The Cause and Significance of Crack and Structural Defect Assessment of School Building in Dire Dawa City (Case Study on Legehare Cracked School Building in Dire Dawa City)

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This is to certify that the thesis prepared by Hana Getachew, entitled: *The Cause and Significance of Crack and Structural Defect Assessment of School Building in Dire Dawa City (Case Study on Legehare Cracked School Building in Dire Dawa City)* and submitted in partial fulfillment of the requirement for the degree of Master of Sciences in Civil Engineering (Structures) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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ABSTRACT

This thesis focuses on investigating the cause of significant crack on a five story School Building located in Dire Dawa city. The building is constructed with reinforced concrete frame structure with infill hollow block concrete walls. Large diagonal crack is observed on many walls coupled with minor crack on some structural members. The fact that this defect is observed within three to five service years of the building brought attention for investigation.

In this paper different approaches are followed to reach to a conclusion. Diagonal crack on many walls is the first cue for a possible differential settlement problem. Hence, Soil samples were collected around the building and different laboratory tests related to settlement have been carried out. According to the tests and settlement estimation result, there is a significant differential settlement among the foundations.

In addition to the settlement analysis, conventional structural analysis and design has been conducted to verify whether the building structure satisfies the revised Ethiopian code of ES EN 1992 and ES EN 1998:2015. The result shows that most of the structural members are found to be safe with the exception of only few columns and beams were over stressed.

The building frame was further analyzed by considering the effect of foundation settlement. The total settlement obtained from the soil investigation was used as a vertical ground displacement at the corresponding foundation levels and the building was redesigned for this displaced condition. The result from the design showed that significant number of structural members (beams and columns) do not satisfy the requirement of the Ethiopian building code.

1. INTRODUCTION

1.1 GENERAL

Dire Dawa is one of the administrative cities in Ethiopia. It is located at 515km away from the capital city Addis Ababa. It covers about 1,213 km^2 of land. It is geographically located at $7^0$44’30.48’’north and $39^0$13’41.52’’east. Currently the city is in the phase of development and industrialization where many construction projects are in progress. Such as commercial, residential, public buildings and railway transport. Among public buildings, there are hospitals, schools, recreation centers and many with other public services.

In a place where major concern is for fast coverage of service and also with limited resource, it takes a careful feasibility study and skill to keep construction quality high. Hence, problems have been encountered related to public buildings. And cracks on structural and non structural members of buildings after a short service life are common.

Cracks are critical concern to structural engineers because they bring impairment to the service, in fact they make the building itself a threat for its occupants as the building causes damage, serious injury and fatality when it collapses. Also it is important to notice, easing the risk of failure for a cracked building just from visual observation is risky by itself. Thus, such defect on buildings needs to be investigated to the minimum to make sure that the cracks are not progressive.

There are five similar educational buildings in Dire Dawa city owned by Ministry of Education. All the buildings are constructed about the same time frame and opened for service in 2008EC. They have almost similar structural design and construction time except built at different location. But among them Legehare School building in Legehare Kebele is one with significant defects.

Currently, most of the upper stories’ rooms in the building are out of service, but due to shortage of class in the school, the school society are still taking class in ground and first floor rooms. But based on the buildings records, there is no investigation has been taken to identify the cause of these defects. Therefore a quick remedial plan is needed to stop any aggravating condition and to bring the building to acceptable and safer zone.

1.2 OBJECTIVE

The main objective of this study is to investigate the causes of sever cracks on Legehare school building, in Dire Dawa city, which appears just after few service life. And also to deal with the best possible solution that could go with the current condition of the building if it is not beyond repair.

But most importantly, the results of this study should be use as an input to take precaution while constructing future school buildings with similar design in different places. Because constructing similar design at different places is a very common practice when it comes to government projects.

1.3 METHODOLOGY

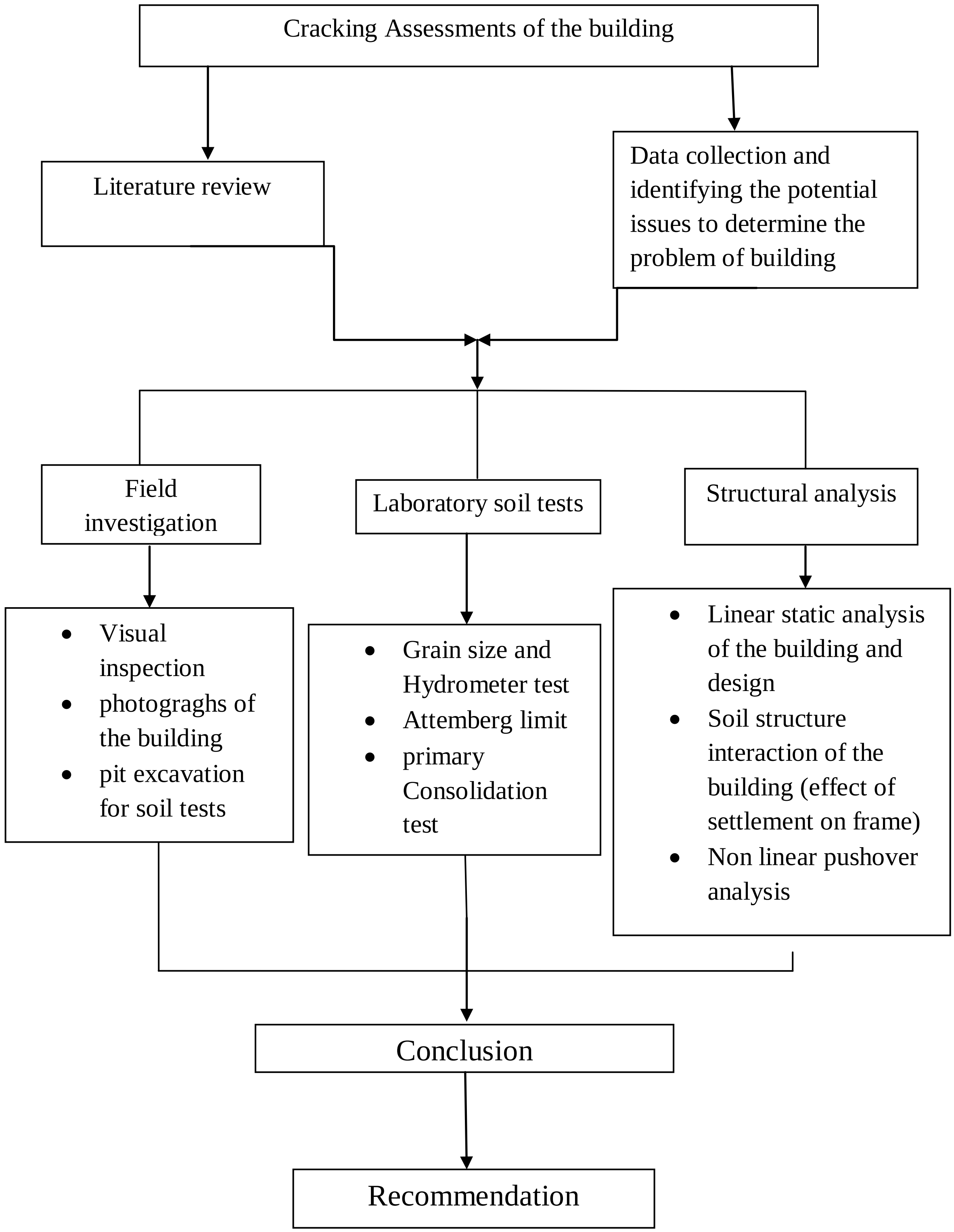
In achieving the above objectives the following methodology is used. Literature review is the initial stage. Numbers of studies from different source have been reviewed to form the framework of the paper and to look for what is important during site inspection phase of the study. It is suggested in many literatures that the existence of diagonal cracks is related to differential settlement. Hence, performing geotechnical investigation was necessary. Accordingly, various soil laboratory tests are conducted. Then differential settlement conditions of the foundations under the structural load are studied based on test results and collected data.

There is a possibility for the structural design to have significant errors which may reduce the required capacity. Thus a design review was made to check whether standard design procedures are used and at the same time to identify any significant design error that contributed to the observed cracks.

The source for almost all the data is Ministry of Education Governmental Office which is the client. The basic reference materials are textbooks, reports and building codes (ES EN 1992-1-1:2015 and ES EN 1998:2015) for structural analysis and design. The modeling of the building for linear and nonlinear (pushover) structural analysis is prepared according to ES EN 1992:2015, FEMA440 and ATC40 recommendations.

Finally, the effect of foundation settlement back to the frame structure is reviewed under soil structural interaction concept. This helps to analyze the safety of the building under the estimated settlement condition. Then summery of all the findings together with proper recommendation concludes this study.

General frame work of methodology



1.4 SCOPE AND LIMITATION

This study mainly focuses on investigating the cause of crack on Legehare shool building. However, the analysis of the super structure analysis made for this building is also applicable to other similar design practice in other place. Effort is made to contain many settlement analyses in the study as to see the soil property from different angles. Although all materials and data collected for this study are found to be enough to reach at conclusion, the following limitations make the study very specific.

- Pits boring for soil investigation are extended only up to 4.5 m depth due to lack of boring equipments.
- In this investigation the issues with respect to building construction qualities are neglected because relevant information were not available. Also the quality of the building construction material is not included in this thesis.
- Some soil characteristics data which are not carried out in laboratory test are gathered from the previous soil test report and literature review.
- Only primary consolidation settlement test is performed for soil laboratory settlement test.

2. LITERATURE REVIEW

Providing a detail review of literature concerning the cause of cracks in general is a good background to see the problem clearly. However, it is difficult to address all in this thesis. So the following are directly linked with the current condition of the building.

2.1 CRACKS

A crack is a complete or incomplete separation of any brittle material into two or more parts, produced by breaking or fracturing as a result of the strains that induce tensile stress in excess of the material's capacity. The crack in concrete and building walls is an inherent feature, which cannot be completely prevented but can only be controlled and minimized. The crack propagate without limit it may cause the structural failure.

Cracks in building could be broadly classified as structural and non structural cracks. And it causes by a multiple reason, specifically diagonal crack can appear in many structural materials and components including block, brick, and concrete foundation walls, chimneys and building interior drywall or plaster. Most studies shown that diagonal cracking in masonry structures usually caused by differential settlements, occasionally frost heaves and more rarely the development of a sink hole under or near the foundation.

Generally, incorrect analysis during design, faulty construction and related to foundation overloading and differential settlements are some of the main reasons to structural and non structural cracks of building.

2.2 EVALUATION OF CRACKS

Before proceeding with repairs an evaluation should be made to determine the location and extent of cracking, the cause of the cracking and the need for repair.

Cracks need to be repaired if they reduce the strength, stiffness, or durability of the appearance of the building structural elements otherwise it can be ignored. Location and extent of cracking as well as information on the general condition of the concrete can be determined by different evaluation methods. The evaluation of cracks is necessary for the following purpose

- ✓ To identify the cause of cracking
- ✓ To assess the structure for its safety and serviceability
- ✓ To establish the extent of the cracking
- ✓ To establish the likely extent of further deterioration
- ✓ To study the suitable of various remedial measures
- ✓ To make a final assessment for serviceability after repairs

The commonly used methods in cracking evaluation are

- ✓ Visual examination
- ✓ Non destructive testing
- ✓ Various laboratory and field soil tests
- ✓ Review of designs for details

Identifying the cause of the cracks should be the first priority to find the solution. And it can be caused by various reasons.

Cause of cracking due to

- Thermal cracking
- chemical reaction
- weathering
- corrosion of reinforcement
- poor construction practice
- errors in analysis, design and detailing
- settlement of foundation

2.2.1 DIAGONAL CRACKS

Diagonal cracks in buildings such as the one shown in Fig 2.1 occur when there is large differential settlement of foundation either due to unequal bearing pressure under different parts of the structure or due to bearing pressure on soil being in excess of safe bearing strength of the soil or due to low factor of safety in the design of foundation. Moreover, there are other major causes such as earthquakes and floods which are not covered in this thesis. [41]

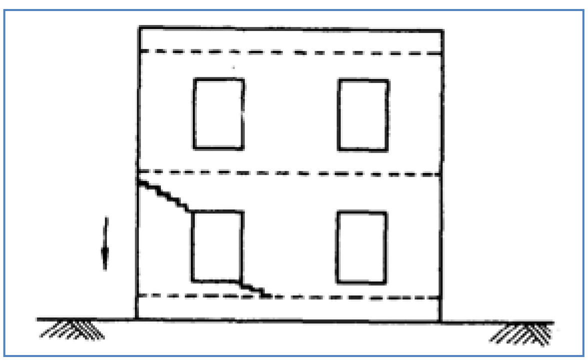


Figure 2.1Building diagonal cracks

And also mainly, differential settlements in buildings occur when there are local variations in the nature of supporting soil ‘and such variations are not detected and taken care of in the foundation design at the time of construction. In order to avoid settlement cracks in buildings, it is essential that designs for their foundations are based on sound engineering principles and good practice.

2.3 SETTLEMENT ANALYSIS

Settlement is defines as the downward displacement of foundation as a result of stress increase caused by the construction of foundations or other loads compresses soil layers.

Two basic types of settlement:

- *Settlement due directly to the weight of the structure.* The first type of settlement is directly caused by the weight of the structure. For example, the weight of a building may cause compression of an underlying sand deposit or consolidation of an underlying clay layer. The settlement analysis must include the actual dead load of the structure. The dead load is defined as the structural weight due to beams, columns, floors, roofs, and other fixed members.
- *Settlement due to secondary influences.* The second basic type of settlement of a building is caused by secondary influence, which may develop at a time long after the completion of the structure. This type of settlement is not directly caused by the weight of the structure. For example, the foundation may settle as water expelled out of the soil under the footings and causes unstable soils to collapse. The foundation may also settle due to yielding of adjacent excavations or the collapse of limestone cavities or underground mines and tunnels. Other causes of settlement that would be included in this category are natural disasters, such as settlement caused by earthquakes or undermining of the foundation from floods. [19]

Virtually all references agree that the total settlement of a foundation is composed of three components graphically as shown in Fig 2.2 they are immediate settlement, consolidation or primary settlement and secondary settlement or creep

1. *Elastic settlement* (or *immediate settlement*), which is caused by the elastic deformation of dry soil and of moist and saturated soils without any change in the moisture content. Elastic settlement calculations generally are based on equations derived from the theory of elasticity.
2. *Primary consolidation settlement,* which is the result of a volume change in saturated cohesive soils because of expulsion of the water that occupies the void spaces.
3. *Secondary consolidation settlement,* which is observed in saturated cohesive soils and is the result of the plastic adjustment of soil fabrics. It is an additional form of compression that occurs at constant effective stress. [14]

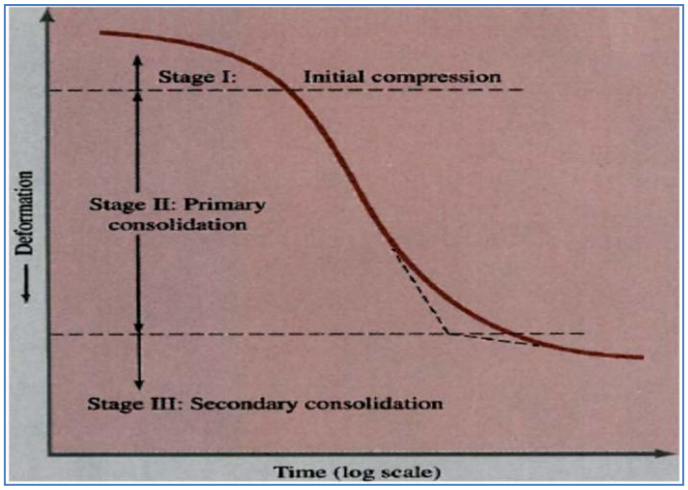


Figure 2.2Typical time vs. deformation relationship for fine-grain soil [17]

The immediate settlement is also sometimes called *volume distortion* settlement. Essentially, this is the rearrangement of grains due to changing stress, resulting in a reduction in void ratio and instant settlement. In most texts, immediate settlement is considered to be an elastic process. Therefore the settlement can be calculated from elastic theory. The important consideration, this immediate settlement calculation from elastic theory will ***underestimate*** the actual settlement. Bowles cited Holtz and Kovaks [1981] state, ″The immediate, or distortion, settlement, although not actually elastic is usually estimated by using elastic theory″. It is recommended that a lower value of *E* be used to account for the plastic flow. [13]

The most difficult part of a settlement analysis is the evaluation of the modulus of elasticity *Es* that would conform to the soil condition in the field. and it can be determine form both laboratory and field tests but due to the disturbance of the sample and other reasons the laboratory test do not represent the actual conditions and normally give low values. [52]

One of the field test use to estimate elastic modulus of the soil is Standard penetration test (STP), the standard penetration test (STP) boring is the most popular sampling and in-situ penetration resistance testing method. The number of blow required to drive the split spoon 30cm is recorded and called “The STP N-value”.[20]

The net elastic settlement equation for a flexible surface footing [52];

$$S_e = q_n B \frac{(1-\mu^2)}{E_s} I_f \qquad (2.1)$$

The Primary consolidation is the gradual re-arrangement of grains as water is expelled. This component of settlement is usually dominant in fine grained saturated clays. But consolidation also occurs in other soil types.

2.3.1 STRESS DISTRIBUTION IN SOIL DUE TO FOOTING PRESSURE

Estimation of vertical stresses at any point in a soil-mass due to external vertical loadings has a great significance in the prediction of settlements of buildings, bridges, embankments and many other structures. Equations have been developed to compute stresses at any point in a soil mass on the basis of the theory of elasticity.

When a load is applied to the soil surface as shown in Fig 2.3, it increases the vertical stresses within the soil mass. The increased stresses are greatest directly under the loaded area, but extend indefinitely in all directions. Many formulas based on the theory of elasticity have been used to compute stresses in soils. They are all similar and differ only in the assumptions made to represent the elastic conditions of the soil mass. The formulas that are most widely used are the Boussinesq and Westergaard formulas. [52]

Boussinesq's equation considers a point load on the surface of a semi-infinite, homogeneous, isotropic, weightless, elastic half-space to obtain q_v [13]

$$q_v = \frac{3Q}{2\pi z^2}\cos^5\theta = \frac{Q}{z^2} A_b \tag{2.2}$$

A_b has the maximum value of 0.477 at r/z = 0

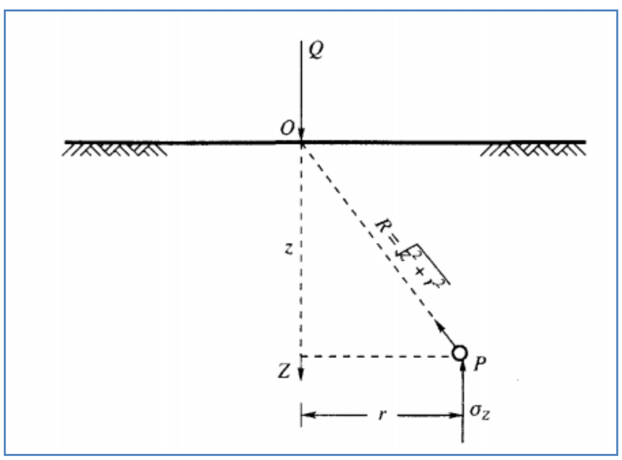


Figure 2.3Vertical pressure within an earth mass [50]

Boussinesq assumed that the soil is elastic, isotropic and homogeneous for the development of a point load formula. However, the soil is neither isotropic nor homogeneous. The most common type of soils that are met in nature are the water deposited sedimentary soils. [52]

When the soil mass consists of layered strata of fine and coarse materials, as beneath a road pavement, or alternating layers of clay and sand, some authorities are of the opinion the Westergaard (1938) equations give a better estimate of the stress q_v.[13]

The Westergaard equations, unlike those of Boussinesq, include Poisson's ratio μ, and the following is one of several forms given for a point load *Q:* [13]

$$q_v = \frac{Q}{2\pi} \frac{\sqrt{a}}{[a+(r/z)^2]^{3/2}} = \frac{Q}{z^2} A_w \tag{2.3}$$

Where $a = (1 - \mu)/ (2 - \mu)$, for r/z = 0 taken A_w=0.32

2.3.2 CONSOLIDATION THEORY

If a fully saturated soil subjected to an increased total stress is allowed to gradually reduce in volume due to dissipation of excess pore pressure, it is said to have primary consolidated. This process might take a long time and when all excess pore pressure has dissipated, the soil is considered fully consolidated.

A clay is said to be normally consolidated if the present effective overburden pressure is the maximum pressure to which the layer has ever been subjected at any time in its history, whereas a clay layer is said to be over consolidated if the layer was subjected at one time in its history to a greater effective overburden pressure, than the present pressure. The ratio overburden pressure,Po to present pressure or preconsolidation pressure, Pc is called the overconsolidation ratio (OCR).[52]

Several methods have been proposed for determining the value of the maximum preconsolidation pressure. They fall under field method and graphical procedure based on consolidation test results.

The earliest and the most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B, on the laboratory e-log *p* curve of an undisturbed sample as shown in Fig 2.4. From *B*, a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the preconsolidation pressure Pc. [52]

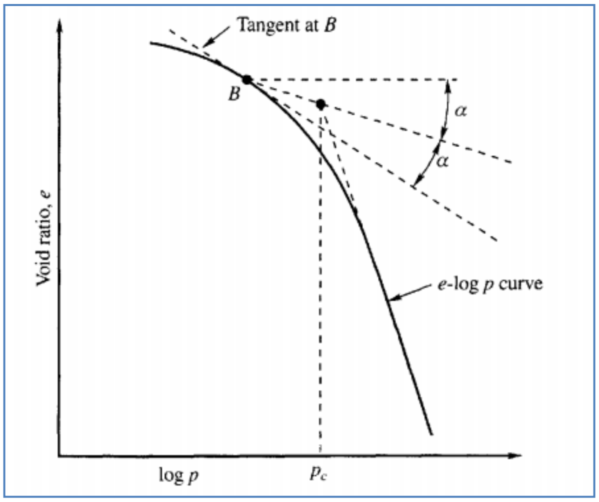


Figure 2.4 Method of determining p by Casagrande method

Therefore, equation for computing primary consolidation settlement for normally consolidated clay, OCR=1 is

$$S_c = H\frac{Cc}{1+e_o}\log\frac{p_o+\Delta p}{p_o} \tag{2.4}$$

For over consolidated clay the settlement calculation depends upon the excess foundation pressure *Pc* over and above the existing overburden pressure *Po*, OCR > 1

(a) If Po + ΔP < Pc, $S_c = H\frac{Cs}{1+e_o}\log\frac{p_o+\Delta p}{p_o}$ (2.5a)

(b) If Po < Pc < Po + ΔP, $S_c = H\frac{Cs}{1+e_o}\log\frac{Pc}{p_o} + H\frac{Cc}{1+eo}\log\frac{\text{Po}+\Delta\text{P}}{\text{p}_\text{c}}$ (2.5b)

The secondary settlement represents time-dependent settlement, or creep, that occurs under a *constant effective stress*. This component of settlement is important in organic soils.

Generally the analysis of settlement consists of two major components (elastic settlement and primary consolidation) which are the value of total settlement.

Computation of settlement of a soil deposit in the field is consist of two parts

- ✓ computation of magnitude of final settlement
- ✓ determination of the time-rate of settlement

For the computation of final settlement, the coefficient of volume change or the compression index is required, determine from void ratio and the effective stress curve. For the time rate of computation, coefficient of consolidation requires using the Terzaghi theory.

2.3.3 DEFINING DIFFERENTIAL SETTLEMENT

Differential or uneven settlement occurs when the soil beneath a structure cannot bear the weights imposed. The settlement of a structure is the amount that the structure will “sink” during and after construction. Differential settlements become a big problem when the foundation settles unevenly. The more uneven the settlement is, the greater the problems are to the building's structure.

If the structure as a whole settles uniformly into the ground there will not be any detrimental effect on the structure as such. The only effect it can have is on the service lines, such as water and sanitary pipe connections, telephone and electric cables etc. which can break if the settlement is considerable. But the differential settlements between the footings have a big effect on any structures. [52]

Differential settlement is a general term used to describe the differences in vertical displacement of foundations. However differential settlement on its own does not give any indication of the spatial variation. It is the magnitude of differential settlement combined with the spatial variation that influences the behavior of the structure.

Angular distortion is defined as the ratio of the differential settlement (δ) and the distance (l) between two points after eliminating the effect of tilt of the building. And also they defined a slope, equivalent to angular distortion; and relative deflection as the ratio of deflection to the length of the deflected part. Subsequently similar definitions have been stated by a number of authors. [50]

Determining the settlement behavior of the proposed structure is one of the primary obligations of the geotechnical engineer. The following parameters are often required:

- *Total settlement.* Also known as the maximum settlement, it is the largest amount of settlement experienced by any part of the foundation, such as shown in fig.
- *Maximum differential settlement* δ. The maximum differential settlement is the largest difference in settlement between two different foundation locations, such as shown in fig. The maximum differential settlement does not necessarily occur at the same location as the total settlement.
- *Rate of settlement.* It is often desirable to know if the settlement will occur during construction as the dead load is applied to the soil, or if the settlement will occur over the life of the project, in which case there may be on-going cracking to the structure.
- *Maximum angular distortion d/l.* The angular distortion is defined as the differential settlement between two points divided by the distance between them less the tilt, where tilt equals rotation of the entire building [48]. As shown in fig. The highest value of d/l would be the maximum angular

distortion. The location of the maximum angular distortion does not necessarily occur at the location of the total settlement or maximum differential settlement.

2.3.4 FACTORS PRODUCING DIFFERENTIAL SETTLEMENT

Differential settlement between the footings as seen in fig 2.5 may cause damage to structures. A good understanding of mechanisms and factors producing differential settlement will result in a better understanding of the behavior of the structure and will therefore allow for a more optimal design. There have many reasons to the cause of differential settlement such as, soil variability, loads and their variability, foundation load-displacement response and building stiffness on differential settlement.

Since varying stiffness and strength of soil beneath foundations is a potential cause of differential settlement.

Soils are heterogeneous materials created by complex geological processes. Soil properties vary from point to point, even in the same strata. [50] Discussed how soil variability can be linked to complex depositional conditions. And also [28] presented the value of a comprehensive geological model in understanding soil conditions on a site, but also states that, regardless of the detail and the amount of work involved, the geological model is unlikely to achieve the same qualitative accuracy as the structural engineering design because of the inherent complexity and in homogeneity of the soil. Other researchers have investigated and quantified the spatial variability of natural soils.

Soil variability to have an effect on settlement, it must be within the stress influence zone of the foundation. For a flexible square foundation, the vertical stress at a depth of 3B, where B equals the width of the foundation, is less than 6% of the surface stress Therefore most soil compression will occur within a 3B depth and the focus should be on soil variations within this zone. [9]

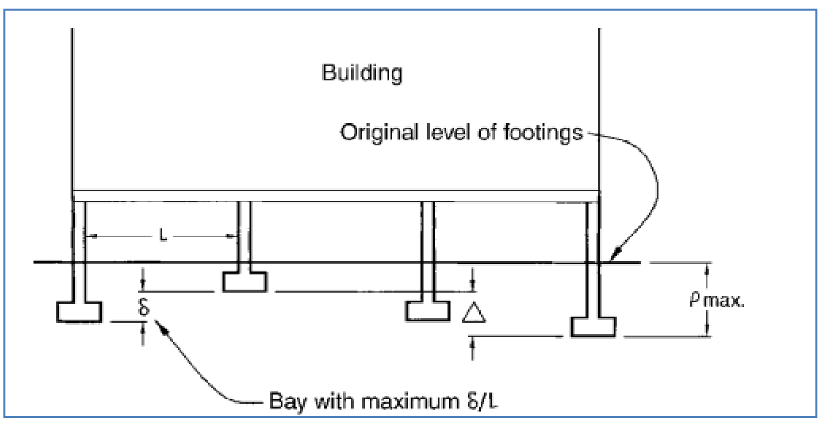


Figure 2.5 Differential foundation settlement

2.3.5 ALLOWABLE SETTLEMENT

Settlements are quantified either by a measurement of the total settlement or differential settlement. Differential settlements are more damaging than uniform settlements. Total settlements, therefore, are often of less interest than differential settlements. Differential settlements can be measured in multiple ways, but the most commonly used measurement is angular distortion.

According to the revised code ES EN 1997 the maximum acceptable relative rotations θ_{max} shown in Fig 2.6 for open framed structures, infilled frames and load bearing or continuous brick walls are unlikely to be the same but are likely to range from about 1/2000 to about 1/300, to prevent the occurrence of a serviceability limit state in the structure. A maximum relative rotation of 1/500 is acceptable for many structures. The relative rotation likely to cause an ultimate limit state is about 1/150. [23]

Unequal settlement of the subsoil may lead to cracks in the structural components and rotation. It may be cause by

i) Non uniform nature of the subsoil throughout the foundation
ii) Eccentric loading
iii) Improper design of the base footing

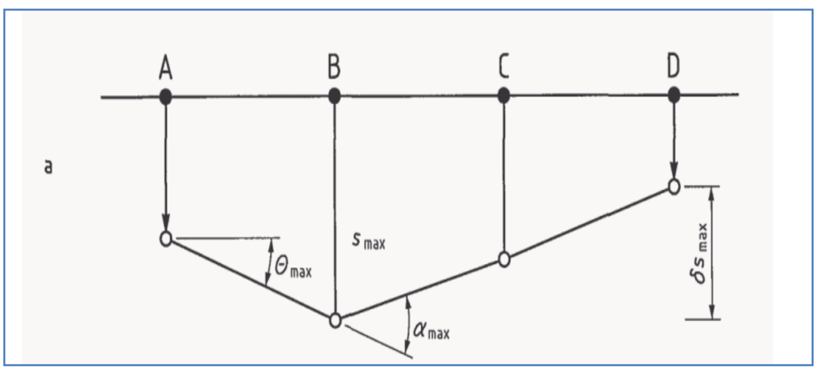


Figure 2.6 Settlement s, differential settlement δs, rotation θ and angular strain α

2.3.6 LABORATORY SOIL TEST

Mechanical analysis is the determination of the size range of particles present in a soil, expressed as a percentage of the total dry weight. Two methods generally are used to find the particle-size distribution of soil: (1) *sieve analysis*—for particle sizes larger than 0.075 mm in diameter, and (2) *hydrometer analysis*—for particle sizes smaller than 0.075 mm in diameter. The basic principles of sieve analysis and hydrometer analysis are described briefly in the following two sections. [13]

Sieve analysis consists of shaking the soil sample through a set of sieves that have progressively smaller openings.

Hydrometer analysis is based on the principle of sedimentation of soil grains in water. When a soil specimen is dispersed in water, the particles settle at different velocities, depending on their shape, size, weight, and the viscosity of the water.

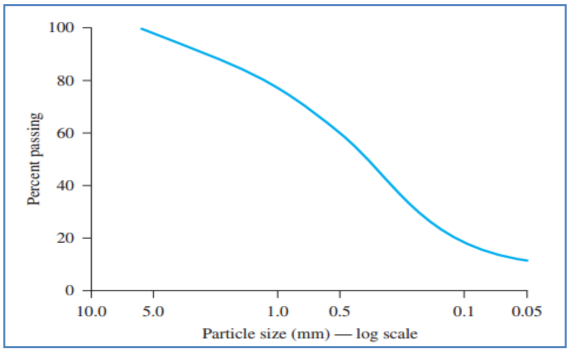


Figure 2.7 Particle-size distribution curve

Once the percent finer for each sieve is calculated, the calculations are plotted on semi logarithmic graph paper with percent finer as the ordinate (arithmetic scale) and sieve opening size as the abscissa (logarithmic scale). This plot shown in Fig 2.7 is referred to as the *particle-size distribution curve.*

I. Consistency of soil and determination of consistency limits plasticity of soils

The plasticity of a soil is its ability to undergo deformation without cracking or fracturing. A plastic soil can be molded in to various shapes when it is wet. Plasticity is an important index property of fine grained soil, especially clayey soils. Plasticity is due to the presence of clay minerals. The absorbed water helps particles to slip one over the other plastically. If water content decreases, the plasticity of soil decreases. The soil becomes plastic only when it has clay minerals. Non-clay minerals, with whatever finer, cannot become plastic. It cannot be rolled in to thread.

a. Liquid limit

A soil containing high water content is in a liquid state. It can flow like liquids (no resistance to deformation). If the water content is reduced gradually, at particular water content the soil becomes plastic (*show small shearing strength*). The water content at which the soil changes from the liquid state to the plastic state is called liquid limit (LL). Or the liquid limit is the water content at which the soil ceases to be liquid.

b. Plastic limit

The soil in the plastic state can be molded in various shapes. When water content is further reduced gradually, the soil cracks when molded (it becomes semi-solid).the water content at which the soil becomes semi-solid is known as plastic limit (PL). Or the plastic limit is the water content at which the soil just fails to behave plastically. Or plastic limit is the minimum moisture content at which the soil clay can be rolled in to a thread of 3 mm diameter without cracking.

c. Plasticity index

The numerical difference between the liquid limit and the plastic limit is known as plasticity index (PI).

It is the range of water content over which the soil exhibits plasticity. PI = LL – PL

Table 2.1; Description of the Strength of Fine-Grained Soils Based on Liquidity Index [41]

Values of LI	Description of soil strength
LI < 0	Semisolid state—high strength, brittle, (sudden) fracture is expected
0 < LI < 1	Plastic state—intermediate strength, soil deforms like a plastic material
LI >1	Liquid state—low strength, soil deforms like a viscous fluid

II. The standard one dimensional consolidation test

The main purpose of the consolidation test on soil samples is to obtain the necessary information about the compressibility properties of a saturated soil for use in determining the magnitude and rate of settlement of structures. The following test procedure is applied to any type of soil in the table consolidation test apparatus shown on Fig 2.8. Loads are applied in steps in such a way that the successive load intensity, *P*, is twice the preceding one. The load intensities commonly used 1/4, 1/2, 1, 2, 4, 8, and 16 tons/ft^2 (25, 50, 100,200,400, 800 and 1600 kN/m^2). Each load is allowed to stand until compression has practically ceased (no longer than 24 hours). The dial readings are taken at elapsed times of 1/4, 1/2, *1,2,4,* 8, 15, 30, 60, 120, 240, 480 and 1440 minutes from the time the new increment of load is put on the sample (or at elapsed times as per requirements). ***Sandy samples*** are compressed in a relatively short time as compared to clay samples and the use of one day duration is common for the latter. After the greatest load required for the test has been applied to the soil sample, the load is removed in decrements to provide data for plotting the expansion curve of the soil in order to learn [51]

Figure 2.8 Table consolidation test apparatus

Incremental loads including unloading sequences are applied to the platen and the settlement of the soil at various fixed times under each load increment is measured by a displacement gauge. Each load increment is allowed to remain on the soil until the change in settlement is negligible and the excess pore water pressure developed under the current load increment has dissipated. For many soils, this usually occurs within 24 hours. [37]

Finally, it is necessary to analyze the test results in the relation of pressure versus void ratio curve as shown in Fig 2.9.

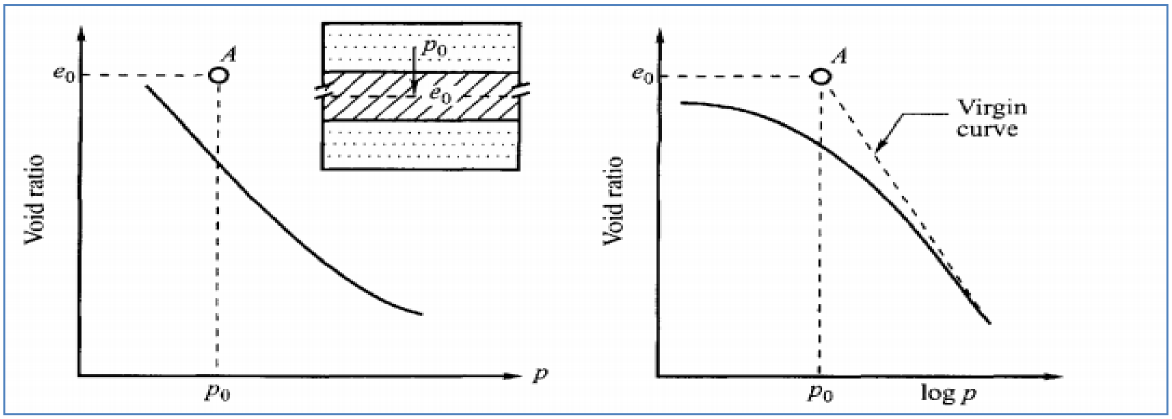


Figure 2.9 Pressure void ratio curve

2.4 CRACKING DUE TO ANALYSIS AND DESIGN ERROR

Design errors are unavoidable in any building designs and can negatively affect cost, schedule and safety performance of the structures. The different types of design drawings may have various levels of design errors due to many factors such as unclear overview of the designs, lack of coordination process, and human mistakes.

This design and detailing errors that may results in unacceptable cracking include use of poorly detailed re-entered corners in walls, slabs, restraint of members subjected to volume changes by vibration in temperature and moisture, lack of adequate contraction joint and improper design of foundation resulting in differential settlement within the structure. Re-entrant corners provide location for stress concentration is the prime location for initial cracks, in the case of window and door opening in concrete walls and beams. Additional properly anchored diagonal reinforcement is required to keep inevitable crack narrow and prevent them from propagating further.

Inadequate amount of reinforcement may results in excessive cracking. A common mistake is to lightly reinforce an element because it is a non-structural element and tying it to rest of the structure in such a manner that it is required to carry a major portion of the load once the structure begin to deform. The non structural elements carry the load. Since this element is not detailed to act structurally, unsightly cracking may results even thought the safety of the structure is not threatened. The restrained members subjected to volume change frequently develop cracks. A slab, wall or a beam restrained against shortening, even if pre stressed, can easily develop tensile stress sufficient to cause cracking. Beams should be allowed to move.

2.5 EVALUATING THE PERFORMANCE OF THE EXISTING BUILDING

Existing building structures deform inelastically when subjected to earthquake loads, so seismic performance evaluation of structures should be conducted considering post-elastic behavior. Therefore, a nonlinear analysis procedure must be used for evaluation purpose as post-elastic behavior cannot be determined directly by an elastic analysis. Moreover, maximum inelastic displacement demand of structures should be determined to adequately estimate the seismically induced demands on structures that exhibit inelastic behavior

Four distinct analytical procedures can be used in systematic rehabilitation: linear static, linear dynamic, nonlinear static and nonlinear dynamic procedures. The choice of analytical method is subject to limitations based on building characteristics. The linear procedures maintain the traditional use of a linear stress-strain relationship, but incorporate adjustments to overall building deformations and material acceptance criteria to permit better consideration of the probable nonlinear characteristics of seismic response. The nonlinear static procedure, often called "pushover analysis," uses simplified nonlinear techniques to estimate seismic structural deformations. The nonlinear dynamic procedure, commonly known as nonlinear time history analysis, requires considerable judgment and experience to perform, and may only be used within the limitations described. [27]

2.6 SOIL STRUCTURAL INTERACTION

Soil-structure interaction is interdisciplinary field which involves structural and geotechnical engineering. In the conventional analysis of building frame, the structural designer assumes that columns are resting on unyielding rigid base. Similarly, in foundation design, foundation settlements are calculated without considering the influence of the structural, stiffness. Although, interaction effect is ignored to simplify the mathematical model but neglecting the interaction between soils and structures may result in a design that is either unnecessarily expensive or unsafe. A more rational solution of soil-structure interaction problem can be achieved with computational validity and accuracy by appropriate analysis.

The building frame, foundation and soil mass form a complete structural system to resist the external loads. The mechanics of soil-structure interaction takes place between these components. The superstructure, foundation and soil mass can be considered as a single integral compatible structural unit for carrying out the interaction analysis to predict more realistic behavior. The stress-strain characteristics of the supporting soil play a vital role in the interaction analysis. The resulting differential settlements of the soil mass are also responsible for the redistribution of forces in the superstructure.

Most of the civil engineering structures involve some type of structural element with direct contact with ground. When the forces, such as foundation settlement, earthquakes, act on these systems, neither the structural displacements nor the ground displacements, are independent of each other. The process in which the response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil is termed as *soil-structure interaction (SSI).* [10]

Several investigators have studied the influence of the phenomenon of soil-structure interaction in framed structures and investigated that the force quantities are revised due to interaction. [7] Provided a highlight about how to use the spring in static analysis with account long and short term deformation of the sub grade soil. [34] proposed an economical iterative procedure for building frames and found significant reduction in differential settlements and consequent additional moments.[37] presented an interaction analysis of a seven-storey; three-bay framed structure in which the soil mass was treated as a Wrinkler's or elastic half space medium.

2.6.1 EFFECT OF DIFFERENTIAL SETTLEMENT ON FRAME STRUCTURE

The dynamic nature of loads, seasonal variation of soil properties, uneven soil strata below footings or inappropriate design of foundation makes differential settlement inevitable. Differential settlement is largely responsible for developing forces or changing the existing forces in the structure and is often considered as the underlying cause of many structural failures.

Differential settlement can cause a significant tilt in the structure, making the occupants uncomfortable. Cracks in the foundation and interior walls, non-uniform settling of doors and windows, sinking of chimneys, bulging walls and sunken slabs are often considered as the adverse effects of differential settlement and can be devastating to a building. These effects are a result of increased axial force, shear force and bending moments in the structure. It is common to implement repair and maintenance measures to prevent or reduce the effects of differential settlement. [52]

The responses of structure in terms of axial forces and vertical displacements of columns in each floor, bending moments and shear forces in beams were analyzed when the corner, edge and center columns were subjected to a settlement of 25 mm. The deformation in structure is elastic for settlement up to 25 mm and the most critical case was that of the center column. It is also inferred that the adjacent beam of the settling column develops significant bending moment, the settling column develops tensile forces while the adjacent column develops compressive forces. The effect of differential settlement is limited to one span from the settling column. [13]

3. VISUAL INSPECTION OF THE SELECTED BUILDING

Most valuable or important data are obtained from visual analysis of a building. A building itself can speak the problem underneath it. The visual observation leads to the proper methods of investigation which helps to identify the cause of a failure. Therefore multi site visits has been done to observe current situation of Legehare school building shown in Fig 3.1.

Figure 3.1Existing building over view

3.1 DAMAGES OBSERVED AT THE SELECTED BUILDING

There are five school buildings which are constructed and opened to service in Dire Dawa city have the same design, construction time and cost at a various place. But on one of those buildings, crack observed within three up to five years after construction completion as can be seen in Fig.3.2 up to Fig 3.9 The four other buildings, except very few nonstructural defects, they are still safe and in a proper service condition.

The Diagonal cracks on wall are constant width throughout their lengths and seem to have occurred at the same time possibly caused by differential settlement or as a result of analysis and design or other. Diagonal cracks emerging from the corners of beams pocket, door and windows edges, and the vertical ones were identified. The slab and some of the beams and columns also suffer severe hair cracks extending through the walls possibly caused by; uneven settlement in the foundation soil putting the building under tension some of these cracks are shown from Fig 3.2 up to Fig 3.9

Figure 3.2 Diagonal cracks

Figure 3.3 External infill walls diagonal cracks along Axis B

Figure 3.4 Internal infill walls diagonal cracks along Axis B

Figure 3.5 Infill walls diagonal cracks along the Axis C

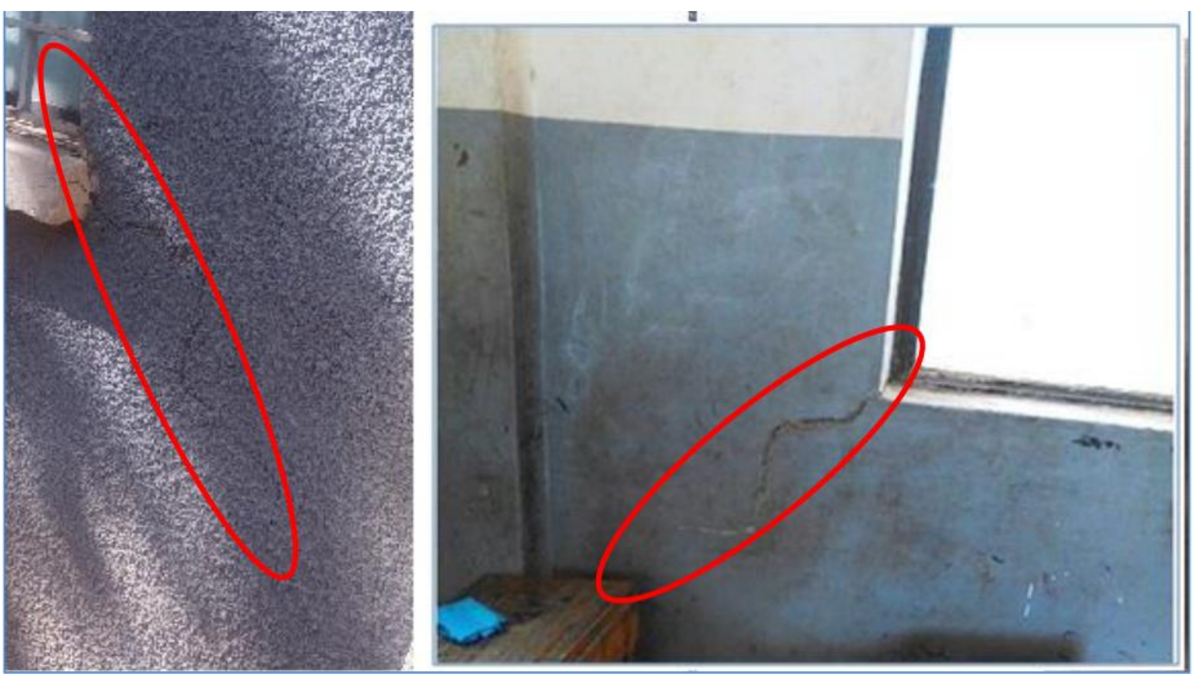


Figure 3.6 External and internal side’s window corner diagonal cracks

Figure 3.7Along axis C column crack

Figure 3.8 Some hair cracks on axis B and cracks at the edge along the axis C columns

Figure 3.9 Building's steel doors distortion

It would be difficult to address the entirely causes of the building cracks in once. Therefore, in this thesis the investigation focused on main reasons of the defects based on the cracks observed in the building. These are

- Cracking due to differential settlement of foundation discussed in part one under soil analysis part
- The other one is cracking due to linear analysis and design errors and non linear analysis of the building structure covered in part two under structural analysis part. And finally the building under super structural load and the sub structural reaction combined effect are studied by applying a vertical displacement at the base of each column.

4. RESULT AND DISCUSSION

4.1 EXPERIMENTAL AND NUMERICAL ANALYSIS OF DIFFERENTIAL SETTLEMENT

Previous studies showed us diagonal cracks are the very likely results of differential settlement. And as shown in the previous figures most of the defects appear as diagonal crack. Therefore to avoid any assumption and study the actual cause of damage, settlement test has to be made.

4.1.1 BORING AND SAMPLING OF SOIL

The soil used for various tests were collected from Legehare site in Dire Dawa. Number of pit trial required is recommended differently by different Codes depending on the type and dimension of the building. For this investigation four pits were excavated within an appropriate spacing for horizontal stratification of soil as shown in Fig 4.1. Due to the scarcity of drilling machine at proper time, exploring the soil was conducted up to 4.5m with the help of different hand boring equipments. The vertical soil stratification was observed by taking three samples within 1.5m spacing for both disturbed and undisturbed samples of each pit.

In each pit, along the vertical strata, the soil character varied. For the safety of the existing building each excavation was conducted at least 3m away from the building's columns. The disturbed and undisturbed samples were wrapped and contained in plastic bags. Undisturbed samples were covered with wax at the top and bottom of the sampler tubes which protects the soil samples from losing moisture content. Then the samples were transported safely to AAiT soil laboratory.

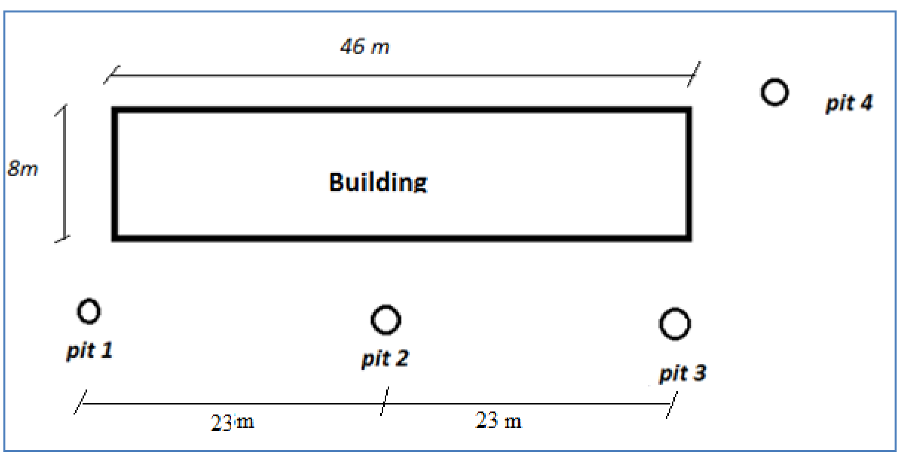


Figure 4.1 Excavated pit location around the building

Figure 4.2 Pit 1 and 4 excavations around the building and taking soil samples

4.1.2 LABORATORY TESTING

Laboratory test is an essential aspect of structural and geotechnical engineering. It gives an approximation about the settlement behavior of a foundation. In this study, laboratory test was conducted using ASTM standard procedures. It provides a clear understanding and knowledge of the soil properties.

Sieve and hydrometer, Attemberg limits and one dimensional consolidation tests are conducted to study the behavior of the soil from each pit.

Site investigation and soil laboratory test had been performed before the design and construction of the building. So soil tests and report data gathered from Ministry of Education Office in Dire Dawa (owner) were helpful to fulfill some missing information about the soil condition.

a. Index properties and soil classification tests

A series of index property tests were conducted on the soil samples to identify the types of soil. The main physical index property of the soils investigated in this study was soil classification, which depends on Atterberg limits and particle size distribution.

The sieve analysis of the soil samples were done as per ASTM D22, first the disturbed samples were naturally dried for more than two days. The dry sample was taken to pass through a series of 4.75, 2.36, 2.00, 1.8, 0.6, 0.3, 0.15 mm sieves. The pan was attached at the bottom of the sieve stack. The sample was poured on the top sieve and stirred for about 10 minutes. Soil retained at each sieve was weighed. The weights of the sample on each sieve were added to compare it with the initial weight of sample. The difference should not be more than 1%.

Hydrometer analysis was performed on the same sample which was finer than 0.075mm sieve size. Therefore, the retained soils on various sieves from sieve analysis and hydrometer analysis were calculated and a particle distribution was obtained as shown in the appendix.

For classification of soil, atterberg limits tests (liquid limit and plastic limit) were performed on the collected samples.

The laboratory soil test result about the properties of soil from each pit is summarized in the Table 4.1.

Table 4.1 Summarized soil property and description

Pits	Depth (m)	Soil description	Strength of fine grained soil	Ll	Pl	Pi
1	2.5	Brown sandy clay with silt	Semisolid state	36.8	15.3	21.53
	4	Brown sandy clay	Semisolid state	32	17.4	14.37
2, 3 and 4	2.5	Whitish spot silty, brown sandy clay	Semisolid state	27.8	12	15.8
	4.5	White clay with sand and silt	Semisolid state	54	23	31

Therefore, from the soil laboratory test result and pervious study data, beneath the foundation there is sandy clay and silty clay soil.

b. One dimensional consolidation test

Consolidation tests for each of four pits at different depths were performed. The soil sample was kept inside a stiff steel ring. The stiff steel ring prevents lateral expansion of the sample and allows only vertical downward deformation. And the two porous stones allow water drainage. The sample was fully submerged in water and remained fully saturated during the test. The different pits samples were prepared in a 5cm diameter and 2 cm height ring where they were placed to be tested by consolidation test apparatus. An initial sitting pressure of 7 kPa was applied on each sample from each pit and then the samples were allowed to consolidate for 24 hours. The initial setting loads brought some acceptable swell in all samples. Dial gauge reading was noted under initial sitting pressure and in each incremental loadings i.e. 50 kPa, 100 kPa, 200 kPa, 400 kPa, 800 kPa, 1600 kPa. and at a certain time intervals i.e. 6secs, 15 secs, 30 secs, 1min, 2 mins, 4 mins, 8 mins, 15 mins, 30 mins, 60 mins, 120 mins, 240 mins, 480 mins and 1440 mins. And also the gauge reading was noted in the event of both loading and unloading.

The consolidation tests were performed on the *undisturbed soil* sample to determine the settlement characteristics of soil at different depths of varies pits.

Varies pits consolidation Test results

The e - log P curve from consolidation test results of each pit is presented in this section. The pre consolidation pressures/stress (σc) is determined from the laboratory e-log P curves using a Casagrande method. The rest of the test results are available in the Appendix section.

Fig 4.3 shows e – log P of Pit 1 at 3m, Fig 4.4 shows e – log P of Pit 1 at 4.5m, Fig 4.6 shows e – log P of Pit 2, 3 and 4 at 3m and Fig 4.6 shows e – log P of Pit 2, 3 and 4 at 4.5m.

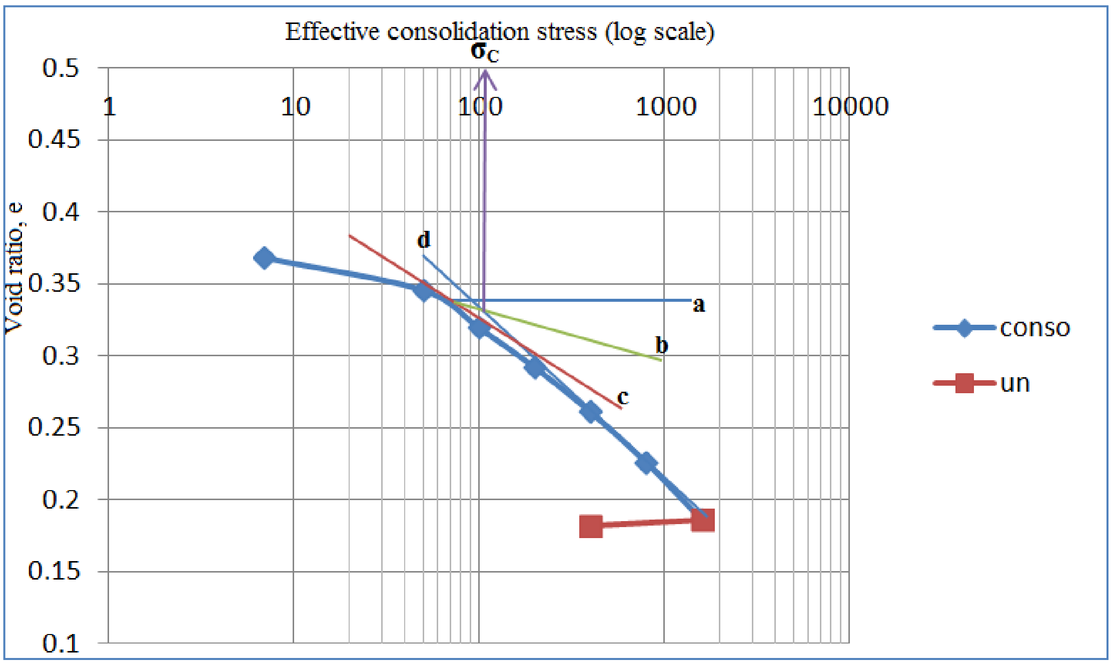


Figure 4.3 Pit 1 at 3m e –log P curve

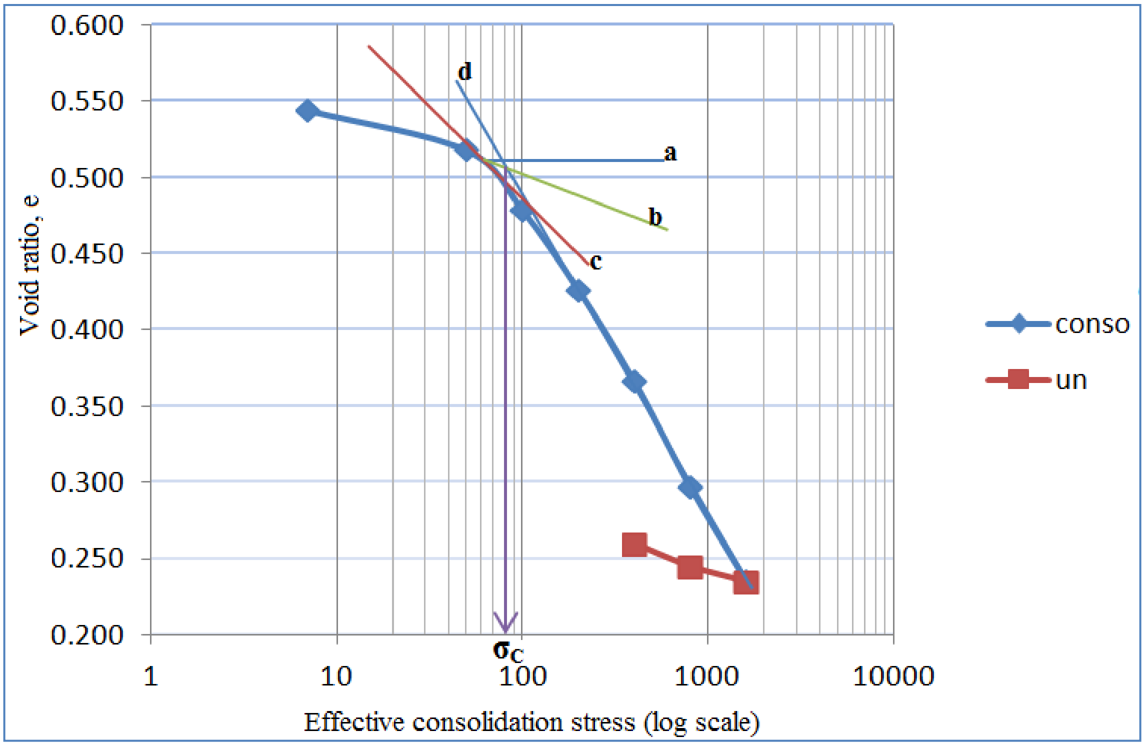


Figure 4.4 Pit 1 at 4.5m e –log P curve

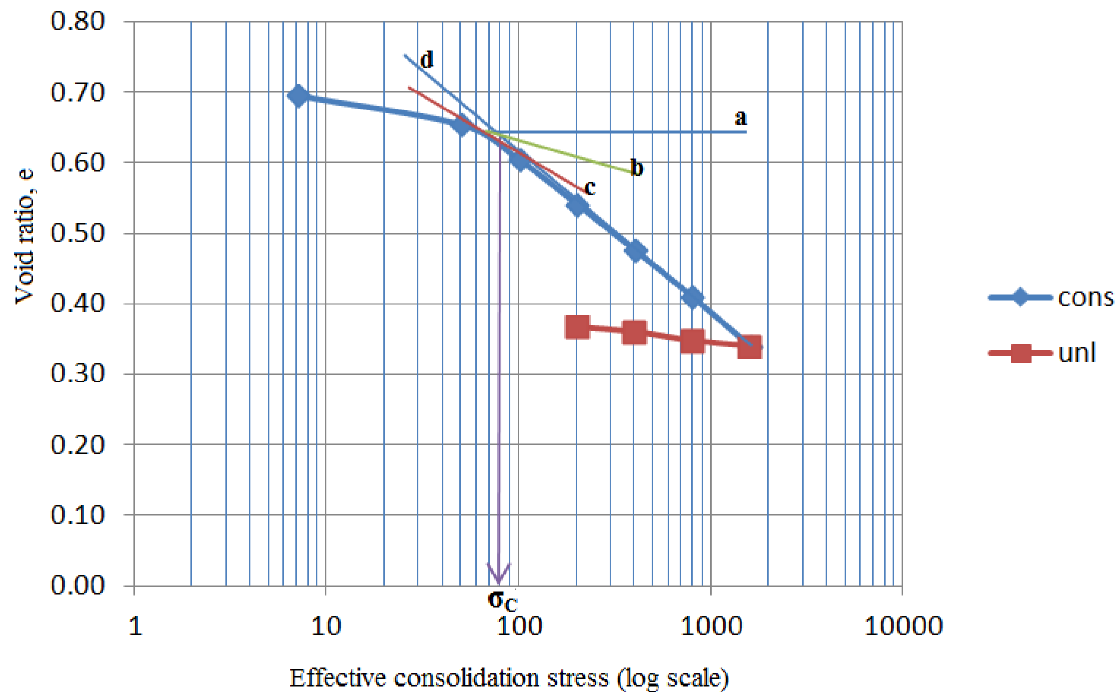


Figure 4.5 Pit 3 at 3m e –log P curve

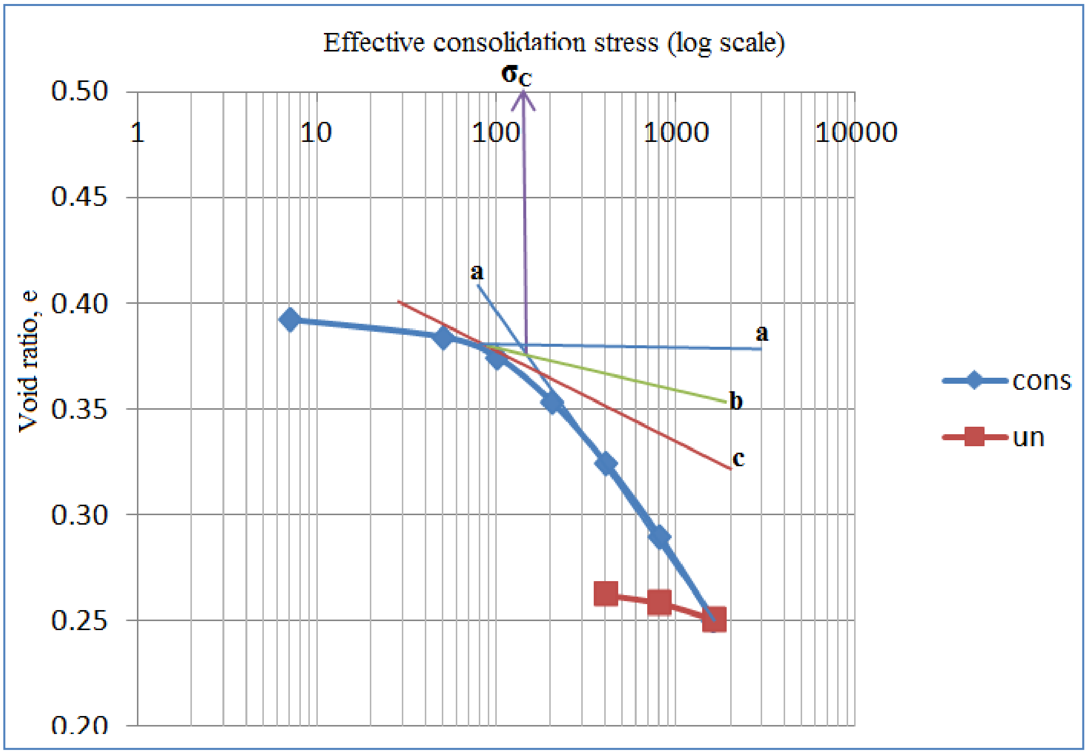


Figure 4.6 Pit 2, 3 and 4 at 4.5m e –log P curve

The soil layers observed in pits 2, 3 and 4; specifically below 4m from ground level show significantly varying property as compared to layers up to -4m level.

On the other hand it is also observed that layers in pit 1 are significantly different from layers in pit 2,3 & 4. Generally, the soil property of layers from pit 1 to pit 4 observed to be stronger.

As can be seen in Fig 4.7, the color texture of soil layers observed in pits 2, 3 & 4 from NGL level 0.00m to -4m is brown with sparsely dotted white spots. Whereas for layers below -4m of pits 2, 3 & 4 as seen in Fig 4.8, the color texture is generally white and relatively stiffer than the layers found above-4m level.

Figure 4.7 Pit 1 soil at 4.5m

Figure 4.8 Pit 2, 3, and 4 soil at 4m up to 4.5m

Therefore, only from the visual and laboratory test, pit 2, pit 3 and pit 4 have almost similar soil type and settlement property.

4.1.3 NUMERICAL ANALYSIS AND DETERMINATION OF FOUNDATIONS SETTLEMENTS

Total settlement is the vertical movement of the foundation from its as constructed position to its loaded position. The total settlement of the foundation due to the imposed load is determined with respect to the application of elastic or immediate, primary consolidation and secondary settlements. But secondary settlement is important in organic soil deposit and can be significant in highly plastic clays, organic soils, and sanitary landfills. Therefore, this component is not discussed further in this study.

a. Immediate Settlement analysis

Immediate settlement is generally assumed to be the settlement that occurs quickly after the construction of foundation.

The immediate settlement may be estimated from elastic theory. A realistic analysis is difficult to perform because of the non linear characteristics of soil. Thus, result from theory of elasticity is generally used in practice by assuming the soil is homogeneous and isotropic.

The calculation for immediate settlement for homogeneous soil layer is carried out using the following formula [17]

$$S_e = q_n \alpha B \frac{(1-\mu^2)}{E_s} Is I_f \quad (3.1)$$

Where

e = elastic settlement

B = width of foundation,

Es = modulus of elasticity of soil,

μ = Poisson's ratio,

q_n = net foundation pressure,

I_f = influence factor.

Is = shape factor

Modulus of elasticity

The modulus of elasticity (Es) of a soil is a soil parameter that is commonly used in the estimation of settlement. Many literatures are providing the range of modulus of elasticity for a various soil type. And also several laboratory and field test methods are available to estimate the strain-stress modulus of elasticity. Some are;

Laboratory test

- Unconfined compression tests
- Triaxial compression tests

Field test

- STP (standard penetration test)
- CPT (cone penetration test)

To estimate the elastic modulus of the soil, some of the field test data previously recorded which are not found as the main areas of error are used to compare and support recommended in literatures. These data are closer to the specific condition of the soil than others generally provided in literatures. Also the best course of action is to perform the test again and see the results directly. But material and resource constrains brings the results less than 100% certainty. However, the other course of action is to make through literature review, which is performed, as to have strong stand for the next analysis. This assures no significant variation occurs on the output of the analysis from the actual behavior of soil. Therefore, to compute the modulus of elasticity recommended N-value result from the previous soil test data used initially as an input.

Some equations for a wide range of soil properties, from the coarse to fine grained soils [13] are described in Table 4.2 to Table 4.5;

Table 4.2 Es estimation using STP N-value

Soil	**Es (MPa)**
Gravelly sand	Es = 0.6(N+6), N < 15 Es = 0.6(N + 6), N > 15
Clayey sand	Es = 0.32(N+15)
Silts, sandy silt, or clayey silt	Es = 0.3(N + 6)

Table 4.3 Standard penetration test N value from the original soil test data and Es

Bore holes	**depth at**	**STP N-value**	**Es = 0.3(N + 6) in MPa**
1			
	3m	15	6.3
	4.5m	41	14.1
2			
	3m	19	7.5
	4.5m	17	6.9

Table 4.4 Modulus of elasticity for granular soils [17]

Soil type	**Es (kN/m^2)**
Soft clay	1,800–3,500
Hard clay	6,000–14,000
Loose sand	10,000–28,000
Dense sand	35,000–70,000

Table 4.5 Representative value of Poisson ratio [17]

Type of soil	**Poisson's ratio, μs**
Loose sand	0.2–0.4
Medium sand	0.25–0.4
Dense sand	0.3–0.45
Silty sand	0.2–0.4
Soft clay	0.15–0.25
Medium clay	0.2–0.5

The poisson's ratio of the sub grade soil assumed 0.3 for all pits.

The total immediate settlement of foundations along axis B and axis C for sandy clay soil are determined based on the estimated settlement parameters. And the results are summarized in the Table 4.6.

Table 4.6 Immediate settlement of the foundation along Axis B and Axis C

Pit no	Axial load	Es	µs	S
Axis B	(kN)	(MPa)		(mm)
Pit 1				
at 3m	686	6300	0.3	44.29
at 4.5m	686	14100	0.3	12.92
			S_{total}	**57.21**
at 3m	1594	6300	0.3	20.67
at 4.5m	1594	14100	0.3	23.35
			S_{total}	**44.03**
Pit 2				
at 3m	686	7500	0.3	37.2
at 4.5m	686	6900	0.3	26.41
			S_{total}	**63.61**
at 3m	1594	7500	0.3	17.37
at 4.5m	1594	6900	0.3	47.72
			S_{total}	**65.09**

Pit no	Axial load	Es	µs	S
Axis C	(kN)	(MPa)		(mm)
Pit 1				
at 3m	925	6300	0.3	13.565
at 4.5m	925	14100	0.3	11.687
			S_{total}	**25.252**
at 3m	1051	6300	0.3	15.412
at 4.5m	1051	14100	0.3	13.279
			S_{total}	**28.692**
Pit 2				
at 3m	925	7500	0.3	11.394
at 4.5m	925	6900	0.3	23.883
			S_{total}	**35.277**
at 3m	1051	7500	0.3	12.946
at 4.5m	1051	6900	0.3	27.136
			S_{total}	**40.083**

b. Primary consolidation settlement

Consolidation settlement parameters for the soil stratum are computed from one dimensional consolidation test.

The numerical analysis for normally consolidated clay is determined by the following formula;

$$\text{Settlement} = \frac{\Delta e}{1+e_0}\text{H} \qquad (3.2)$$

Δe= change in void ratio; can be calculated by using e-log σ_v plot which is obtained from laboratory test result.

H = thickness of the soil layer

The change in void ratio;

$$\Delta e = C_c \log \frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_{vo}} \quad (3.3)$$

Cc= slope of the *e*–log s plot and is defined as a compression index

$$C_C = \frac{e_o - e}{\log\sigma - \log\sigma_o} \quad (3.4)$$

σ_{vo}= average effective stress kN/m^2

$\Delta\sigma$=the net change in pressure produce by the structure in kN/m^2

In the other hand, for overconsolidated clay soil the settlement is determined based on overcosolidation ratio, OCR.

The slopes of swell index line segment, Cs (or recompression index, Cr) value can be found based on the following slope formula read from the graph shown in Fig 4.9.

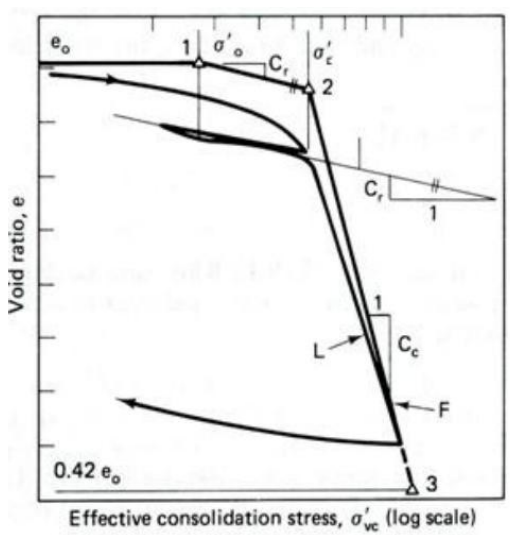


Figure 4.9 e – log P curve

$$C_s = \frac{e_o - e}{\log\sigma - \log\sigma_o} \quad (3.5)$$

$$\text{OCR} = \frac{\sigma c}{\sigma o} \quad (3.6)$$

Two conditions are considered to compute settlements of the soil:

1. if the overconsolidation ratio, OCR =1 then the soil is normally consolidated therefore the settlements are determined by using

$$S_c = H\frac{Cc}{1+e_o}\log\frac{\sigma_o+\Delta\sigma}{\sigma_o} \tag{3.7}$$

2. if the overconsolidation ratio, OCR >1 then the soil is over consolidated but,
   a. if $\sigma o + \Delta\sigma < \sigma c$ used $S = Cs\ \frac{Ho}{1+eo}\log\frac{\sigma_D}{\sigma o}$ (3.8)

   b. if $\sigma o < \sigma_C < \sigma_D + \Delta\sigma$ used $S = Cs\ \frac{Ho}{1+eo}\log\frac{\sigma_C}{\sigma o} + Cc\ \frac{Ho}{1+eo}\log\frac{\sigma_D}{\sigma c}$ (3.9)

Where, $\sigma c(Pc)$ = Preconsolidation stress (determined by Casagrande's method),σo = current over burden stress, $\Delta\sigma$ = stress due to external load comes from the structure, $\sigma_D = \sigma o + \Delta\sigma$.

Generally, after the laboratory test and numerical analysis of consolidation settlement parameters, the final settlement results are calculated as shown in the Table 4.7 and Table 4.8;

Table 4.7 Settlements analysis result of pit 1 at the various depths

soil stratum thickness,	Consolidation parameters						Footing size		Westergaard's coefficient	Axial load	Stress Increment,	S(m)	Remark
H(m)	eo	Cc	Cs	Po(kN/m2)	Pc	OCR	B(m)	L(m)	Iw	P(kN)	Δσ (kN/m2)		
Axis B													
1	0.37	0.12	0.027	54	100	1.852	1.8	1.8		686	211.728395	0.044	over consolidated soil
3	0.51	0.21		81	83	1.025			0.32	686	97.5644444	0.14	Normally consolidated soil
											Total S(mm)	**183.7**	
1	0.37	0.12	0.027	54	100	1.852	3.2	3.2		1594	155.664063	0.034	over consolidated soil
3	0.51	0.21		81	83	1.0247			0.32	1594	226.702222	0.239	Normally consolidated soil
											Total S(mm)	**273.8**	
Axis C													
1	0.37	0.12	0.027	54	105	1.944	1.8	3.2		1257.57	218.328125	0.043	over consolidated soil
3	0.51	0.21		81	83	1.025			0.32	1257.57	178.8544	0.213	Normally consolidated soil
											Total S(mm)	**256.2**	
1	0.37	0.12	0.027	54	100	1.852	2.4	2.4		1051.68	182.583333	0.039	
3	0.51	0.21		81	83	1.025			0.32	1051.68	149.572267	0.187	
											Total S(mm)	**225.8**	
Axis C'													
1	0.37	0.12	0.027	54	105	1.944	1.8	1.8		493.5	152.314815	0.032	over consolidated soil
3	0.51	0.21		81	83	1.025			0.32	493.5	70.1866667	0.114	Normally consolidated soil
											Total S(mm)	**146.3**	

Table 4.8 Settlements result for pit 2, 3 and 4 at various depths

soil stratum	Consolidation parameters						Footing size		Westergaard's coefficient	Axial load	Stress Increment,	S(m)	Remark
thickness,	eo	Cc	Cs	Po(kN/m2)	Pc	OCR	B(m)	L(m)	Iw	P(kN)	Δσ (kN/m2)		
Axis B													
1	0.7	0.223	0.048	54	80	1.4815	1.8	1.8		686	211.728395	0.07321	over consolidated
3	0.41	0.115	0.0095	85.005	140	1.647			0.32	686	97.5644444	0.03261	normally consolidated
											Total S(mm)	**105.818**	
1	0.7	0.223	0.048	54	80	1.4815	3.2	3.2		1594	155.664063	0.05971	over consolidated
3	0.41	0.115	0.0095	85.005	130	1.5293			0.32	1594	226.702222	0.09668	normally consolidated
											Total S(mm)	**156.384**	
Axic C													
1	0.7	0.223	0.048	54	80	1.4815	1.8	3.2		1257.57	218.328125	0.07461	over consolidated
3	0.41	0.115	0.0095	85.005	140	1.647			0.32	1257.57	178.8544	0.07175	normally consolidated
											Total S(mm)	**146.351**	
1	0.7	0.223	0.048	54	80	1.4815	2.4	2.4		925	160.590278	0.06103	over consolidated
3	0.41	0.115	0.0095	85.005	140	1.647			0.32	925	131.555556	0.05075	normally consolidated
											Total S(mm)	**111.785**	
Axic C'													
1	0.7	0.223	0.048	54	80	1.4815	1.8	1.8		493.5	152.314815	0.05879	over consolidated
3	0.41	0.115	0.0095	85.005	140	1.647			0.32	493.5	70.1866667	0.01535	normally consolidated
											Total S(mm)	**74.1363**	

The total settlement of the foundation is the maximum value obtained from the sum of immediate and primary consolidation settlements of the foundation with respect to its original position.

$$S_{Total} = S_{immediate} + S_{consolidated} \tag{3.10}$$

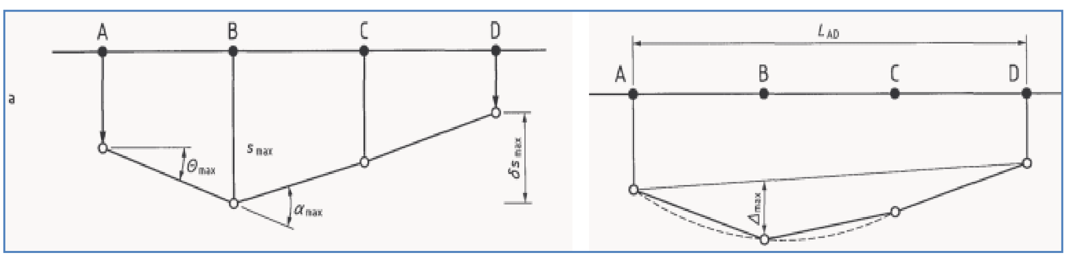


Figure 4.10 Angular Distortion

From the Geotechnical point of view, as discussed previously in the literature section no significant damage will come to a structure if its foundations are uniformly settled as a whole regardless of how large the settlement may be. In this case only the connections of the underground utility lines are affected. However, when the settlement is non-uniform (differential), as it’s always the case, it tends to break structure apart. The settlement analysis result of this study shows the differential settlements of the foundations due to varying property of soil underneath. The level of damage to the structure can be evaluated in terms of angular distortion.

As shown in Fig 4.10 the angular distortion is the settlement different between the maximum point to the minimum point divided by the length between them. Different codes recommend different damage levels.

Table 4.9 ES EN 1997 [32] Angular distortion limit

limit states	Angular Distortion limits
To prevent the occurrence of a serviceability limit state	1/300 - 1/2000
A maximum relative rotation acceptable for many structures	1/500
likely to cause an ultimate limit state	1/150

The maximum angular distortion between Pit 1 and 2 is 0.0052. It is greater than the code recommended for maximum acceptable limit 1/300 as can seen in Table 4.9.

It has been eight years since the building constructed and open to the service. Within this years, according to time-rate of the settlement calculation result 80% and 70% around pit 1 and the rest of the pits respectively settlements are takes placed from the total or final settlement.

4.2 ASSESSMENT OF CRACKING CAUSED BY ANALYSIS AND DESIGN FAULT

One of many reasons building can be cracked unexpectedly within short life time is structural analysis and design errors. Some of these errors lead to over consideration of a design load which will result in the so called under design, also have a direct impact on the cost of the building. But most errors are about collecting insufficient or wrong pre design data, neglecting safety factors, the variation between the new and old design code and other paper work errors related with copying and editing. And by their nature they make things missed and design results insufficient. So the probability of under design is very high. Considering their gradual consequence, both wrong design results are treat for the safety of the building but mostly under design will result a born weak structure that could fall apart anytime soon. Therefore, in this study effort is made for the analysis and design of the existing building from the given architectural and structural drawings and compares it with the original building designed data.

4.2.1 DESCRIPTION OF CASE STUDY

This study involves modeling a 6-story school building which is constructed from a reinforced concrete structural frame with hollow concrete block (HCB) walls. The walls are 20mm thick. The structural members are made of in-situ reinforced concrete. All the floor slabs are solid reinforced concrete with 200mm thickness. The substructure contains spread and combined footings.

The building is laid on 46m by 8m rectangular area raising 16m from ground as shown in Fig 4.11 plan view from ETABS2016. The whole structural system is essentially symmetrical. Also each frame of building is designed as gravity frame. Regarding the material strength, the characteristic compressive strengths of concrete constituting the columns and beams is 20 MPa. The yield strength of steel deformed bars is 260.87 MPa. The allover structural dimension of the building described in Table 4.10.

Table 4.10 Detail reference of the building

Description	**Value/cross-section**	**Remark**
Story height	3m	All stories
wall thickness	20cm	
Slab thickness	20cm	it's typical to all stories
Beam size		
Top tie beam	20cmx30cm	
Intermediate beams	30cmx40cm	Beams from 1st up to 5th stories
Grade beam	25cmx40cm	
Columns		
C1 and C4	30cmx30cm	4th floor columns
	30cmx30cm	3rd floor columns
	30cmx30cm	2nd floor columns
	30cmx30cm	1st floor columns
	35cmx35cm	Ground floor columns
	35cmx35cm	Foundation columns
C2	30cmx30cm	4th floor columns
	35cmx35cm	3rd floor columns
	35cmx35cm	2nd floor columns
	40cmx40cm	1st floor columns
	40cmx40cm	Ground floor columns
	40cmx40cm	Foundation columns
C3	30cmx30cm	4th floor columns
	40cmx40cm	3rd floor columns
	40cmx40cm	2nd floor columns
	45cmx45cm	1st floor columns
	50cmx50cm	Ground floor columns
	50cmx50cm	Foundation columns

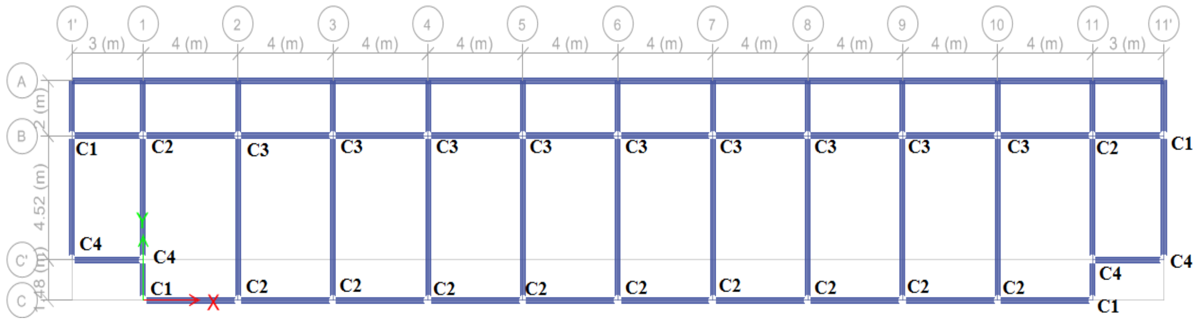


Figure 4.11 The overall building dimension

4.2.2 MODELING OF THE BUILDING

The super structure building model is copied from architectural drawings to ETABS 2015 and 2016. Three dimensional static linear analyses are done according to ES EN 1998:2015 new code recommendations. All of the structural elements such as beams, columns, and slab sections are used directly from the given original structural drawings data which designed and prepared before the building is constructed. The building is regular in plan and elevation. The 1st, 2nd, 3rd and 4th floors of the building are similar.

a. Earthquake static analysis

The revised ES EN 1998-1:2015 part 1 at Table: D2 shown that Dire Dawa city is categorized under seismic hazard zone 3 and the ground acceleration 0.1g is determined using the reference peak ground acceleration for Dire Dawa and building importance considered class II. The behavior factor q for ductility class low is 1.5 is used for this analysis.

As per the new code recommendation type 2 spectrum type is used for the selected building structural seismic analysis. And the soil is assumed as type C.

It is also necessary to define effective elastic properties since the elastic part of the member cracks when the member enters into inelastic range. Hence, the stiffness reduction factors are used to reduce the gross stiffness properties of the beams, columns and slabs of the structural members.

The stiffness modification factors used for all section are as follow

- Shear area in 2 & 3 direction = 0.5
- Torsional constant = 0.1
- Moment of inertia in 2 & 3 axis = 0.5

- Slabs
  - Bending in m11, m22 and m33 directions = 0.05

Eurocode8 2004 seismic load pattern are defined to apply the lateral earthquake load on a given structure with using the above data.

Load combinations

Different load combinations are used to analysis the given structure including new seismic load combination. The load combinations which are displayed on Table 4.11 shall be accounted and considered in this study as per the new ES EN 1998:2015 and ES EN 1992:2015 code recommended

Table 4.11 Load combination

Combinations	**Total output combinations**
GRAVITY	1
GRAVITY ± IMPX	2
GRAVITY ± IMPY	2
SEISMIC±GRAVITY(DEAD +SDEAD+PARTITION+0.3 LL)	1
1.35(DEAD+SDEAD+PARTITION) +1.5LL	1
SERVICE	1
[SEISMIC GRAVITY] ± [IMP X,Y] ± [EQXN, EQXP, EQYN,EQYP],± 0.3*[EQXN, EQXP,EQYN,EQYP]	32
ENVELOPE X (all equivalent static load combinations)	1
ENVELOPE Y (all equivalent static load combinations in the X)	1
ENVELOPE	1

The building is modeled and analyzed as a 3D frame system on ETABS2016 as shown in Fig 4.12 and Fig 4.13. The result of this analysis was interpreted as, for the beams member case design the reinforcement and compare with the original design are done, for columns the demand capacity ratio ETABS output are checked. Unlike beams and columns the slab member of the structure are checked by comparing the SAFE2014 deflection output result with the acceptable code recommended limit.

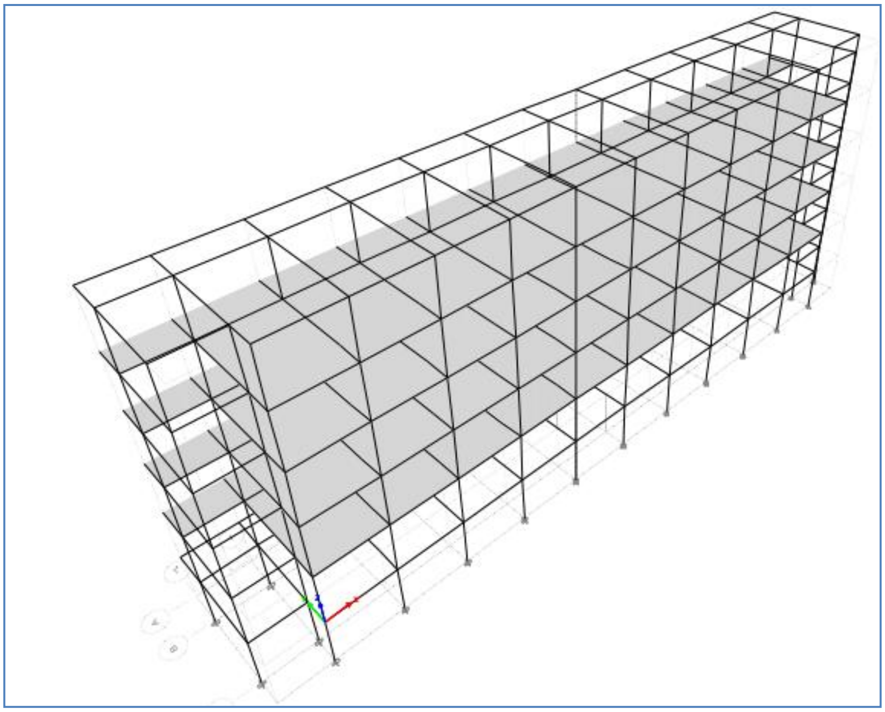


Figure 4.12 Basic structural system ETABS 2016 model

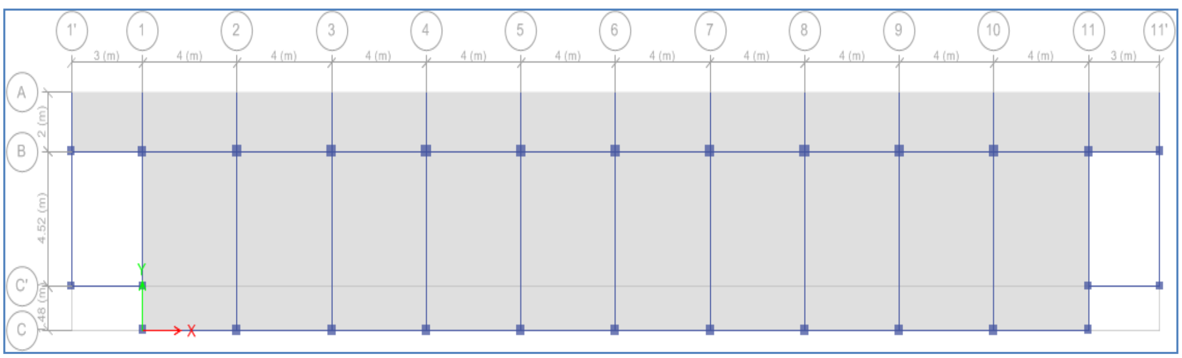


Figure 4.13 Floor plan model

4.2.3 ANALYSIS AND DESIGN RESULT

I. Beam

According to ENVELOPE load combination from the list of different load combinations the result of analysis or sample bending moment diagram over the beams along axis-B are shown in Fig 4.14.

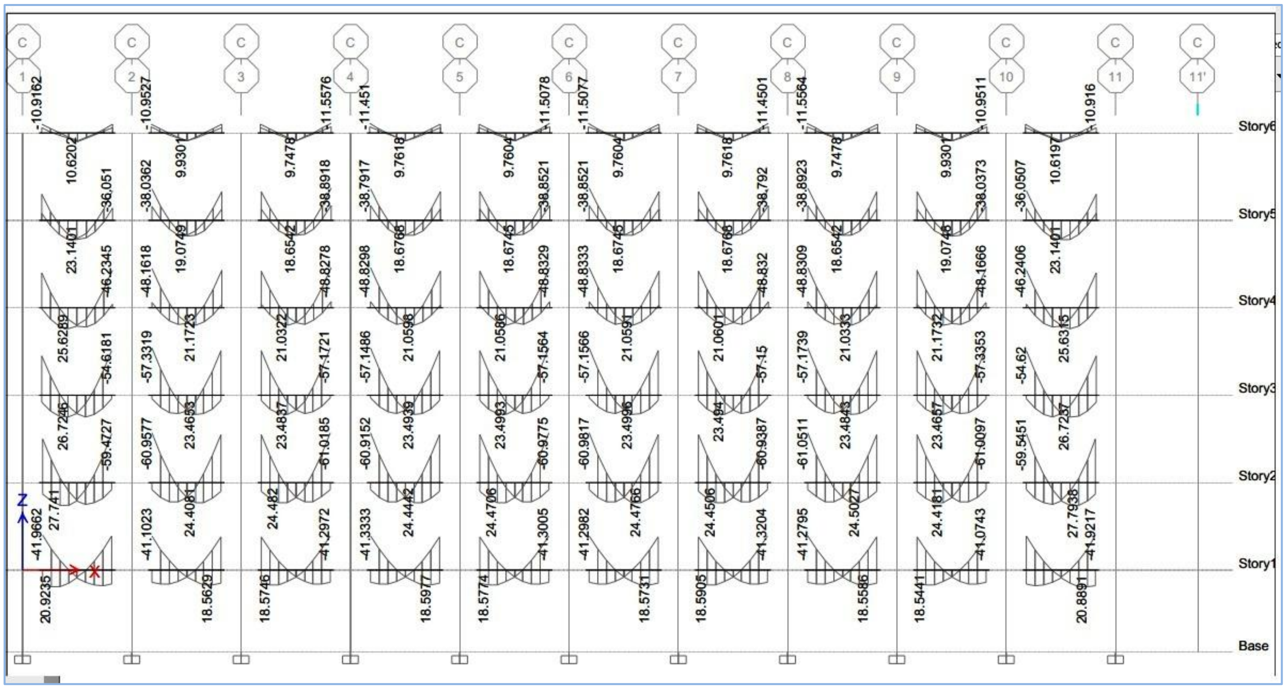


Figure 4.14 Bending moment diagram for beam on axis C

Figure 4.15 General beams longitudinal reinforcement and comparison with the original design

		Provided reinforcement			calculated		Calculated/	Remark
Level	Location	#Bars	Area (mm2)		Moment	R-bar Area	provided	
	& beam section				(kN-m)	(mm2)		
	Axis B							
	(300x400)							
	sec 7-7 (support)	4ϕ20	1256.64		58.84	958	0.762	top bar
	sec 5-5 (span)	3ϕ16	603.2		26.63	477	0.791	Bottom
	Axis C							
	(300x400mm)							
	sec 2-2 (support)	3ϕ16	603		61.1	778.48	1.291	top bar
1st, 2nd, 3rd	sec 3-3 (span)	2ϕ16	402.124		24.5	477	1.186	bottom
and 4th	Axis 2, 4, 6, 8, 10							
typical floor	(300x500mm)							
	Sec 6-6	5ϕ20	1570.8		90.96	1219	0.776	right side top
	Sec 9-9	4ϕ20+2ϕ20	1884.96		212.6	2008	1.065	left side top
	Sec 8-8	3ϕ20	942.5		56.67	822	0.872	bottom
	Axis 3, 5, 7, 9							
	(300x500mm)							
	Sec 10-10	5ϕ20	1570.8		90.96	1446	0.921	right side top
	Sec 12-12	4ϕ20+2ϕ20	2199.115		212.6	2289	1.041	left side top
	Sec 11-11	3ϕ20	1256.63		56.67	1059	0.843	bottom
	Axis1' and 11'							
	(300x500mm)							
	Sec 9-9	6ϕ20	1884		90.96	1198.78	0.636	top
	Sec 8-8	3ϕ20	942		56.67	725.728	0.770	bottom
	Axis 1 and 11							
	(300x500mm)							
	Sec 9-9	6ϕ20	1884		127.64	1740.204	0.924	top
	Sec 8-8	3ϕ20	942		43.89	556.43	0.591	bottom
Top tie beam (200x300mm)		2ϕ14	307.87		14.26	176.81	0.574	top bar
		2ϕ14	307.87		11.9	176.81	0.574	bottom
Grade beam(250x400mm)		2ϕ14	307.87		51.13	406	1.319	top bar
		2ϕ12	226		28.58	250	1.106	bottom

The demand capacity ratio for beams at each floor is display in Table 4.12 as a ratio of calculated to provided rebar area. Most beams show a demand capacity ratio less than 1. However, significant portion beams show exceeded demand capacity ratio greater than 1. Thus it can be deducted that the contribution to observed cracks design error cannot be ignored.

II. Column

The vertical compression members in RC structure subjected to axial load and moment due to unbalanced moments from beams and also the load may not be centered on the columns. Therefore, columns should be designed for axial load and design moment obtained from the analysis result.

Table 4.12 Columns demand capacity ratio ETABS2016 result

Column design							Demand/capacity
Column axis B	**B**	**H**	**Level**	**Nsd**	**M2-2**	**M3-3**	**P-m-m interaction ratio**
Col B6	300	300	4th floor	87.787	17.32	5.4	0.413
Col B6	400	400	3th floor	395.46	48.68	22.2	0.367
Col B6	400	400	2th floor	703.46	39.04	37.94	0.478
Col B6	450	450	1th floor	1016.7	54.68	46.38	0.475
Col B6	500	500	Ground	1334	36.65	66.48	O/S
Col B6	500	500	Foundation	1462.4	41.89	104.87	0.597
Column axis C							
Col C6	300	300	4th floor	46	8.2	3.6	0.233
Col C6	350	350	3th floor	242.49	48.8	18.09	0.584
Col C6	350	350	2th floor	442.62	38.08	28.9	0.55
Col C6	400	400	1th floor	648	56.3	44.6	0.619
Col C6	400	400	Ground	855.2	36.4	42.45	0.514
Col C6	400	400	Foundation	957.08	13.59	53.35	0.587
Column axis C’							
Col C’11	300	300	4th floor	42.22	6.43	12.82	0.348
Col C’11	300	300	3th floor	179.1	8.87	12.6	0.343
Col C’11	300	300	2th floor	305.825	13.89	25.76	0.514
Col C’11	300	300	1th floor	451.684	13.89	33.2	0.606
Col C’11	350	350	Ground	637.78	21.189	53.604	0.644
Col C’11	350	350	Foundation	785.12	29.12	54.08	0.702

As can be seen from Table 4.13 and Fig 4.15, the demand capacity ratios for each column is displayed. It can be seen that most columns have a demand capacity ratio less than 1. I.e. most columns are safe except columns B6 at a ground floor as shown in Fig 4.15. It can be suggested from the result that the contribution to exhibited cracks from design error is minimal.

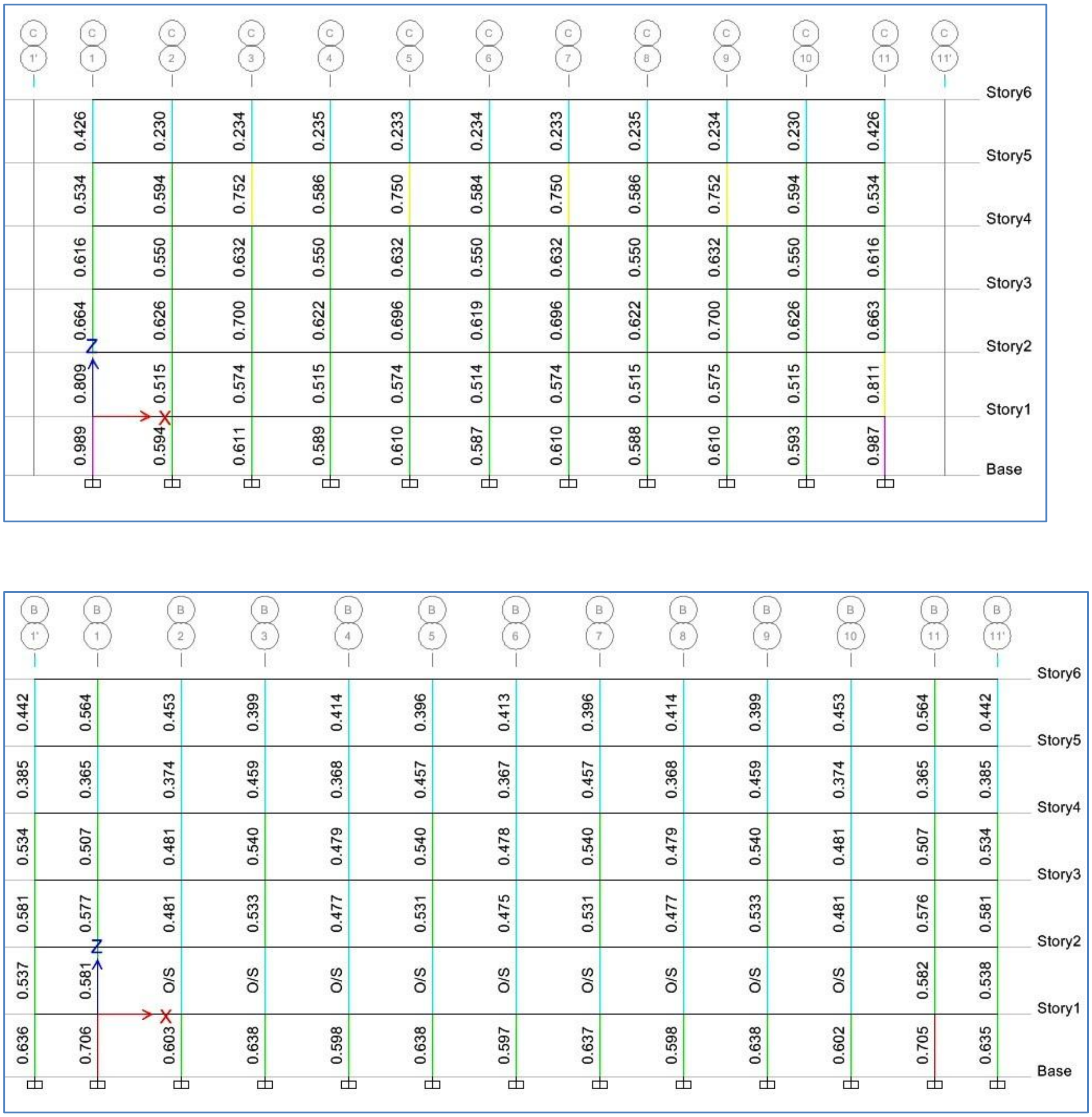


Figure 4.16 Axis B and C columns P-M-M interaction ratio from ETABS 2016 output

III. Slab checking for serviceability

The floor slabs are modeled on SAFE 2014 by exporting directly from ETABS2016 model including loads which are applied on the floor.

The typical floor slabs checking for serviceability (long term deflection) under sustain loads with the allowable deflection. The EUROCODE 2 recommended that the allowable deflections should not exceed Le/250.

Le = effective span length

Le for the typical slabs = 4000mm

The allowable deflection Δ = 16 mm > the maximum deflection as seen in Fig 4.16 from SAFE 2014 output is 11.62mm

Therefore the typical slabs of the floors under the expected given loads are within acceptable deflection limit and the typical slabs are safe under the previous designed condition.

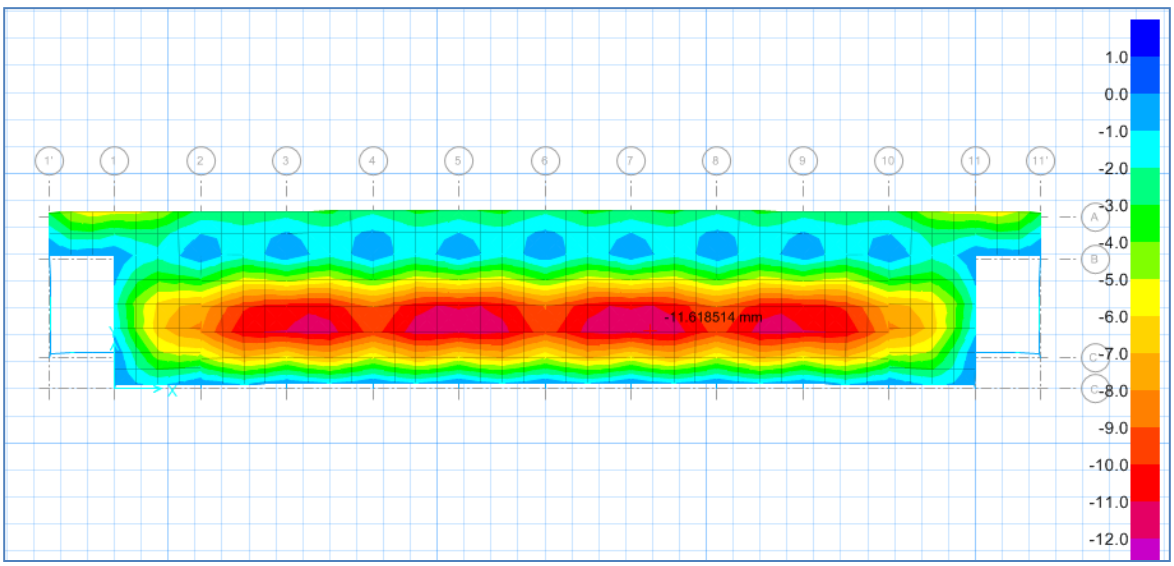


Figure 4.17 Safe 2014 deflection output

4.3 THE EFFECT OF THE DIFFERENTIAL SETTLEMENT ON THE BUILDING FRAME

Foundation design necessitates two different studies; one deals with the bearing capacity of the soil; the other is concerned with the foundation settlements. Considering that the loads transferred from the superstructure to the foundation are non-uniform, differential settlements between the foundation's elements are expected. But the conventional design of building frames is based on the assumption that the settlement of footings has no effect on the load of the corresponding columns and on other structural members of the building. In reality, the differential settlements among various footings result in a redistribution of the column loads, the amount of which depends on the rigidity of the structure and the load-settlement characteristics of the soil.

The settlement analysis result showed that the foundation soil prevailing underneath the building is under differential settlement condition. On the other hand, analyzing and redesigning the entire building structure without settlement consideration brought minor difference from the original design data. Here the difference is small enough not to exceed the safety factor provided for loads and materials at first place. But the cumulative effects of the soil and the building structure has to be evaluated thoroughly. Hence, force quantities and settlement at lately adjusted condition can only be obtained through interactive analysis of the soil–structure– foundation system. This explains the importance of considering soil effects on the whole structural system of the building.

Three dimensional analyses of the superstructure, the foundation and the soil beneath all together constitute a complete system. Also due to differential settlement among various parts of the structure, the axial force, moments and other stresses in the structural members may change. And the extent at which the loads are redistributed depends upon the rigidity of the structure and the load –settlement characteristics of soil.

So far the supper structure and the soil settlement characteristics are analyzed individually. Here in this chapter, the building is modeled and analyzed nearly to simulate its actual existence which means its frame structure and soil behavior/settlement/ all together. Following the analysis, the effect of foundation differential settlement and the building frame structure one on another is discussed. Most importantly, this analysis is performed to observe whether the building structure with the original design can sustain the additional stress from soil differential settlement or not.

4.3.1 MODELING

The conventional model of the frame with fixed bases is analyzed using ETABS 2016. And the outputs are presented in the previous chapter. Now, in this chapter similar model is used to study the soil structure interaction system, except with the introduction of determined settlement values as a ground displacement at the bases of the columns. The analysis is performed in 3D model and parameters are calibrated based on the settlement analysis data. The building has four type of footing in general as described in Table 4.14; three isolated and one combined.

Table 4.13 Building base foundations dimensions

Type	Footing dimension
F1	1.8mx1.8m
F2	2.4mx2.4m
F3	3.2mx3.2m
Combined footing	3.2mx1.8m

During modeling the downward displacements assigned at the column bases are obtained from settlement analysis. And as discussed in the previous chapter, the laboratory result of soil parameters observed in pit 2, 3 and 4 are approximately similar but soil parameters in pit 1 significantly vary to be approximated. Hence, for the soil laid under foundation bases which coincide with the pits, the respective soil parameters are used. And for the soil laid under foundation column in between pit1 and pit 2, the final settlement parameters are interpolated. On the other hand, the respective foundation settlements in between the two pits are determined by using interpolated final settlement parameters and modeled on ETABS 2016 as shown in Fig 4.17.

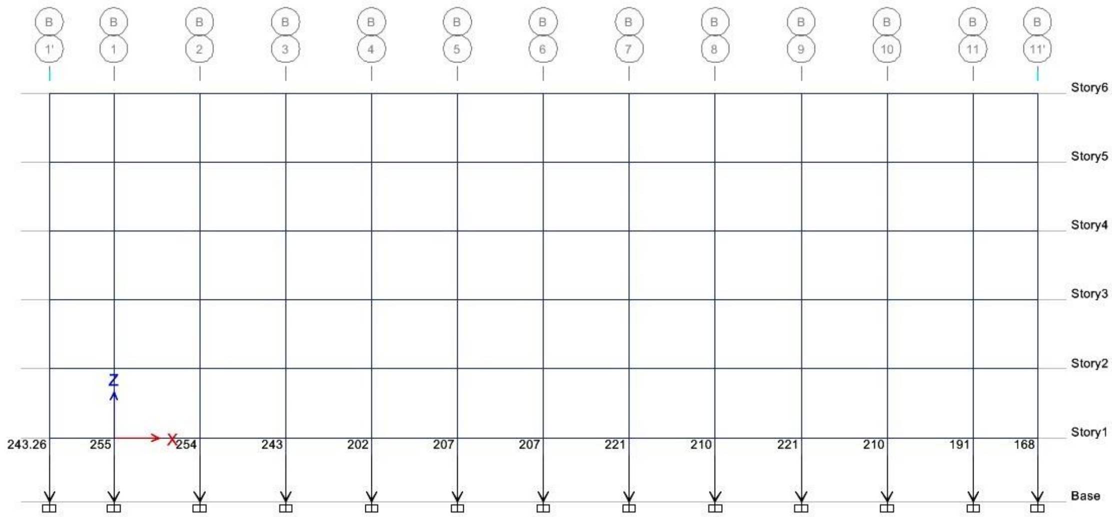


Table 4.14 Axis B assigned ground displacement on ETABS 2016

4.3.2 ANALYSIS RESULT

Figure 4.18 and 4.19 clearly shows the final settlement different at both sides of the building. And unlike the results from visual observation and followed by analysis, here the difference between the two sides of the building in terms of differential settlement is quite noticeable.

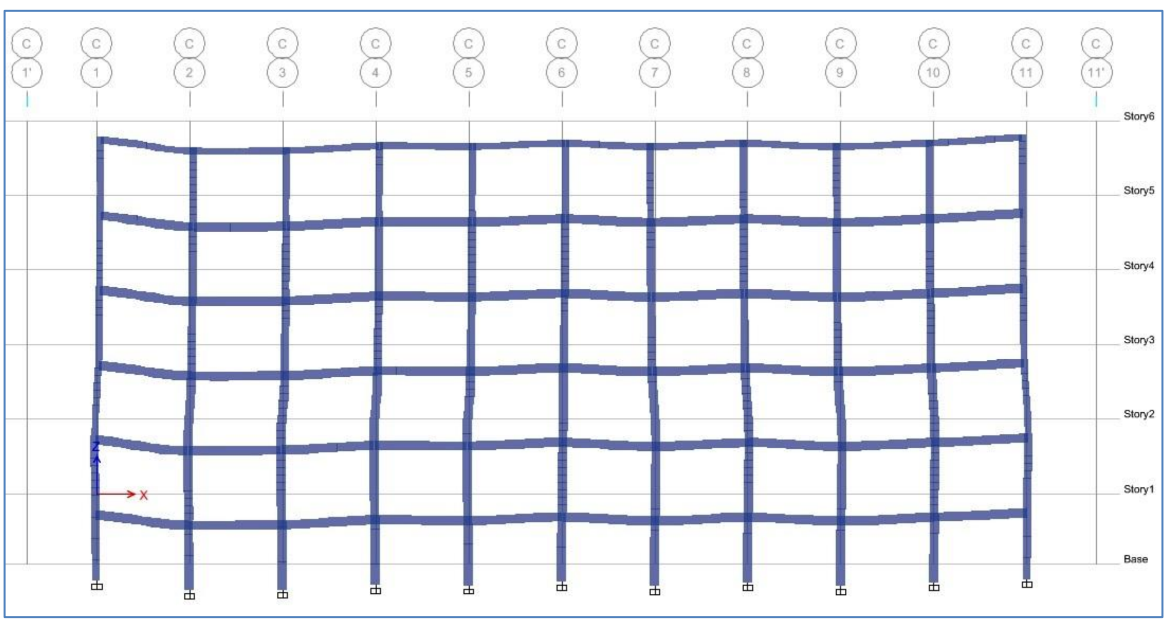


Figure 4.18 ETABS2016 analysis result of Axis C frame deformed shape

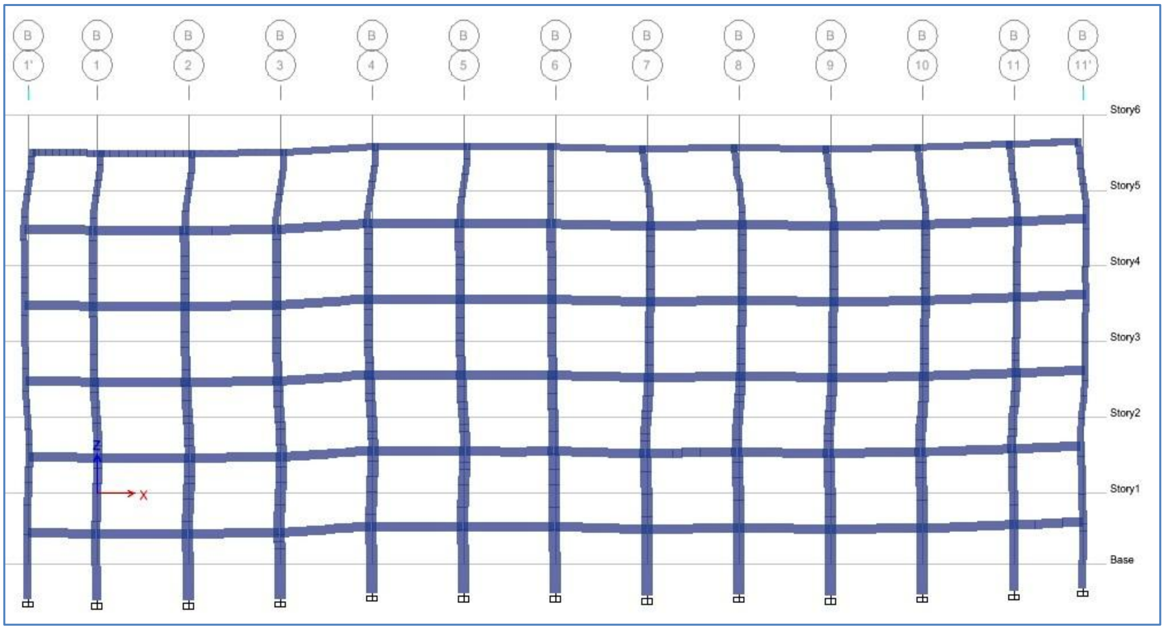


Figure 4.19 ETABS2016 analysis result of Axis B frame deformed shape

Table 4.15 Beams axis B and axis C reinforcement comparison with the original data

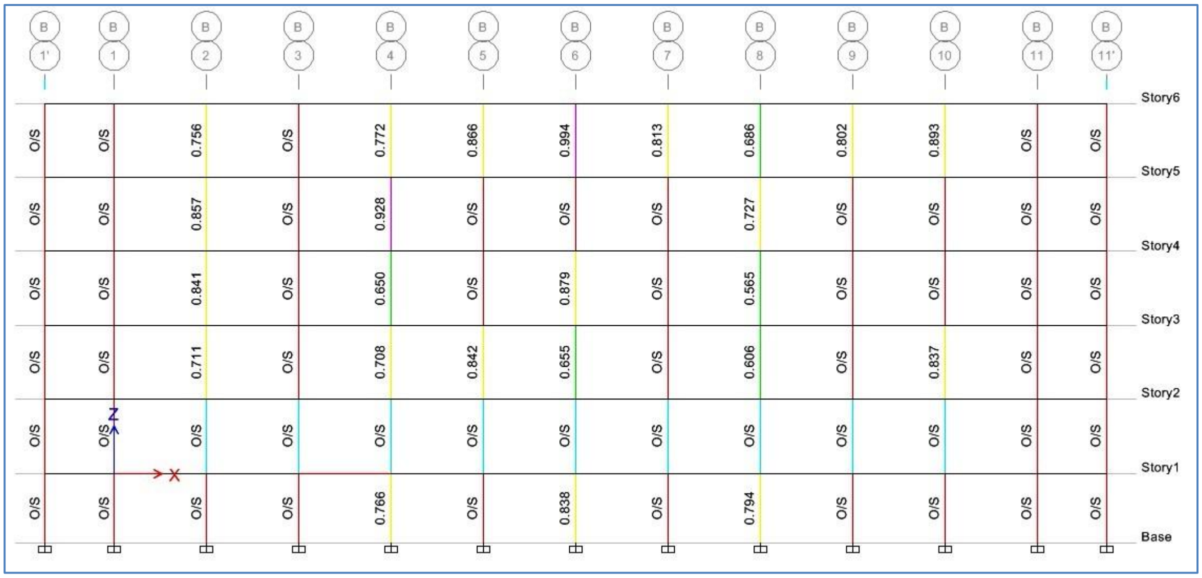
Level	Location & Beam section	Provided reinforcement #bars	Provided reinforcement Area (mm2)	Calculated R-bar area (mm2)	Calculated/ provided	Remark
1st, 2nd, 3rd and 4th typical floor	**Axis B** (300x400mm)					
	Sec 7-7 (support)	4ϕ20	1256.64	1515	1.21	Top bar
	Sec 5-5(span)	3ϕ16	603.2	884	1.47	Bottom
	Axis C (300x400mm)					
	Sec 3-3 (support)	3ϕ16	603	1226	2.03	Top bar
	Sec 2-2(span)	2ϕ16	402.124	717	1.78	Bottom

In Table 4.15 shown that the demand capacity ratio of beams on Axis B and C of typical floors is computed as a ratio rebars area of calculated to provided. The demand capacity ratio values in Table 4.15 indicate that the case study building beam members are highly stressed.

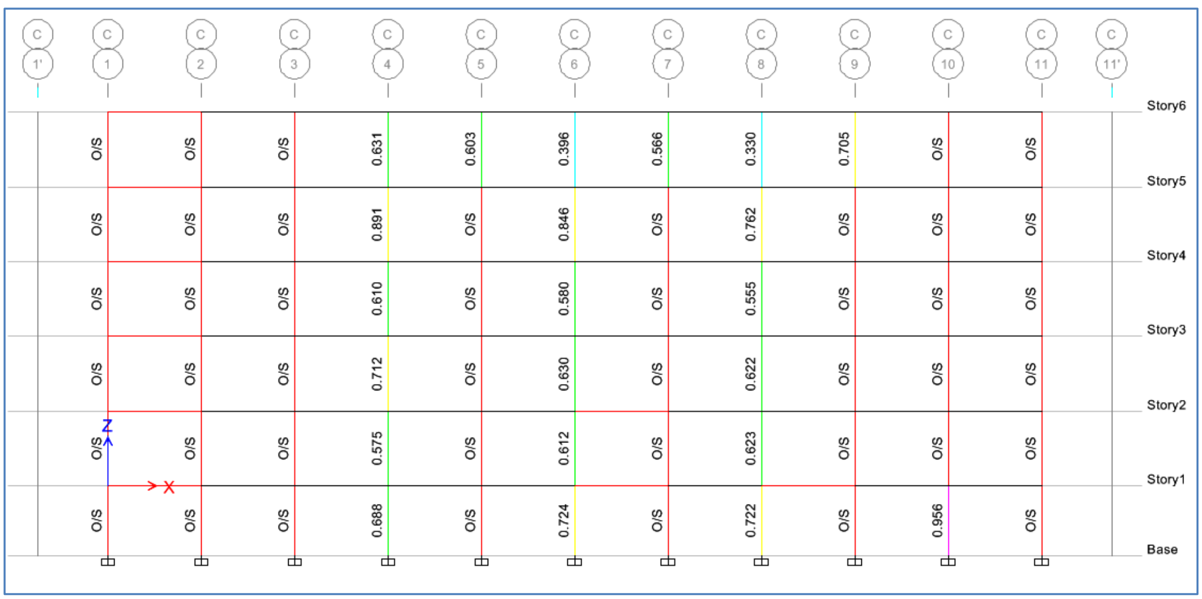
As compare with demand capacity ratio computed for the conventional design shown in Table 4.12 and the result in Table 4.15 demand capacity ratio computed for displacement load due to differential settlement, the lateral exhibits much larger value.

Table 4.16 Column demand capacity ratio with Settlement effect on frame

Column design							Demand/capacity
Column axis B	**B (mm)**	**H (mm)**	**Level**	**Nsd (kN)**	**M2-2 (kN-m)**	**M3-3 (kN-m)**	**P-M-M interaction ratio**
Col B6	300	300	4th floor	90.45	43.74	52.26	0.994
Col B6	400	400	3th floor	394.67	41.55	67.08	O/S
Col B6	400	400	2th floor	702.47	50.61	56.5	0.879
Col B6	450	450	1th floor	1019.51	77.17	67.4	0.655
Col B6	500	500	Ground	1344.34	152.83	87.04	0/s
Col B6	500	500	Foundation	1450.35	109.32	94.71	0.838
Column axis c							
Col C1	300	300	4th floor	21.2	89.63	138.43	O/s
Col C1	300	300	3th floor	75.6	136.65	168.76	O/s
Col C1	300	300	2th floor	155.287	164.51	167.46	O/s
Col C1	300	300	1th floor	245.54	200	146.51	O/s
Col C1	350	350	Ground	363.89	284.	199.05	O/s
Col C1	350	350	Foundation	480.73	166.13	74.97	O/s
Column axis c							
Col C6	300	300	4th floor	48.47	43.824	10.83	0.396
Col C6	350	350	3th floor	233.51	35.97	34.85	0.846
Col C6	350	350	2th floor	426.44	53.21	30.167	0.58
Col C6	400	400	1th floor	628.75	69.11	41.89	0.630
Col C6	400	400	Ground	836.74	75.4	38.15	0.612
Col C6	400	400	Foundation	970.4	45.02	48.46	0.724
Column axis c’							
Col C’11	300	300	4th floor	201.75	50.66	16.05	O/S
Col C’11	300	300	3th floor	1060.1	91.13	28.1	O/S
Col C’11	300	300	2th floor	2073	104.83	29	O/S
Col C’11	300	300	1th floor	3303	137.844	28.3915	O/S
Col C’11	350	350	Ground	4966	219.56	36.76	O/S
Col C’11	350	350	Foundation	6222.47	119.189	23.0597	O/S



(a)



(b)

Figure 4.20 Column P-M-M interaction ratio under the effect of settlement effect on frame (a) Axis B; (b) Axis C

The demand to capacity ratio ETABS2016 output result as shown in Fig 4.20 for vertical displacement loaded due to differential settlement revealed that most of the existing column structures are overstressed. These indicate that there is significant deformation in columns. When comparing result from conventional design, Table 4.13 with analysis result from differential settlement analysis as shown in Table 4.16, there is large difference such that in the later most column structure showed overstressed status than in the former.

4.4 NON LINEAR ANALYSIS

Although there was no history of seismic event occurred near the case study building, non linear pushover analysis was conducted to observe the pushover analysis was conducted to observe the ultimate failure behavior of the sample building. Detail analysis procedures, corresponding analysis calculations and detail result are shown in the Appendix B.

5. CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

Four different assessment approaches have been used to evaluate the performance of the building, which then lead to the cause of the crack. After a careful analysis and result discussion the following conclusions are drawn from each assessment.

- ✓ There is variation in property of soil along vertical and horizontal strata. This brought relative rotation of the structure which exceeded the maximum acceptable limit. Therefore, differential settlement is the major cause for the crack.

  Based on results from settlement time rate analysis, major portion of the final settlement has already been exhausted. Thus, the probability which further structural crack could occur due to settlement is less.
- ✓ From design review result /as per the revised codes/, some of the building's columns and beams are found over stressed. This condition makes the building less resistant against settlement effect. Thus, design error is the contributing factor which aggravates the width and length of the crack.
- ✓ Finally from SSI analysis, many columns are overstressed. These are more overstressed columns found in the second assessment approach (design review result). Furthermore, many beams exceeded their acceptable deflection limits. Therefore, SSI analysis results comprehensively indicate that the effect of foundation differential settlement significantly contributed to the existing condition of the building.
- ✓ Though no seismic activity has occurred near the building, results from pushover analysis points that the building is not safe for such event. Specifically structural elements along the shorten direction exceeded the collapse prevention limit which is not acceptable for the service category of the building/school building/.

5.2 RECOMMENDATION

Conventionally, soil investigation is carried out to determine the bearing capacity and decide the location of the foundation. It is not common to conduct settlement analysis during the conventional soil investigation. The result of the study shows the importance of conducting such investigation especially for site consisting of clay layers.

Therefore the author suggests that concerned government office involved in design permit should make the settlement analysis as one of the requirement.

Generally speaking, foundation affected by soil settlement can be repaired. But introducing such method in context of low cost building will be unacceptably expensive and in some cases unnecessary as it may even exceed the total cost of the building.

REFERENCE

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APPENDIX A- SOIL INVESTIGATION AND ANALYSIS DATA

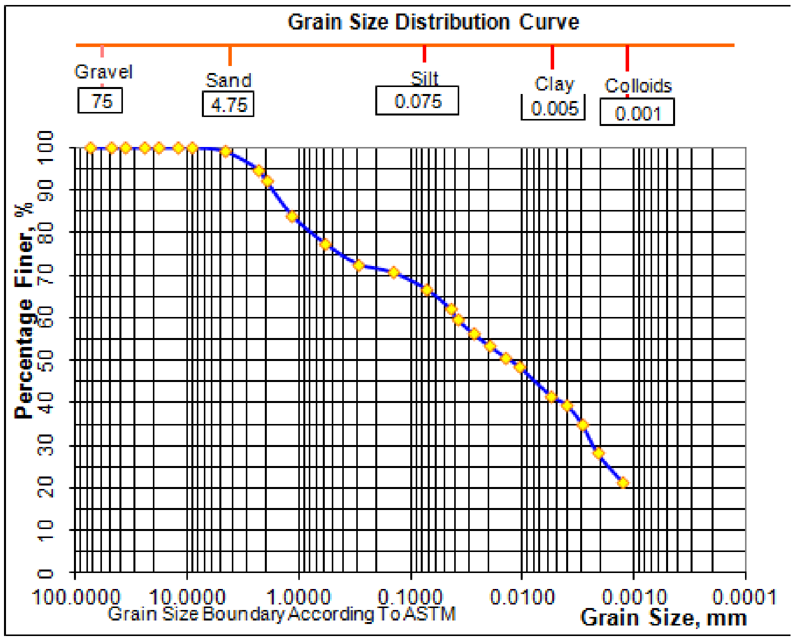
Table AASHTO Soil Classification System

CLASSIFICATION OF HIGHWAY SUBGRADE MATERIALS (With suggested subgroups)											
General Classification	Granular Materials (35% or less passing No. 200)							Silt-Clay Materials (More than 35% passing #200)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, Percent Passing:											
No. 10	0-50										
No. 40	0-30	0-50	51-100								
No. 200	0-15	0-25	0-10	0-35	0-35	0-35	0-35	36-100	36-100	36-100	36-100
Characteristics of fraction passing # 40:											
Liquid Limit				0-40	41+	0-40	41+	0-40	41+	0-40	41+
Plasticity Index	0-6		N.P.	0-10	0-10	11+	11+	0-10	0-10	11+	11+
Group Index	0		0	0		0-4		0-8	0-12	0-16	0-20
Usual Types of Significant Constituent Materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good					Fair to Poor					

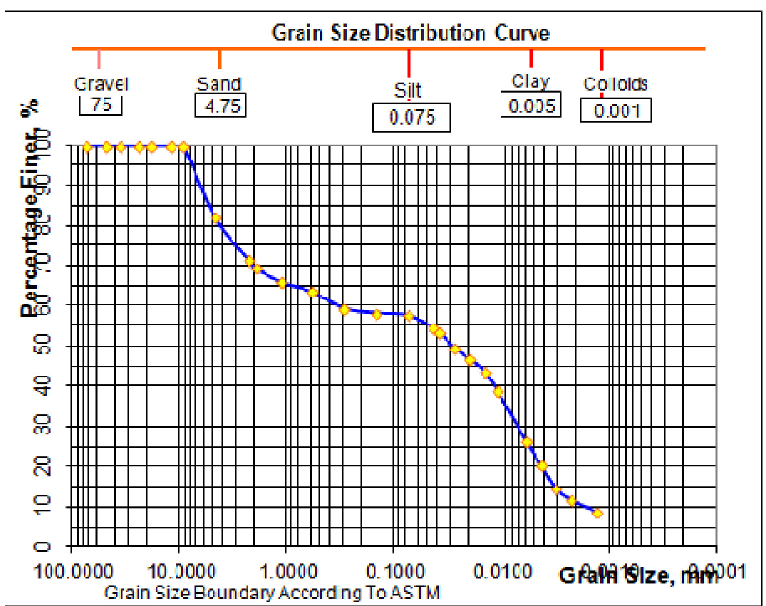
Soil laboratory test results

Varies Pits Grain Size Distribution

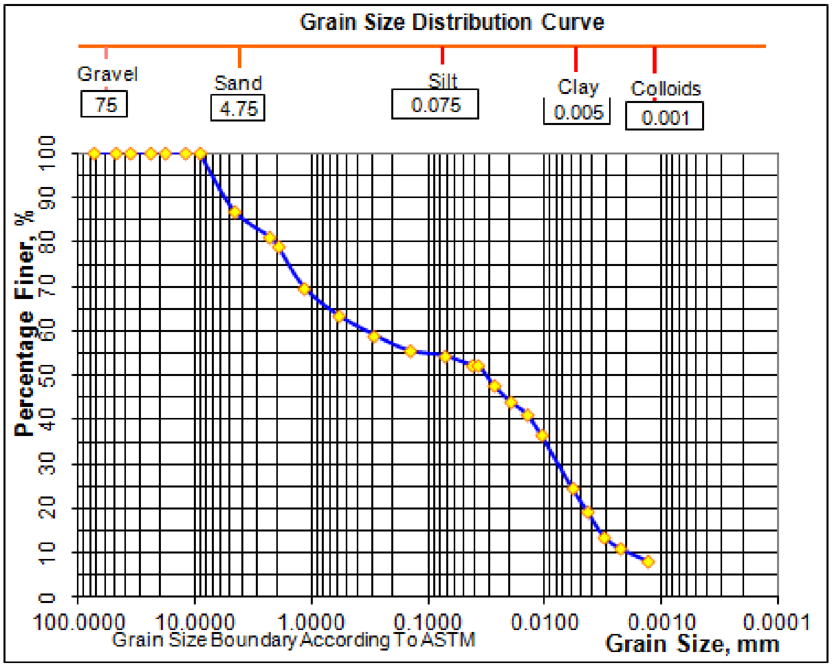
Pit 1 at 3m



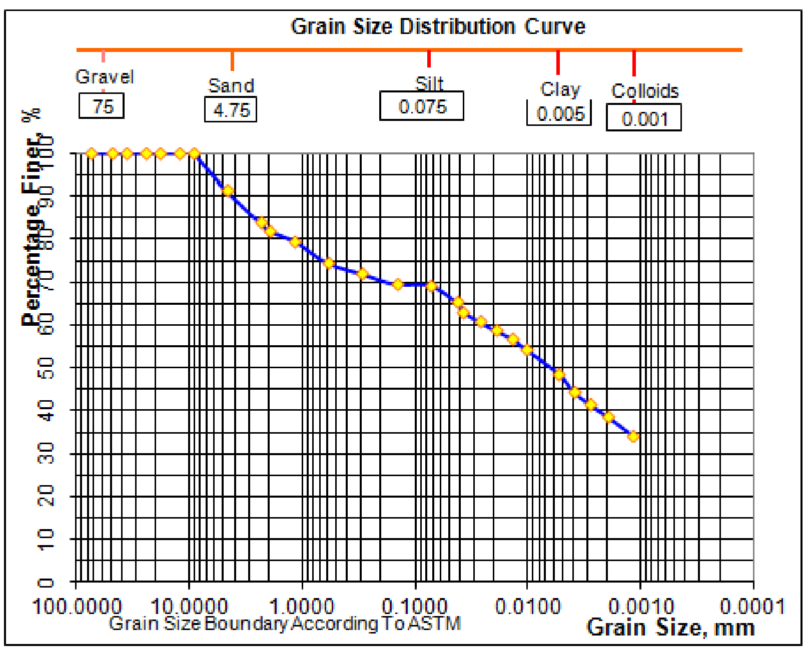
Pit 1 at 4.5m



Pit at 2, 3 and 4 at 3m



Pit at 3 and 4 at 4.5m



Consolidation test cumulative gauge readings of each pits from consolidation test

Pit 1

At 3m

Time(min.)	√time	Dial Guage Reading, mm 7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	3.39	3.102	3.428	3.806	4.204	5.022	5.474
0.1	0.32		3.324	3.762	4.310	4.758	5.326	5.842
0.25	0.50	-	3.342	3.618	4.330	4.780	5.338	5.866
0.50	0.71	-	3.348	3.642	4.370	4.806	5.350	5.890
1	1.00	-	3.354	3.662	4.396	4.828	5.364	5.922
2	1.41	-	3.566	3.644	4.224	4.852	5.378	5.946
4	2.00	-	3.570	3.702	4.266	4.878	5.390	5.980
8	2.83	-	3.596	3.720	4.290	4.906	5.402	6.018
15	3.87	-	3.418	3.734	4.310	4.928	5.414	6.060
30	5.48	-	3.404	3.952	4.328	4.950	5.428	6.110
60	7.75	-	3.408	3.966	4.346	4.970	5.438	6.162
120	10.95	-	3.418	3.980	4.362	4.986	5.448	6.208
240	15.49	-	3.402	3.990	4.378	5.000	5.454	6.236
480	21.91	-	3.422	3.804	4.390	5.010	5.462	6.258
1440	37.95	3.10	3.428	3.806	4.204	5.022	5.474	6.278

Calculation table:

Applied Pressure P (kpa)	Final Dial reading (mm)	Change In specimen Height (mm)	Final Specimen Height (mm)	Void Height, h_v (mm)	Void Ratio, e
Loading					
7		0.00	20.00	8.20	0.70
7	2.300	0.15	20.00	8.20	0.70
50	4.798	-0.49	19.51	7.72	0.65
100	5.380	-0.58	18.93	7.13	0.60
200	6.124	-0.74	18.19	6.39	0.54
400	6.898	-0.77	17.42	5.62	0.48
800	7.466	-0.63	16.79	4.99	0.42
1600	8.252	-0.83	15.96	4.16	0.35
Unloading					
1600	8.652	-0.83	15.96	4.16	0.35
800	8.558	0.09	16.05	4.25	0.36
400	8.406	0.15	16.20	4.41	0.37
200	8.324	0.08	16.29	4.49	0.38

At 4.5m

		Dial Guage Reading, mm						
Time(min.)	√time	7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	2.88	2.462	2.809	3.328	4.020	4.814	5.733
0.1	0.32		2.650	3.032	3.636	4.460	5.280	6.070
0.25	0.50	-	2.644	3.060	3.672	4.484	5.302	6.092
0.50	0.71	-	2.686	3.096	3.714	4.514	5.336	6.122
1	1.00	-	2.696	3.127	3.762	4.552	5.369	6.146
2	1.41	-	2.706	3.158	3.810	4.584	5.402	6.178
4	2.00	-	2.730	3.186	3.850	5.624	5.442	6.212
8	2.83	-	2.733	3.214	3.892	5.662	5.488	6.252
15	3.87	-	2.745	3.236	3.923	4.692	5.534	6.278
30	5.48	-	2.760	3.258	3.948	4.722	5.600	6.350
60	7.75	-	2.769	3.279	3.967	4.748	5.636	6.412
120	10.95	-	2.790	3.294	3.982	4.764	5.676	6.466
240	15.49	-	2.799	3.302	3.996	4.780	5.702	6.507
480	21.91	-	2.804	3.315	4.006	4.796	5.720	6.532
1440	37.95	2.46	2.809	3.328	4.020	4.814	5.733	6.550

Calculation table:

Applied Pressure P (kpa)	Final Dial reading (mm)	Change In specimen Height (mm)	Final Specimen Height (mm)	Void Height, h_v (mm)	Void Ratio, e
Loading					
7	2.880	0.000	20.000	6.775	0.512
7	2.460	0.420	20.420	7.195	0.544
50	2.809	-0.347	20.073	6.848	0.518
100	3.328	-0.519	19.554	6.329	0.479
200	4.020	-0.692	18.862	5.637	0.426
400	4.814	-0.794	18.068	4.843	0.366
800	5.733	-0.919	17.149	3.924	0.297
1600	6.550	-0.817	16.332	3.107	0.235
Unloading					
1600	6.550	-0.817	16.332	3.107	0.235
800	6.494	0.056	16.614	3.389	0.245
400	6.414	0.080	16.694	3.469	0.260
200	6.316	0.098	16.792	3.567	0.270

Pit 2

At 3m

Time(min.)	√time	Dial Guage Reading, mm 7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	2.44	2.296	2.652	3.098	3.840	4.606	5.636
0.1	0.32		2.308	2.804	3.698	4.242	5.070	5.646
0.25	0.50	-	2.310	2.826	3.710	4.286	5.088	5.672
0.50	0.71	-	2.520	2.848	3.748	4.324	5.110	5.698
1	1.00	-	2.618	2.870	3.780	4.348	5.136	5.724
2	1.41	-	2.728	2.892	3.618	4.380	5.168	5.756
4	2.00	-	2.746	2.912	3.650	4.410	5.202	5.792
8	2.83	-	2.756	2.930	3.680	4.442	5.238	5.830
15	3.87	-	2.770	2.948	3.712	4.478	5.274	5.884
30	5.48	-	2.784	2.990	3.738	4.508	5.312	5.924
60	7.75	-	2.800	3.026	3.758	4.534	5.346	5.978
120	10.95	-	2.812	3.050	3.780	4.554	5.376	6.028
240	15.49	-	2.824	3.066	3.796	4.572	5.598	6.064
480	21.91	-	2.832	3.082	3.812	4.588	5.614	6.091
1440	37.95	2.30	2.840	3.098	3.840	4.606	5.636	6.136

Calculation table:

Applied Pressure P (kpa)	Final Dial reading (mm)	Change In specimen Height (mm)	Final Specimen Height (mm)	Void Height, h_v (mm)	Void Ratio, e
Loading					
7		0.00	20.00	3.88	0.24
7	2.300	0.15	20.00	3.88	0.24
50	2.840	-0.54	19.46	3.34	0.21
100	3.098	-0.45	19.01	2.89	0.18
200	3.840	-0.59	18.42	2.30	0.14
400	4.606	-0.74	17.68	1.56	0.10
800	5.636	-0.61	17.07	0.95	0.06
1600	6.210	-0.72	16.35	0.23	0.01
Unloading					
1600	6.210	-0.72	16.35	0.23	0.01
800	6.022	0.19	16.54	7.38	0.02
400	5.724	0.30	16.61	2.06	0.03

Pit 2, 3 and 4

At 3m

Time(min.)	√time	Dial Guage Reading, mm 7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	2.44	4.312	4.798	5.380	6.124	6.898	7.466
0.1	0.32		4.606	5.040		6.360	7.148	7.682
0.25	0.50	-	4.614	5.084		6.398	7.118	7.708
0.50	0.71	-	4.626	5.124		6.632	7.208	7.832
1	1.00	-	4.632	5.154		6.674	7.232	7.972
2	1.41	-	4.642	5.186		6.706	7.260	7.996
4	2.00	-	4.652	5.216		6.738	7.290	8.032
8	2.83	-	4.664	5.248		6.766	7.322	8.070
15	3.87	-	4.676	5.268		6.788	7.348	8.102
30	5.48	-	4.690	5.292		6.812	7.376	8.136
60	7.75	-	4.722	5.313		6.834	7.390	8.166
120	10.95	-	4.752	5.333		6.852	7.415	8.188
240	15.49	-	4.766	5.348		6.866	7.430	8.208
480	21.91	-	4.780	5.362		6.891	7.447	8.225
1440	37.95	2.30	4.798	5.380	6.124	6.898	7.466	8.252

Calculation table:

Applied Pressure P (kpa)	Final Dial reading (mm)	Change In specimen Height (mm)	Final Specimen Height (mm)	Void Height,h_v (mm)	Void Ratio, e
Loading					
7		0.00	20.00	8.20	0.70
7	2.300	0.15	20.00	8.20	0.70
50	4.798	-0.49	19.51	7.72	0.65
100	5.380	-0.58	18.93	7.13	0.60
200	6.124	-0.74	18.19	6.39	0.54
400	6.898	-0.77	17.42	5.62	0.48
800	7.466	-0.63	16.79	4.99	0.42
1600	8.252	-0.83	15.96	4.16	0.35
Unloading					
1600	8.652	-0.83	15.96	4.16	0.35
800	8.558	0.09	16.05	4.25	0.36
400	8.406	0.15	16.20	4.41	0.37
200	8.324	0.08	16.29	4.49	0.38

At 4.5 m

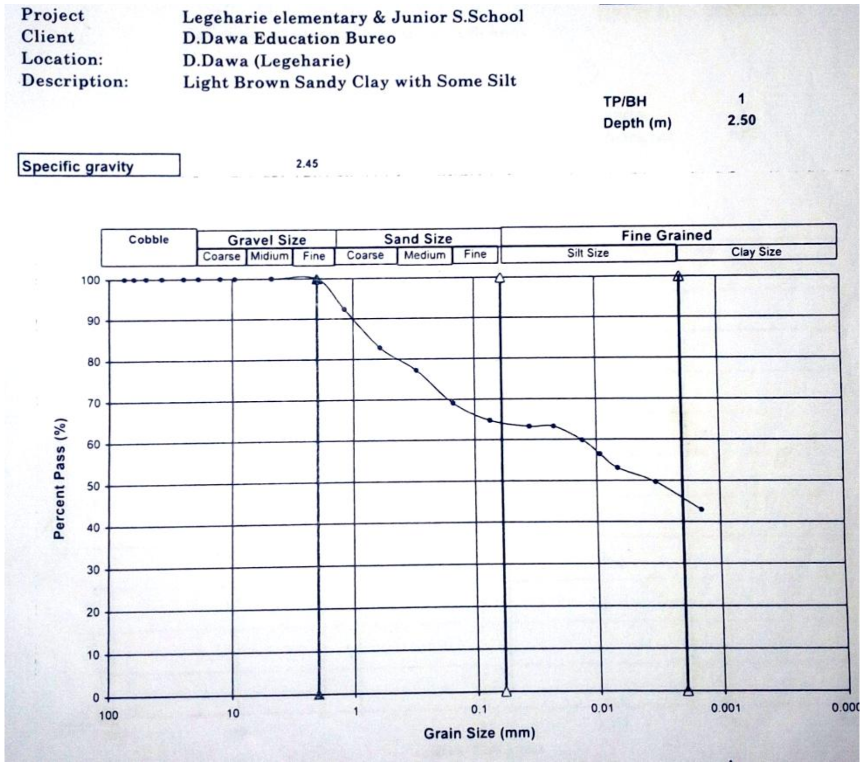
		Dial Guage Reading, mm						
		7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
Time(min.)	√time							
0	0.00	3.39	2.901	3.018	3.153	3.453	3.924	4.360
0.1	0.32		2.974	3.066	3.330	3.834	4.152	4.664
0.25	0.50	-	2.976	3.070	3.338	3.838	4.266	4.692
0.50	0.71	-	2.979	3.076	3.344	3.844	4.182	4.714
1	1.00	-	2.982	3.082	3.358	3.852	4.200	4.741
2	1.41	-	2.986	3.089	3.370	3.860	4.218	4.762
4	2.00	-	2.989	3.096	3.382	3.870	4.240	4.788
8	2.83	-	2.992	3.103	3.392	3.877	4.264	4.813
15	3.87	-	2.994	3.110	3.402	3.885	4.282	4.835
30	5.48	-	2.998	3.118	3.412	3.892	4.302	4.859
60	7.75	-	3.000	3.125	3.421	3.900	4.316	4.880
120	10.95	-	3.004	3.132	3.428	3.906	4.329	4.895
240	15.49	-	0.000	3.138	3.437	3.912	4.340	4.908
480	21.91	-	0.000	3.142	3.440	3.917	4.350	4.921
1440	37.95	2.46	3.018	3.153	3.453	3.924	4.360	4.934

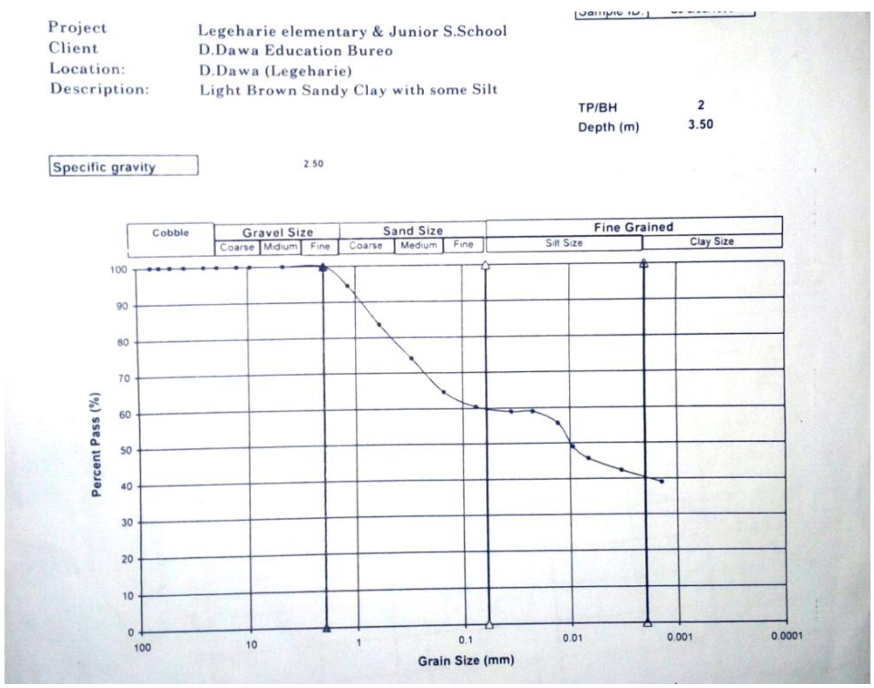
Calculation table:

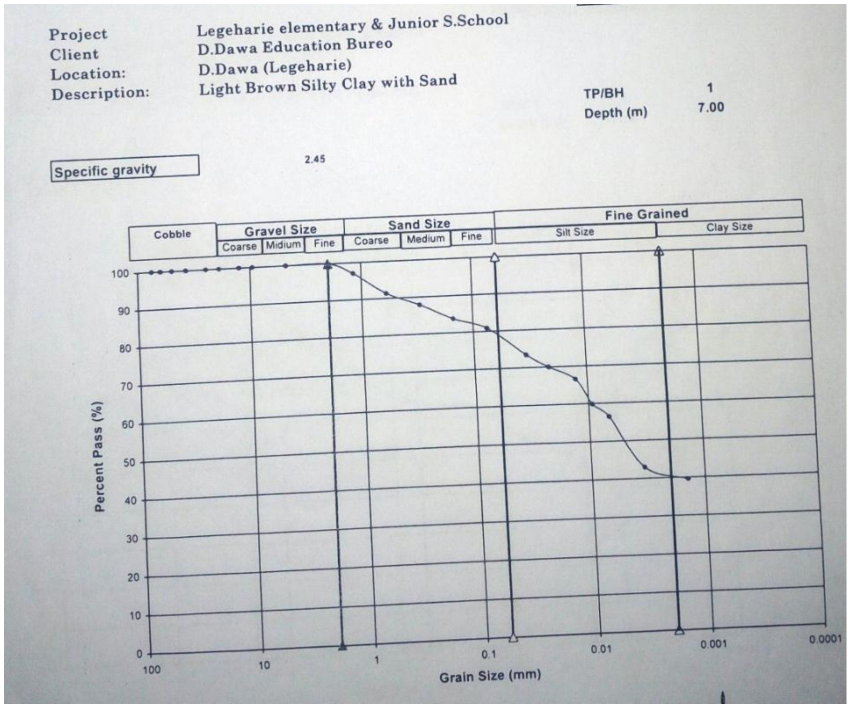
Applied Pressure P (kpa)	Final Dial reading (mm)	Change In specimen Height (mm)	Final Specimen Height (mm)	Void Height, h_v (mm)	Void Ratio, e
Loading					
7	0	0.00	20.00	6.70	0.50
7	3.100	0.92	20.92	7.62	0.57
50	5.214	-0.12	20.80	7.51	0.56
100	5.870	-0.14	20.67	7.37	0.55
200	6.458	-0.30	20.37	7.07	0.53
400	7.000	-0.42	19.95	6.66	0.50
800	7.894	-0.44	19.52	6.22	0.47
1600	8.602	-0.57	18.94	5.65	0.44
Unloading					
1600	8.602	0.00	18.94	5.65	0.44
800	8.550	0.05	18.99	9.83	0.44
400	8.232	0.32	16.61	2.06	0.45

Soil index properties from the previous studied Soil test report

Grain size distribution at a varies depth





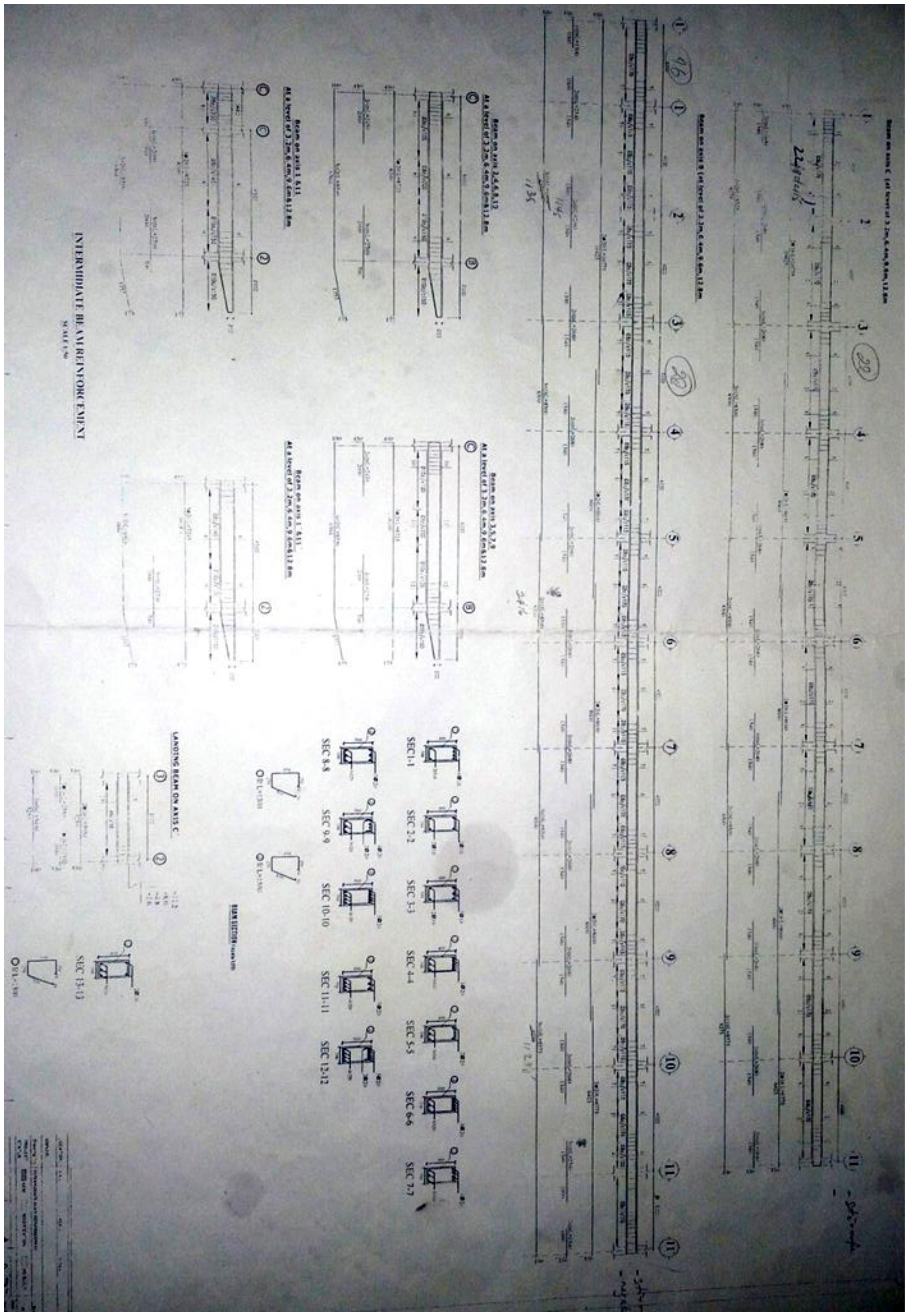


Attemberg Limit at a varies depth

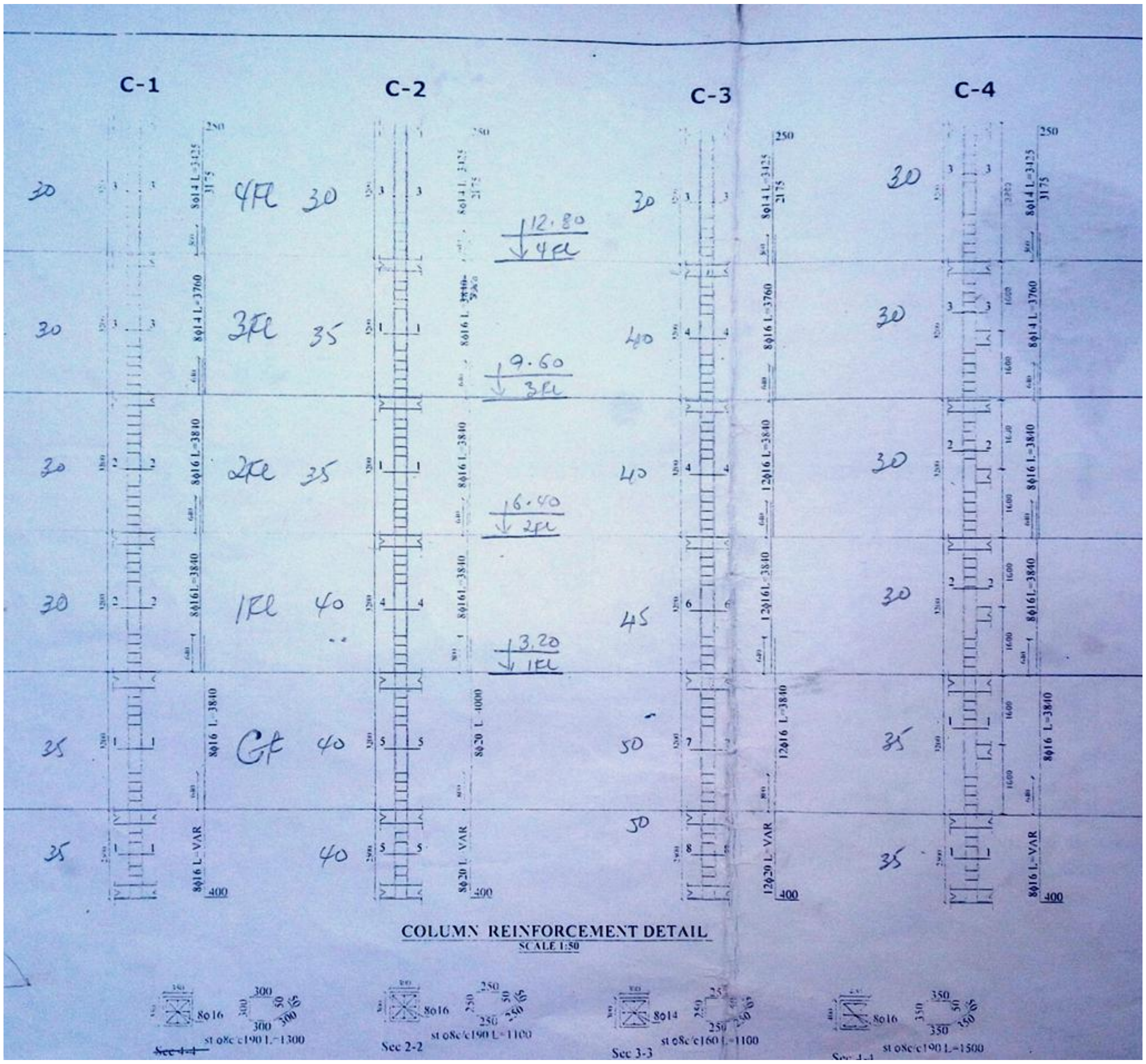
N°	BH N°	Depth (m)	Specific gravity	Bulk Unit weight Kg/m3	Moisture content (%)	LL (%)	PL (%)	PI (%)	Free Swell %	Shear Strength C KN/m^2	Shear Strength Ø Degree
1	1	2.50	2.45	1865	10.07	32.20	20.96	11.24	30	13	14
2	1	5.00	2.45	1829	8.14	27.80	17.43	10.37	10	16.00	17
3	1	9.20	2.50			28.80	21.34	7.46			
4	2	3.50	2.45	1889	13.08	28.20	15.27	12.93	30	12	18
5	2	6.80	2.45			25.20	18.39	6.81			
6	2	9.30	2.50			28.80	18.19	10.61	30		

The building structural drawings

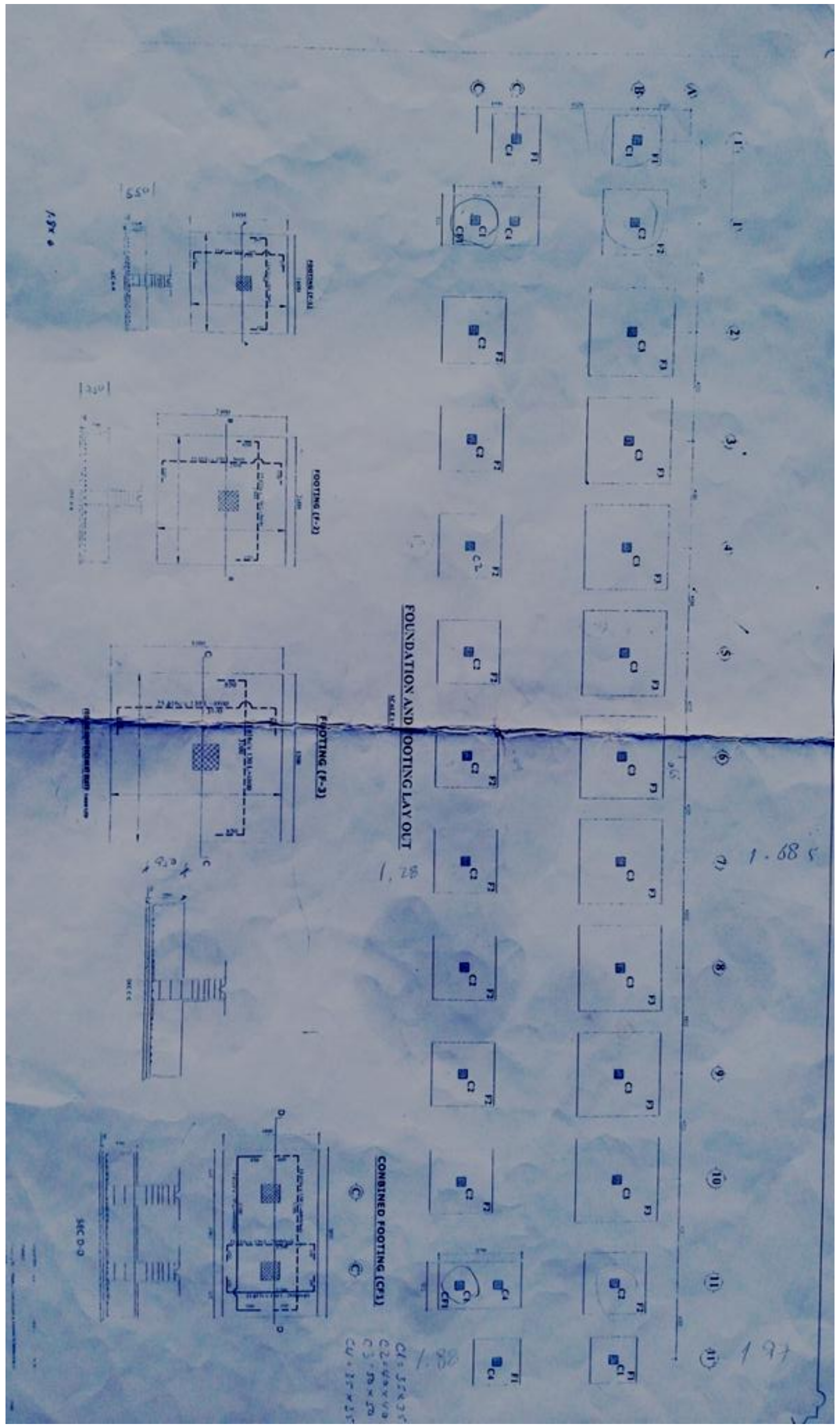
Typical floors beam detail



Columns


C-1
C-2
C-3
C-4
COLUMN REINFORCEMENT DETAIL
SCALE 1:50
4FL
3FL
2FL
1FL
GF
12.80
4FL
9.60
3FL
6.40
2FL
3.20
1FL
Sec 2-2
Sec 3-3

Foundation detail



APPENDIX B NON LINEAR PUSHOVER ASSESSMENT

B.1. PUSHOVER ANALYSIS RESULT AND DISCUSSION

B.1.1 Base shear versus top displacement (pushover curve)

A static nonlinear (pushover) analysis performed using SAP2000 version 18.1.0. The loads were applied independently in the global X and Y directions. The control node to monitor the displacement of the building was selected at the centre of mass of the building. In the present paper, a displacement control based analysis was performed. A pushover analysis was carried out separately in the X and Y directions and a maximum roof target displacement of 0.28m was chosen to be applied. Due to an incremental lateral displacement, the pushover curve slope gradually changed with the progressive plastic hinge formation in beams and columns throughout the building structure. The resulting pushover curves, in terms of base shear roof displacement (V-U) are displayed as shown in Fig B.1 for both X and Y directions.

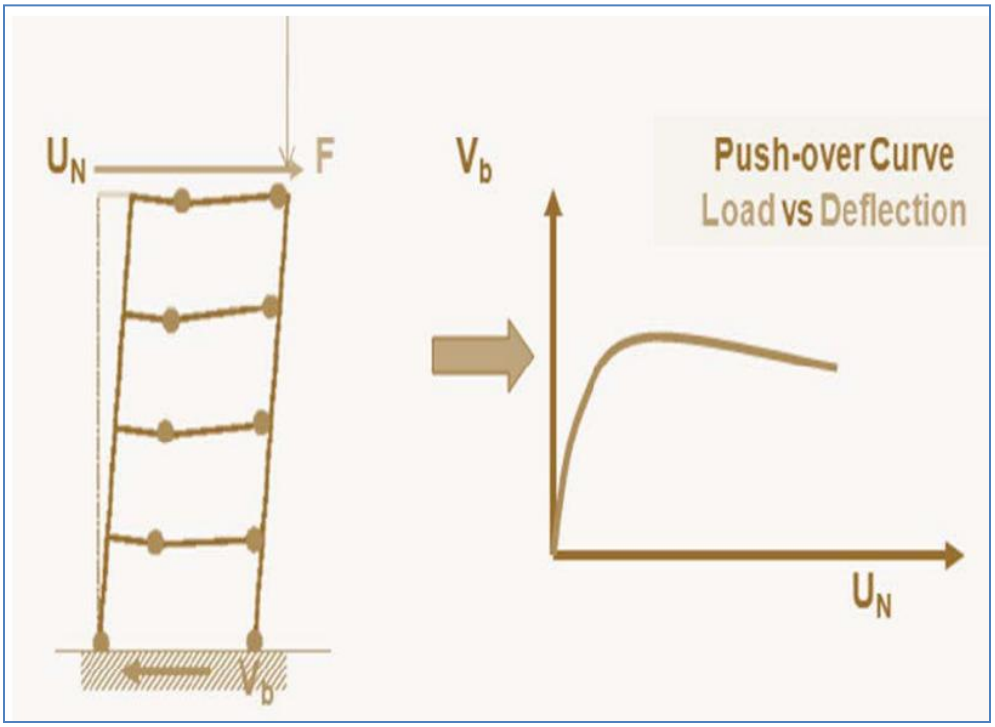


Figure B.1: Pushover curve, load vs deflection

X- Direction

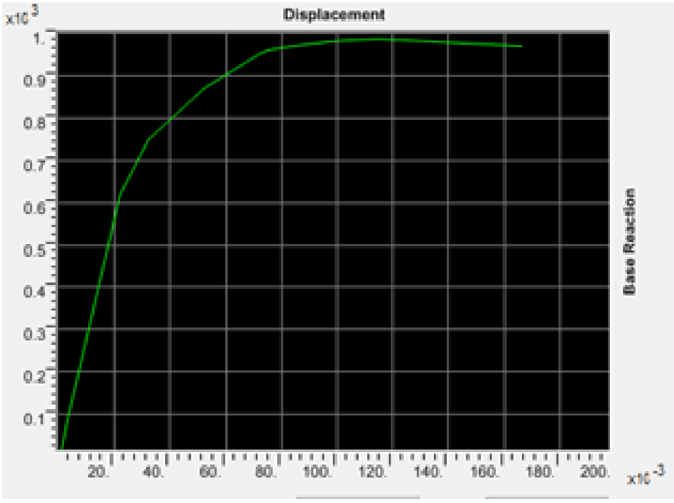


Figure B.2: Base shear-roof displacement in X direction

The maximum base shear of the structure in the X direction from Fig B.2 is V= 886 kN and ultimate roof displacement δ = 0.1153 m

Y-Direction

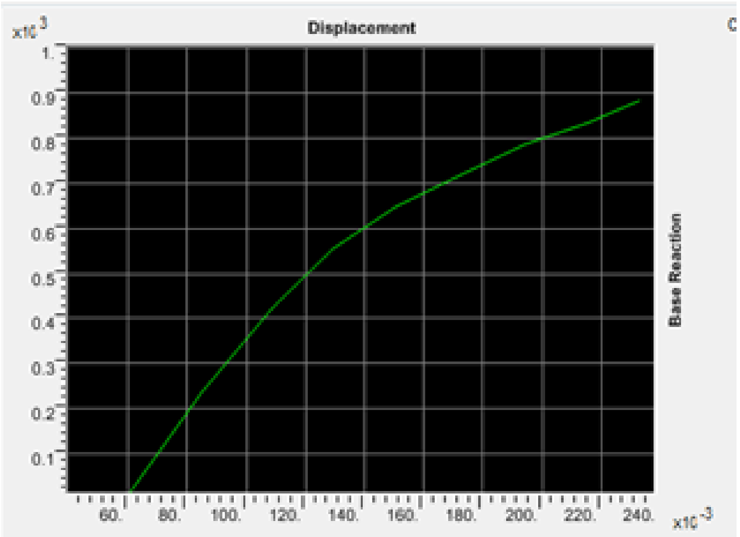


Figure B.3: Base shear-roof displacement in Y direction

The maximum base shear of the structure in the Y direction from Fig B.3 is V= 886 kN and ultimate roof displacement δ = 0.2323 m.

From the pushover curve result, X direction receive similar force within lesser displacement than Y direction. Therefore X direction is stiffer than y direction. This happened by the fact in the X direction more number of columns are exist than Y.

Hence, based on the above result, if any earthquake will come about, most of the damage will first occur on Y-axis (lesser number of columns exists).

a. Performance level

As can be seen in Fig B.4 the performance point is the point where the capacity curve crosses the demand curve according to FEMA356 and ATC40. For each pushover analysis, a corresponding capacity curve was thus obtained as shown in the Fig B.5 and Fig B.6. The yellow line represent demand curve, the green one is the capacity. This analysis was completed within 17 and 10 (last) steps along in the X and Y direction respectively.

The main output of a pushover analysis is in terms of response demand versus capacity. If the demand curve intersects the capacity envelope near the elastic range, then the structure has a good resistance. If the demand curve intersects the capacity curve with little reserve of strength and deformation capacity, then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse. From this analysis in both directions the performance point as shown in the Fig. and Fig occurs at the plastic zone which means the structure is not safe under the expected lateral earthquake load.

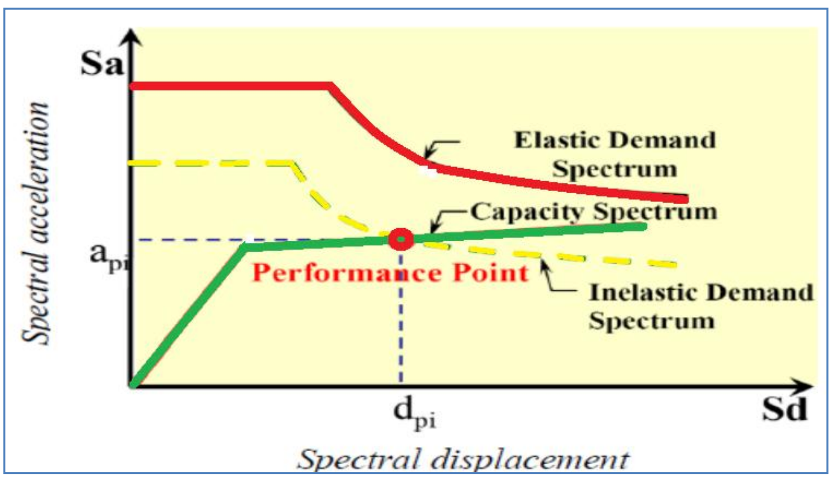


Figure B.4: Performance point and damage level

X direction

This analysis was completed in 13 steps and performance point was set between step 4 and 5 of the analysis. The performance point D is equal to 0.065m.

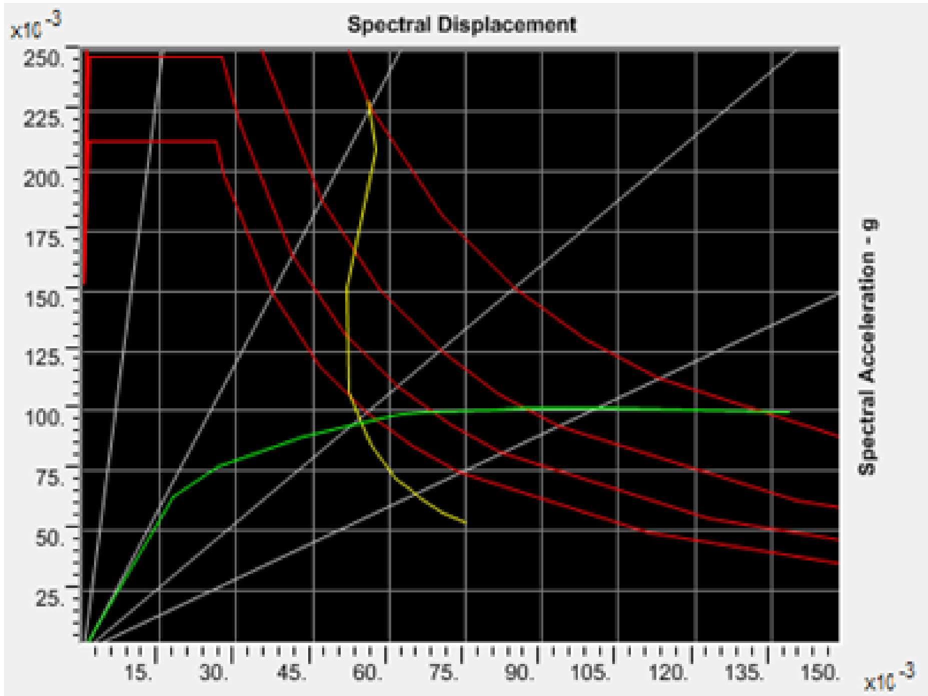


Figure B.5: Performance point due to push in the X direction according to ATC 40

Table B.1: Push capacity curve damage level in the X direction

TABLE: Pushover Capacity Curve												
LoadCase	Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
Text	Unitless	m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSH X	0	0.000439	0	1000	10	0	0	0	0	0	0	1010
PUSH X	1	0.00315	85.69	999	11	0	0	0	0	0	0	1010
PUSH X	2	0.022166	617.959	923	75	12	0	0	0	0	0	1010
PUSH X	3	0.032145	748.345	860	88	62	0	0	0	0	0	1010
PUSH X	4	0.052248	870.163	824	48	117	9	0	12	0	0	1010
PUSH X	5	0.072459	951.872	799	66	56	16	0	71	2	0	1010
PUSH X	6	0.074959	958.702	789	75	43	18	0	82	3	0	1010
PUSH X	7	0.079959	966.754	777	87	21	13	0	106	6	0	1010
PUSH X	8	0.096209	979.781	755	93	24	1	0	129	8	0	1010
PUSH X	9	0.103084	982.966	750	95	24	3	0	129	9	0	1010
PUSH X	10	0.114998	986.216	743	100	24	1	0	117	25	0	1010
PUSH X	11	0.126248	984.663	738	99	29	1	0	96	47	0	1010
PUSH X	12	0.146248	978.06	734	82	50	0	0	87	57	0	1010
PUSH X	13	0.166248	968.366	729	70	67	0	0	70	74	0	1010

From table B.1 can be concluded that most elements have entered in the plastic zone in the X direction. More than 9 elements of the building are in the collapse prevention (CP-C) limit state at the performance point. This means that the building of those elements requires strengthening.

Y direction

This analysis was completed in 9 steps and as show in Fig B.2 performance point was set between steps 6 and 7 of the analysis. The performance point d is equal to 0.18m.

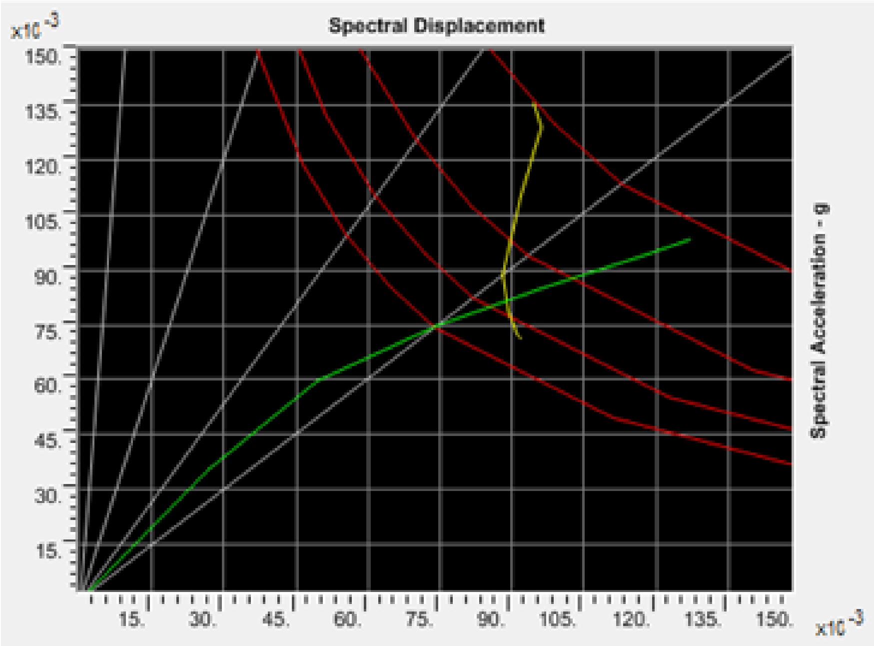


Figure B.6: Performance point due to push in the y direction according to ATC 40

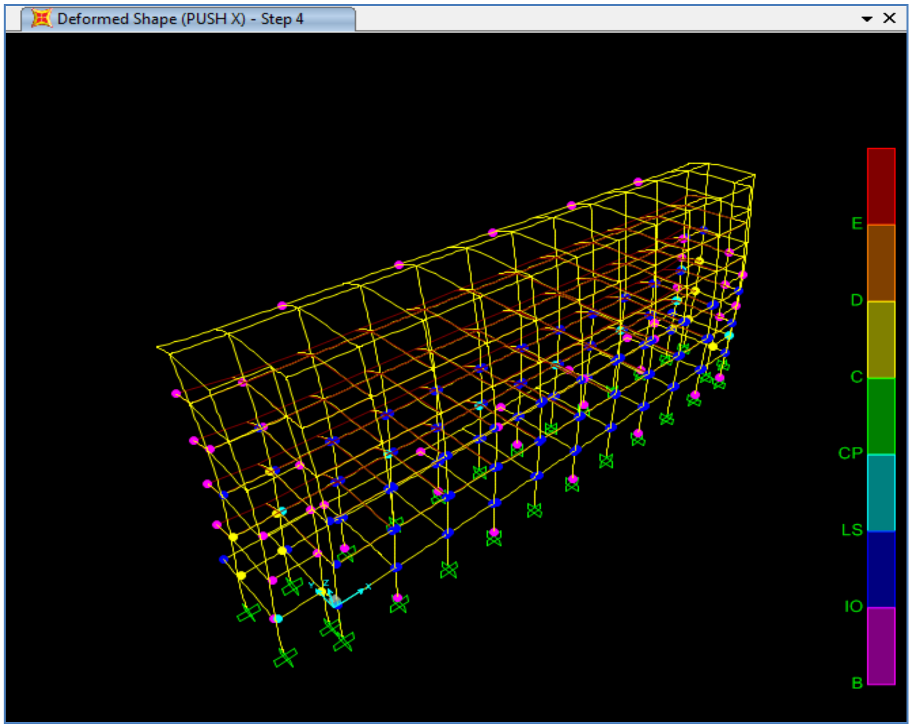
Table B.2: Push capacity curve damage level in the Y direction

TABLE: Pushover Capacity Curve												
LoadCase	Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
Text	Unitless	m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSH Y	0	0.058714	0	1000	10	0	0	0	0	0	0	1010
PUSH Y	1	0.062773	37.822	999	11	0	0	0	0	0	0	1010
PUSH Y	2	0.084781	236.245	988	18	4	0	0	0	0	0	1010
PUSH Y	3	0.108802	424.89	963	28	19	0	0	0	0	0	1010
PUSH Y	4	0.129439	556.711	937	30	29	6	0	6	2	0	1010
PUSH Y	5	0.150492	649.404	921	26	39	1	0	18	5	0	1010
PUSH Y	6	0.17284	719.14	909	12	60	4	0	16	9	0	1010
PUSH Y	7	0.194336	784.849	902	18	46	9	0	22	13	0	1010
PUSH Y	8	0.214878	833.501	893	20	33	20	0	26	18	0	1010
PUSH Y	9	0.232808	882.163	887	21	26	16	0	40	18	2	1010

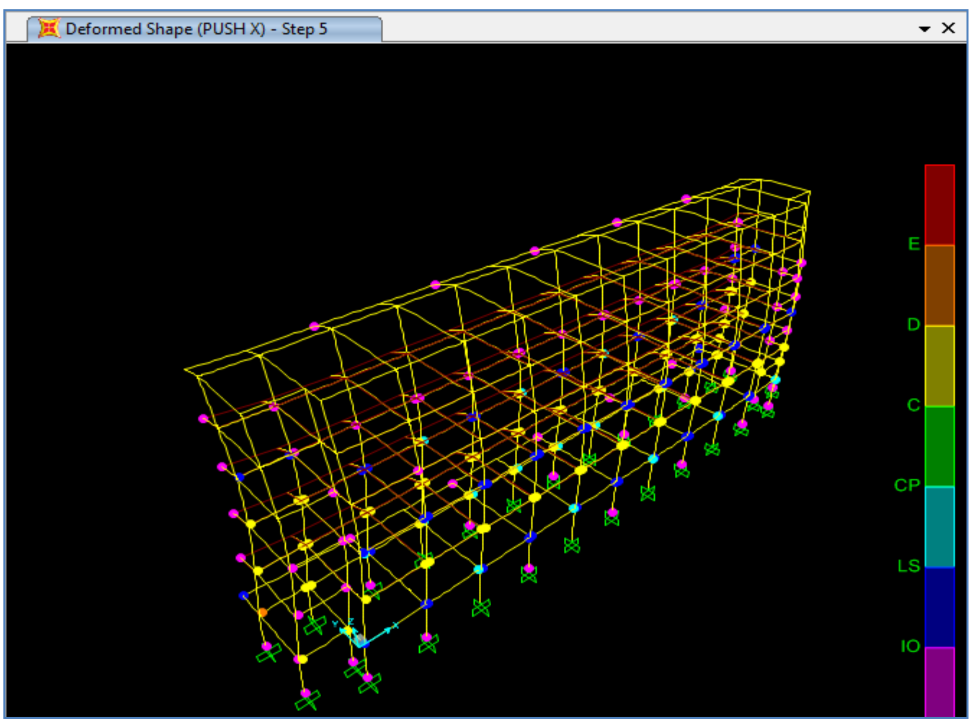
From table B.2 Y direction can be concluded that most elements have entered in the plastic zone. More than 60 elements of the building are in the collapse prevention (CP-C) limit state at the performance point. This means that the building of those elements requires strengthening.

Plastic hinge formation at the performance point

Plastic hinge formations for the selected building mechanisms have been obtained at different displacements levels. The formation of hinges at the performance point step of pushover analysis is plotted in Fig B.7 at X direction and Fig B.8 at Y direction for a given story's building. Plastic hinge formation starts with beam ends and at top columns of lower stories, then consecutively to upper stories and continue with yielding of interior intermediate columns in the upper stories. The life safety performance level is selected because the buildings are designed according to EBCS-8, 1995 which is intended to protect the life safety of the people. But as shown in the fig, most of the selected building hinges are formed at the non linear plastic range LS and CP performance level which are shown the building is not safe for expected earthquake load.

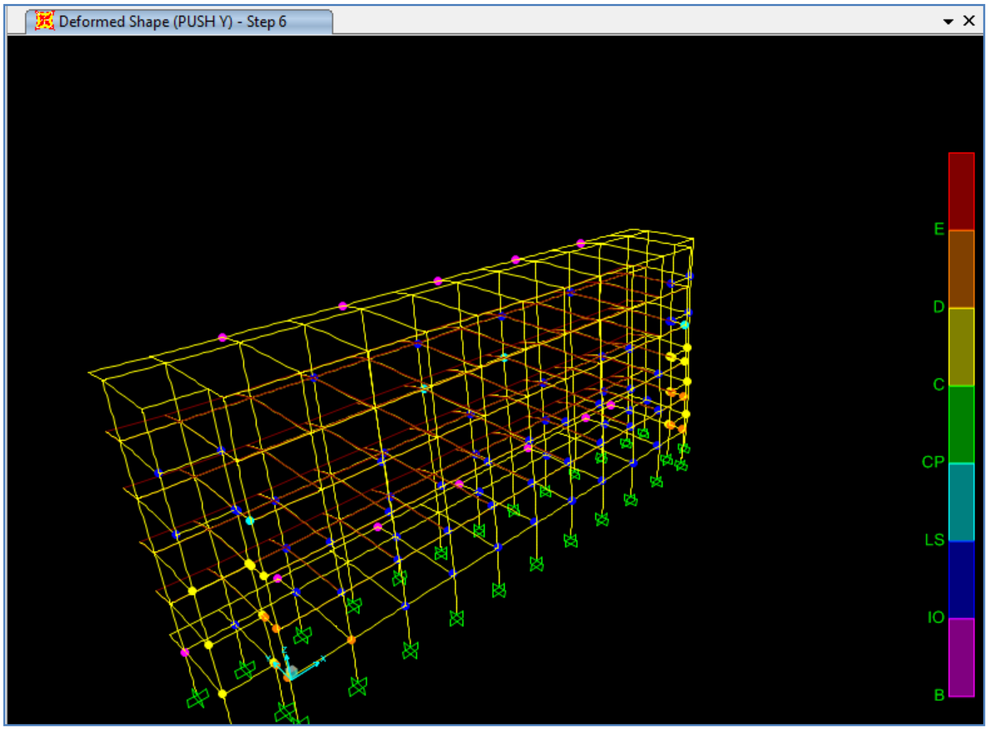


(a)

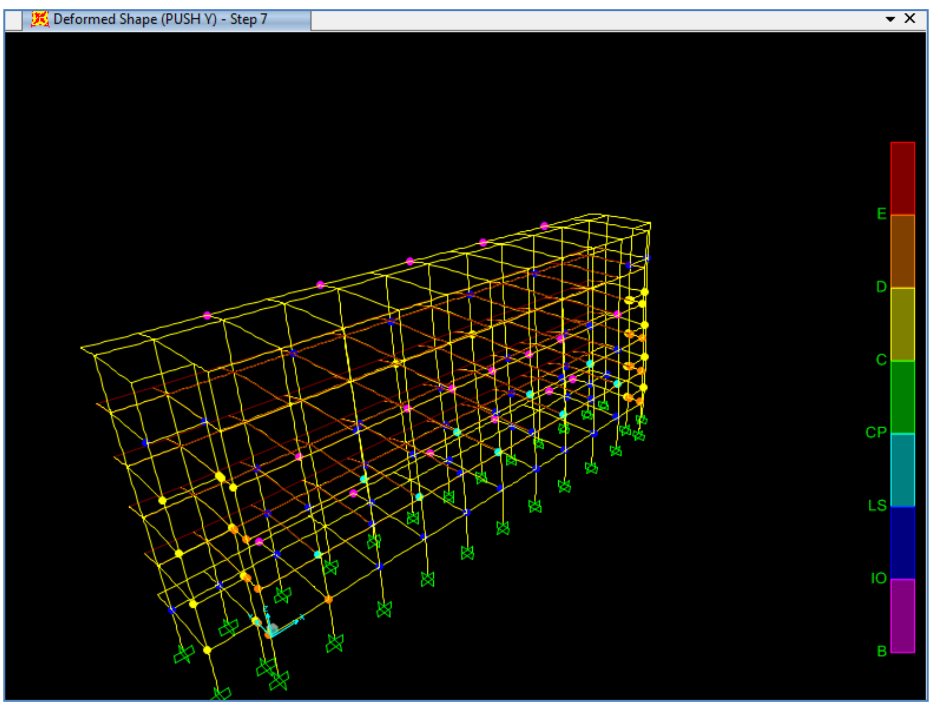


(b)

Figure B.7: Yielding pattern of the structure at the performance point in x direction; (a) step 4 (b) step 5



(a)



(b)

Figure B.8: Yielding pattern of the structure at the performance point in x direction; (a) step 6 (b) step 7

ATC 40 seismic coefficient C_A and Cv

Table B.3; Seismic coefficient C_A

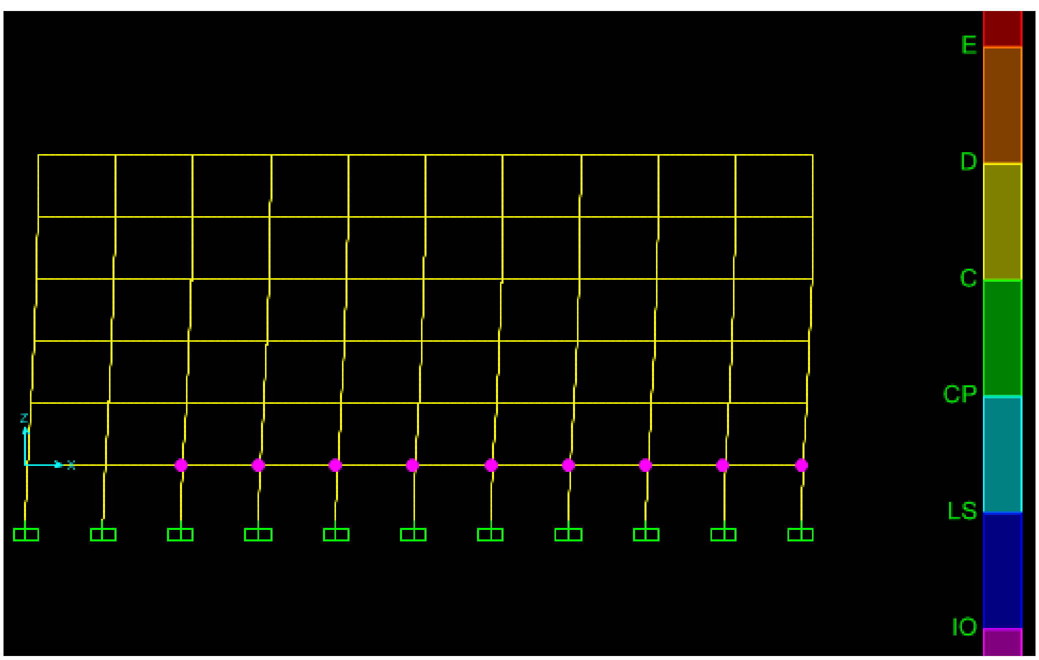
Soil Profile Type	Shaking Intensity, ZEN[1,2]					
	= 0.075	= 0.15	= 0.20	= 0.30	= 0.40	> 0.40
S_B	0.08	0.15	0.20	0.30	0.40	1.0(ZEN)
S_C	0.09	0.18	0.24	0.33	0.40	1.0(ZEN)
S_D	0.12	0.22	0.28	0.36	0.44	1.1(ZEN)
S_E	0.19	0.30	0.34	0.36	0.36	0.9(ZEN)
S_F	Site-specific geotechnical investigation required to determine C_A					

Table B.4; Seismic coefficient Cv

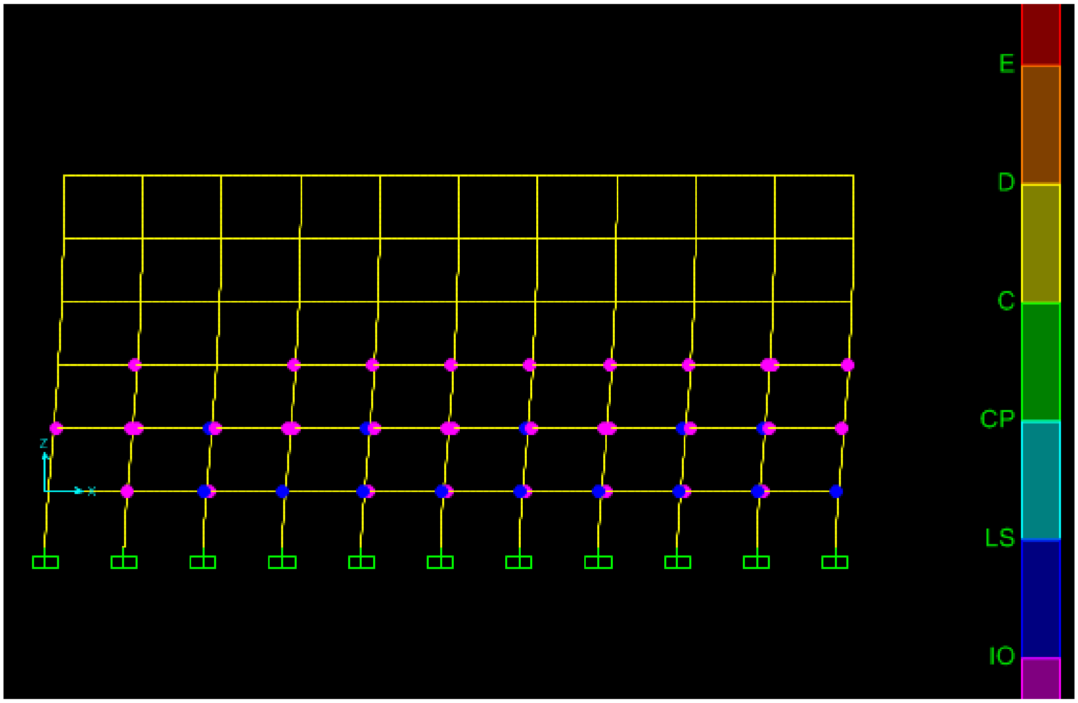
Soil Profile Type	Shaking Intensity, ZEN[1,2]					
	= 0.075	= 0.15	= 0.20	= 0.30	= 0.40	> 0.40
S_B	0.08	0.15	0.20	0.30	0.40	1.0(ZEN)
S_C	0.13	0.25	0.32	0.45	0.56	1.4(ZEN)
S_D	0.18	0.32	0.40	0.54	0.64	1.6(ZEN)
S_E	0.26	0.50	0.64	0.84	0.96	2.4(ZEN)
S_F	Site-specific geotechnical investigation required to determine C_V					

Some progressive step of plastic hinge formation

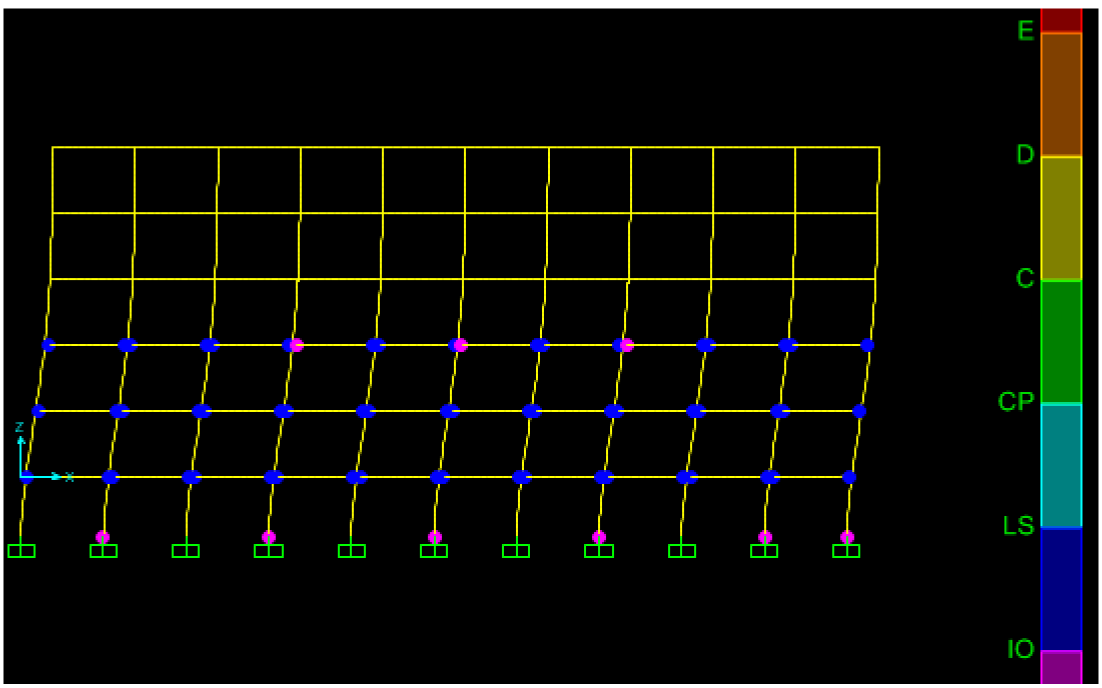
X direction



Step 2

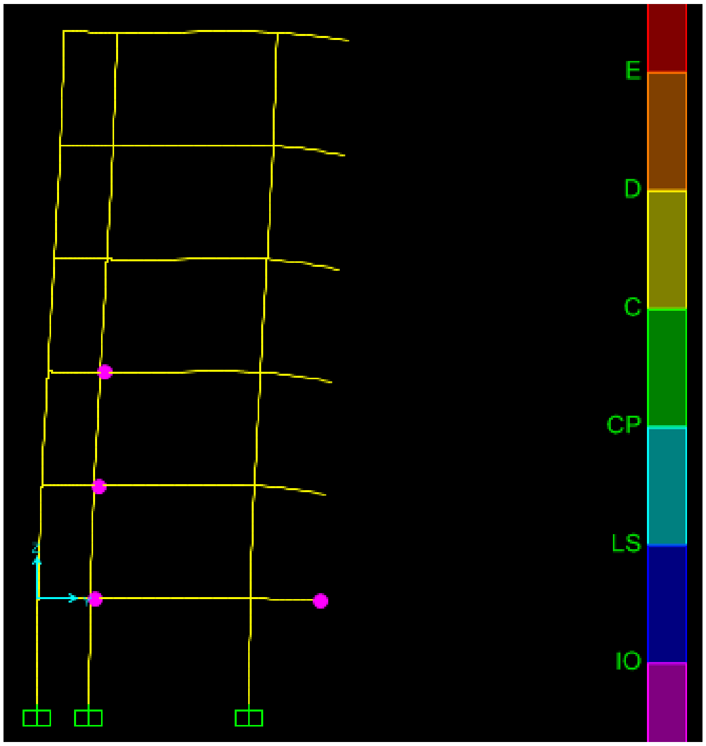


Step 4



Step 6

Y direction



Step 3

Step 4

At step 6