-value will be causing liquefaction at depth 10 feet of the mentioned places. In the same manner, Table 4.4 and 4.5 are listed for the maximum limit of corrected N-value causing liquefaction for the depth of 15 feet and 20 respectively.

TABLE - 4.1

Results of Cyclic Stress Ratio to the Cerrelated Pentration Value for Different Area

Sl. No.	Location of borehole	Depth taken into consideration	Depth water level (H)	Magnitude of Earthquake of Nearest Epicentre	a max on Rock	a max in sand	τ_h / σ_o'	Remarks
1.	Sylhet	26	10	6.0	0.2 g	0.18 g	0.158	No liquefaction
2.	Sherpur	26	10	6.0 7.6	0.16 g 0.28 g	0.24 g	0.2133	Liquefaction
3.	Maulvi Bazar	26	10	6.0	0.3 g 0.36 g	0.27 g	0.237	Liquefaction
4.	Sarail	26	10	5.1 7.6	0.04 g 0.16 g	0.15 g	0.133	Liquefaction
5.	Bhairab Bazar	26	10	5.1	0.04 g	0.03 g	0.0257	No liquefaction
6.	Narsingdi	26	10	6.0	0.03 g	0.015 g	0.0133	No liquefaction
7.	Muktagacha	26	10	6.0	0.05 g	0.04 g	0.035	No liquefaction

TABLE -4.2

CALCULATED N_1-VALUE CAUSING LIQUEFACTION

AT DEPTH 5 FEET

(Blows /ft)

Sl. No.	Location of Bore Whole	Approx Lat. Long.	Depth taken into consideration	Depth Water Level	Magnitude of Earthquake of Nearest epicenter	a_{max} on Rock	a_{max} on Sand	τ_h/σ_o'	N_{max} limit for/N_1 value causing Liquefaction
1.	Sylhet	24.89° 91.86°	5	5	6.0	0.2g	0.18g	0.117	7.0
2.	Sherpur	24.63° 91.70°	5	5	7.6	0.28g	0.24g	0.156	15.0
3.	Maulvibazar		5	5	6.[illegible]	0.3g	0.27g	0.175	13.0
4.	Sarail		5	5	5.1	0.145g	o.13g	0.084	5.0
5.	Bhairab Bazar	24.01° 91.42°	5	5	5.1	0.04g	0.03g	0.019	1.0
6.	Narsingdi	23.90° 90.74°	5	5	6.0	0.03g	0.015g	0.009	-
7.	Muktagacha		5	5	6.0	0.05g	0.04g	0.026	1.0

TABLE -4.3

CALCULATED N_1-VALUE CAUSING LIQUEFACTION

AT DEPTH 10 FEET

(Blows /ft)

Sl. No.	Location of Bore Whole	Approx Lat. Long.	Depth taken into consideration	Depth Water Level	Magnitude of Earthquake of Nearest epicenter	a_{max} on Rock	a_{max} on Sand	τ_h/σ_o'	N_{min} limit for/N_1 value causing Liquefaction
1.	Sylhet	24.89° 91.86°	10	5	6.0	0.2g	0.18g	0.155	10
2.	Sherpur	24.63° 91.70°	10	5	7.6	0.28g	0.24g	0.206	19
3.	Maulvibazar		10	5	6.0	0.30g	0.27g	D.232	15
4.	Sarail		10	5	5.1	0.145g	o.13g	0.112	8
5.	Bhairab Bazar	24.01° 91.42°	10	5	5.1	0.04g	0.03g	0.269	2
6.	Narsingdi	23.90° 90.74°	5	5	6.0	0.03g	0.015g	0.009	-
7.	Muktagacha		5	5	6.0	0.05g	0.04g	0.026	1.0

TABLE -4.4

CALCULATED N_1-VALUE CAUSING LIQUEFACTION

AT DEPTH 15 FEET

(Blows /ft)

Sl. No.	Location of Bore Whole	Approx Lat. Long.	Depth taken into consideration	Depth Water Level	Magnitude of Earthquake of Nearest epicenter	a_{max} on Rock	a_{max} on Sand	τ_h/σ_0'	N_{max} limit for/N_1 value causing Liquefaction
1.	Sylhet	24.89° 91.86°	15	5	6.0	0.2g	0.18g	0.172	12
2.	Sherpur	24.63° 91.70°	15	5	7.6	0.28g	0.24g	0.229	22
3.	Maulvibasar		15	5	6.4	0.30g	0.27g	0.260	19
4.	Sarail		15	5	5.1	0.145g	0.13g	0.125	10
5.	Bhairab Basar	24.01° 91.42°	15	5	5.1	0.04g	0.03g	0.029	3
6.	Narsingdi	23.90° 90.74°	15	5	6.0	0.03g	0.015g	0.014	1
7.	Muktagacha		15	5	6.0	0.05g	0.04g	0.038	3

TABLE -4.5

CALCULATED N_1-VALUE CAUSING LIQUEFACTION

AT DEPTH 20 FEET

(Blows /ft)

Sl. No.	Location of Bore Whole	Approx Lat. Long.	Depth taken into consideration	Depth Water Level	Magnitude of Earthquake of Nearest epicenter	a_{max} on Rock	a_{max} on Sand	τ_h/σ_0'	N_{min} limit for/N_1 value causing Liquefaction
1.	Sylhet	24.89° 91.86°	20	5	6.0	0.2g	0.18g	0.184	14
2.	Sherpur	24.63° 91.70°	20	5	7.6	0.28g	0.24g	0.243	23
3.	Maulvibazar		20	5	6.7	0.30g	0.27g	0.276	21
4.	Sarail		20	5	5.1	0.145g	0.13g	0.133	11
5.	Bhairab Bazar	24.01° 91.42°	20	5	5.1	0.04g	0.03g	0.031	3
6.	Narsingdi	23.90° 90.74°	20	5	6.0	0.03g	0.015g	0.015	1
7.	Muktagacha		20	5	6.0	0.05g	0.04g	0.041	4

CHAPTER - 5

5.1 Conclusions :

In this study an attemtpt has been made to summarize the current state of the art for evaluating the liquefation potential of a soil deposits on level ground due to earthquake shaking. From the historical background of earthquake hazards on the Bengal basin and it's surroundings, it is interpreted that the western and south-eastern portions of Bangladesh are not so vulnerable to earthquake effect. An attempt has been made to investigate the liquefaction possibility of some locations within the north eastern zone of Bangladesh.

It seems that the design engineer confornted with the need to evaluate the liquefaction potential of a deposit has two basic choices if he consider it appropriate to neglect the possible effects of drainage occuring during the period of cyclic stress applications.

1) For the first choice he has calculate the stesses induced in the ground by the design earthquake using either ground response analysis or simplified procedures and to compare these stresse with those required to cause liquefaction of representative sampler in the laboratory.

Since laboratory facility is not so good, simplified procedure is taken to evaluate the liquefaction potential of several places within north eastern places in Bangladesh. By this procedure, the liquefaction potential of any places may be evaluated from the field penetration resistance value and the available earthquake history. The results of the evaluation of liquefaction potential are summarized in Table 4.2, 4.3, 4.4 and 4.5 in previous chapter. The maximum limits of corrected N-Value i.e N_1 causing liquefaction at different depths are shown in different table. From geological point of view the places of Sylhet, Sherpur, Maulvibazar Bhairab-bazar, Muktagacha within the north-eastern region are of recent deposits containing loose fine sand. Major earthquake epicentres are also located within north-eastern region of Bangladesh. The place Sylhet may, therefore, be liquefied at depth 5 feet during an earthquake of magnitude 6.0 if the corrected N-value is 7 or less and also liquefied at depth 10 feet if the N_1-value is 10. Similar way, the place Sherpur, Maulvibazar may be liquefied at depth 5 feet during an earthquake of magnitude 6.3 if the corrected N-value is 7 and 13 respectively. On the basis of these analysis, it can be interpreted that the place Maulovibazar if more suseeptible to liquefy than the places of Sylhet and Sarail. Liquefaction potential is, therfore, dependant on four major factors. These are the field penetration resistance, Magnitude of earthquakes, distance of the place under consideration from the known epicentre and the water table. If the distance of any place from the known epicentre is longer and the water table is lower, then equation 2.5 shows that the cyclic stress ratio of that place at depth taken into consideration will lower indicating no liquefaction. But if the N-value is too low, liquefaction will occur. In other case, if the place taken into consideration is very close to the known epicentre and water-table is high, the cyclic stress ratio automatically will be higher causing liquefaction. But the N-value of that place is too high i.e. dense condition, the place does not undergo liqufaction. Therefore, from the result of liquefaction analysis it can be interpreted that the place of Sylhet, Sherpur, Maulvibazar and Sarail may undergo liquefaction at depth 5 feet or 10 feet, if the soil is very loose fine sand under high water table.

5.2 Recommendation for Further Study :

The liquefaction potential measure has been carried out only on the basis of correlation between the cyclic stress ratio and the modified penetration resistance. The cyclic stress ratio are also correlated with the available attenuation curves. This may some times either in over estimation or under estimation of lique faction potential.

Therefore, on the basis of correlation, liquefaction potential measure may either in overestimation or underestimation. So, to establish a better judgement on liquefaction potential on any site, it is better to varify the correlated results with the result of cyclic triaxial test result done in laboratory.

APPENDIX-A

CLACULATIONS FOR EVALUATION OF LIQUEFACTION POTENTIAL

For investigation of liquefaction potentiality of soil under north-eastern region of Bangladesh, a group of soil boring logs are collected from River Research Institution, BWDB, Dhaka. These boring logs are shown in fig.A_1 to fig. A_{12}. These boring logs are taken as a representative sample of the respective districts. On the basis of these representative samples, liquefaction potential analysis was made by using the method suggested by Seed and Idriss (1982).

CALCULATIONS

A sample calculation for evaluation of liquefaction Potential at Sherpur is given below.

The magnitude of Earthquakes of the nearest Epicentres are 6.1 and 7.6. The distance of the boring log from these epicentres are 16 miles and 22 miles respectively. Now from figure 2.5 (shown in chapter-2) the acceleration on rock are 0.18g and 0.22g respectively. From Fig-2.7 the maxm *acceleration on cohesionless Soil is 0.24g.*

Assume the water table at 10 ft.

Now, at 26 ft. $\sigma_o' = 10x120+16x57.6$

$= 2.1216$ *ksf.*

and N - value at 26 ft. = 12

From Fig-4.8, the value of $C_N = 1$

Hence $N_1 = C_N.N = 1x12 = 12$

From Fig -2.11 the value of $r_d = 0.93$

The cyclic stress ratio, $\frac{\tau_{av}}{\sigma_o'} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma_o'} , r_d$

$= 0.65 \frac{0.24g}{g} \frac{26x120}{2121.6} x0.93$

$= 0.2733$

For $N_1 = 12$ *and cyclic stress ratio = 0.2733 condition (in Fig-4.9) plots just below the curve for M=7.6. Hence ,with* $a_{max} = 0.24g$ *the sand would liquefy.*

Another example : Calculation for evaluation of Liquefaction Potential at Maulvibazar.

The magnitudes of earthquakes of the nearest epicentres are 6.7 and 7.6. The distance of the boring log from these epicentres are 9 and 14 miles respectively. Therefore, from Fig -2.5, the acceleration on rock are 0.3g and0.36g respectively. Hence from Fig-2.7 the maxm acceleration on cohesionless soil is 0.27g.

Assume the water table at 10 ft.

Now, at 26ft., $\sigma_o' = 10x120+16x57.6 = 2.1216$ ksf.

From Fig-2.11, the value of $r_d = 0.93$

The cyclic stress ratio, $\frac{\tau_{av}}{\sigma_o'} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma_o'}$

$= 0.65 \frac{0.27g}{g} \frac{26x120}{2121.6} x 0.93$

$= 0.24$

The N - value at 26 ft. = 10

From Fig -4.8 ,the value of $C_N = 1$

Hence $N_1 = N.C_N = 10.1 = 10$

For N_1, = 10 and cyclic stress ratio 0.24 condition plots above the curve shown in Fig -4.9 for M=7.6, hence with a_{max}=0.27g sand would liquefy for M=7.6

SUMMARY OF CALCULATIONS :

Calculation for Liquefaction Potential for Different Location are made in Manner Specified in the Expamles.The various steps of the calculation are shown in tabular form in Table (A-1):

TABLE A - 1

Various Steps of Calculation for Liquefaction Potential Analysis

Soil condition at different place / Step of calculation	A_1	A_4	A_5	A_6	A_9	A_{10}
The magnitude of earthquake of the nearest epicentres	6.0	6.1 7.6	5.1 7.6	5.4 7.6	5.1	6.0
Distance of the boring log from the nearest epientres	14miles	18miles 19miles	26miles 43miles	48miles 68miles	32miles	64miles
The maxm accl. on rock taken from Fig. 2.5	0.2 g	0.20 g 0.30 g	0.04 g 0.16 g	0.02 g 0.04 g	0.04 g	0.03 g
The maxm accl. on sand (taken from Fig. 2.7)	0.18 g	0.25 g	0.15 g	0.04 g	0.04 g	0.03 g
Depth under consideration	57ft	104ft	26ft	26ft	26ft	26ft
Depth of Water - Table	10ft	10ft	10ft	10ft	10ft	10ft
Value of σ_0	3.9 Ksf	6.6144 Ksf	2.1216 Ksf	2.1216 Ksf	2.1216 Ksf	2.1216 Ksf
Value of r_d (From Fig. - 2.11)	0.70	0.50	0.93	0.93	0.93	0.93
Cyclic stress ratio (using equation 2.5)	0.1436	0.1533	0.1333	0.0355	0.0355	0.026
N-value at the depth under consideration	32	52	16	17	23	28
The Corrected N-value i.e. N_1 (From Fig. 4.8)	22.4	24.44	16	17	23	28
Criteria for liquefaction or non-liquefaction condition (From Fig.4.9	Not liquefy	Not liquefy	liquefy	Not liquefy	Not liquefy	Not liquefy

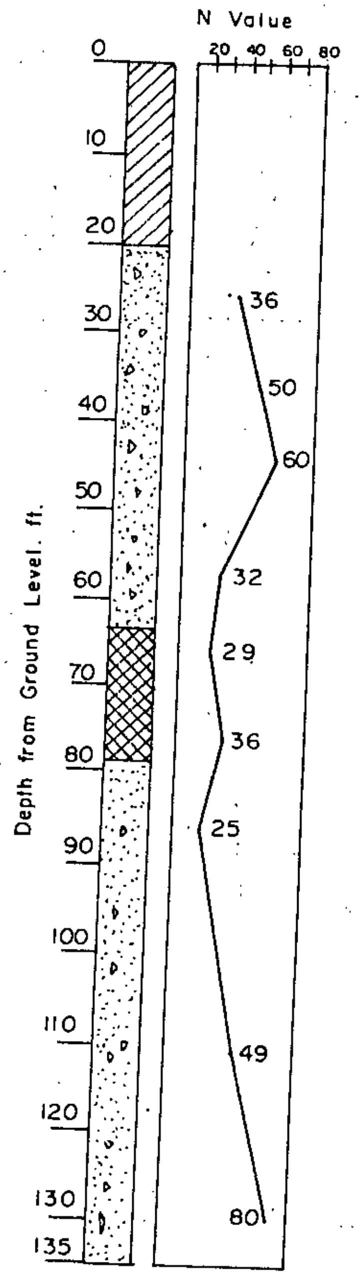


Fig-A1 TYPICAL BORING LOG AT SYLHET

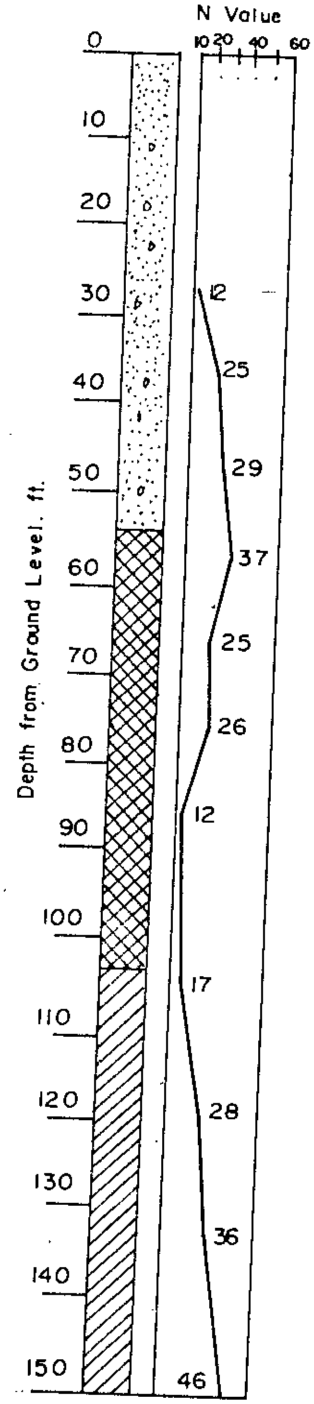


Fig-A2 TYPICAL BORING LOG AT SHERPUR

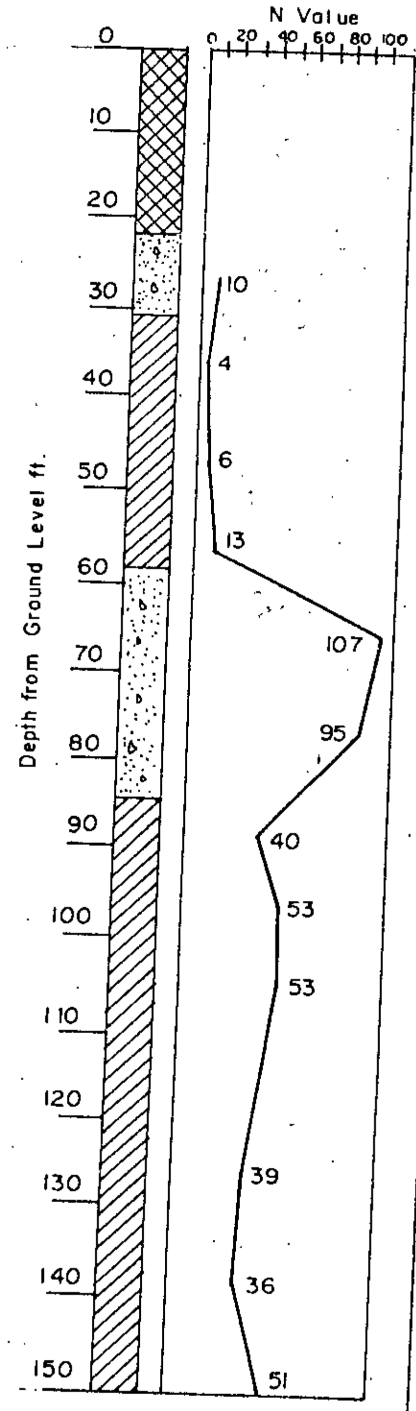


Fig-A3 TYPICAL BORING LOG AT MAULVI BAZAR

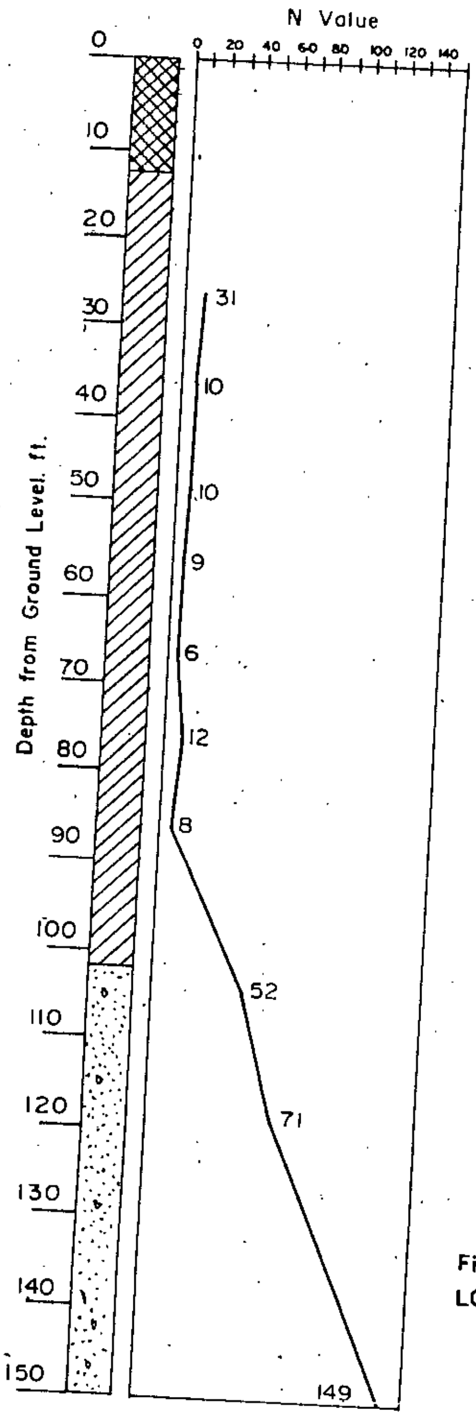


Fig – A4 TYPICAL BORING LOG AT HOBIGONJ

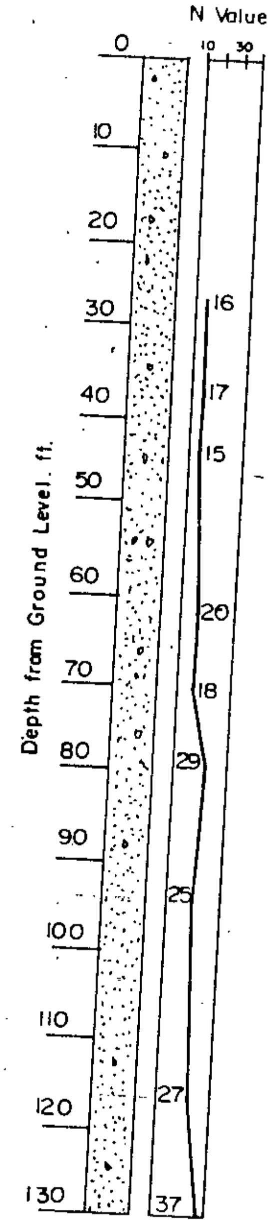


Fig -A5 TYPICAL BORING LOG AT SARAIL

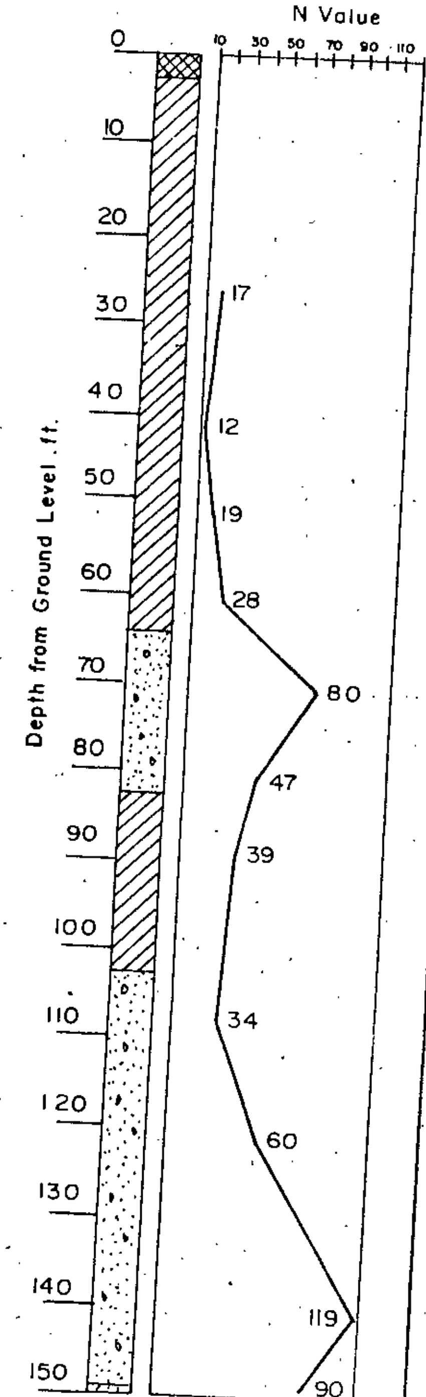
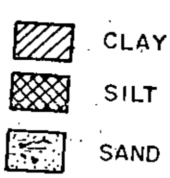


Fig.– A6 TYPICAL BORING LOG AT COMILLA



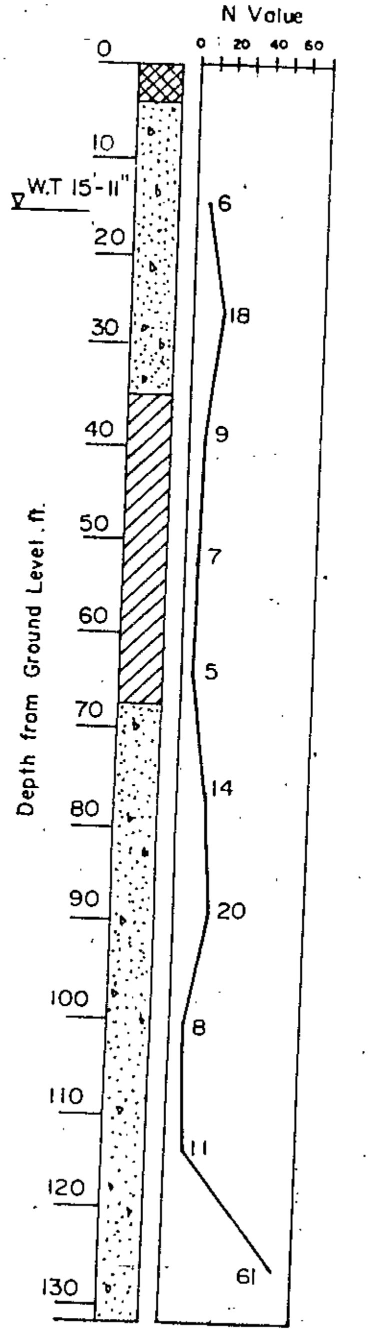


Fig-A7 TYPICAL BORING LOG AT KAMAR KHALI

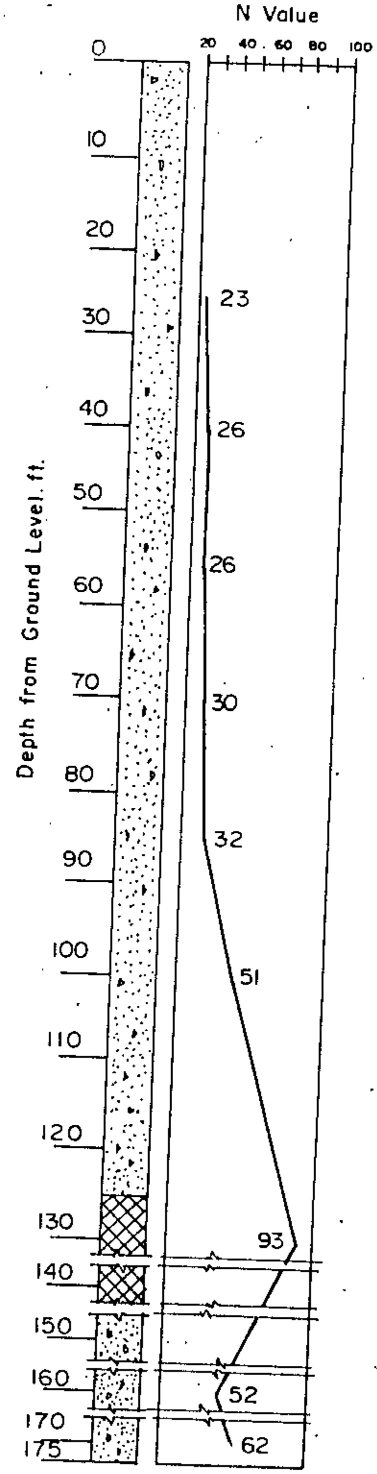


Fig-A8 TYPICAL BORING LOG AT BHAIRAB BAZAR

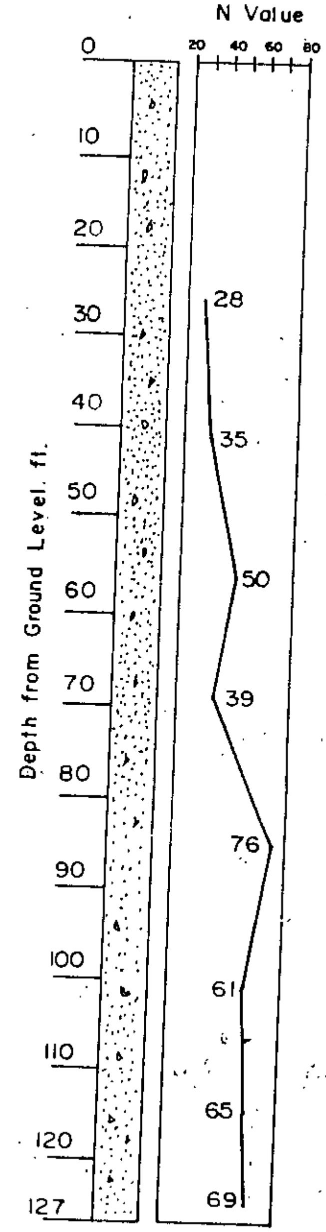


Fig-A9 TYPICAL BORING LOG AT NARSINGDI

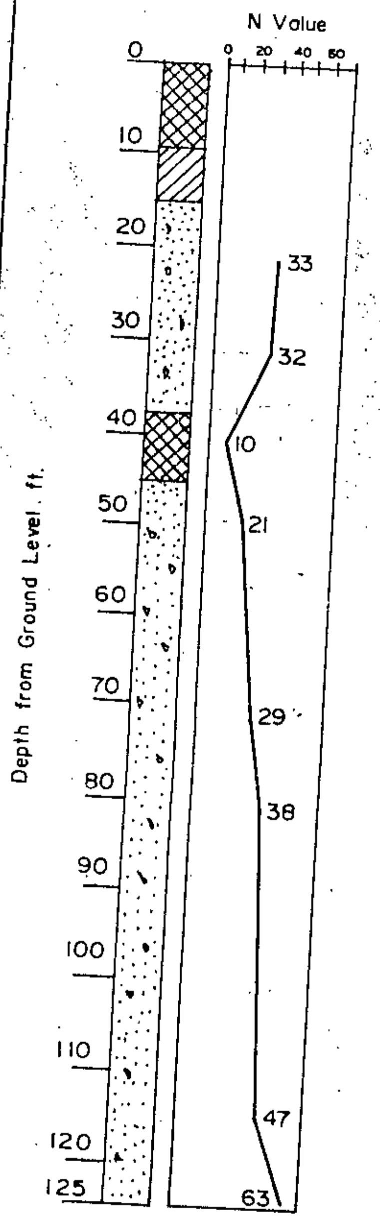


Fig- A10 TYPICAL BORING LOG AT LAKSHMIPUR

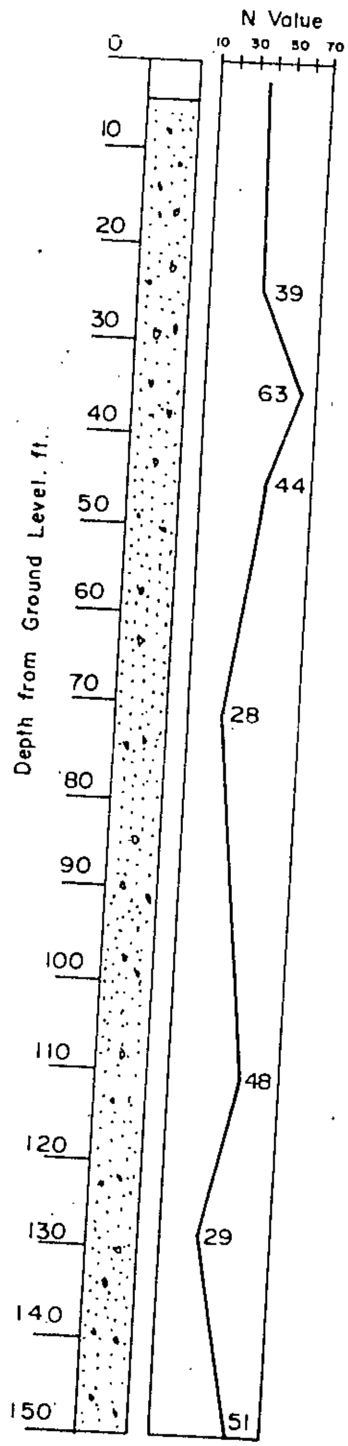


Fig-A11 TYPICAL BORING LOG AT CHANDPUR

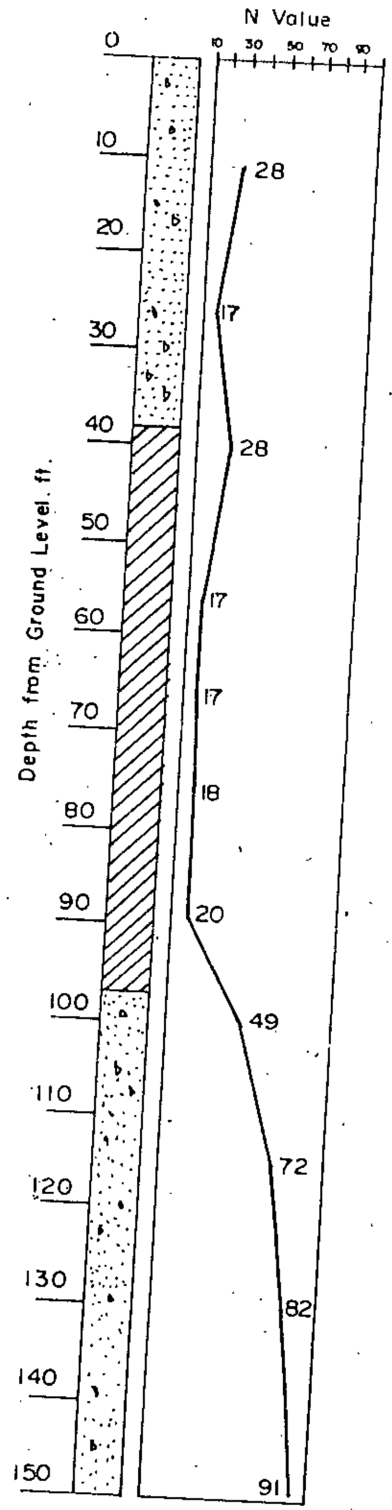


Fig-A12 TYPICAL BORING LOG AT MUKTAGACHA

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