

**THE STUDY OF BUILDING COLLAPSE USING
SAINT THOMAS'S CHURCH HALL, AKURE AS A
CASE STUDY**

BY

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CVE/00/7339**

**A THESIS SUMMITTED TO THE SCHOOL OF POSTGRADUATES STUDIES
FOR THE AWARD OF MASTER OF ENGINEERING DEGREE IN CIVIL
ENGINEERING (STRUCTURAL ENGINEERING OPTION)**

IN



**THE DEPARTMENT OF CIVIL ENGINEERING, SCHOOL OF
ENGINEERING AND ENGINEERING TECHNOLOGY, FEDERAL
UNIVERSITY OF TECHNOLOGY AKURE, NIGERIA**

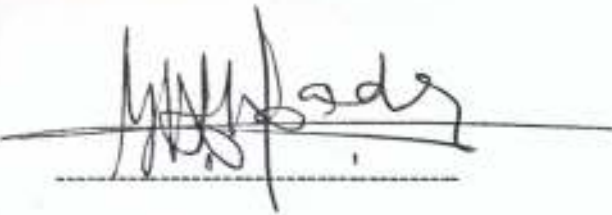
JUNE, 2005

CERTIFICATION

I certify that this project was carried out under my supervision by FAKERE, Rufus Adelanke (CVE/00/7339) of the Department of Civil Engineering, School of Engineering and Engineering Technology, Federal University of Technology, Akure, Ondo State, Nigeria.



24th June, 2005


DR. G.A. ALADE

Supervisor.

Date

DEDICATION

This project is dedicated to my brothers:

Pastor Bayo Fakere

And

Apostle (Dr.) Joseph Oluwaokere.



ABSTRACT

In the last few years, there have been incessant reports of the collapse of buildings resulting in the loss of lives and properties in Nigeria. There has, however, been a dearth of information regarding any findings about the collapse of the building structures.

An extensive study of causes of selected building collapse in Nigeria and abroad is carried out in the work. Questionnaires were used in the investigation. Also used were oral interviews, site inspections, appraisal of the structural drawings and laboratory tests on materials collected from the collapsed building sites.

The results show that over-reinforcement and over loading contributed to the collapse of the buildings. The average concrete crushing strength of these materials was measured as 4.9N/mm^2 , which is much lower than the code required minimum strength of 21N/mm^2 for the assumed 1:2: 4 mix proportion. In addition, the coarse aggregate used in the construction contained impurities like biotite and muscovite, which are easy oxidizing agents capable of accelerating building collapse. However, the bearing capacity of the soil measured between $140\text{kN/m}^2 - 260\text{kN/m}^2$ indicating that the soil is adequate to support the building foundations.

The followings are also identified as some of the causes of building collapse: the use of inferior building materials, inadequate planning and design, poor management and workmanship, improper supervision during construction, occurrence of natural disasters such as earthquakes and improper use of explosives. Based on the results obtained through this study as well as the recommendations on them, the further occurrence of building collapse will be greatly reduced.

ACKNOWLEDGEMENT

I am particularly thrilled to extend my appreciation to my supervisor, Dr. G.A. Alade, for taking the courage to supervise this work and always sparing the time for discussion even during his leisure hours. To be candid, his tremendous effort, contribution and unflinching support helped so much towards the completion of this project. Without exaggeration, his fatherly advice during the period had made this thesis a remarkable success. Once again, I am very grateful to him.

I also thank Dr. C. Arum, Dr. A.M. Oguntuase, Dr. J.O. Babatola, and Engr. Aderinlewo who are lecturers in the Civil Engineering Department, Federal University of Technology, Akure, for their assistance in this project. In addition, I extend my gratitude to Pastor Bayo Fakere, my brother, and Engr. L. A Akinyemi for the encouragement and inspiration given to me so as to make this work a reality. More grease to their elbows.

I also gratefully acknowledge those whose names could not be mentioned here who really prayed for me during the course.

However, except the Lord builds the house, all those that build it labour in vain. Therefore, I thank the Almighty God, who was always by my side throughout the time of doing these works, for His protection, guidance and making me to be in good condition of health during the period of working on this thesis. So, thanks be to God.



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CHAPTER ONE

INTRODUCTION

1.1 Preamble

A structure is a whole building, complex, framework or essential part of a building. Marshall and Nelson (1981) defined structure as a body capable of resisting applied loads without any appreciable deformation of one part relative to another. In a simpler form, a structure is that which carries load and transfers the load from the point of load application to the point of load support. The structure of the building is therefore that part of the building construction which gives the construction sufficient strength to withstand the load to which the whole building is subjected. A building structure does this by carrying the load imposed on it and transferring them safely to the foundations and hence into the ground. There are two broad sub-divisions of the structure. The first type is the framed structures, which resist applied loads by virtue of their geometry. The second type is the mass structures, which resist applied loads by virtue of their weight. According to MacGinley (1998) buildings are utilized primarily for living, working and storage. He further categorized them into three broad types. The first is the monumental structures which comprise the churches, city halls and sports arena. The second is the institutional structures represented by the more usual kind, such as block of flats, tertiary institutional buildings for academic and administrative purposes. The third

group comprises the industrial structures represented by the ordinary small scale industrial types.

1.2 Building Loads

Mosley and Bungey (1989) remarked that the loads a building has to withstand are classified as follows:

- (i) **Dead Loads:** Those whose magnitude and position do not change with time. Examples are the weights of the structural elements, plumbing fixtures, air-conditioning fixtures and other forms of mechanical and electrical equipment.
- (ii) **Live Loads:** Those whose magnitude and position do not remain constant. Examples are the weights of people, movable objects within a building and temporary partitions.
- (iii) **Lateral loads:** These include wind loads and seismic forces. They are dynamic loads.

1.3 Basic Requirements of Buildings:

Ogunsemi (2002) disclosed the basic requirements that a building must satisfy. Each and every member of a structural system should be able to resist, without failure or collapse, the applied loads under the service conditions. In other words, it must possess adequate strength. This demands that the materials of the structure must be adequate to

resist the stresses generated by the loads. The shape and size of the structure must be adequate also.

The components of the structure should be able to resist deformation under loading conditions. Deformation implies a change in size or shape when a body is subjected to stress. This means that the component should possess adequate stiffness. Thus the stiffness of a beam or column is a measure of its resistance to bending or buckling. A material or structure that is very strong but lacking in stiffness will so much *deform that it will not be able to resist the applied loads.*

All the structural members of the building must be stable, otherwise the whole structure is assumed to be unstable. Structural stability is needed to maintain shape since it is the ability of a structure to retain under load, its original state of equilibrium. It can mean anything from resistance to a minor degree of movement to resistance to sliding, overturning, partial or complete collapse. Any phenomenon that can alter the load carrying behavior of a structure, if not properly taken care of can lead to instability, a condition in which the support reaction is less than applied load. Thus to ensure stability, loads must be balanced by reactions, and the moments due to loads must be balanced by the moments due to reactions.

Any building that cannot withstand the load applied to it will show signs of distress which may lead to failure and invariably total collapse.



The possibility of building collapse should not be underestimated. Its occurrence is usually accompanied by loss of properties and lives. A building may collapse when one or more of the essential components of the structure fails. When the building collapses, the professional bodies such as Council for the Regulation of Engineering in Nigeria (COREN), The Nigerian Society of Engineers (NSE), Builders Associates and even Governments usually set-up panels of enquiry to determine the immediate and remote causes of such failure and if possible recommend sanctions against those culpable.

The increase in the number of collapsed buildings in Nigeria has been a serious concern to professionals, like Builders, Architects and Structural Engineers. Building collapses have so often been associated with structural failures. Therefore, structural failures, no doubt, are very dangerous and should be avoided using all necessary precautions and machineries possible. Ultimately, the effect of collapse of a building structure, particularly at its completion stage and when it is occupied by users is devastating on human lives. Even when lives are not lost, much financial investment is wasted. Once the specifications of the building including its materials and components are not complied with during construction, the result is building failure.

1.4 Factors Responsible for Building Collapse:

Building failure can be attributed to many factors. Many buildings in Nigeria have collapsed due to some of the following reasons.

- (i) **Inadequate preliminary works:** Preliminary works are operations which include site investigation, and foundation. Building collapse is imminent where these operations are carried out shoddily.

Site investigation is to determine the properties of the soil strata. Seeley (1987) said that all potential building sites would need to be investigated to determine their suitability for buildings and the nature and extent of the preliminary work that would be needed. Particular attention should be given to the nature of the soil and its probable load-bearing capacities, as there may be variations over the site. The past history of the site should be investigated with particular reference to the former existence of trees, water level, borehole log, underneath soil strata and waste dumps. A careful study should be made of adjacent structure to ascertain whether failure can result due to localized conditions. According to Bell (1987), soil is an unconsolidated assemblage of soil particles between which are voids. These voids may contain water, air or both. Soil is derived from the breakdown of rock materials by weathering and erosion and may have suffered some amount of transportation prior to deposition. Neville and Chatterton (1978) concluded that the development of soil mechanics which relates to the understanding of the physical properties of any particular soil type in relation to loads was really the main stepping stone towards a scientific approach to foundation problem and construction.

However strong, rigid or structurally stable a building may be, its satisfactory performance depends exclusively upon the ground which supports it.

(ii) **Adoption of wrong foundation:** Lambe and Whitman (1979)

defined foundation as the part of the structure in direct contact with the ground and which transmits the load of the structure to the ground which plays an important role in the construction of building structures. Foundation is expected to carry all the dead, super imposed and wind loads from a building to the soil on which the building rests in such a way that settlement of the structure is limited, so that failure of the underlying soil is prevented. The depth of soil strata in response to the loadings from the structure has to be located properly in order to safely bear the foundation of building. Otherwise, the structure will fail.

- (iii) **Poor Concrete Mix Ratio:** Usually Concrete is a mixture of cement, sand, gravel and water in definite proportions. In providing support for a building, concrete is the most common material used. Tomlinson (1980) reiterated that poor material do not make good concrete. The cement, sand and stone must all be sound and have the types and qualities specified. The result of poor concrete works is building collapse. The steel reinforcements are embedded in concrete, so that compressive stresses are taken by concrete, while tensile stresses are catered for by steel reinforcements. For

this purpose to be fulfilled, steel rods must be bent in accordance to design. Otherwise, it will fail and cause collapse.

- (iv) **Improper walling:** The wall is a very important part of the building that also provides support. Other function is to enclose or divide space. A wall that will adequately provide support is a load-bearing wall which must provide adequate strength and stability, weather resistance and durability. The commonest walling material in Nigeria now is sandcrete blocks of various sizes. As a matter of fact, walls which provide support to buildings must be straight, perpendicular and produced of sound materials. The appearance of a crack line in a building is a sign of failure. Failure in block laying may lead to eventual collapse of the building.
- (v) **Lack of approved structural design:** According to Ataev (1985), the basic requirement of any structural component of a building is that it should be strong enough to carry and support all possible types of loads to which it is likely to be subjected. Therefore, building design is not just the Architectural design, it also includes structural, electrical and mechanical engineering. A building that is poorly designed structurally will eventually collapse. The final object of structural analysis is to enable the Engineer design and construct a building structure, which is satisfactory in service, and that such design must be approved by the approving body. This means that it

must not collapse when loads are applied and the deformation must not be excessive.

(vi) **Poor Building Material Specification:** The uses of poor building material specifications have been possible root causes of collapse. In buildings, the materials that are essentially used on construction sites are cement, sand, gravel, granite chipping, timber, iron rods and sandcrete blocks. Other materials are aluminum, glass and ceramics. Good building constructions are enhanced by materials of good quality. Proper handling and storage must be given to building materials. Material specifications must relate exactly to the intended construction and must be of adequate standard. Specifications are to prescribe what materials should be used and where there is a deviation, failure, that is, building collapse should be expected.

(vii) **Ineffective Supervision:** Averting building collapse depends largely on effective supervision of works. Hence, improper supervision will lead to the collapse of the building structures. Supervision involves the intricate knowledge of workmanship and materials while inspection is only to ensure adherence to contract documents especially the drawings. The object of supervision is primarily to ensure that employer's requirements as expressed in the contract documents are correctly interpreted and the problems which are bound to arise are satisfactorily resolved.

(viii) **Climate:** Apart from failures arising from negligence or negligent behaviour, many of our designed buildings have failed due to the persistent incidence of weather. Ogunsemi (2002) remarked that a good building is not that which merely fulfils the purpose for which it is designed and erected, but a building comely and able to withstand the onslaught of weather conditions.

Collapse of Saint Thomas's Church Hall:

The collapse of the Saint Thomas's Church Hall, 2 – storey building, Arakale, Akure occurred on the 30th September in 1998. This excruciating event which wounded and killed so many people drew the attention of some Government functionaries, Engineers and other professionals from within and outside the state to sympathize with the Government of Ondo State and members of the Saint Thomas's Church, Arakale, Akure. The tragedy generated so many palpitating questions in the Akure metropolis on who should be blamed and held responsible for the collapse of the building. Are they the designers, builders, supervisors and the client? Was the contract mismanaged? In fact, nobody has really diagnosed what went wrong with the building except few allegations that there were some structural defects that were not taken into consideration by the construction Engineers. According to Abidakun (1998), the collapse was an unusual and unprecedented happening.

The collapse of the Saint Thomas's Church Hall is no doubt an unfortunate one. It is supposed to serve as a lesson to our building

Engineers that negligence on their part could cause a monumental loss to both human and material resources.

This project is intended to find out the real causes of the collapse and hence serve as a reference document to Engineers, Designers and builders in building profession. The recommendations will also help to reduce the rate and severity of building collapse in Nigeria.

1.6 Objectives

The objectives of this research are as enumerated below:

- (i) To determine the general causes of some buildings' collapse and specifically the reason for the collapse of Saint Thomas's 2-storey Church Hall, Akure.
- (ii) To determine the adequacy of the foundation soil and construction aggregate materials through various laboratory tests.
- (iii) To evaluate the extent of loss in terms of life and property.
- (iv) To make some suggestions on safe delivery of structure particularly building projects.

CHAPTER TWO

LITERATURE REVIEW

2.1 Preamble

Tunstall (2000) defined building as any structure for whatsoever purpose and of whatsoever materials constructed and every part thereof whether used as human habitation or not. It includes foundations, plinth, walls, floors, roofs, chimneys, plumbing and building services, fixed platforms, verandah, balcony, cornice or projection, part of a building or anything affixed thereto or any wall enclosing or intended to enclose any land or space and signs and outdoor display structures. Powell and Billington (1984) defined building as any permanent or temporary structure and, unless the context otherwise requires, it includes any other erection of whatever kind or nature. On the other hand, buildings are enclosures for spaces designed for specific uses meant to control local climate, distribute services and evacuate wastes. They exist to meet the primary physical human needs of shelter. They are seen as structural entities capable of securing self and applied weights on to the ground.

Past records have shown that the early man's methods of constructing shelters are quite different from those of the modern technological era. Izoma (2001) admitted that shelter represents one of the most basic human needs and has no doubt a profound impact on the health, welfare and productivity of the individual. In spite of its importance, there is unfortunately a universal shortage of desperately needed dwelling units. Punmia (1987) submitted that the early men were using mud blocks, bamboos and palm trees and that the first hut with bamboos and leaves can be taken as the first civil engineering construction carried out to satisfy the needs for a shelter. Before that, caves were his early abode.

The methods used by the early men to build their house could be criticized to some extent. Though the period is referred to as the dark ages, when modern technological inventions and discoveries were yet to be made. A house built with mud block, as a matter of fact, without

good concrete foundation is not durable. Also, during that period cement had not been in existence. The moment there are cracks generated on the walls as a result of differential settlement could easily lead to endangering the lives of the users. In the event of serious rain storm and wind, the house built with mud, bamboo and palm trees will not stand the tests of time. Such houses are not conducive for living. Any time there is an outbreak of fire, these houses could be easily burnt, destroying lives and properties. However, the early man's methods of building their houses were dangerous and risky to lives and properties.

The development of housing facilities reveals that man has been moulding his environment throughout the ages, for more comfortable living. India for example still has many old cave temples with halls and rooms having beautiful carvings. Egyptians constructed huge pyramids. The Greeks developed a style of proportions of building elements. Romans developed arches for vaults and domes, and used pozzolana sand, mortar plaster and concrete. The emancipation of science and technology has brought about increase in knowledge, which also contributes to the development of a country. Nigeria is still a developing country, which always imports most of the construction materials both superior and inferior from the neighboring countries. Nigeria experiences collapses of buildings like other countries.

Syal and Goel (1993) disclosed that most of the structures in the past were made either in masonry, steel or timber depending upon the availability of the materials and the nature of the structure. Very recently, concrete, as building material, has come into existence. In a short period, concrete has gained so much importance that today more than 65 percent of the structures coming up in the world are constructed with concrete. During the Gothic period of Architecture (1100-1500 A.D.) churches with pointed arches ribs supporting masonry vaults were constructed. The arched ribs were supported by stone pillars strengthened by buttresses. These structures led to the idea of framed structures. The great pyramid is one of the most extraordinary works of Architecture ever executed. Approximately, two-

thirds the size of Hoover Dam, it stood for more than four millennia as the largest man-made structure on earth, rising to a height of 137.4m from the base that was carved from the solid bedrock of the Giza plateau that covers 5.2 hectares and constructed of an estimated 2 million blocks of limestone and granite.

The period from 1750 A.D onward is known as the period of modern Architecture and structural Engineering. Due to economic pressure after the war, and due to industrial development, many new methods and materials of construction were developed. The use of reinforced concrete construction triggered the rapid development of modern structural Engineering. Adedeji (2001) stated that Ferro-cement has been used in house construction mainly for roofing because of its relatively low cost, durability and weather resistance. Because it is very easily shaped into curved forms, it has been used in domed roofs, hyperbolic paraboloids, vaults, and other extruded type shapes which might be too expensive with conventional construction methods because of prohibitive cost of form work. Precast roofing with ferro cement was used in the Universidal Autonoma Metropolitana in Mexico to solve the serious dwelling problem affecting large numbers of people in the area. Ferro cement is simply a combination of ferrous material with cement sand mortar, resulting in a very thin, stiff and homogenous structural form. It can be considered as a very special form of reinforced concrete because the ferrous material consists of very small diameter reinforcing steel wire mesh, which are closely bound together, and uniformly distributed in the cement sand mortar matrix. Functional structural components such as columns, beams, canopies, and reinforced concrete construction slab became increasingly popular because of the increasing speed in construction.

The idea of using concrete in the building construction is a welcome one and right step in the right direction. This is because concrete is very strong in compression though weak in tension. So, steel reinforcement is used to take up tensile stresses at places where section is subjected to tensile stresses. Such concrete is known as reinforced concrete. Cement concrete shrinks a little and grips very fast

on the steel reinforcement. There is bond between concrete and steel, and tensile stresses are transferred to steel reinforcement. Steel is used as reinforcement to take up tensile stresses in reinforced concrete structures because of its high tensile strength, good bond with concrete, coefficient of expansion, which is nearly same as for concrete and free availability.

However, failure of structural members and foundation of buildings which normally lead to collapse in most cases have begun since the inception of technology materials. The news of collapse of buildings has always been on the increase in the Daily News papers not only in Nigeria but also in the world as a whole. Records have shown recently that multi-storey buildings collapsed in Lagos, Ibadan, Kano, Kaduna, Akure and many other towns in Nigeria where so many people lost their lives. These have also happened in various parts of the world.

2.2 Earlier conditions of the collapsed building site.

The site of the collapsed building as reliably gathered from the reports of the questionnaire returned is a built up area. A built up ground is a refuse, excavated rock and soil deposited for the purpose of filling a deep depression, valley and gorge or for raising the site above its natural level. The site terrain was initially undulating and sloppy. Part of the site was originally a swampy area but later reclaimed and developed. A building was first constructed on the site using mud block in the early 1960 which was used as classrooms by Saint Thomas's Primary School and Sunday School by the Saint Thomas's Church. The partitions in the building were later removed to convert the whole building to a place of worship. The building was later demolished and the place was left open for many years before the plan to build this collapsed church hall came up.

It was also gathered that part of the site was used as a graveyard. Abidakun (1998) reported that "Among the notable dignitaries who have visited the place since the incident happened was Dr. Akintola Aguda who disclosed that he is a member of the church

and that his father was buried in the church cemetery". Today, there is no indication to prove that the site had been used for a cemetery.

The 2-storey framed structure collapsed after the casting of the 2nd floor slab on 30th of September 1998. There were some signs noticed prior to the collapse. Concreting of the 2nd floor slab had already been completed that day when suddenly the pillars at the ground floor started making sounds of strain, particles of concrete started flying out from the body of the ground floor columns. The columns started wangling and twisting. Also, buckling of the steel at the base, crushing of materials and concrete cover giving way exposing the column reinforcement which are caused by overloading were noticed. Eventually, sound of explosion perhaps, after reaching the yield point occurred.

According to some respondents to the questionnaire, the notice, failure and collapse were sudden under 60 seconds. Out of about 120 people who were working in the building at the time of the collapse, at least 5 people died and many people were seriously wounded. But most of the workers had already finished and were about to go home when the incident occurred. That was why most of the workers escaped from being trapped. According to Abidakun (1998), it was by stroke of luck that some people came out of the rubble alive.

The architectural and structural designs of the collapsed building were done by FUTA ventures Limited at the Federal University of Technology, Akure. The building was designed for a single storey building. During the construction, the number of floors was increased to two to become a two-storey building without any corresponding modification to the structural design.

The project was not contracted out to any contractor. It was done by a direct labour following the directive of the Church Parish Council. Also, there was no Bill of Quantities prepared but the contract sum was estimated to be about five million Naira (N5m) in 1995. The method of supervision was adhoc type, taking up by the engineers in the Church Development Building Committee. According to the report collected from the questionnaires' distributed, full time bricklayers,

carpenters, electricians and iron benders were employed to work in the construction of the building. Since there was no consultancy services for the supervision, site meetings were not held regularly during the construction so as to attend to practical problems on site. There were also no regular daily, weekly and monthly reports written to determine the periodic progress of works. In fact, no soil investigation tests were located out to determine the bearing capacity of the soil and the type of foundation required for the structure. Consequently, the masonry works started from very shallow foundations. Strip shallow foundation of about one metre depth was used for the construction. There were no laboratory tests for quality control to determine the cube strength of the concrete materials.

However, information collected from the questionnaire's report about causes of the collapsed building were attributed to foundation problem, bad quality of construction materials, column failure, poor workmanship and lack of thorough supervision.

2.3 CAUSES OF BUILDING COLLAPSE.

Building collapse has been a source of concern to so many people particularly those associated with building industry. This is because there are so many cases of building collapse all over the world and particularly Nigeria. Most of these cases had resulted into colossal economic losses in terms of lives and property. Building collapse are some of the cardinal issues, which have created serious concern to all the professionals like Structural Engineers, Architects and Builders. The Government too is worried on frequency of collapse of buildings in this country. However, Aderibigbe (2001) admitted that the recurring event of building collapse has forced some state Governments to enforce and enact some laws recommending forfeiture of such buildings and prosecution of their owners.

2.3.1 SOME SELECTED REPORTED INCIDENTS OF BUILDING COLLAPSE IN THE PAST (1977-2003)

Nigeria has witnessed in the recent times a spate of collapse of buildings in some cities. When buildings collapse, they create deep

emotional reactions from every segment of the population. Consequently, very rigorous attempts have been made by various professionals connected with building construction to identify the causes of the collapse of buildings.

Simire (2001) disclosed the collapse of a three-storey, Mosque, building where so many people died as a result of a heavy rain downpour. The structure was located at Buhari Street, Mushin, Lagos and collapsed on 20th April, 2001. It was gathered that the structure was formerly a bungalow and that the addition of one floor was too much for the foundation which was not initially designed to carry such a load.

Aderibigbe (2001) reported the collapse of a four storey structure located at Akewusola Street, off Oduduwa street, Oworosonki, in the Kosofe Local Government Area of Lagos State. The incident claimed at least one life with three others seriously wounded. According to the report, the structure was initially built as a bungalow without an approved building plan. Information revealed that the addition of more floors led to the collapse before the completion. Also it was gathered that the landlord of the house was the Architect, the builder and the supervisor.

Table 1: Some Collapsed buildings in Nigeria. (1977 - 2003)

Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
School Building	Government Secondary School, Markafi Kaduna State	July, 1977	Lack of maintenance	7 people died
Residential Buildings	Bamawa Housing Estate, Kaduna, Kaduna State	August 1977	Faulty Design	28 people died
3 Residential Buildings	Bamawa Housing Estate, Kaduna	July 1980	Faulty Structural design	6 people died and 184 units pulled down

Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
One storey building	75, Ijoka Road Oluwatuyi Qtrs, Akure	1982	Under reinforcement, lack of qualified professional to supervise	No casualty
One storey building	12, Gbogi Street, Akure	August 1983	Fire disaster	2 died
4 – storey commercial building (LACCO)	Oba Adesida Road, Akure	October, 1983	Fire disaster	No casualty
Residential Building	Allen Avenue, Lagos	January, 1985	Excessive Loading	No casualty recorded
Residential Building	Adeniji Adele, Lagos	January, 1985	Excessive loading	2 died including owner
Residential Building	Ojuelegba, Lagos	May 18, 1985	Rain storm	No casualty
Uncompleted 4-storey building	Iponri, Lagos	May 20, 1985	Structural failure	13 people reported dead
Residential Building	Victoria Island, Lagos	July 18, 1985	Excessive loading	13 people reported dead
One-storey building	14, Ayegunle street off Arakale, Akure	1985	Foundation problem	No casualty, occupant already abandoned the building
Residential building	Gboko, Benue State	September, 1985	Lack of maintenance	1 person died
High Court Building	Isiala, Imo State	February, 1986	Collapsed ceiling	No casualty
Mosque building	Osogbo, Osun State	May, 1986	Structural Failure	No death recorded
2-storey building under construction	Agege, Lagos	May 9, 1987	No structural design	2 people died including owner

Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
Residential building	Idusagbe lane, Idumota Lagos	September 14, 1987	No structural design	17 people died, 12 persons injured
Commerical Building	Ikorodu Road, Lagos	September 29, 1987	Storm	3 people died
Residential Building	Calabar, Cross River	October, 9 1987	Storm	3 people died.
School building	Diobu, Port Harcourt	April, 1990	No structural design and approval of town planning	Over 50 people reported dead
One-storey building	Shetima Avenue, Kano State	July, 1991	Lack of maintenance of roof leakage	No casualty.
Un-completed 2-storey building	Saint Thomas's Anglican Church, Arakale, Akure	30 th September, 1998	Failure of structural members	5 persons reported dead and many wounded
2- Storey Residential building	Road 3, plot 10 Funbi Fagun St. Abeokuta.	November, 1998	Use of poor Quality building materials	No death recorded.
Multi-storey building	Ojuelegba, Lagos	April 28, 1999	Use of poor quality material	3 people died
One storey (Mud building)	6, stadium road, General Hospital Ondo	May, 1999	Lack of reinforced structural members	No casualty
3-storey Residential Building	Salisu street, Iju Ishaga, Lagos	August 18 th , 1999	Rain storm	35 people died
2-storey building	Dawodu street, Ifo, Ogun State	October 16, 1999	Rain storm	20 people died
One-storey building	4 Oke-Bola st , Ado Ekiti	1999	Fire disaster and dereliction of the building	No casualty
One-storey building	Bishop's Court Diocese, Ife Road, Ondo	1999	Under-reinforcement, poor workmanship, improper supervision	No casualty

S/N	Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
30.	One-storey building	No. 23 Odi Olowo, Akure	1999	No site investigation and foundation problem	Sank and collapsed but no casualty
31.	Luxury flat	Ajah, (Eleganza), Lagos	April 2000	Faulty/ improper supervision	2 people reported dead
32.	One-storey building	Oke-Bola st., Ado Ekiti	August 14 th , 2000	Poor quality control, rain storm	No casualty
33.	2-storey building mosque	21, Buhari street, Mushin, Lagos	18 April, 2001	Former bungalow converted to storey building	7 person reported dead
34.	2Nos, One-storey building	Owa Ale of Ikare, Ogbagi Street, Ikare	2001	Fire disaster	No casualty
35.	One-storey residential building	69, odo Ikoyi, Akure	September, 2001	Foundation problem	No casualty
36.	Residential building	Igborokanla Street, Owo	2001	Fire disaster	No casualty
38.	Many buildings	Odoso compound, Ikare	February, 26 2002	Fire disaster	No casualty
39.	Residential Building	Irele township Ondo State	May 2002	Rainfall and flooding	No casualty
40.	One storey building (Ogbo's House)	16, Oyemekun road, Akure	Nov. 22, 2002	Fire disaster	No casualty
41.	Methodist Primary School building	12 Methodist Church road, Gbogi, Akure	16 April, 2003	Rain storm	No casualty
42.	Church building porch deck	15, Ojuelegba, Akure	April 25, 2003	Poor workmanship and under-reinforcement	No casualty

S/N	Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
43	One Storey and bungalow buildings	Nos. 8 & 10 Oshinle road, Akure	1995 to 2003	Poor foundation	Sank but no casualty
44	2-storey and 2-bungalow buildings	No 30, 32 and 34 Odi Olowo, Akure	1995 to 2003	Foundation problem	Sank but no casualty
45	One-storey building	52, Odojoka street, stadium road, Akure	Feb. 2003	No structural members	No casualty
46	Portal framed structure	Onyearugbulem market, Akure	May 2003	Poor workmanship and under-reinforcement of the cantilevering end	No casualty

Sources:

1. Ogunsemi (2002)
2. Nigeria Daily News papers (1977 – 2003)
3. Personal Investigation

Table 2: Some Collapsed buildings in Abroad (1978 – 2003)

S/N	Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
	Residential buildings	Saravvero Yugoslavia	November, 1978	Earthquakes	1,160 people died
	Hotel Continental	Mexico City, Mexico	15 th September, 1985	Earthquakes	Many died
	12 Storey central communication center	Mexico City	15 th September, 1985	Earthquakes	Many died
	Commercial building	Mexico City	15 th September, 1985	Structural failure as a result of Earthquakes	Many died
	Top floors collapse of office building complex	Mexico City	1985	Earthquakes	Many died

S/N	Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
6.	Mid-floor failure of Hotel de Carlo	Mexico city	1985	Earthquakes	Many died
7.	Multi-floor parking Garage	"	"	"	"
8.	Bottom level failure of 7 – storey office building	"	"	Weak first floor	"
9.	6-storey Residential building	"	"	Poor Foundation	"
10	School building	"	"	Lack of adequate stiffening in shear wall	No death because it occurred before the school hour
11.	Collapsed Floors punctured by load bearing column	"	"	Severe resonance oscillations of the building	Many died
12.	Juarez Hospital Framed structure	Mexico city	1985	Localized failures at the beam-to-beam joints of each floor	400 medical personnel and patients were trapped
13.	21-storey steel framed office building	"	"	Earthquakes	Many died
14.	Buildings collapse	Middle East Israel	January, 1991	Missile and Bomb attack	Many died due to building hit by Saddam Hussein's sent missiles
5.	A luxury apartment block	Alexandria	1992	Structural failure	20 people died
6	Two separate buildings	Egypt	1992	Structural failure	30 people died

S/N	Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
17	Buildings collapse	Western Asia, Turkey	February, 1992	Avalanches, caused by snowfalls in the western Asia	More than 100 people died
18	Buildings collapse	Turkey	March, 1992	Twin Earthquakes	Over 4,000 people died
19	Café and coffee house and many others	Jerusalem, Israel	March, 1992	Mudslides caused by some of the same unusual weather that caused blizzard and flood in the middle east	Over 20 people were killed
20	Israel Embassy	Argentina	March 1992	Car Bomb donated by Islamic Jihad	More than 27 people died
21	A Tower and many office buildings	Sanfernado valley, Northern Los Angeles	17 th January, 1994	Earthquakes	57 died and 1,500 injured
22	A shopping center	Seoul, South Korea	1995	Structural failure	More than 500 people died
23	Apartment block	Italy	1998	Structural failure	27 people died
24	Three-storey wood frames	New York	1998	Fire outbreak	2 person died
25	A duplex, 3-decker type of wooden frame	Blue Hill Avenue Dorchester field section Boston	26 th August 1998	Outbreak of fire as a result of Gas explosion	15 civilians and 3 fire fighters injured
26	Apartment block	Foggia, Rome	December, 1998	Structural failure	17 people died
27	Six-storey block	Southern Italy, Europe	11 th Nov 1999	Structural failure	34 people died many wounded
28	World Trade Centre	Washington DC, America	11 th September	Terrorist attack	Over 1000 people dead

S/N	Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
29.	Many tall buildings	Paraguay	9 th Nov. 2001	Earthquakes	125 people dead
30.	Many buildings	Brazil	16 th February, 2002	Seismic forces due to Earthquakes	Over 1000 people dead
31.	Buildings collapse	St. Petersburg, Russia	5 th June, 2002	Foundation problem	3 people died and 1 girl missing
32.	School building	Turkey	18 th April, 2003	Earthquakes	Over 200 students died
33.	3-storey faculty building including many other buildings	Rouiba University, Northern Algeria	21 st May 2003	Earthquakes	707 people died 4,600 injured
34.	Library Building	Rouiba, Algeria	21 st May 2003	Earthquakes	Many students died
35.	10-storey tower	Regbaia, Algeria	"	"	Many died
36.	Many Structural building	Boumerdes, Algeria	"	"	Thousands feared dead
37.	Partially built supermarket	Southern city of Jiangmen, China	8 th October, 2003	Structural problem	More than 10 people died, 5 people missing, 5 people injured
38.	25 feet Tower	Toccoa	24 th October, 2003	Structural problem	12 people wounded
39.	10- storey Atlantic City Garage	Atlantic City, South Jersey, America	30 th October, 2003	Cracks developed in the concrete floor/columns	4 people died and 21 persons injured

S/N	Type of Building Structure	Location of Building	Failure/ Collapse Date	Suspected Causes	Remarks
40.	Butchery Mill	200, Main street, Somersworth	6 th Nov. 2003	Fire outbreak	More than a dozen families became homeless
41.	Women's Day Stage roof	South Africa	7 th November, 2003	Gust of Wind	A woman paralyzed
42.	A 3-storey church under Refurbishment	1713 Arctic Avenue, Atlantic City, U.S	9 th November, 2003	A steel support brace gave out	No death recorded

Sources:

1. Internet Browsing
2. Nigeria Daily News papers (1999-2003)

2.3.2 Some specific reasons for the collapse of building structures

(i). Poor Site Investigation:

This is the case when construction is carried out with insufficient or non-existent geotechnical studies. Inadequate soil investigation before embarking on a new building work may result in structural failure and eventual collapse. Ogunsemi (2002) remarked that sulphate in clay soil is deleterious to buried concrete, iron or steel. High water table which is seasonal produces a lower soil bearing capacity than when the soil is dry. However, before the commencement of any building structure, the first and most crucial aspect of the project is to carry out the soil investigation so as to know the nature, type and properties of the soil that is expected to carry the structure. Also, the site investigation must be carried out not by a novice who doesn't know anything but by Engineer and Geologist who are to advise on the type of structure which the site could carry.

Maurice and Clive (1986) concluded that the soil type will influence its bearing capacity, it will determine the amount of excavation necessary to achieve a reasonable bearing capacity, how

much this changes with moisture content the thickness of load spreading sub-base needed, the thickness of concrete foundations, how much heave or lift with frost penetration. Therefore, thorough investigation is essential. It would be unwise to assume either uniformity of soil or knowledge of soil type. Exploratory pits at least one metre deep should be dug. It is better to dig them on the perimeter of the proposed building area but some should be taken out within the building if retaining walls or column foundations are involved. The purpose of this is to ensure that site investigation is properly and satisfactorily done.

The example of this occurred at Odi Olowo Street in Akure, Ondo State where a storey building that was erected on a swampy and water logged area sank and collapsed. This was as a result of site investigation which was not carried out before the construction of the building. The site investigation, if carried out would have helped the builder to know the type of foundation and some construction parameters to be carried out before a building could stay in such a location.

ii. Improper supervision of project during construction.

Improper supervision leads to the collapse of some building structures in this country. Akeju (1984) admitted that the involvement of competent professionals to handle the planning and design of a project does not entirely guarantee its stability. This is because some incompetent and unprofessional site supervisors are being engaged in the construction of the project who could not even interpret drawing details correctly. The project must be faithfully and accurately reproduced on the site. Every stage of the work must be supervised by appropriate qualified professionals. For example a four storey structure located at Akewusola Street off Oduduwa Street Oworonsoki, Lagos, collapsed and claimed several lives due to lack of proper supervision. Aderibigbe (2001) reported that the evidence obtained from neighbours in the next buildings revealed that the landlord was the Architect, builder and the supervisor because the job was done by a direct labour. Also, the luxury flat that collapsed at Ajah, Eleganza, Lagos on

April 2000 that claimed two lives was as a result of improper supervision. In addition, the one storey building that collapsed at Ijoka Road in Akure in the year 1982 was as a result of improper supervision.

iii. Foundation problem.

Some buildings constructed on improper foundation have collapsed in various part of the country. Walton (1987) reiterated that the type of foundation to be used for a building depends on the structural form of the building, the magnitude of the load, the nature of the soil and its capacity to support the load.

In some cases, wrong foundation is being used for a particular building construction due to inexperience of the designer and builder. Ogunsemi (2002) remarked that foundation failure may be due to any or combination of

- (a) Absence of a proper investigation of the site or wrong interpretation of the results of such investigation.
- (b) Faulty design of foundation
- (c) Bad workmanship in the construction of foundation
- (d) Poor construction materials during the construction of the foundation due to financial constraints.

According to Gaylord et. al. (1970) the subsidence caused by the removal of underground oil, water, salt, coal, limestone and other minerals may cause settlement of structural foundation.

The collapse of a one-storey residential building from personal investigation at Odo Ikoyi Akure in September, 2001 was as a result of foundation problem. The collapse of a 2-storey building and one storey building located at Odi Olowo in Akure between 1995 and 1999 was as a result of foundation problem and swampy soil which were not properly investigated before the buildings were constructed.

iv. Inadequate / lack of structural design.

This is when incompetent personnel is in charge of the structural design. The Architectural design must pass through the Architect for endorsement before final approval is given if the building is just a

bungalow. If the building is a one-storey and above, Architectural design must be accompanied with the structural drawings duly endorsed by a COREN registered Structural Engineer before final approval can be granted. It is improper for an Architect after completing his own aspect of the design to also take up the structural design. Also, some Draughtsmen are in the habit of taking up a structural design. Some building projects too are being constructed without any structural design at all. These could lead to failure and collapse of a building structure. George and Arthur (1968) remarked that the final goal of the Structural Engineer is to design structures safely, economically, and efficiently. Hence, with this basic understanding as a firm foundation, familiarity with current design procedures and skill in employing them is of the importance. Ambrose (1979) emphasized that designing building structures involves the consideration of a wide range of factors. Building structural designers, must not only understand structural behaviour and how to provide for it adequately, but must also be knowledgeable about building construction materials and processes, building codes and standards, and the economics of building must all be considered in the design stage.

The collapse of a 2-storey church building belonging to St. Thomas's Church, Arakale, Akure was as a result of over reinforcement and over loading of the structural members.

The collapsed three residential buildings at Barmawa housing Estate, Kaduna in July 1980 and the loss of lives of 6 people including the pulling down of 184 units were as a result of faulty structural design.

Also, the collapse of a School building at Diobu Port-Harcourt by April, 1990, that claimed over 50 lives, Residential building at Idusagbe lane Idumota Lagos that occurred in September 14th 1987, which claimed 17 lives including 12 persons injured and a 2-storey building under construction at Agege Lagos by 9th May, 1987 that claimed 2 lives including the owner were all as a result of lack of structural design. This shows that inadequate and lack of structural designs are some of the causes of buildings collapse in Nigeria.

v. **Poor workmanship**

Some client and contractor are fond of employing unqualified staff to handle the materials during construction. This contributes to poor workmanship. Some professional bricklayers if they are not supervised by qualified professional do not even know how to interpret the concrete mix specifications in the contract drawings. Eventually, they mix what they like to produce weak structural members which could not stand the test of time.

Simire (2001) admitted that ignorance and poverty are the main contributory factors why people are not using competent hands to execute their building projects. Besides, Government agencies do not have enough personnel as well as the funding to be able to monitor the numerous construction works going on at a particular time. Some workers in the construction company steal away the construction materials like cement and reinforcement. The result is shoddy and poor quality work. Some contractors too in an attempt to maximize their gain reduce the quality of work expected. However, poor workmanship makes it difficult or impossible for workers to perceive and apply the concepts of quality control.

Ogunsemi (2002) inferred that the basic mechanism for deterioration of reinforced concrete could result from insufficient cover to reinforcement or unsatisfactory quality of reinforcing material. All these are poor workmanship. Sometimes, formworks were not adequately braced to provide the designed strength to the structural member. Also, inadequate compaction of concrete and faulty sequence of operation by the contractors can lead to collapse of the building. An Engineer having designed his building put the production in care of an inexperienced person on the site. The building collapsed because he did not cross check the activities of his men.

An example of this is the failure and collapse of the first floor slab of a two-storey building at Bishop's Court Diocese, Ondo, along Ife Road, Ondo, that occurred in 1999. The first floor slab of the two-storey building caved in and sagged some weeks after the casting because of under reinforcement and poor workmanship.

vi. Use of substandard and poor quality building materials

Sometimes, the owner in an attempt to save cost wants to build by himself using the so called direct labour method. The owner of the property purchased the materials by himself buying cheap and inferior building materials without any idea of what type of materials to be bought for a particular job. As a matter of fact, poor construction work could also occur through the use of substandard materials such as unwashed gravel, contractor not following instructions of the Engineer as per the aggregate ratio of cement and sand.

Waddell (1974) remarked that aggregate may contain mineral particles which in some exposure condition of the concrete undergo excessive volume changes causing rupture of the concrete surface, or they may create sufficient interior stress so as to cause cracking and impair the structural integrity of the concrete. Aderibigbe (2001) ascertained that statistics of collapse buildings, particularly in the Lagos metropolis, have gained a frightening dimension within the last decade, with a minimum of four buildings going down as a result of poor or inferior building components and questionable construction personnel always leading to loss of lives.

Examples of this are the collapse of a 2-storey residential building located at road 3 plot 10 Funbi Fagun Street Abeokunta Ogun State that occurred in November, 1998 and multi-storey building at Ojuelegba Lagos that occurred on April 28, 1999 which claimed the lives of three people, happened as a result of use of poor quality building materials. Also, the collapse of 2 storey residential building that occurred at Oke Bola, Ado Ekiti on 14th August, 2000 was due to the use of poor quality materials.

vii. Wrong conversion of building

This happens when alteration made to the structure are not taken into consideration during design. This usually occurs when there is an upward economic change in the value of the building location. A building which might have been designed and constructed as residential might be converted into a school or a supermarket requiring

large free space or sometimes, more floors may be added thereby leading to over-loading the existing foundation. Foundation and structural member's inadequacy are usually the cause of collapse in this regard. Ogunsemi (2002) gave an example of this as the collapse of Sague School in Port-Harcourt claiming over 50 lives.

A building originally designed as a single-storey suddenly without any modification, becomes a two or three storey from the owner's decision is a wrong step which could lead to the failure and collapse of the building.

On the collapse of a two-storey building belonging to Saint Thomas's Church Arakale, Akure, it was gathered and confirmed from the structural drawings that the building was designed for a storey building. During the construction, it was increased to 2-storey without making any alteration to the one storey originally designed for the building. This attitude caused overloading to the structure. Eventually, it collapsed claiming several lives.

Aderibigbe (2001) cited an alleged illegal bungalow at Akewusola, off oduduwa Street, Lagos that was extended to a four floor and later collapsed. The incident claimed one life with three others seriously wounded. Also, Simire (2001) on the collapse of Mushin mosque, reported that the collapse structure was for quite some time a bungalow and that the addition of fresh floors might have been too much for the foundation which was not initially designed to carry such a load.

viii. Lack of qualified and appropriate professional.

An attempt by the client and contractors to use unqualified professionals to ensure quality job and cost control could lead to failure of any building construction. This is when incompetent personnel is in charged of either design, construction or supervision.

Ogunsemi (2002) revealed that a two-storey building at Iju, Lagos State, under construction had ordinary sandcrete hollow blocks as load bearing wall without a single column. The design of this building was done by Architect and Engineers. The builders on this site

were mere bricklayers and carpenters and some draughtsmen calling themselves Architects and Engineers. The building collapsed at roof level towards the end of November, 1999. Also, the collapse of a storey residential building at Ijoka road, Akure in the year 1982 was as a result of unqualified professionals who took up the supervision of the building contract.

ix. **Rain storm**

Some buildings collapsed due to natural disaster like rainstorm and wind. The structural members of some of these buildings, such as slabs, beams, columns and foundation, have inadequate strength owing mostly to poor workmanship and lack of consideration for soil conditions at design stage. Some of these buildings are constructed with sun-dried mud block without any reinforced concrete structural members. Omenge (1988) disclosed that clay soil is readily and abundantly available in Nigeria and has been used as sun-dried block in traditional buildings. Despite the Advantages of high fire resistance, good thermal comfort are characterized by low mechanical strength, high shrinkage and low durability.

Osunade (1990) reported the shear and tensile strengths of unreinforced laterized concrete. According to him, the results obtained showed that the values of shear and tensile strengths of laterized concrete specimens are lower than those of concrete. For concrete grade 25, the shear and tensile strength values of reinforced concrete are 3.75Nmm^{-2} and 1.65Nmm^{-2} respectively while for unreinforced laterized concrete specimens the values are 2.84Nmm^{-2} and 1.40Nmm^{-2} respectively.

An example of this is a storey building constructed with mud situated at Stadium Road, Ondo, which collapsed in November, 1988 due to lack of reinforced concrete structural members. Also the Residential mud house and bungalow at Irele township, Ondo-Okitipupa Road, collapsed on May 2002 as a result of rainfall and flooding.

Also, a storey residential building situated at Oke Bola, Ado Ekiti collapsed by August, 2000 as a result of poor workmanship and

construction materials used in the construction of the deck. As reliably gathered, cracks and deflections were first noticed in the building. During heavy rain down pour, the deck which had already deflected retained water pondage for some days and later collapsed. This happened before the roofing of the building. In addition, the collapse of 2-storey building, located at Dawodu street Ifo, Ogun State that occurred in October 1999 where 20 people died, a 3-storey residential building at Salisu street, Iju Ishaga, Lagos that occurred in August 18, 1999 that claimed 35 lives, and also a commercial building along Ikorodu road, Lagos which happened on September, 29, 1987 that claimed 4 lives with 15 people injured all happened as a result of rain storm. Also outside Nigeria, the collapse of Café and Coffee house with many others located at Jerusalem in Isreal by March, 1992 in which over 20 people were killed were caused by some blizzard and flood.

x. Inadequate quality control

Some building structures collapse as a result of inadequate quality control during construction. Some examples are the use of sand heaps for concrete, which have high clay content, poor workmanship resulting in concrete honey combs and eccentric loading of columns and segregation due to dropping mix through a great height.

Onwusonye (2002) reported that quality control during field construction is concerned with ensuring that the work is accomplished in accordance with the requirements specified in the contract documents. A construction quality control programme involves inspection, testing and documentation for the control of quality of materials, workmanship and methods.

The collapse of a storey building at Oke Bola, Ado Ekiti that occurred on August 14, 2000 was due to poor quality control of the construction materials.

xi. Bomb explosion

Explosions of bomb have occurred in some parts of this country and outside this country in the recent time. Most of these bomb explosions are being caused by terrorist. Some structural buildings have collapsed as a result of these.

Dixon (2002) reported the deflection of the Ikeja Military Cantonment, Lagos that occurred on January 29th, 2002. According to the report the lounge is the transit point of top level dignitaries to Lagos. The ceilings of the key rooms and offices within the 50 metres by 30 metres lounge building including the inner chamber room caved in, exposing the concrete structure of the top roofing due to the bomb explosion. Also, on 11th September, 2001, the terrorists attacked the twin tower of the over 100 storey of the world Trade Centre and the Pentagon, in Washington DC, America. This bomb explosion attack led to the collapse of the building and destruction of thousands of lives and property. According to Brouwer (2002), the National Institute of Standards and Technology (NIST) was inaugurated to conduct a long-term investigation into the collapse of the three of the buildings at the world trade Center.

In a similar occurrence, internet browsing (1998) in Buenos Aires, church and school buildings all in Argentina, collapsed due to missile and car Bomb attack sponsored by Islamic Jihad that occurred by March 1992 in which more than 27 people died. Also, many buildings collapsed in the Middle East, Israel by January, 1991 as a result of missile and Bomb attacks that claimed so many lives due to buildings hit by Saddam Hussein's Scud missiles.

xii. Fire disaster

Fire disaster mostly caused by men or terrorists as a result of inter tribal war and communal clashes have led to collapse of so many buildings in Nigeria.

The Ife/Modakeke war that occurred between 1995 and 1999 led to destruction of lives, properties and the collapse of so many buildings in Ife and Modakeke, Osun State, Nigeria.

The collapse of the roof of a 4-storey LACCO building in 1983, one storey residential building at Gbogi and one storey building at Oke-Bola Street all in Akure were as a result of fire disaster.

Also, the collapse of some buildings located at Odoso compound in Ikare on February 26, 2002, two residential one storey buildings located at Owa Ale of Ikare street which occurred in 2001 as a result of the chieftaincy rift and clashes between the Olukare of Ikare and Owa Ale of Ikare were all due to fire disasters.

From Internet Browsing (1998), a duplex, 3-decker, type of wooden frame building located at Blue Hill Avenue Dorchester field section Boston got burnt by outbreak of fire as a result of Gas explosion on 26th August, 1998. Infact what was once a large fully occupied wooden apartment building was reduced to a large pile of burning debris. During this incidence, 15 Civilians and 3 fire fighters were injured.

xiii. Lack of maintenance culture

Some of the causes of collapse of building structures during the service period can be attributed to misuse and lack of maintenance culture. Adenuga (1999) remarked that maintenance of a structure should start from the day the excavation is dug. For example, if the excavation for foundation shears before or after placement of concrete, it must be cleared and maintained because earth impurities impair the strength of concrete. Therefore, maintenance is a continuous exercise which ensures that the defects are rectified as soon as they appear before further damage is done.

According to Crocker (1990), failure to maintain a building correctly may lead to premature faults, but in some circumstances this may not be the responsibility of the building owner. It could be the consequence of tenants not carrying out some of the obligation of a lease. It could also be due to the designers of the building or component manufacturers not properly (or at all) advising the owner to particular maintenance requirements. The collapse of a one-storey

building that occurred at Shetima Avenue, Kano State, by July 1999 was as a result of lack of maintenance of the roof leakage.

xiv. **Earthquakes**

Deaths resulting from earthquakes are usually caused by buildings collapse and the direct effects of the earth's movements. An earthquake is a sudden release of energy in the earth's crust, caused by movements deep beneath the surface.

Earthquakes according to Babatunde (2003) occur where large stresses build up in the rock along fault lines which are associated with movements of the crystal or tectonic and plates. As the plates jostle against each other, becoming distorted, tremendous strain builds up. From time to time, this is discharged in zones where the rocks are weakest, causing a series of shock waves known as seismic waves. These can have highly destructive effects on the earth's surface as they cause the ground to undulate and shake from side to side. Most fault lines are located around the Pacific Rim, giving rise to regular earthquakes in the West of the United States and Latin America, China, Japan and the South Pacific region. Also, North Africa, Italy, the Balkans and Turkey are also located near fault-lines involving the African plate and the Eurasian plate. Earthquakes can occasionally be predicted by knowledge of locations and cycles of past earthquake activity, by minor tremors immediately prior to a major event, and sometimes by abnormal animal behaviour.

There are so many examples of buildings that collapse as a result of earthquakes in the recent times. Earthquakes reduced some structures around Algiers to piles of crumbled masonry, leaving many residents trapped inside. In the University town of Rouiba, on the far eastern outskirts of the Algerian capital, a three-storey faculty building had simply folded in on itself. Also, in the town of Reghaia in Algeria, a 10-storey tower had collapsed completely killing several people of which 540 were reported dead and about 4,600 people were injured. Also, earthquakes where many buildings sunk in Northern Algeria, at least 707 people were known to have been killed on 21st May 2003.

Infact, a school building collapsed on 18th April, 2003 at Turkey of which over 200 students died.

From Internet Browsing (1985) the failure of top floors, Hotel continental, Mexico City in September, 1985 was as a result of Earthquakes. Top floors of buildings are particularly vulnerable because of the following reasons.

- (a) Upper floors are displaced more than ground level floors during earthquakes.
- (b) The resonance frequencies of the buildings coincide with the ground vibrations leading to large amplification of oscillations.
- (c) Upper floors often have smaller and weak load bearing components.
- (d) The length of this earthquake event gave time for development of tensional vibrations due to asymmetric distribution of masses and elasticity in the high – rise buildings.

In contrast to the totally destroyed office building in the foreground, the 44-storey Torre Latinoamericana office building in the background on the right, remained almost totally undamaged, as it did in a 1957 earthquake. The building is a symmetrical steel frame structure built to resist earthquakes. It has 200 piles extending down about 35 metres to the top most stable and load-bearing earth stratum. The tall transiently designed to withstand large horizontal forces.

The causes of building collapse generally may be attributed to a number of reasons among which are lack of Town planning approval, alterations to the approved design, negligence to specification, ignorance and greed on the part of the client and Contractors, lack of proper soil investigation of the site, poor quality design, inefficient supervision, inadequate quality control, differential settlement of foundation and wind forces causing lateral deflection. It has also been pointed out that the use of bad quality materials, overloading, lack of adequate care in design of structures, poor workmanship, earthquakes, bomb blast and fire disaster have been possible root causes of building collapse.

CHAPTER THREE

METHODOLOGY

3.1 Review of Literature and past works on collapse of buildings

Some books, past works and News papers on building collapse were read so as to get some examples and causes of buildings collapse. Also, internet browsing on reasons for the collapse of some buildings abroad was done. Moreover, locations of some building collapse were visited in Ikare, Akure, Ondo, Irele and Ado-Ekiti, Nigeria.

3.2 Use of questionnaire

Some information and data related to the collapse of Saint Thomas's Church Hall, Arakale, Akure were acquired through questionnaires and oral interviews. Precisely, 120 copies of questionnaires were sent out to people to answer some technical questions on the collapse of the Hall. A sample of the questionnaires sent out is attached in appendix No. 8. Therefore, 95 copies of answered questionnaires were collected, collated and analyzed using Bar and Pie charts. Some of the respondents to the questionnaires are professionals. They comprise 10 Architects, 8 Builders, 7 Surveyors and 47 Engineers including three Engineers who took part in the supervision of the collapsed building. Others are civil servant, students, businessmen and members of the Church.

The Isikan community leader, the Iralepo of Isikan, whose palace is very close to the Saint Thomas's Church Akure and who also witnessed the collapse was contacted to answer some oral questions about the collapse of the Church Hall and the past records of the site.

3.3 Excavation of Soil Samples from the Building Site

The soil materials used in the construction of the Saint Thomas's Church Hall were collected for laboratory analysis. The laboratory tests include Atterberg Limits, Moisture content, Dry density, Compaction, California bearing ratio and Triaxial undrained tests. These were done to determine the bearing capacity and the

underground water conditions that could have affected the building foundation.

Soil samples were taken from 4 trial pits towards the four corners of the location where the building collapsed. The site plan showing the locations of the soil pits is shown in fig. 3.3.1. Samples could not be taken at the centre of the location because the raft foundation of the new building under construction has been completed.

Disturbed samples were taken from trial pits 1 and 2 at a depth of 1.5m while undisturbed samples were taken at a depth of 1.8m to 2m from the same pits. Also disturbed samples were taken from pits 3 and 4 at a depth of 1.5m and 1.8m respectively while undisturbed samples were taken from pits 3 and 4 at a depth of 1.8m and 2m respectively. However, the water table was not reached in any of the trial pits.

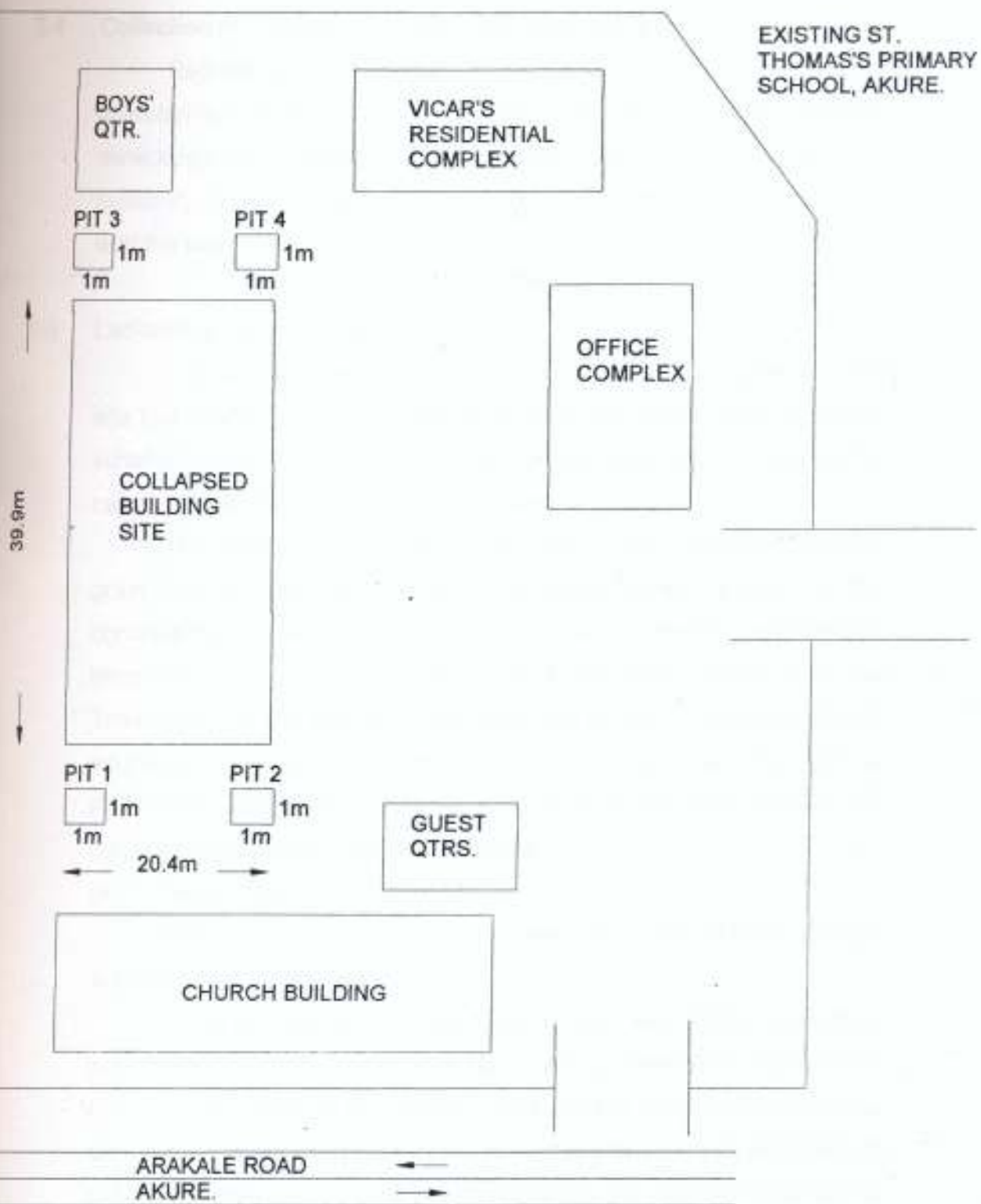


Fig 3.3.1 : Site Plan showing the locations of the soil pits

3.4 Collection of Construction Materials from the Building Site

Representative samples of the concrete rubbles were also collected and analyzed in the laboratory. The tests include crushing, mineralogy and alkalinity test. From these, compressive strength and suitability of the building materials were determined for comparison with the standards

3.5 Laboratory Test Procedures

The laboratory tests carried out were aimed at enumerating the site stratigraphy (soil layers), soil properties and construction materials suitability with a view to determining the appropriate engineering causes of the collapsed building structure.

The Atterberg limit tests, compaction test, moisture content, grain size analysis, dry density, and compressive strength of the concrete rubbles were carried out at the material testing and research laboratory of the Ministry of Works and Transport, Akure while the Triaxial tests of the soil and mineralogy test of the construction coarse aggregate were carried out at the material testing laboratory, Federal Polytechnic Ado Ekiti in Nigeria. According to BS 1377 (1975), the laboratory test procedures are as follows:

(a) Liquid limit

The liquid limit is the minimum moisture content at which the soil will flow under its own weight.

Using the spatula, the dried soil sample was mixed thoroughly with distilled water to form a stiff consistency. Thereafter, a portion of this soil was placed in the circular brass dish of the liquid limit device and leveled off parallel to the base. Also, the grooving tool was used to cut through the centre of the portion of soil in the dish. By means of a handle-rotating can, the dish was raised and dropped on the rubber base at the rate of two revolutions per second until the bottom of the groove closed. Then, the number of blows at which the groove closed was recorded. Afterwards, a small portion of the soil in the dish was taken for moisture content determination.

This process was repeated several times. On the chart, the moisture content values were plotted on the Arithmetical scale against the number of blows on the logarithmic scale to determine the liquid limit. (See Appendix two)

(b) Plastic limit

The plastic limit is the minimum moisture content at which the soil can be rolled into a thread of 3mm diameter without breaking up.

A 20gm of dried soil was mixed with distilled water and moulded into a ball. The ball of soil was rolled with the palm of the hand on the glass plate to form a thread. The rolling continued until the thread became 3mm thick. The soil was then kneaded together and rolled out again. This process continued until the thread crumbled. Afterwards, some portion of soil was taken for moisture content determination. The process was repeated 3 times. The average of the moisture contents was taken as the plastic limit.

The plastic index of the soil is the difference between the liquid limit and plastic limit.

(c) Compaction test

Compaction is a method by which a soil is placed in a dense state in order to decrease future settlements, increase shear strength and decrease permeability.

A 3kg of air-dried soil was mixed with 5% of its weight of water. The mixed soil was compacted in 3 layers in mould of known weight with collar attached, each layer was compacted with 25 blows of the 2.5kg hammer dropping at 300mm. Later, the collar was removed so as to level and weigh the soil. The soil was extruded from the mould and the moisture content was determined. The compacted soil was broken up and mixed with the rest of the sample. Also, more water was added and mixed thoroughly.

The above was repeated several times until the weight of the compacted soil reached its maximum and decrease.

The graph of dry density was plotted against moisture content to determine the optimum moisture content and maximum dry density.

(b) **Moisture content**

The moisture content is the ratio of the weight of dry solids in the same volume usually expressed on percentage basis.

The tin container was weighed and denoted as W_1 . The sample of soil was put in the container, weighed and denoted as W_2 . Later the tin and content was placed in the oven, maintained at 105°C and kept for 24 hours. There after, the tin and content was reweighed and denoted as W_3 . These were repeated 4 times and the moisture content was determined.

(e) **Particle size distribution**

For coarse-grained material, the distribution of various sized particles in the soil may be determined by sieving.

The oven-dried soil sample was weighed. The sample of soil was sieved through succession of B.S. Sieves 38mm ($1\frac{1}{2}$ "), 25mm (1"), 6.25mm ($\frac{1}{4}$ "), 5mm ($\frac{3}{16}$ ") No. 4, No. 7, No. 14, No. 36, No. 52, No. 72, No. 100, and No. 200. The material retained on each sieve was weighted. The percentages of the total sample passing each of the sieve were calculated. Afterwards, the results were plotted against each sieve size on the particles size distribution chart to determine the type of soil.

(f) **Triaxial compression test**

A cylindrical soil sample is subjected to a lateral pressure and a vertical load is applied at a constant rate of strain by a steel ram attached to a proving ring. This vertical load increases until the sample fails in shear.

A constant rate strain compression machine composed of a loading frame and hydraulic pressure apparatus including an air compressor and water reservoir.

A 200mm by 200mm undisturbed sample brought from site was reduced in size, cut and shaped to a specimen. The specimen was then covered with a rubber membrane to prevent loss of moisture. The prepared sample was placed in the position in the transparent triaxial cell. The cell was filled with water. The air pressure was then increased to the required amount. During this time, the air pressure was kept

constant. The level of the cell was adjusted so that the proving ring just touched the cell piston. The machine was allowed to be in motion. The proving ring and the strain gauge reading were recorded as the vertical loads were applied until failure occurred. These were repeated 3 more times using different specimen. Eventually, the ultimate bearing capacity of the soil sample from Tomlinson (1980) was calculated using this formular;

$$q(ult) = 1.3 CN_i + \delta DN_q + 0.4\delta BN_\delta$$

Where

C	=	Cohesion of soil
D	=	Depth of foundation
B	=	Width of foundation
δ	=	Density of soil

N_i , N_q and N_δ are bearing capacity factors

(g) **California bearing ratio (CBR) test**

The California bearing ratio determines a penetration resistance of a compacted specimen of the soil compared to a standard resistance for crushed stone. The resistance divided by the standard for the stone multiplied by 100 is called the California bearing ratio (CBR). The test measures the shearing resistance of a soil used for construction under controlled moisture and density conditions.

A quantity of soil was prepared to compact two CBR moulds of soil at the optimum moisture content of the soil. The two moulds less base plates and collars were weighed. So, a mould to the base plate was clamped. Then the spacer disk in the mould was inserted and covered with a piece of filter paper. The soil was compacted using 36 blows per 3 layers. Afterwards, the collar was removed, and the sample was trimmed smoothly and flushed with the mould. The base plate and spacer disk were removed. The mould, and compacted soil were weighed. Eventually, the wet mass, density and unit weight were determined.

The above was repeated three times. The sufficient slotted weight of 4.5kg was placed on the sample to simulate the required over burden pressure. The specimen was placed in the compression

machine and the piston using a seating load of 4.5kg. Penetration readings as outlined together with the corresponding load readings were taken. As a result, a curve of penetration resistance versus penetration in millimeters of the sample was plotted to obtain the penetration resistance for 2.5 and 5.00mm to compute the two CBR numbers.

(h) Compressive strength of concrete

The compressive strength of the concrete rubbles was determined in the laboratory to assess the quality of the concrete.

The mass of the concrete rubble was determined using weighing machine to calculate the density. The concrete rubbles were crushed and readings were taken on the crushing machine.

(i) Mineralogy test

This test determines the composition of all the constituents present in the coarse aggregate material.

The concrete rubble was first broken down by mechanical method so as to extract all the stone aggregate. The stone was crushed and grinded to powder. The powder was poured into the testing machine to give the various percentages of feldspar, quartz, biotite and muscovite present.

3.6 Over view of the structural design of the collapsed building

The structural drawings presented in 9 sheets with the serial numbers drg. No. 1, drg. No. 2, drg. No. 4, drg. No. 5, drg. No. 6, drg. No. 7, drg. No. 8, drg. No. 9, drg. No. 10 containing the foundation details, column details, layout of floor beams, floor slab details, floor beam details, floor beam continued, beams continued, case details, roof and beams details respectively designed by FUTA Ventures Limited, in 1993 were provided as addendum for this project on the collapsed building. In the drawings, the foundation, columns and beams layouts are clearly shown (see Figs 3.3.2 and 3.3.3).

The analysis and design of some of the beams were done. In all, there are 17 beams designed from the first floor to the roof level of the collapsed building. Out of these, beams 6,7,8,9,12,13,14 and 15 were studied and checked for possible errors and construction discrepancies.

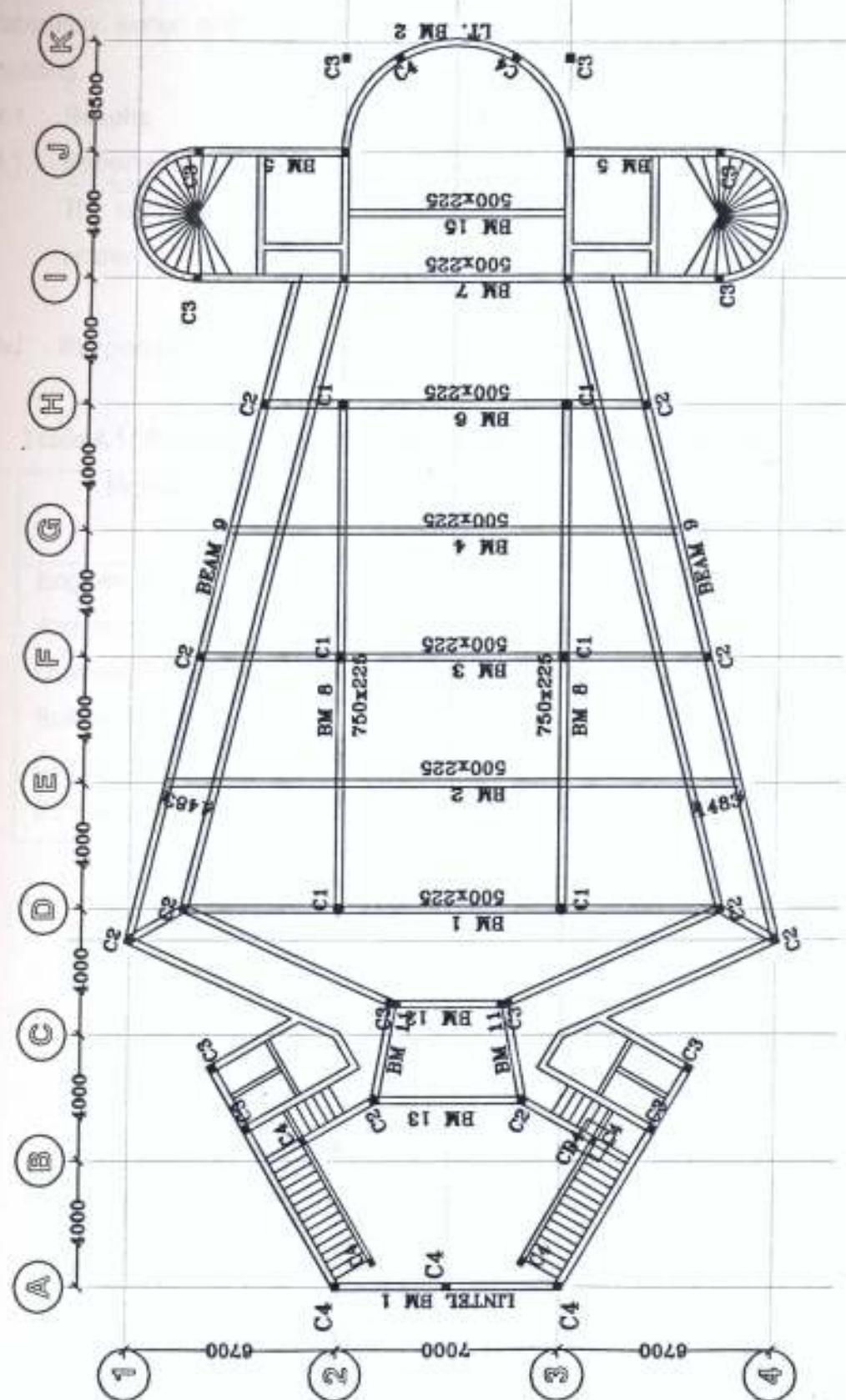


Fig. 3.3.3: BEAMS AND COLUMNS LAYOUT
SCALE 1:100
42

CHAPTER FOUR

RESULTS AND DISCUSSIONS

This chapter comprises all the summary of the site investigations, laboratory works and checking of the structural design of the collapsed building.

4.1 Results

4.1.1 Reports from the questionnaires

The summary of the reports collated from the respondents are shown below:

(a) Response from professionals.

Table 4.1: Professionals who responded to the questionnaire

Professionals	Number	Percentage (%)
Engineers	47	50
Architects	10	11
Quantity Surveyors	7	7
Builders	8	8
Clergy men	3	3
Others	20	21

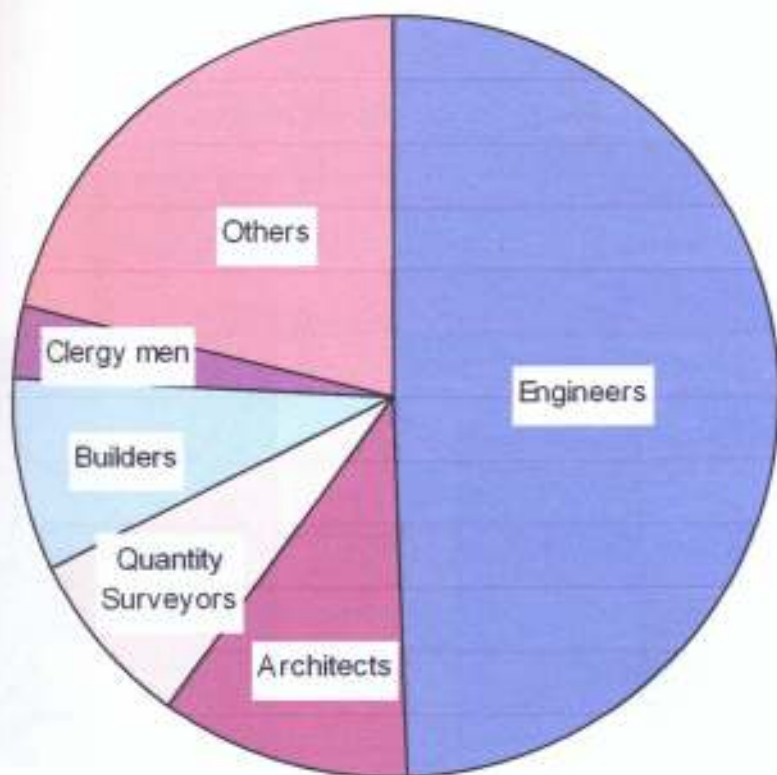


Figure 4.1: A Chart showing Professionals who responded to the questionnaire

(b) Reasons for the collapse

Table 4.2: Response on reasons for the collapse of Saint Thomas's Church Hall:

Causes of Collapse	Engineers	Architects	Builders	Quantity surveyors
Foundation problem	13	3	-	-
Poor workmanship	18	5	3	3
Overloading	7	2	3	1
Inadequate supervision	10	-	4	2
Poor design	5	2	-	-

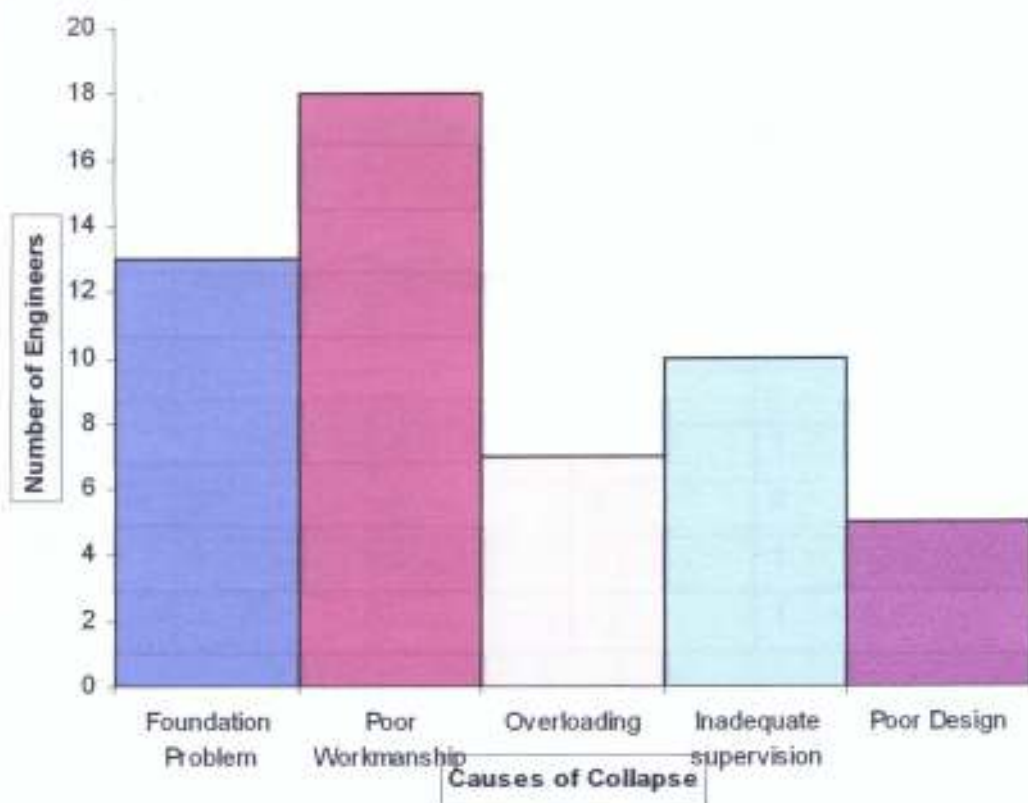


Figure 4.2: A Histogram showing Causes of Collapse indicated by engineers

(c) Probable estimated cost of the building

In an attempt to know the extent of loss in terms of property involved in the project, the response from the Engineers, Architects, Builders and Quantity Surveyors on the estimated cost of the collapsed building is tabulated below. From Table 4.3, the highest ranking from each profession is five million naira (N5 million) followed by seven million naira (N7 million). Also, four out of the six professionals who responded to the questionnaire and also members of the collapsed church, all estimated the collapsed building as five million Naira (N5 million). However, based on all the evidences above, the estimated cost of the collapsed building is five million Naira (N5 million) as at 1998.

Table 4.3: Response on probable estimated Cost of the building

Cost N(million)	Respondents				
	Engineers	Architects	Builders	Quantity Surveyors	Others
2.00	5	-	-	-	-
3.00	-	2	1	1	-
5.00	20	3	3	4	5
7.00	5	2	2	1	6
10.00	3	-	-	1	5

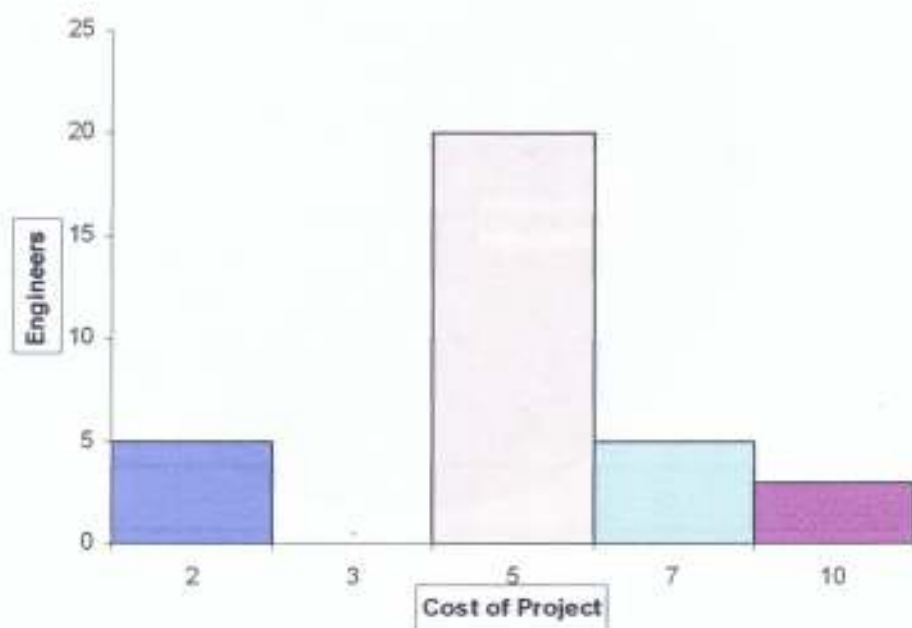


Fig. 4.3: A Histogram showing Number of Engineers against estimated cost of the project

Table 4.4: Professionals who estimated the cost of the collapsed building as five million.

Professionals	Number	Percentage (%)
Engineers	20	57
Architects	3	9
Builders	3	9
Quantity Surveyors	4	11
Others	3	14

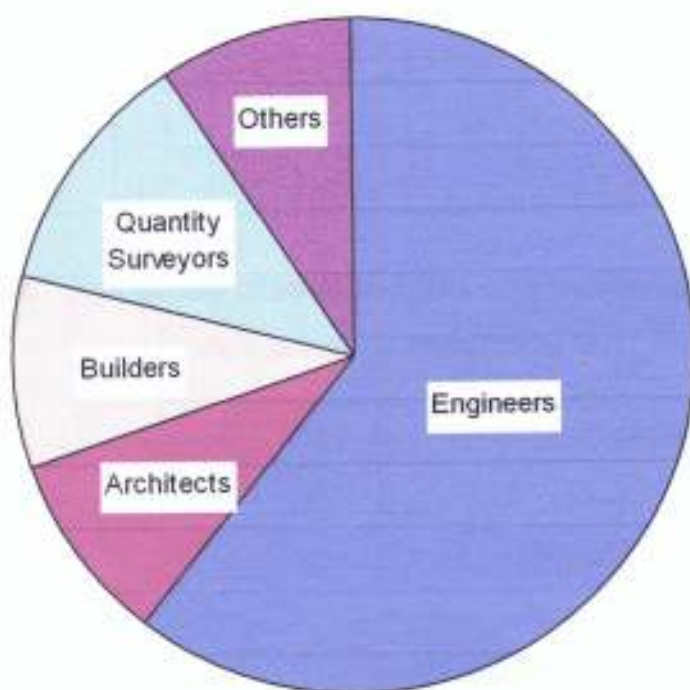


Fig. 4.4: A Chart showing Professionals who estimated the cost of the project as Five Million

(d) **Response on the number of people who died in the collapsed building**

Most of the answers received in the questionnaire's report on the actual number of people who died were mere assumptions. However, there were some of the Church members and people who were present on the day when the building collapsed who put 5 as the number of people that died. According to Abidakun (1998), manual labour had to be employed to retrieve dead bodies and rescue lives of those who were trapped. So, based on these evidences five people died in the collapsed building.

Table 4.5 (a): Number of people who died in the Collapsed building.

Age group (Years)	Number of Death									
	1	2	3	4	5	6	7	8	9	No Idea
20 – 29	-	-	-	1	5	6	-	1	4	3
30 – 39	1	-	1	-	7	2	1	3	3	12
40 – 69	1	-	2	1	18	1	2	1	3	14
70 – Above	1	-	-	-	-	-	-	-	-	-

Table 4.5(b): Age group of respondents who confirmed five people dead

Age group	Respondents	Percentage (%)
20 – 29	5	17
30 – 39	7	23
40 – 69	18	60

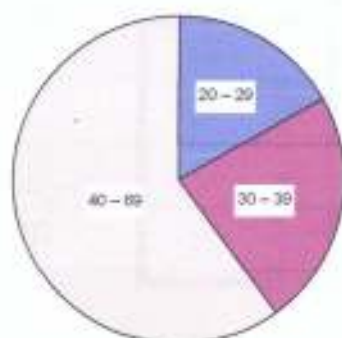
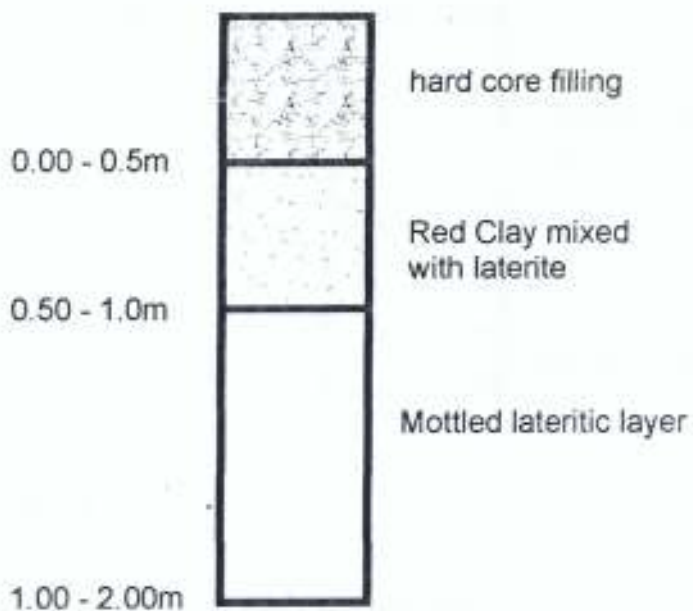


Fig. 4.5: A Chart showing Age group of respondents who confirmed five people dead.

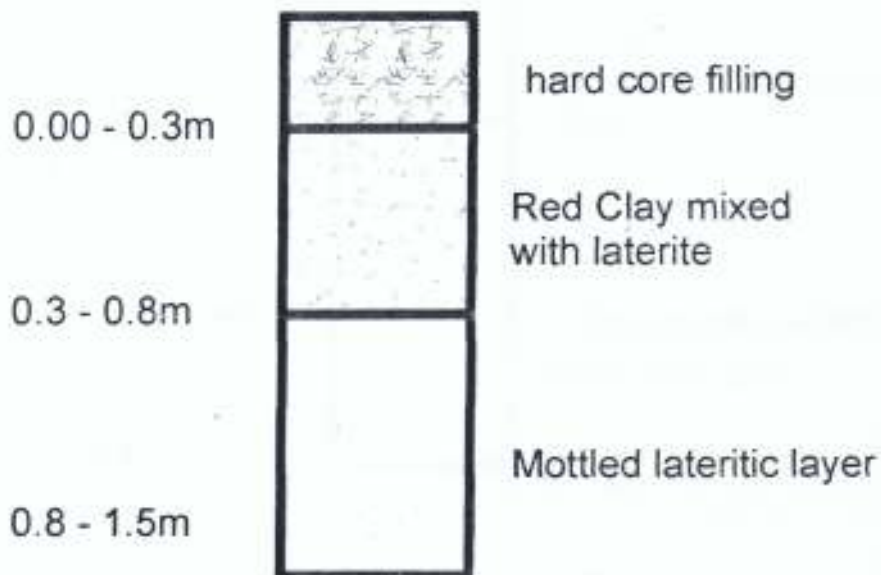
4.1.2 Summary of the Laboratory Results

Table 4.6 : Sub soil profile

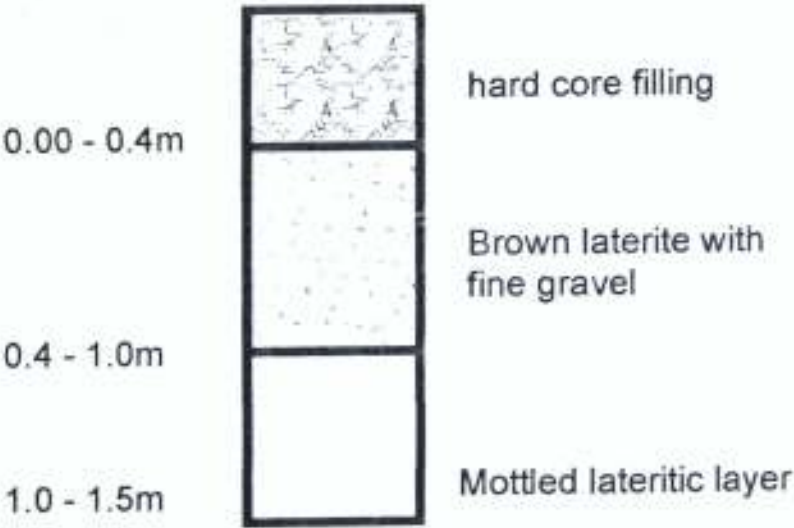
(i) Pit No. 1



(ii) Pit No. 2



(iii) Pit No. 3



(iv) Pit No. 4

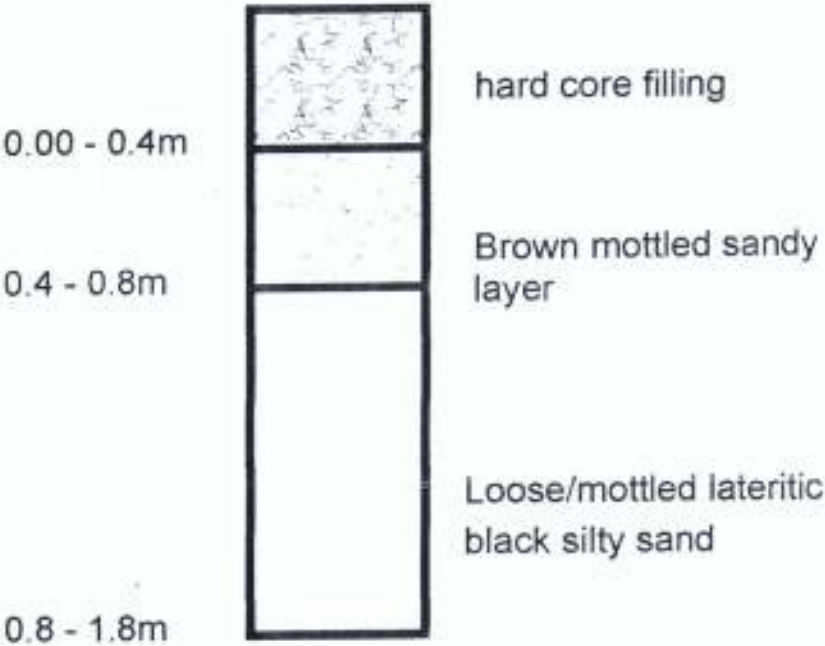


Table 4.7: Atterberg Limits

Locations	Pit 1	Pit 2	Pit 3	Pit 4	Average	Standard value range
Depth of Materials	1m	1m	1m	1m		
Liquid limits	31	25.5	22	26	26.13%	25% - 27%
Plastic limits	16	14	18	11	14.8%	10% - 15%
Plastic index	15	11.5	4	15	11.3%	
Linear shrinkage	1.6	1.4	0.2	1.0	1.05%	0.9% - 1.5%

Table 4.8: Moisture Content

Locations	Pit 1	Pit 2	Pit 3	Pit 4
Depth of Materials	1m	1m	1m	1m
	8.1	7.0	8.6	3.6
	9.3	10.0	10.2	6.8
	15.5	12.9	11.8	9.2
	19.2	14.0	17.5	12.3
		18.0		16.8
Average	13.03	12.38	12.03	9.74
Standard value range	10% - 15%			

Table 4.9: Dry Density

Locations	Pit 1	Pit 2	Pit 3	Pit 4
Depth of Materials	1m	1m	1m	1m
	1.86	1.83	1.77	1.45
	1.91	1.90	1.91	1.70
	1.99	2.01	1.96	1.90
	1.83	1.98	1.80	2.01
		1.85		1.70
Average	1.90	2.39	1.86	2.19
Standard value range			1.8 – 2.1	

Table 4.10: Compactions

Table 4.10: Compactions

Locations	Pit 1	Pit 2	Pit 3	Pit 4	Average	Standard value range
Max. Dry Density	2.0	2.0	1.97	2.00	1.99	1.8-2.10
Opt. Moisture Content	14.3%	12.5%	12.5%	11.8%	12.78%	10 - 15

Table 4.11: California Bearing Ratio

Locations	Pit 1	Pit 2	Pit 3	Pit 4	Average	Standard value range
0.0025 Penetration	62.3	64.57	62.94	76.77	66.65	80 – 100
0.005 Penetration	65.58	67.95	66.70	75.77	69.00	80 -100

Table 4.12(a) :Unconsolidated Undrained Triaxial Test (Pit No. 1 Sample)

Shear Stress δ_1 kN/m ²	Shear Stress δ_2 kN/m ²	$\frac{\delta_2 - \delta_1}{2}$ kN/m ²	Allowable Bearing Pressure	Standard Value Range
299	464	82.5	147.7 kN/m ²	Must not be less than 120 kN/m ²
438	683	122.5		
559	891	166		
452	1030	289		

Table 4.12(b) :Unconsolidated Undrained Triaxial Test (Pit No. 2 Sample)

Shear Stress δ_1 kN/m ²	Shear Stress δ_2 kN/m ²	$\frac{\delta_2 - \delta_1}{2}$ kN/m ²	Allowable Bearing Pressure	Standard Value Range
327	404	38.5	252 kN/m ²	Must not be less than 120 kN/m ²
464	645	90.5		
655	772	58.5		
731	974	121.5		
846	1064	108.0		
688	1165	238.5		

Table 4.12 (c) :Unconsolidated Undrained Triaxial Test (Pit No. 3 Sample)

Shear Stress δ_1 kN/m ²	Shear Stress δ_2 kN/m ²	$\frac{\delta_2 - \delta_1}{2}$ kN/m ²	Allowable Bearing Pressure	Standard Value range
114	262	74	218kN/m ²	Must not be less than 120kN/m ²
286	448	81		
410	576	83		
581	654	36.5		
537	694	78.5		

Table 4.12 (d) :Unconsolidated Undrained Triaxial Test (Pit No. 4 Sample)

Shear Stress δ_1 kN/m ²	Shear Stress δ_2 kN/m ²	$\frac{\delta_2 - \delta_1}{2}$ kN/m ²	Allowable Bearing Pressure	Standard Value range
147.6	236	44.2	213kN/m ²	Must not be less than 120 kN/m ²
294	359	32.5		
370	568	99		
501	635	134		
546	663	58.5		
560	672	56		
498	670	86		

4.13: Mineralogy Test

Sample	Minerals	Percentage (%)
Sample A Granite but feldspathic	Feldspar	70
	Muscovite	8
	Quartz	6
	Biotite	6
	Montmorillonites	4
	Kaolinite	3
	Illites	3
Sample B Porphyritic Granite but Quartzitic	Quartz	60
	Biotite	30
	Muscovite	10
Sample c Charnockitic	Mainly Biotite with minute particles of muscovite and quartz	Over 80% Biotite

Table 4.14: Compressive Strength of Concrete

Cube No	Age for testing	Date tested	Weight of cube (gm)	Density kg/m ³	Crushing load (N)	Crushing strength N/mm ²
A	Over 28days	18/12/03	4958	8471	20,000	5.2
B	"	"	3630	6694	15,000	2.8
C	"	"	4150	6072	20,000	3.6
D	"	"	4170	6043	15,000	3.2
E	"	"	3040	7737	5,000	6.9
F	"	"	6658	4326	15,000	3.8
G	"	"	2795	3864	3,000	6.2
H	"	"	2870	4014	5,000	4.9
I	"	"	3590	4308	3,000	7.0
J	"	"	3504	2354	5,000	5.8

Average Crushing Strength = 4.9N/mm²

Standard value Range = 21 – 22 N/mm²

4.2 Discussion of Laboratory Test Results

4.2.1 Atterberg Limits

The Atterberg limits are very useful characteristics of assemblages of soil particles. The limits are based on the concept that a fine-grained soil can exist in any of four states depending on its water contents. The greater the amount of water a soil contains, the less interaction there will be between adjacent particles and the more the soil should behave like a liquid. It is expected that the water that is attracted to surfaces of the soil particles should not behave as a liquid. The significances of the atterberg limit tests are as follows:

- The limits are used directly in specifications for controlling soil for use in fill and in semi-empirical method of design.
- To determine the rate of water flowing through a sample of soil, measure the permeability of the soil.
- To determine the settlement of a building, measure the compressibility of the soil, and use this value in settlement equations based on Terzaghi's theory of consolidation.

- (d) To evaluate the stability of a slope, measure the shear strength of the soil and substitute this value in an expression based on the law of static.

From the laboratory results, average values of liquid limit, plastic limit, plastic index and linear shrinkage are approximately 26.13, 14.8, 11.3 and 1.05 respectively (see Table 4.7). These values fall within the value expected for normal soil which the rate of water flowing and settlement in a building are very much negligible. These conclude the fact that the collapse of the building was not as a result of excess of water contents, liquid limit and linear shrinkage in the soil material of the soil underneath the collapsed building site.

4.2.2 Compaction test

The soil samples were compacted in the laboratory at various water contents and compaction curves drawn. (See appendix 3) compaction curves show that as the molding water content is increased, the dry density increases to a peak and then decreases. The density and moisture content at peak density are respectively maximum dry density and optimum moisture content for the soil samples. The maximum dry density are 2.0, 2.0, 1.97 and 2.0 while the optimum moisture content are 14.3%, 12.5%, 12.5%, and 11.8% respectively. (See Table 4.10).

Increasing the moisture content tends to increase the inter-particle repulsions, thereby permitting a more orderly arrangement of the soil particles to be obtained with a given amount of effort. Increasing the compactive effort at a given moisture content tends to work the particles into a more nearly parallel arrangement.

In general, compaction causes a significant change in the total stresses and pore pressures within the compacted soil. The nature and magnitude of these stresses depend on the soil and the amount of compaction imparted to the soil. Also, it increases the lateral effective stress, rearranges soil particles and moves them closer together, and increases the ratio of horizontal effective stress to vertical effective stress. Compaction also increases soil strength and reduces its permeability, compressibility and the voids available for flow.

Therefore, a high proportion of settlement of foundation on loose soil takes place as the load is applied. In soft compressible silts and clays,

much more settlement would be encountered. If the whole of the foundation area of a structure settles to the same extent there is no detrimental effect on the super structure. However, if there is a relative movement between various parts of the foundation due to the soil strata, stresses are set up in the structure. Serious cracking and even collapse of the structure may occur if the differential settlements are excessive. In the case of the collapse of the Saint Thomas's Church building under investigation, since the compaction tests carried out show that moisture content and dry density of all the trial pits are almost the same and without any excessive value, settlement due to consolidation of the foundation soils and the immediate settlement of the soil which took place during application of the load would be negligible. Also, differential settlement encountered in the soil would also be negligible. In addition, the consolidation settlement which takes place as a result of volume reduction of the soil caused by extrusion of some of the pore water from the soil would be uniform in all the four corners of the soil and could not cause collapse of the building.

4.2.3 Unconsolidated Undrained Triaxial Test

The triaxial test is used to determine the stress-strain properties of soil. The effective stress-strain behaviour of granular soil is virtually the same for dry and saturated conditions. The stress-strain behaviour of clay is greatly dependent on the stress history of the sample. The higher the over consolidation ratio, the stiffer the soil.

The basic criterion governing the design of foundation is that the settlement must not exceed some permissible value. This value will vary from structure to structure. In order to ensure that this basic criterion is met, two things have to be considered : First, for any foundation there is some value of the applied stress at which the settlements start to become very large and difficult to predict. This load is called the bearing capacity. The foundation must be designed so that the actual bearing stress is less than the bearing capacity, with an appropriate margin of safety to cover uncertainties in the estimate of both the bearing stress and the bearing capacity. Generally, the bearing capacity is taken as the bearing stress causing local shear failure.

Also, after determining the bearing capacity and ensuring that the bearing capacity exceeds the expected applied bearing stress with an adequate margin of safety, the settlement that will occur under the expected load must be estimated and compared with the permissible value.

Settlement can detract the appearance of a building by causing cracks in exterior masonry walls and interior plaster walls. It can also cause a structure to tilt enough for the tilt to be detected by the human eye. Settlement can cause a structure to fail structurally and collapse even though the factor of safety against shear failure in the foundation is high.

In case of the building under investigation, the collapse was sudden under 60 seconds and that there was no trace of cracks either in the interior or exterior masonry and structural members, there was also no tilting of the members to serve as warning before the collapse. The absence of these signals showed that there was no differential settlement in any part of the building. Also the results of the laboratory tests done show that the bearing capacity of sample one is 148kN/m^2 , sample 2 is 252kN/m^2 , sample 3 is 218kN/m^2 , sample 4 is 213kN/m^2 (See tables 4.12 (a) – 4.12 (d)). With these results, it could be concluded that the bearing capacity of the site ranging from 148kN/m^2 - 260kN/m^2 is adequate for a two storey building structure. Therefore, the collapse of the building is not as a result of low bearing capacity of the soil underneath.

4.2.4 Grain size analysis

The results of the grain size analysis were as tabulated in appendix 4 from the graph plotted, the description of the samples are as shown below:

Sample 1, Red gravelly sandy soil

Sample 2, Red gravelly sandy soil

Sample 3, Brown gravelly sandy soil

Sample 4, Brown gravelly sandy soil

The particle size distribution test is a form of classification test in which sieve analysis or a combination of sieve analysis and sedimentation or hydrometer analysis is used to obtain grading curves which can be plotted on the chart. The particle-size distribution test is of particular value in the investigation of problems of excavating permeable soils below the water table. It

could be deduced from the result that the soil is suitable for the foundation designed for the collapsed structure.

4.2.5 Compressive strength of the concrete rubbles of the collapsed building.

It was reliably gathered that no quality control test was done on the construction materials for the collapsed building. Also, no concrete was cast during the construction for laboratory test. However, in an attempt to assess the quality of the concrete materials used for the construction so as to determine the level of workmanship, some of the concrete rubbles and remains of the collapse were taken to the laboratory for compressive strength crushing test. Eventually the average value of 4.9 N/mm^2 is obtained (see table 4.14). This value is too much below the cube strength expected for ratio 1:2:4 mix design after 28 days of curing. Though the concrete samples collected for testing had been exposed in the atmosphere for about 4 to 5 years which invariably might have reduced the crushing strength of the concrete. Also, the effect of the collapse would also reduce the strength of the concrete. Notwithstanding, the value would not have been reduced drastically. Higher results should have been obtained. It could be remarked that mix ratio of the concrete materials is obviously less than 1:2:4 mix ratio. Therefore it could be concluded that the workmanship during the construction of the collapsed building was poor. This as a matter of fact must have contributed to the weak structural members like columns, beams and slab which led to the collapse of the building. (See plate 4.1)



Plate 4.1: Poor Concrete Mix

Tomlinson (1980) pointed out that the principal cause of deterioration of concrete in foundation is attack by sulphates present in the soil, in the ground water or in sea water. Other agencies causing deterioration include chemical waste, organic acids, frost, sea action, certain deleterious aggregates and corrosion of reinforcement. All these could reduce the strength of concrete and eventually lead to collapse of building.

Expatriating further, sulphates in solution react with portland cement to form insoluble calcium sulphate and calcium sulfo-aluminate. The crystallization of the new compounds is accompanied by an increase in molecular volume which caused expansion and disintegration of the concrete at the surface. The essential feature of resisting sulphate attack is to produce a dense, impermeable concrete. Also, the disruptive effect of frost is due to expansion of the water while it is freezing in the pores in the concrete. The effect is most severe if the freezing is rapid when there is no time for water to be extruded from the surface.

Moreover, corrosion of reinforcement occurred in the first floor structural members already completed 3 years before the commencement of the last floor as a result of insufficient cover. Also the presence of chlorides in the aggregates and in certain proprietary chemical admixtures and the penetration of water into cracks and honey combed concrete weakens the concrete. Stray electric

currents can also cause electrolytic corrosion. It can be avoided by using the correct cover, by careful selection of aggregates and by the use of dense well-compacted concrete.

It was alleged that vibration machine was not used during the casting of all the concrete works in the collapsed building. Also, insufficient cover in the structural members would have led to corrosion of reinforcement in the concrete. All these amounted to poor workmanship as part of the principal factors that led to the collapse of the structure. (See plates 4.2 and 4.3)



Plate 4.2: In-adequate Cover and large spacing of reinforcement



Plate 4.3: Non monolithic Construction

4.2.6 Mineralogy test of aggregates

According to Twidale (1982), Granite chipping is the end product of granitic rock after crushing. Aggregate stone is composed of quartz, feldspar and mica. The mica is made up of Biotite and muscovite. Granite is an important building structure stone because of its good appearance, hardness and resistance to weathering except when crystals of mica (Biotite and muscovite) are large and weathered leaving void at the surface. In general terms, a granite rock consists of quartz, feldspar, Biotite and muscovite. All these have their significant role in building construction.

(a) **Quartz:** This is regarded as most stable of the common rock-forming minerals and certainly of those that occur in granite. Quartz of granitic rocks have a very high resistance in weathering profiles and is longer than any of the other essential rock-forming minerals.

(b) **Feldspar:** This is one of the dominant rock-forming minerals which provide the basis for classification of granites and many other crystalline rocks. Feldspar like other minerals has a very high resistance to weathering. Therefore, it is recommended that granite for building construction may be rich in alkaline feldspar and quite minute in biotite content, otherwise such building may have problem after construction.

(c) **Mica (Biotite and Muscovite)**

Biotite is particularly susceptible to moisture attack. In an environment exposed to moisture, mica takes water into its lattice and expands, particularly along basal cleavage. The fractures developed in the expanded biotite extend as micro-fissures through adjacent quartz and feldspar crystal. The biotite is altered to hydrobiotite. The chemical changes are slight involving oxidation of Iron II to Iron III and the replacement of potassium by water molecules. This leads to expansion and disintegration. When this problem occurs, the concrete is weakened and eventually leads to structural failure. In the mineralogy test carried out on the Aggregate used for the construction of the structural members of the collapsed building, (See table 4.13) sample A contains 70% Feldspar, 8% Muscovite, 6% Biotite, and 6% Quartz, 4% Montmorillonites, 3% Kaolinite and 3% illites. In sample B, Quartz is 60%, Biotite is 30% and muscovite is 10%. In sample C, the dominant is mainly Biotite with minute

accessories of muscovite and quartz. From the above analysis, Biotite is considered to be the most dominant mineral of the granite material, followed by feldspar and muscovite. However, we can conclude that Biotite is dominant of the aggregate used for the construction of the collapsed building and therefore is one of the factors that led to the failure of the structural members of the building.

4.3 Checking of the design

According to BS 8110 (1985), the sum of the area of Reinforcement in a beam divided by the product of width and depth (A_s/bd) should not exceed 4.0 percent. On the other hand, too little reinforcement is also undesirable; therefore A_s/bd should not be less than 0.25 percent for mild steel or 0.15 percent for high yield steel. Also, when the slab forms the flange of a T or L – beam, the area of reinforcement in the flange and at right angles to the beam should not be less than 0.3 percent of the longitudinal cross-section of the flange. Using the above, the comparison of the selected beams analyzed with those designed by FUTA ventures are as calculated below:

(i) Beam 6

Beam 6 is a flanged T – beam. Section analysis showed the sufficiency of the following areas of steel: Top Reinforcement of 3Y20 (942mm^2) plus 2Y16 (402mm^2) and bottom reinforcement of 3Y16 (603mm^2) with total area of 1947mm^2 at the support. This Beam as designed by FUTA Ventures has a top reinforcement of 4Y25 (1964mm^2) and bottom reinforcement of 3Y20 (942mm^2) with the total area of 2906mm^2 also at the support.

VERIFICATION: For the steel obtained:

$$(a.) \quad \frac{100A_s}{bd} = \frac{100 \times 1947}{225 \times 425} = 2.04$$

$$= 2.04 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} < 4.0$$

Therefore, the steel percentage is within the limits specified by the code

(b.) FUTA Ventures (steel actually provided):

$$\frac{100A_s}{bd} = \frac{100 \times 2906}{225 \times 425} = 3.04$$

$$\text{So } 3.04 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} < 4.0$$

The steel percentage is therefore within the limits specified by the code

(ii.) Beam 7

Beam 7 is a flanged T – beam. Section analysis showed the sufficiency of the following areas of steel: Top reinforcement of 4Y20 (1257mm²) and bottom reinforcement of 3Y16 (603mm²) with the total area of 1860mm² at the support. This beam as designed by FUTA Ventures has a top reinforcement of 4Y25 (1964mm²) and bottom reinforcement of 3Y20 (942mm²) with the total area of 2906mm² also at the support.

(a.) For the steel obtained:

$$\frac{100A_s}{bd} = \frac{100 \times 1860}{225 \times 425} = 1.95$$

$$\text{So } 1.95 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} < 4.0$$

Therefore the steel percentage is within the limits specified by the code.

(b.) FUTA Ventures (steel actually provided):

$$\frac{100A_s}{bd} = \frac{100 \times 2906}{225 \times 425} = 3.04$$

$$\text{So } 3.04 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} < 4.0$$

The steel percentage is therefore within the limits specified by the code.

(iii.) **Beam 8**

Beam 8 is a flanged T – beam. Section analysis showed the sufficiency of the following areas of steel: Top reinforcement of 3Y20 (942mm²) and bottom reinforcement of 3Y20 (942mm²) with total area of 1884mm² at the support. This beam as designed by FUTA ventures has a top reinforcement of 4Y25 (1964mm²) and bottom reinforcement of 4Y32 (3217mm²) plus 4Y32 (3217mm²) with total area of 8398mm² at the support

VERIFICATION:

(a.) For the steel obtained:

$$\frac{100A_s}{bd} = \frac{100 \times 1884}{225 \times 739} = 1.13 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} < 4.0$$

The steel percentage is within the limit specified by the code

(b.) FUTA Ventures (steel actually provided):

$$\frac{100A_s}{bd} = \frac{100 \times 8398}{225 \times 739} = 5.05 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} \geq 4.0$$

Therefore beam 8 is over-reinforced because the steel percentage provided by FUTA Ventures is higher than the maximum steel percentage allowed for beam as specified by the code.

(iv.) **Beam 9**

Beam 9 is a flanged L – beam. Top reinforcement of 3Y20 (942mm²) and bottom reinforcement of 3Y16 (603mm²) with total area of 1545mm² at the support. This beam as designed by FUTA Ventures has top reinforcement of 4Y25 (1964mm²) and bottom reinforcement of 3Y20 (942mm²) with the total area of 2906mm² also at the support.

VERIFICATION

(a.) For the steel obtained:

$$\frac{100A_s}{bd} = \frac{100 \times 1545}{225 \times 425} = 1.62 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} < 4.0$$

The steel percentage is within the limit specified by the code.

(b.) FUTA Ventures (steel actually provided)

$$\frac{100A_s}{bd} = \frac{100 \times 2906}{225 \times 425} = 3.04 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} < 4.0$$

The steel percentage is within the limit specified by the code.

(v.) **Beam 12**

Beam 12 is a flanged T – beam. Top reinforcement of 3Y20 (942mm²) plus 3Y16 (603mm²) and bottom reinforcement of 3Y16 (603mm²) with total area of 2148mm² at the support. This beam as designed by FUTA Ventures has 4Y25 (1964mm²) reinforcement on top and 3Y16 (603mm²) reinforcement at the bottom with the total area of 2567mm² also at the support.

VERIFICATION

For the steel obtained:

$$\frac{100A_s}{bd} = \frac{100 \times 2148}{225 \times 425} = 2.25 > 0.3$$

$$\text{Also } \frac{100A_s}{bd} < 4.0$$

(b.) FUTA VENTURES (Steel actually provided):

$$\frac{100A_s}{bd} = \frac{100 \times 2567}{225 \times 425} = 2.68$$
$$= 2.68 > 0.3$$

Also $\frac{100A_s}{bd} < 4.0$

The steel provided is within the limit.

(vi.) **Beam 13**

Beam is a flanged T – beam. Section analysis showed the sufficiency of the following areas of steel: Top reinforcement of 3Y16 (603mm²) and bottom reinforcement of 3Y20 (942mm²) with total area of 1947mm² at the mid span. This beam as designed by FUTA Ventures has a top reinforcement of 3Y16 (603mm²) and bottom reinforcement of 3Y16 (603mm²) with the total area of 1206mm² also at the mid span.

VERIFICATION: For the steel obtained

(a.) $\frac{100A_s}{bd} = \frac{100 \times 1947}{225 \times 467} = 1.14 > 0.3$

Also $\frac{100A_s}{bd} < 4.0$

The steel obtained is within the limit

(b.) FUTA Ventures (steel actually provided)

$$\frac{100A_s}{bd} = \frac{100 \times 1206}{225 \times 467} = 0.71 > 0.3$$

Also $\frac{100A_s}{bd} < 4.0$

Therefore the steel percentage is within the limit

(vii.) **Beam 14**

Beam 14 is flanged L – beam. Section analysis showed the sufficiency of the following areas of steel: Top reinforcement of 3Y16 (603mm²) and bottom reinforcement of 3Y16 (603mm²) with total area of 1206mm² at the mid span. This beam as designed by FUTA Ventures has 3Y16 (603mm²) reinforcement at the bottom and 3Y16 (603mm²) at the top with the total area of 1206mm² also at the mid span.

VERIFICATION: For the steel obtained:

(a.) $\frac{100A_s}{bd} = \frac{100 \times 1206}{225 \times 425} = 0.78 > 0.3$

Also $\frac{100A_s}{bd} < 4.0$

bd

The steel percentage is within the limit

(b.) FUTA Ventures (steel actually provided)

Since the area of reinforcement is the same, 100 As/bd

$$\frac{100A_s}{bd} < 4.0$$

The steel percentage is within the limit

(viii.) **Beam 15**

Beam 15 is a flanged L – beam. Section analysis showed the sufficiency of the following areas of steel. Bottom reinforcement of 3Y20 (942mm²) plus 3Y16 (603mm²) and top reinforcement of 2Y16 (402mm²) with total area of 1947mm² at the mid span. This beam as designed by FUTA Ventures has 4Y20 (1257mm²) reinforcement in the bottom and 2Y16 (402mm²) at the top with the total area of 1659mm² also at the mid span.

VERIFICATION: For the steel obtained

$$(a.) \quad \frac{100A_s}{bd} = \frac{100 \times 1947}{225 \times 425} = 1.26 > 0.3$$

$$\text{Also} \quad \frac{100A_s}{bd} < 4.0$$

The steel is within the limit

(b.) FUTA Ventures (steel actually provided)

$$\frac{100A_s}{bd} = \frac{100 \times 1659}{225 \times 425} = 1.07 > 0.3$$

$$\text{Also} \quad \frac{100A_s}{bd} < 4.0$$

The steel percentage is within the limit.

Table 4.15: The summary after checking the selected beams designed by FUTA ventures.

BEAMS SELECTED	REMARKS
6	Normally Reinforced
7	Normally Reinforced
8	Over – Reinforced
9	Normally Reinforced
12	Normally Reinforced
13	Normally Reinforced
14	Normally Reinforced
15	Normally Reinforced

Comments

Most of the beams designed by FUTA Ventures are normally reinforced except beam 8 which is over-reinforced as shown in the Table 4.15 above. As a matter of fact, beam 8 is the most critical of all the Beams in the building. The beam has the longest depth of 575mm and overall length of 24000mm supported at 4m intervals compared with all the other beams. Also, the beam is in pair and spanned almost through the whole length along the centre of the structure. The over-reinforcement of these most essential beams was dangerous and therefore contributed much to the failure and collapse of the building. Therefore it can be concluded that part of the reasons of the collapse of the building is as a result of over-reinforcement of some of the structural members in the design done by FUTA Ventures. (See plate 4.4)



Plate 4.4: Over-reinforcement of beams

4.4 Effect of Over-and Under-Reinforced Beams on Building Structures.

The results and comparison of the checking, analysis and design of some of the beams with the beams designed by FUTA Ventures show that some of the collapsed building beams were over and under reinforced while some of the beams have a normal or balanced reinforcement. However, over and under-reinforcement of beams have an adverse effect on building construction generally.

Balanced design is a design that would result in the simultaneous failure of concrete and steel if failure were to occur. Such a design is the most economical in terms of material usage. Here, the maximum moment of resistance in concrete and steel would be equal. In practice, however, most section or designs are unbalanced, since the dimensions and reinforcement areas must be rounded off to suitable practical values. Non-balanced sections

are of two types. The first one is over-reinforced sections while the second one is under-reinforced sections.

Under-reinforced section will fail in flexural tension while an over-reinforced section will fail in flexural compression. In an under reinforced section, when the stresses in the tension reinforcement attain their yield value, this makes the cracks to open so that the depth of the beam available to resist flexural compression is reduced (the neutral axis moves up the compression face) and final collapse occurs by the crushing of the compression zone. Unlike the Under-reinforced sections which contain two little reinforcement to permit the balanced section to be developed, over reinforced section are those that contain more reinforcement than is required to give a balanced section. Hence, for an over-reinforced section as the applied moment is increased, the concrete attains its ultimate stress value first and by the time reinforcement reaches its yield stress, the concrete is overstressed. If the load is continually increased, since overstress in the concrete occurs earlier than in the reinforcement, any failure is sudden, as the concrete crushes in compression. It is pertinent to note that unlike for the under-reinforced section where the depth of the compression zone is reduced, in an over-reinforced section the depth of the compression zone increase to a value greater than of a balanced section. In an under-reinforced section, less steel material is used and besides since when overstressed and the reinforcement yields, it is still able to support the yield stress, failure here is more gradual. From the above, it can be seen that the over-reinforced section has a depth to the neutral axis greater than that of the balanced section while for under-reinforced sections the depth to the neutral axis is less than the balanced section. In order to determine whether a section is over-or under-reinforced, therefore, it is necessary to calculate the position of both the balanced and the actual neutral axes considering or to compare the total area of reinforcement divided by the product of width and depth with the values specified by BS 8110 (1995).

4.5 Foundation Layout and Arrangement of Columns

Going through the foundation details of the designed drawings provided, the followings are discovered.

- (i.) The Spacing between columns are too large. In some areas, they are at 7m span intervals. In other areas, at the outer walls, the spacing is over 8m span. All these are too large for a framed structure.
- (ii.) In the partition walls on the foundation layout, the spacing between the columns is also too large. It is about 20m intervals. This is also too large for a framed structure.
- (iii.) At the auditorium, from the foundation layout, the spacing between the columns is too large also, it is between 7m to 8m intervals

Generally, the number of columns provided in the foundation layout is too small for a framed structure. Also it was discovered that the structure designed for the collapsed building prepared by FUTA Ventures was for a single storey building only. During the construction, the floors were increased by one to become a 2-storey building causing an overloading to the Foundation base without any modification to the design.

4.6 Effect of Over Loading on Building Structure.

In members which sustain chiefly or exclusively axial compression loads, such as building columns, it is economical to make the concrete carry most of the load. Still, some steel reinforcement is always provided for various reasons. For one, very few members are truly axially loaded, steel is essential for resisting any bending that may exist. For another, if part of the total load is carried by steel with its much greater strength, the cross-sectional dimensions of the member can be reduced, the more the cross sectional dimensions the larger the amount of reinforcement.

In the square column, the four longitudinal bars serve as main reinforcement. They are held in place by transverse small-diameter steel which prevent displacement of the main bars during construction operations and counteract any tendency of the compression- loaded bars to buckle out of the concrete by bursting the thin outer cover. When axial load is applied, the compression strain is the same over the entire cross section and in view of the bonding between concrete and steel is the same in the two materials.

Information on stresses, strains and similar quantities serves chiefly as a tool for determining carrying capacity. The performance of column discussed so far indicates two things. Firstly, in the range of large stresses and strains which precede attainment of ultimate strength and subsequent failure, elastic relationships cannot be utilized. Secondly, the member behaves differently

under fast and under slow or sustained loading and show less resistance to the latter than to the former.

As the ultimate strength which is the maximum load the structure or member will carry is approached, there occurs a redistribution of the relative share of the load resisted, respectively, by concrete and steel, the latter taking an increasing amount. The ultimate load at which the member is on the point of failure consists of the contribution of the steel when it is stressed to the yield point plus that of the concrete when its stress has attained the ultimate strength.

When the tension force in the member is small enough for the stress in the concrete to be considerably below its tensile strength, both steel and concrete behave elastically. However when the load is further increased, the concrete reaches its tensile strength at a stress and strain of the order of one tenth of what it could sustain in compression. At this stage the concrete cracks across the entire cross section. When this happens, it ceases to resist any part of the applied tension force, since, evidently, no force can be transmitted across the air gap in the crack. At any load larger than that which caused the concrete to crack, the steel is called upon to resist the entire tension force. With further increased load, the tensile stress in the steel reaches the yield point. When this occurs, then tension member cease to exhibit small elastic deformations but instead stretch a sizeable and permanent amount at substantially constant load.

This does not impair the strength of the member. Its elongation, however, becomes so large as to render it useless. Therefore the maximum useful strength of a tension member is that force which will just cause the steel stress to reach the yield point.

From the reports collected, the failure of the collapsed building started from the columns. This occurred within the 30 seconds prior the collapse as the columns started twisting, wangling and buckling with the throwing of stones from the existing columns and removal of the cover provided in the columns. According to Mosley and Bungey (1982) short column usually fail by crushing and slender column is liable to fail by buckling. If the failure started from the columns as indicated above, most of the columns must have been wrongly designed. Perhaps, all were designed as short columns. Hence, columns three and four about 25 numbers which should have been designed as slender columns would fail by buckling. We can deduce that part of the reasons of the

collapse is buckling of the columns due to overloading of the few columns provided.

Also, the addition of one floor to the one storey designed during construction would have some effect on the foundation base. Overloading would cause the earth pressure of the foundation base to increase and bending failure of the columns base would occur.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

From this study, the following conclusions are made:

The results of the soil laboratory tests carried out on soil materials collected from Saint Thomas's Church Hall collapsed building site showed that the sub soil material is gravelly sandy soil with bearing capacity in the range 140-260kN/m². Also, the average liquid limit is 26.13%, plastic limit is 14.8%, plasticity index is 11.3% and linear shrinkage is 1.05. Compared with the standard values of liquid limit of the range 25% - 27%, plastic limit 10% - 15% and shrinkage between 0.9 – 1.5%, the inference from the soil tests is that the collapse of the building is not as a result of poor soil material underneath or differential settlement of the foundation.

The overview and checking of the structural design for the collapsed building showed that the design was not properly done. Some of the structural members were over-reinforced which made the building collapse to be sudden under 60 seconds and without giving any prior warning.

The low crushing strength of the concrete rubbles which is 4.9N/mm² is too much below expected value for 1:2:4 mix design standard for concrete after 28 days of curing. The average value should be in the range 21N/mm²-22N/mm². This shows poor quality control arising from the adoption of poor mix ratios and the presence of clay or laterite deposits in the aggregates, poor concrete work arising from poor formwork resulting in reduced cross – section of members and inadequate placing and compaction of concrete resulting in voids and poor bonding between concrete and reinforcement.

Inability of the client, the authority of the Saint Thomas's Church, to engage competent, registered contractors and full time supervising professional Consultants who would have enforced good quality control but preferred quacks, sub contractors and direct labour cheap job which led to bad workmanship and eventual collapse of the building

The mineralogy test on samples of coarse aggregate used for the construction of the collapsed building showed that the aggregate contained certain percentage of impurities like Biotite and muscovite, which are injurious to concrete in building structure. The oxidation of the Iron II present in the

Biotite to Iron III weakened the concrete and reduced the strength. This invariably contributed as part of the reasons for the collapse.

Consequently, the works carried out on the collapse of Saint Thomas's Anglican Church hall, Akure where five people died and properties worth of Five Million Naira was destroyed was caused by poor design, poor workmanship, lack of thorough supervision, overloading through the addition of one floor and defect of the coarse aggregate materials.

5.2 Recommendations

From the conclusion of this study, the followings are recommended:

All clients or building developers should be compelled to comply with approved building plans before the construction of their building and that all building construction works should be well designed and supervised by a registered member of Council for the Regulation of Engineering in Nigeria (COREN), Architects Registration Council of Nigeria (ARCON) and Council of Registered Builders of Nigeria (CORBON). Competent registered Contractors should be employed to supervise construction works or projects.

Regular workshops such as the one by the Nigerian Society of Engineers (NSE) and COREN should be held to improve the professional competence of members.

There should be a law providing heavy penalties for Contractors who fail to have registered professionals in supervisory capacity in major building projects. Systematic inspection of building works should be enforced at the Local Government level and penalties for failure to comply with the building standard regulation should be provided for.

All building construction materials like sand, cement, aggregate, reinforcement bars and particularly foundation soil should be tested before the commencement of any construction. The mineralogy and Alkalinity tests of coarse aggregates should be done to know whether the material contains some percentage of impurities, which are deleterious and injurious to cement and reinforcing rods.

Government should quickly promulgate a national building regulation for the elimination or considerable reduction of the incessant collapse of

buildings in Nigeria and quick response committee for investigating incidents of building collapse should be formed.

The law governing all approved structural details of buildings, materials and effective supervision by the local Town Planning Authorities should be enforced and not be compromised. Section 2(1) of the Ondo State Building and sub division regulations 1984 demands that no building or structure or any part thereof should be erected, converted, altered or enlarged unless a development permit has been obtained by the owner or his agent from the ministry. This provision, which is also contained in section 30(1) of the Nigerian Urban and Regional Planning Decree 88 of 1982, should be enforced. The Government should fund this board established through the decree so as to ensure adequate monitoring of the building approval and construction in our society.

Government should exercise leadership in protecting and enhancing the quality of all the buildings by encouraging regular maintenance so as to sustain human life.

All Government functionaries and building developers should be properly trained and encouraged to always give construction of large scale buildings to competent and registered Contractors who will also be supervised by a registered structural engineering Consultant and Architect who preferably have designed such projects.

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Analysis and Design of Selected Beams

DESIGN DATA

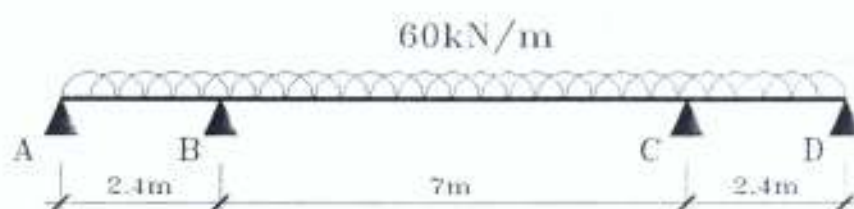
Live load on slab	=	4.0kN/m ² (church load)
Allowable bearing stress	=	160kN/m ²
Partition and finishes	=	1.5 kN/m ²
Density of reinforced concrete	=	24 kN/m ²
Characteristic strength (f_y)	=	250 N/mm ² (mild steel)
Characteristic strength (f_y)	=	460 N/mm ² (high yield)
Code of practice (cp)	=	BS 8110 Part 1:1985

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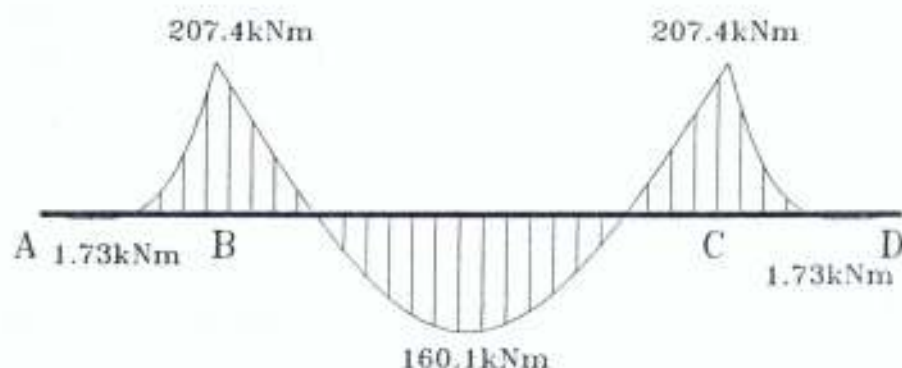
REF BS 8110	CALCULATION	OUTPUT
	Loading of the Slab Slab loading = $0.175 \times 24 = 3.6 \text{ kN/m}^2$ Partition and finishes = 1.5 kN/m^2 Dead load = $3.6 + 1.5 = 5.1 \text{ kN/m}^2$ Imposed load = 4.0 kN/m^2 Design load = $1.4 (5.1) + 1.6 (4.0)$ = 13.54 kN/m^2 $\approx 14 \text{ kN/m}^2$ Design load of the slab = 14 kN/m^2	$d=149\text{mm}$ Slab Design Load $=14\text{kN/m}^2$

REF BS 8110	CALCULATION	OUTPUT
	(a) Analysis of Beam No.6 Beam rib = $225\text{mm} \times 325\text{mm}$ slab Thickness = 175mm Depth of beam = 500mm Effective depth assuming 25mm and cover of 20mm reinforcement $d = 500 - 20 - 10 = 470\text{mm}$ Beam center = 3.5m Loading Self weight = $0.225 \times 0.325 \times 24 = 1.755\text{kN/m}$ Finishes = $2 \times 0.325 \times (20 \times 0.02) = 0.26\text{kN/m}$ Dead load from slab = $3.5 (5.1) = 17.85\text{kN/m}$ Dead load = 26.465kN/m Imposed load = $3.5 \times 4 = 14\text{kN/m}$ Design load = $1.4 \times 26.465 + 1.6 \times 14$ = 59.451kN/m $\approx 60\text{kN/m}$	$d=470\text{mm}$ Design Load $=60\text{kN/m}$

REF BS 8110	CALCULATION	OUTPUT
	Span Stiffness 2.4m span = $\frac{1}{2.4} \times 0.75 = 0.313$ 7.0m span = $\frac{1}{7} = 0.143$ Distribution factors $\frac{0.313}{0.143 + 0.313} = \frac{0.313}{0.456} = 0.69$ $\frac{0.143}{0.143 + 0.313} = \frac{0.143}{0.456} = 0.31$	



Distribution Factor	0	0.69	0.31	0.31	0.69	0
FEM	0	43.2	-245	+245	-43.2	0
DIST.		+139.24	+62.56	-62.56	-139.24	
CO			-31.28	+31.28		
DIST.		+21.58	+9.7	-9.7	-21.58	
CO			-4.9	+4.9		
DIST.		+3.38	+1.52	-1.52	-3.38	
Final Support Moment	0	207.4	-207.4	+207.4	-207.4	



REF BS8110	CALCULATION	OUTPUT
	Longitudinal Reinforcement Beam depth = 5000mm Effective depth = 500 - 30 = 470mm Effective width = $\frac{225 + 0.70 \times 4000}{5}$ b = 785mm²	
	Support Reinforcement Maximum support moment = 207.4kN/m $\frac{M}{bd^2} = \frac{207.4 \times 10^6}{225 \times 470^2} = 4.17\text{N/mm}^2$ From design chart $f_y = 425\text{N/mm}^2$ grade 25 concrete, the $\frac{M}{bd^2}$ value Falls above the curve hence, the $\frac{100 A_s}{bd}$ not available. A thicker Beam has to be selected. Try Beam depth of 575mm d = 575 - 30 = 545mm $\frac{M}{bd^2} = \frac{207.4 \times 10^6}{225 \times 545^2} = 3.13$	

Design Chart No. 2	$\frac{100 A_s}{bd} = 1.05$ $A_s = \frac{1.05 \times 225 \times 545}{100}$ $= 1288\text{mm}^2$ Provide 3Y20 (943mm ²) 2Y16 (402mm ²) 1345mm ²	Support 3Y20 Top 2Y16 Top
REF BS8110	CALCULATION	OUTPUT
Design Chart No. 2	Span Reinforcement Maximum span moment = 160.1kN/m $\frac{M}{bd^2} = \frac{160.1 \times 10^6}{785 \times 545^2} = 0.68$ $\frac{100 A_s}{bd} = 0.108$ Too low. Provide minimum reinforcement $A_s = \frac{0.150 \times 785 \times 545}{100}$ 641mm^2 Provide 3Y20 (943mm ²)	Mid Span 3Y20 Bottom Mid Span 4Y20 Bottom
3.4.6.3 Table 3.10	Check Deflection Basic span/effective depth ratio = 20.8 Modification factor for 0.6 Tension reinforcement = 1.14 $\frac{\text{Allowable span}}{\text{Effective depth ratio}} = \frac{26 \times 1.14 \times 545}{1000}$ $12.92\text{m} > 7.0\text{m}$ Adequate	
3.4.4.4	Moment of Resistance $M_r = 0.15 f_{cu} bd^2$	

$$= 0.15 \times 25 \times 1000 \times 0.785 \times 0.54^2$$

$$= 899 \text{ kN/m} > 160.1 \text{ kN/m}$$

Adequate

Shear Reinforcement

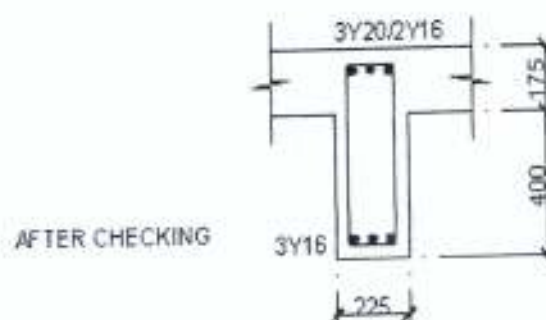
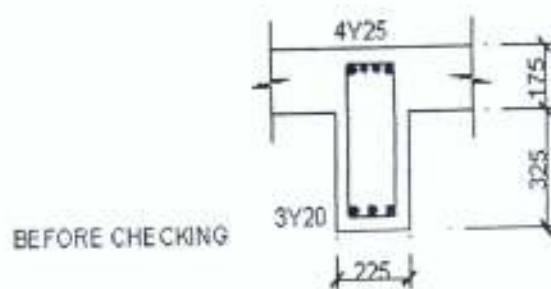
Maximum shear force $V = 210 \text{ kN}$

$210 \text{ kN} < 251 \text{ kN}$ of Beam 12

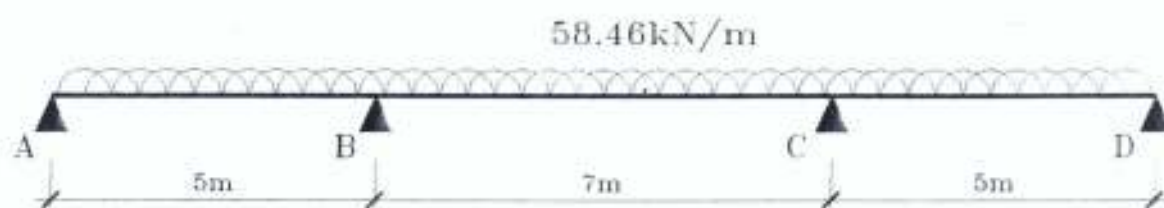
Provide R10 at 225 shear reinforcement

Shear
Reinforcement
R10-225

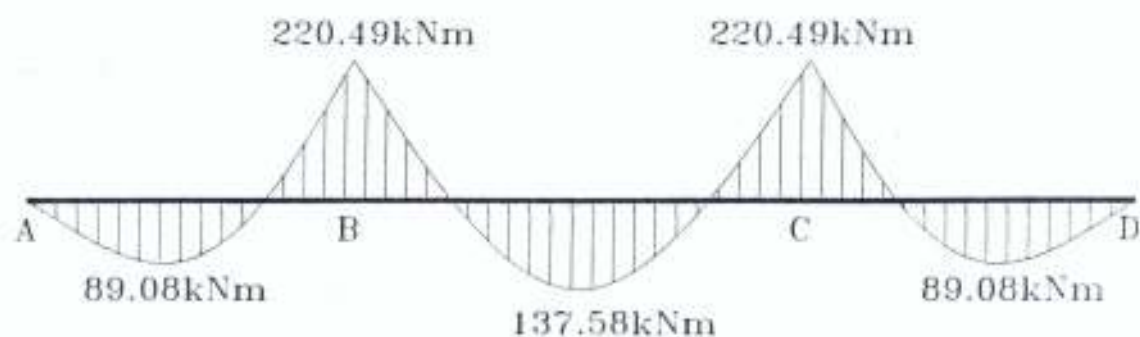
Sections of Beam 6 at the Support



REF BS 8110	CALCULATION	OUTPUT
	(b) The Analysis and design of Beam No.7 Beam centres = 4m Loading Self weight of beam = 2.46kN/m Load from floor Slab = $14 \times 4 = 56\text{kN/m}$ Design load = $2.46 + 56 = 58.46\text{kN/m}$ = 58.46kN/m	Design Load =58.4kN/m



D. F	0	0.52	0.48	0.48	0.52	0
F.E.M	0	182.69	-238.71	+238.71	-182.69	
DIST.		+29.13	+62.89	-26.89	-29.13	
CO			-13.45	+13.45		
DIST.		+6.99	+6.46	-6.46	-6.99	
CO			-3.23	+3.23		
DIST.		+1.68	+1.55	-1.55	-1.68	
Final Support Moment	0	220.49	220.49	220.49	220.49	



REF BS 8110	CALCULATION	OUTPUT
	Bending Moment Diagram Longitudinal Reinforcement Support Reinforcement Maximum support moment = 220.50 kNm $\frac{M}{bd^2} = \frac{220.5 \times 10^6}{225 \times 470^2}$ $= 4.4 \text{ N/mm}^2$ The value of M/bd^2 is above what could be read from the design chart. Hence use a thicker beam. Try a beam of 550mm thick $d = 550 - 20 - 10 = 520 \text{ mm}$ $\frac{M}{Bd^2} = \frac{220.5 \times 10^6}{225 \times 520^2}$ $= 3.62 \text{ N/mm}^2$ Design Chart $\frac{100 A_s}{bd} = 1.14$ No. 2 $A_s = 0.0114 \times 225 \times 520$ $= 1334 \text{ mm}^2$ Provide 4Y20 (Top) $A_s = 1570 \text{ mm}^2$	

REF BS 8110	CALCULATION	OUTPUT
	Span Reinforcement Maximum span moment = 137.6kNm Effective span = $\frac{255 \times 0.7 \times 4000}{5}$ = 785mm $\frac{M}{bd^2} = \frac{137.6 \times 10^6}{785 \times 520^2} = 0.65\text{N/mm}^2$ $\frac{100 A_s}{bd} = 0.18$ $A_s = 0.0018 \times 785 \times 520$ = 735mm² Provide 3Y20 Bottom $A_s = 942\text{mm}^2$ Deflection Span/Effective depth ratio = 20.8 $\frac{100 A_s}{bd} = \frac{100 \times 942}{785 \times 520} = 0.15$ Modification factor for 0.25% Tension reinforcement = 1.41 Allowable span = 1.41 x 26 x 520 = 19.06 > 7m Use 550mm deep beam Shear Reinforcement Max shear force = 204.61kN Shear stress = $\frac{204.61 \times 103}{225 \times 520}$ = 1.75N/mm² $V_c = 0.35\text{N/mm}^2$	
Design Chart No.2		
3.4.6.2		
Table 3.10		
Table 3.11		

3.4.5.4
3.4.5.2

$$\frac{A_{sv}}{S_v} = \frac{225 \left\{ \frac{1.75 - 0.35}{0.87 \times 250} \right\}}{1.45} = 1.45$$

For 2 legs of R10 $A_{sv} = 157$

$$S_v = \frac{157}{1.45} = 108\text{mm}$$

The spacing is too close

Try a higher yield bar of same diameter

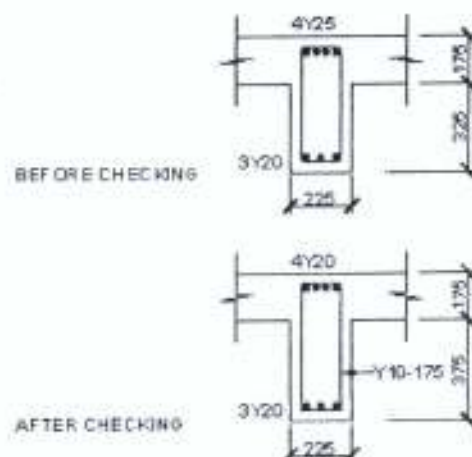
Try Y10

$$A_{sv} = \frac{225 (1.75 - 0.35)}{0.87 \times 460} = 0.79$$

$$S_v = \frac{157}{0.79} = 198.7\text{mm}$$

Provide Y10 – 175 stirrups

Section of Beam 7 at the Support



REF BS 8110	CALCULATION	OUTPUT
	(c) The Analysis and Design of Beam 8 Beam rib = 225mm x 575mm Slab thickness = 175mm d = 750 - 20 - 10 = 720mm Loading Self weight = $0.225 \times 0.575 \times 24 = 3.105\text{kN/m}$ Finishes = $2 \times 0.575 \times (20 \times 0.02) = 0.46\text{kN/m}$ Partition wall = $2.2\text{kN/m}^2 \times 3\text{m} = 6.6\text{kN/m}$ = 6.6kN/m Dead load from slab = $(7/2 + 3/2) \times 5.1 = 25.5$ Dead load = 35.67 Imposed load = $3.5 \times 4 = 14\text{kN/m}$ Design load = $1.4(35.67 \times 1.6 \times 20$ = 50 + 32 = 82kN/m	

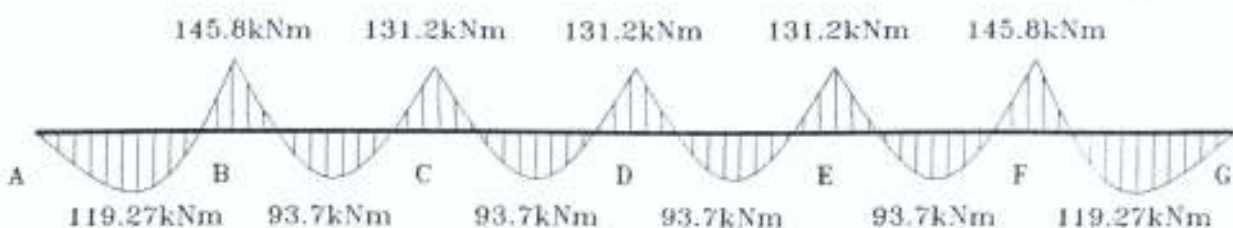
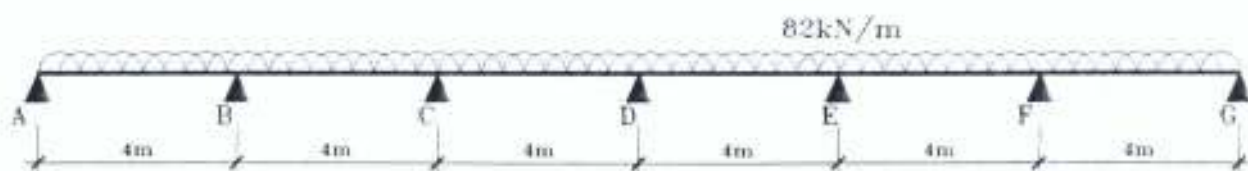


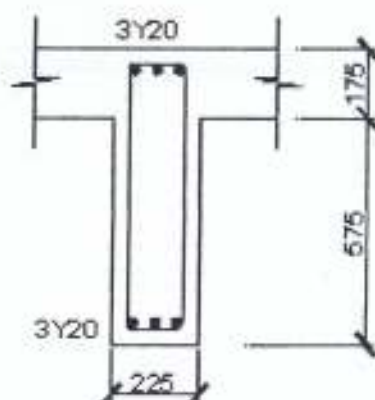
Table 3.6 3.4.3		A	Mid	B	Mid	C	Mid	D	Mid
	Moment	0	119.27	145.8	93.7	131.2	93.7	131.2	93.7
	Shear	147.6	-	196.8	-	180.4	-	180.4	-

	E	Mid	F	Mid	G
Moment	131.2	93.7	145.8	119.27	0
Shear	180.4	-	196.8	-	147.6

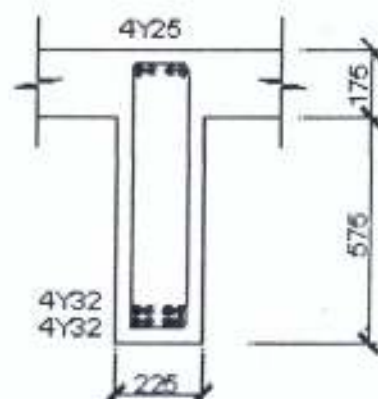
REF BS 8110	CALCULATION	OUTPUT
	Support Reinforcement (Top) Maximum moment = 145.8kN/m Beam size = 225mm x 750mm d = 720mm $\frac{M}{bd^2f_{cu}} = \frac{145.8 \times 10^6}{225 \times 720^2 \times 25}$ $= 0.05 < 0.15$ Design as singly reinforced beam $\frac{M}{bd^2} = \frac{145.8 \times 10^6}{225 \times 720^2}$ $= 1.18 \approx 1.2$ $A_s = \frac{0.42 \times 225 \times 739}{100}$ $= 698.4 \approx 6$ $= 700\text{mm}^2$ Provide 3Y20 (942mm²) Top	Support Reinforcement Top 3Y20

REF BS 8110	CALCULATION	OUTPUT
	Mid Span Reinforcement For span AB, BC, CD, DE, EF, and FG Maximum moment = 119.27kN/m Effective width = $225 + \frac{(0.7 \times 3500)}{5}$ = 225 + 350 = 715mm $\frac{M}{bd^2} = \frac{119.27 \times 10^6}{715 \times 720^2} = 0.32$ $A_s = 0.12bd$ too low Provide Minimum Reinforcement $A_s = \frac{0.15 \times 715 \times 720}{100}$ = 772mm² Provide 3Y20 (942mm²) bottom	Provide 3Y20 Bottom
Table 3.10	Deflection Check $\frac{\text{Span}}{\text{Effective depth}} = 20.8$ $\frac{100 A_s}{bd} = \frac{100 \times 942}{225 \times 720}$ = 0.56 Modification factor for 0.56% = 1.105 Allowable span = 1.105 x 20.8 x 720 = 16,985 > 4000 Okay	

After checking



Before checking



REF BS 8110	CALCULATION	OUTPUT
	(d) The Analysis and Design of Beam 9 Beam centres = 2m Loading Self weight = $0.225 \times 0.385 \times 24 = 2.075 \text{ kN/m}$ Finishes = $2 \times 0.385 \times (20 \times 0.02) = 0.308 \text{ kN/m}$ Partition wall = $2.22 \text{ kN/m}^2 \times 3 \text{ m} = 6.6 \text{ kN/m}$ Dead load from slab = $2(5.1) = 10.2 \text{ kN/m}$ Total = 19.18 kN/m Imposed load = $2 \times 4 = 8 \text{ kN/m}$ Design Load = $1.4(19.18) + 1.6(8)$ = 39.65 kN/m = 40 kN/m	Design Load = 40 kN/m

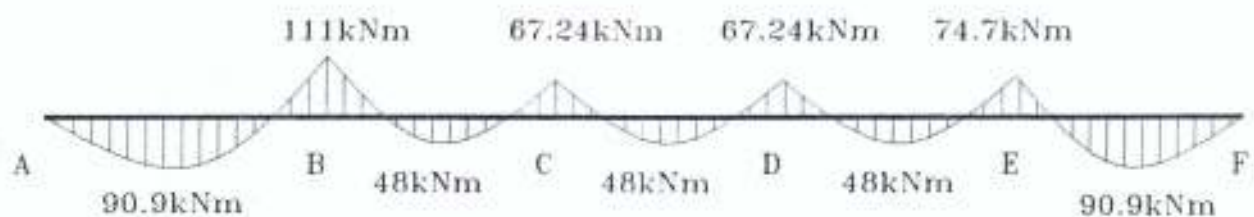
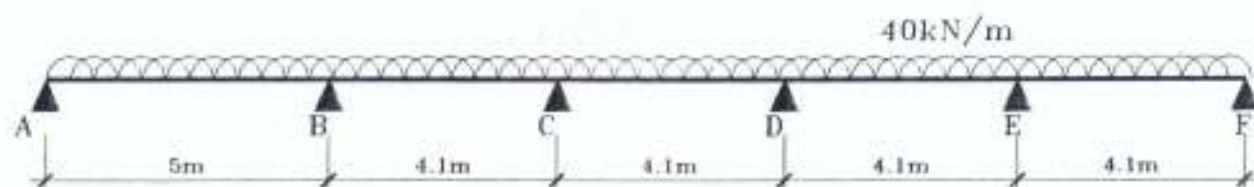
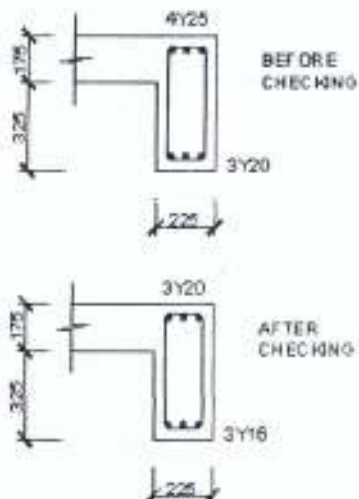
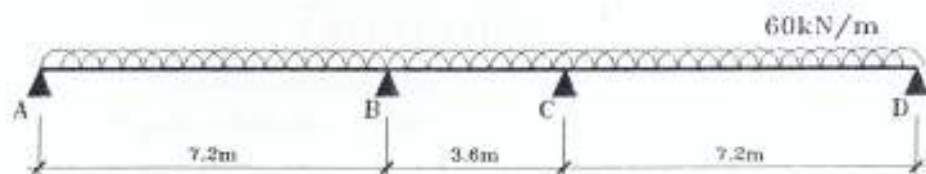


Table 3.6		A	Mid	B	Mid Span	C	Mid Span	D	Mid	E	Mid	F
3.4.3	Moment	0	90.9	111	48	67.24	48	67.24	48	74.7	90.9	0
	Shear	90	-	120	-	90.2	-	90.2	-	120	-	73.8

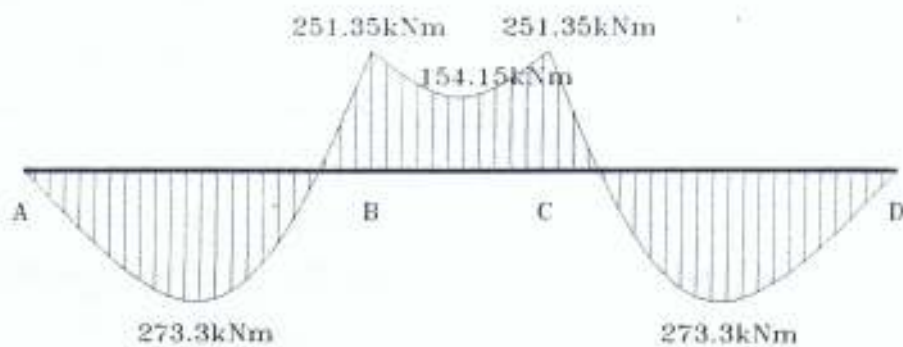
REF BS 8110	CALCULATION	OUTPUT
	Span Reinforcement For A span AB, BC, CD, DE, EF Maximum moment = 50.90kN/m Beam size = 225 x 500mm $d = 500 - 20 - \frac{20}{2} = 470\text{mm}$ Effective width (b) = $225 + \left\{ \frac{0.7 \times 2000}{10} \right\} = 365\text{mm}$ $\frac{M}{bd^2f_{cu}} = \frac{90.9 \times 10^6}{365 \times 470^2 \times 25}$ = 0.05 < 0.15 Design as singly reinforced beam $\frac{M}{bd^2} = \frac{90.9 \times 10^6}{365 \times 170^2} = 1.13$ $A_s = \frac{0.3 \times 365 \times 470}{100}$ = 514mm² Provide 3Y16 (603mm²) bottom	



REF BS 8110	CALCULATION	OUTPUT
	(c) The Analysis and Design of Beam No. 12 Beam centres = 3.5m Loading Self weight = $0.225 \times 0.325 \times 24 = 2.075 \text{ kN/m}$ Finishes = $2 \times 0.385 \times (20 \times 0.02) = 0.308 \text{ kN/m}$ Partition wall = $2.2 \text{ kN/m}^2 \times 3 \text{ m} = 6.6 \text{ kN/m}$ Dead load from slab = $3.5 (5.1) = 17.85 \text{ kN/m}$ Dead load = 26.833 kN.m Imposed load = $3.5 \times 4 = 14 \text{ kN/m}$ Design load = $1.4 (26.833) + 1.6 (14)$ = $59.966 \approx 60 \text{ kN/m}$	Design Load = 60 kN/m
REF BS 8110	CALCULATION	OUTPUT
	Beam depth = 500mm (from the drawing) Effective depth assuming 20mm diameter bar and 25mm cover $d = 500 - 10 - 20 = 470 \text{ mm}$ Effective width of beam $b = 225 + \frac{3500 \times 0.7}{5}$ = 715mm	



D. F	0	0.27	0.73	0.73	0.27	0
FEM	0	+388.8	-64.8	+64.8	-388.8	
DIST.		-87.48	-236.52	+236.52	+87.48	
CO			+118.26	-118.26		
DIST.		-31.98	-86.33	+86.33	+31.98	
CO			+43.17	-43.17		
DIST.		-11.66	-31.51	+31.51	+11.66	
CO			+15.76	-15.76		
DIST.		-4.26	-11.50	+11.50	+4.26	
CO			+5.75	-5.75		
DIST.		-1.55	-4.20	+4.20	+1.55	
CO		-0.57	-1.53	+1.53	+0.57	
Final Support Moment	0	251.35	-251.35	+251.35	-251.35	



REF BS 8110	CALCULATION	OUTPUT
Design Chart No. 2	Support Reinforcement	
	Support moment = 251.35kN/m	
	$\frac{M}{bd^2} = \frac{251.35 \times 10^6}{225 \times 470^2} = 5.05$	
	5.05 N/mm ² is greater than what could be checked up in the chart.	
	Hence the beam depth needs to be increased.	
	Try A beam depth of 600mm	
	Effective depth = 600 – 30	d=575mm
	= 570mm	
	$\frac{M}{bd^2 f_{cu}} = \frac{251.35 \times 10^6}{225 \times 570^2 \times 25} = 0.14$	
	0.14 < 0.15	
Design Chart No. 2	Design as singly reinforced beam	
	$\frac{M}{bd^2} = \frac{251.35 \times 10^6}{225 \times 570^2} = 3.5$	
	$\frac{100 A_s}{bd} = 1.18$ for $f_{cu} = 25\text{N/mm}^2$	
	$A_s = \frac{1.18 \times 225 \times 565}{100}$	
	= 1500mm ²	
	Provide 3Y20 (943mm ²)	
	3Y16 (603mm ²)	Provide
	$A_s = 3Y16$ (943mm ²)	3Y20
	$A_s = 1546\text{mm}^2$	3Y16
	Try a beam of 575mm thick	Top
Design Chart	Effective depth = 575 – 30	
	= 545mm	
	$\frac{M}{bd^2} = \frac{251.35 \times 10^6}{225 \times 545^2} = 3.83$	
Design Chart	$\frac{100 A_s}{bd} = 1.32$	



REF BS 8110	CALCULATION	OUTPUT
No. 2	$A_s = \frac{1.32 \times 225 \times 545}{100}$ $= 1604\text{mm}^2$ Try a beam of 550mm thick effective depth = $550 - 30 = 520\text{mm}$ $\frac{M}{bd^2} = \frac{251.35 \times 10^6}{225 \times 520^2} = 4.21\text{N/mm}^2$ The 4.21 is above what could be read from the design chart. Hence a thicker beam required. ∴ Use 575mm depth beam $A_s = 1546\text{mm}^2$ Provide 3Y20, 3Y16 $A_s = 943 \times 603$ $= 1546\text{mm}^2$	Provide 3Y20, 3Y16 Top
Design Chart No. 2	Span Reinforcement Max. Span moment = 273.3kNm Effective width = $(0.7 \times \frac{3500}{5}) + 225$ $= 715\text{mm}$ $\frac{M}{bd^2} = \frac{273.3 \times 10^6}{715 \times 545^2} = 1.28$ $\frac{100 A_s}{bd} = 0.34$ $A_s = \frac{0.34 \times 715 \times 545}{100}$ $= 1324\text{mm}^2$ Provide 3Y25 Bottom $A_s = 1472\text{mm}^2$	b=715mm d=545mm

Table
3.10**Check Deflection**

$$\frac{\text{Basic span}}{\text{Effective depth}} = 20.8$$

Modification factor for 1.2

$$\text{Tension reinforcement} = 0.94$$

$$\frac{\text{Allowable span}}{\text{Effective depth}} = 20.8 \times 0.94$$

$$\text{Max. Span} = \frac{20.8 \times 0.94 \times 545}{100}$$

$$= 10.6\text{m} > 7.2\text{m}$$

Selected beam section adequate.

Moment of Resistance

$$\begin{aligned} M_r &= 0.15 f_{cu} b d^2 \\ &= 0.156 \times 25 \times 715 \times 0.54^2 \times 1000 \\ &= 819 \text{ kN/m} > 273.3 \text{ kN/m} \end{aligned}$$

Okay

Use 3Y25 Bottom

$$A_s = 1472 \text{ mm}^2$$

Shear Reinforcement

$$\text{Max. Shear Force } V = 251 \text{ kN}$$

$$\text{Max. Stress } (V) = \frac{V}{bd}$$

$$= \frac{251 \times 10^3}{715 \times 545} = 0.6$$

$$\frac{100 A_s}{bd} = \frac{100 \times 1472}{715 \times 545} = 0.37$$

3.4.5.4

$$V_c = 0.41 \text{ N/mm}^2$$

3.4.5.2

$$\frac{A_{sv}}{S_v} = \frac{b(v - v_c)}{0.87 f_{yv}}$$

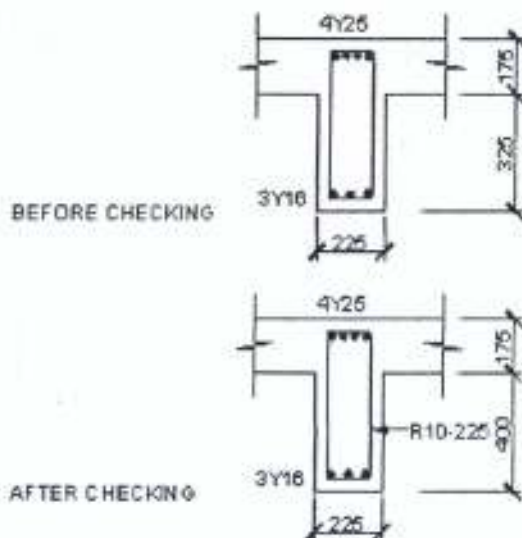
$$\begin{aligned} &= \frac{715(0.6 - 0.41)}{0.87 \times 250} \\ &= 0.69 \end{aligned}$$

Use

3Y25

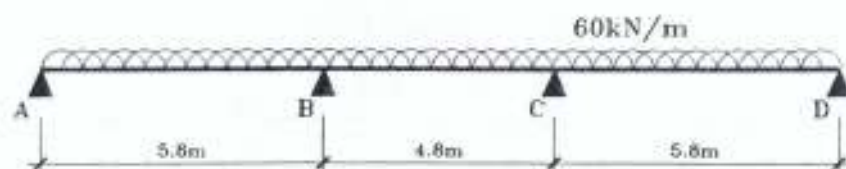
As=1472

REF BS 8110	CALCULATION	OUTPUT
	For R10, $A_s = 157\text{mm}^2$ Hence $S_v = \frac{157}{0.69} = 228\text{mm}^2$ Provide R10 @225 stirrups	

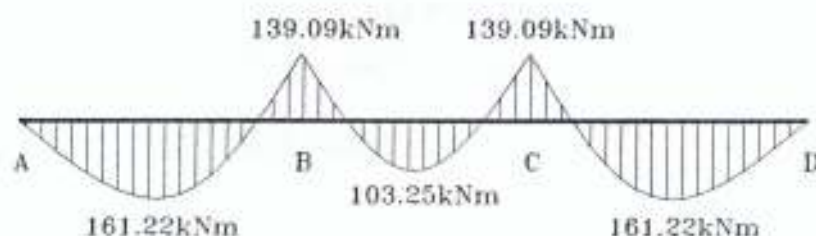


Section of Beam 12 at the Support

REF BS 8110	CALCULATION	OUTPUT
	(f) The Analysis and Design of Beam No. 13 Beam rib = 225mm x 325,, Slab = 175mm $d = 500 - 20 - 10 = 470\text{mm}$ Loading Self weight = $0.225 \times 0.325 \times 24 = 2.075\text{kN/m}$ Finishes = $2 \times 0.385 \times (20 \times 0.02) = 0.308\text{kN/m}$ Partition wall = $2.2\text{kN/m}^2 \times 3\text{m} = 6.6\text{kN/m}$ Dead load from slab = $3.5(5.1) = 17.85\text{kN/m}$ Dead load = 26.833kN/m Imposed load = $3.5 \times 4 = 14\text{kN/m}$ Design load = $1.4(26.833) \times 1.6(14)$ = 59.966 ≈ 60kN/m	



Stiffness	$\frac{3}{4} \frac{1}{L}$	$\frac{1}{L}$	$\frac{3}{4} \frac{1}{L}$
K	$\frac{3}{4} \times \frac{1}{5.8} = 0.13$	$\frac{1}{4.8} = 0.21$	$\frac{3}{4} \times \frac{1}{5.8} = 0.13$
Distribute Factor	0.38	0.62 0.62	0.38
FEM	0 -16.20	+115.2 -115.2	+168.20 0
Balance	+20.14	+32.86 -32.86	-20.14
Carry over		-16.43 +16.43	
Balance	+6.24	+10.19 -10.19	6.24
Carry over		-5.095 +5.095	
Balance	+1.94	+3.16 -3.16	1.98
Carry over		-1.58 +1.58	
Balance	+0.60	+0.98 -0.98	0.60
Carry over		-0.49 +0.49	
Balance	+0.19	+0.30 -0.30	0.19
Total support Moment	-139.09	+139.09 139.09	+139.09



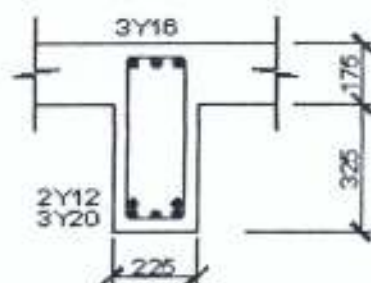
REF BS 8110	CALCULATION	OUTPUT
	Mid span moment = 161.22kNm Support moment = 139.09kNm Design of Mid Span Reinforcement $b = 225 + \left\{ \frac{4000}{5} \times 0.7 \right\}$ $b = 785\text{mm}$ $\frac{M}{bd^2f_{cu}} = \frac{161.22 \times 10^6}{785 \times 470^2 \times 25}$ $= 0.03 < 0.15$ Design as singly reinforced beam $\frac{M}{bd^2} = \frac{161.22 \times 10^6}{785 \times 470^2} = 0.92$ $A_s = \frac{0.30 \times 785 \times 470}{100}$ $= 1106.00\text{mm}^2$ Provide 3Y20, 2Y12 (942 + 226) A_s bottom Reinforcement (1168mm²) Support Reinforcement Moment = 139.09kNm $\frac{M}{bd^2} = \frac{139.09 \times 10^6}{225 \times 470^2}$ $= 2.80$	

REF BS 8110	CALCULATION	OUTPUT
3.4.6.3 Table 3.10	$A_s = \frac{0.88 \times 225 \times 470}{100}$ $= 924 \text{mm}^2$ Provide 3Y20 (942mm²) Top Check Deflection <u>Basic span</u> = 20.8 Effective depth Modification factor for 1.05 Tension reinforcement = 0.96 Allowable span/effective depth ratio $= 20.8 \times 0.96$ Max span = $\frac{20.8 \times 0.96 \times 470}{100}$ $= 9.4\text{m} > 5.8\text{m}$ Selected beam section adequate Moment of Resistance $M_r = 0.156 f_{cu} b d^2$ $= 0.156 \times 25 \times 0.225 \times 470 \times 1000$ $= 184 \text{kN/m} > 139 \text{kN/m}$	

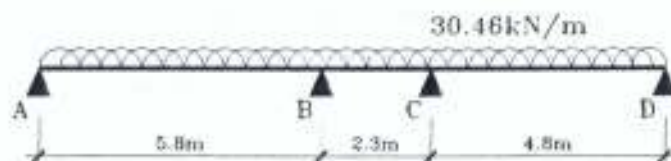
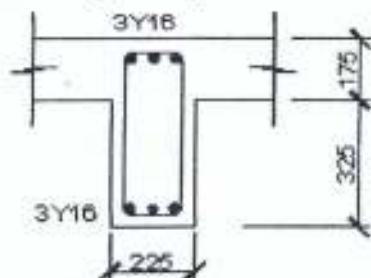
- Use 3Y20, 2Y12 (Bottom) (mid span) and 3Y20 (Top) support

Section of Beam 13 at Mid Span

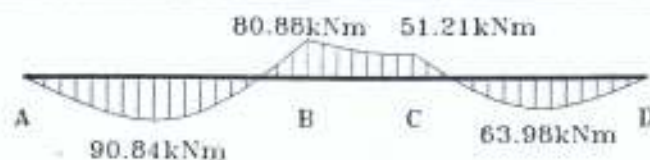
AFTER CHECKING



BEFORE CHECKING



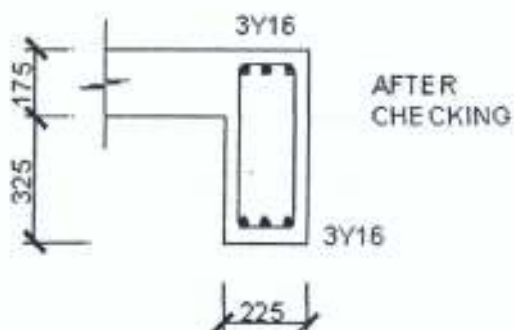
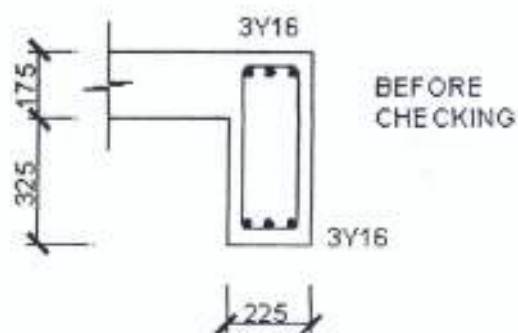
DF	0	0.23	0.77	0.73	0.27	0
FEM	0	128.08	-13.43	13.43	-87.72	0
DIST		-26.37	-88.28	+54.23	+20.06	
CO			27.12	-44.14		
DIST		-6.24	-20.88	+32.22	+11.92	
CO			16.11	-10.44		
DIST		-3.71	-12.4	+7.62	+2.82	
CO			3.81	-6.2		
DIST		-0.88	-2.93	-453	+1.67	
Final Support Moment	0	207.4	-207.4	+207.4	-207.4	



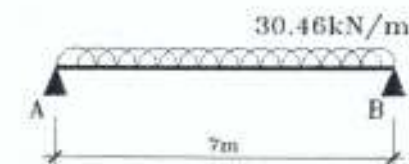
REF BS 8110	CALCULATION	OUTPUT
	(g) <u>The Analysis and Design of Beam 14</u> Beam centres = 2m <u>Loading</u> Design load from floor slab = 14kN/m^2 Self weight of beam = $0.325 \times 24 \times 0.225 \times 1.4$ = 2.46kN/m Design load = $2.46\text{kN/m} + (2 \times 14)\text{kN/m}$ = 30.46kN/m	
Design Chart 2	Longitudinal Reinforcements Support Maximum moment = 80.88kNm Effective depth = $500 - 20 - 10$ $d = 470\text{mm}$ $b = 225\text{mm}$ $\frac{M}{bd^2} = \frac{80.88 \times 10^6}{225 \times 470^2} = 1.63\text{N/mm}^2$ $\frac{100 A_s}{bd} = 0.43$ $A_s = 0.0043 \times 225 \times 470$ = 454mm^2 Provide 3Y16, $A_s = 603\text{mm}^2$	$d=470\text{mm}$ $b=225\text{mm}$ Provide 3Y16 Top
	Span Reinforcement Max. Span moment = 90.84 Effective width = $\frac{225 \times 0.7 \times 2000}{10}$ = 365mm	

REF BS 8110	CALCULATION	OUTPUT
Design Chart 2	$\frac{M}{bd^2} = \frac{90.84 \times 10^6}{365 \times 470^2} = 1.13 \text{ N/mm}^2$ $\frac{100 A_s}{bd} = 0.34$ $A_s = 0.0034 \times 365 \times 470$ $= 584 \text{ mm}^2$ Provide 3Y16 Bottom, $A_s = 603 \text{ mm}^2$	Provide 3Y16 Bottom
3.4.5.4	Shear Reinforcement Max shear force (v) = 102.3kN Shear stress $\frac{v}{bd} = \frac{102.3 \times 10^3}{225 \times 470}$ $= 0.97 \text{ N/mm}^2$ $\frac{100 A_s}{bd} = \frac{100 \times 603}{225 \times 470} = 0.57$ $V_c = 0.52$ $\frac{A_{sv}}{S_v} = \frac{b(v - v_c)}{0.87 f_{yv}}$ $= \frac{225 (0.97 - 0.52)}{0.87 \times 250}$	
3.4.5.2	$\frac{A_{sv}}{S_v} = 0.47$ For 2 legs of 10mm rod, $A_{sv} = 157 \text{ mm}^2$ Hence $S_v = \frac{157}{0.47}$ $= 334 \text{ mm}$ Provide R10 – 250 $\frac{A_{sv}}{S_v} = \frac{157}{250} = 0.63$ $0.63 > 0.47$ Okay	Provide R10-250 Links

REF BS 8110	CALCULATION	OUTPUT
Table 3.10	Deflection $\frac{\text{Span}}{\text{Effective depth ratio}} = 20.8$ $\frac{100 A_s}{bd} = \frac{100 \times 603}{365 \times 470}$ $= 0.35$	
Table 3.11	Modification factor for 0.35% Tension reinforcement = 1.28 Allowable span = $1.28 \times 20.8 \times 470$ = 12.54m > 5.8m Okay	



REF BS 8110	CALCULATION	OUTPUT
	4.4.8 The Analysis and Design of Beam 15 Beam centres = 2.0m Loading Self weight of beam = 2.46kN/m Load from floor slab = $2 \times 14 = 28\text{kNm}$ Design load = $2.46 + 28 = 30.46\text{kN/m}$	



SHEAR FORCE DIAGRAM



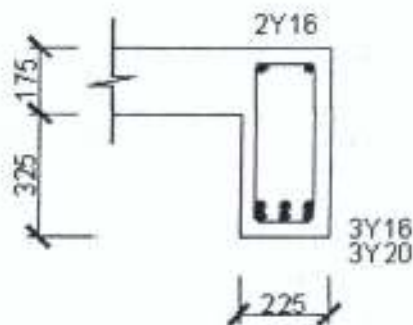
BENDING MOMENT DIAGRAM

REF BS 8110	CALCULATION	OUTPUT
	Shear force reaction $= \frac{30.46}{2} \times \frac{7}{2}$ $= 106.61\text{kN}$ Span moment = Area enclosed by the shear force diagram $= \frac{106.61}{2} \times 3.5$ $= 186.57\text{kNm}$	

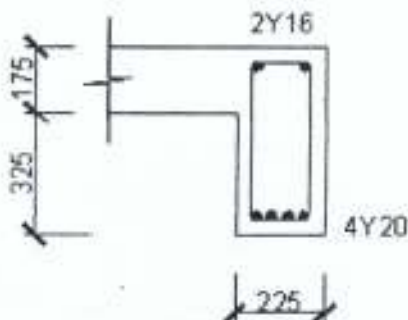
REF BS 8110	CALCULATION	OUTPUT
Design Chart 2	Longitudinal Reinforcement Span Max. Span moment = 186.57kNm Effective width = $\frac{225 \left\{ \frac{0.7 \times 2000}{10} \right\}}{10}$ = 365mm $\frac{M}{bd^2} = \frac{186.57 \times 10^6}{365 \times 470^2}$ = 2.31N/mm² $\frac{100 A_s}{bd} = 0.67$ $A_s = 0.0067 \times 365 \times 470$ = 1150mm² Provide 3Y20, 2Y12 Bottom $A_s = 1169\text{mm}^2$	d=470mm Provide 4Y20 Bottom
3.10 3.4.6.3 Table 3.11	Deflection $\frac{\text{Span}}{\text{Effective depth ratio}} = 16.0$ $\frac{100 A_s}{bd} = \frac{1169 \times 100}{365 \times 470}$ = 0.70 Modification factor for 0.70 Tension reinforcement = 1.0 Allowable span = 1.0 x 20 x 470 = 9.4m > 7m Adequate	

REF BS 8110	CALCULATION	OUTPUT
3.4.5.4	Shear Reinforcement Max. shear force = 107kN Shear stress = $\frac{107 \times 10^3}{225 \times 479}$ = 1.01N/mm² $V_c = 0.7\text{N/mm}^2$ $A_{sv} = \frac{b(v - v_c)}{0.887f_{yv}} = \frac{225(1.01 - 0.57)}{0.87 \times 250} = 0.46$	
3.4.5.2	2 legs of R10, $A_{sv} = 157\text{mm}^2$ $S_v = \frac{157}{0.47} = 334\text{mm}^2$ Provide R10 – 250	

Before checking



After checking



APPENDIX TWO

DETERMINATION OF THE LIQUID AND PLASTIC LIMITS OF A SOIL. DATA AND CALCULATION SHEET

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED BUILDING SITE , ARAKALE AKURE, **JOBS:** POST GRADUATE PROJECT
SAMPLE NO: ONE

DATE: 20/2/2003

Moisture content test to be marked m.
liquid limit test to be marked LL.
Plastic limit test to be marked P.L.
Air-dry moisture content to be marked A.

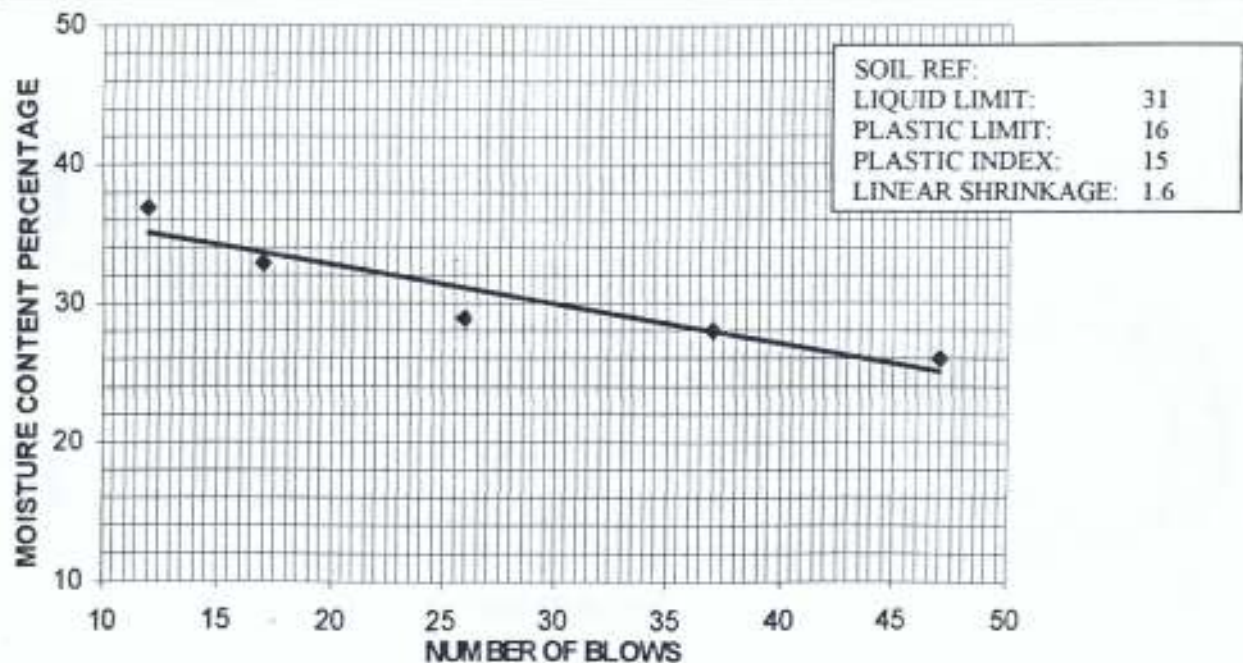
No of blows refers to liquid limit
determination

Moisture contents

Wt. Of moisture

Wt. Of dry soil X 100m%

TEST NO	1	2	3	4	5	6	7
Type of test	LL	LL	LL	LL	LL	PL	PL
No of blows	12	17	26	37	47		
Container No	17 ^A	5	G	61	Q	8	2
Wt. Of wet soil + Container - gm	33	33	34	32	31.5	26.5	25.5
Wt. of dried soil + Container - gm	30	30.5	31.5	30	29.5	26	25
Wt. Container - gm	22	23	23	23	22	23	22
Wt. of moisture - gm	3	2.5	2.5	2	2	.5	.5
Wt. of dry soil - gm	8	7.5	8.5	7	7.5	3	3
Moisture content (m) - %	37	33	29	29	26	16	16



DETERMINATION OF THE LIQUID AND PLASTIC LIMITS OF A SOIL DATA AND CALCULATION SHEET

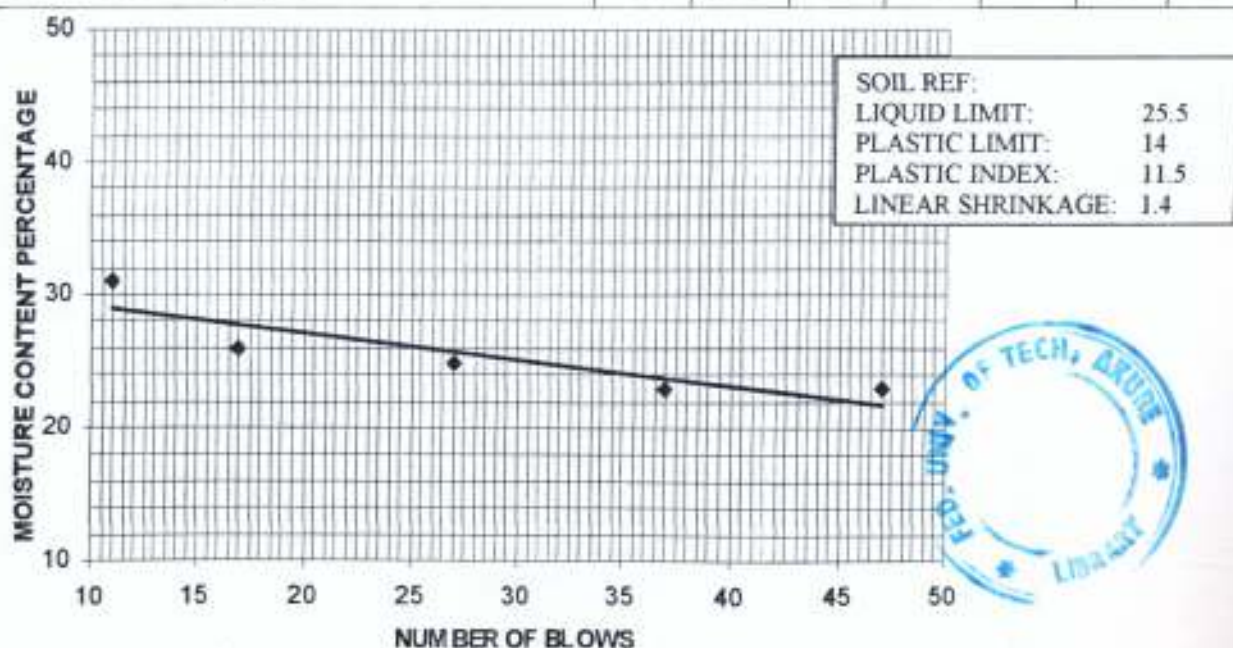
SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED BUILDING SITE, ARAKALE AKURE **JOBS:** POST GRADUATE PROJECT
SAMPLE NO: TWO

DATE: 20/2/2003

Moisture content test to be marked m.
liquid limit test to be marked LL.
Plastic limit test to be marked P.L.
Air-dry moisture content to be marked A.

No of blows refers to liquid limit determination
Moisture contents
 $\frac{\text{Wt. Of moisture}}{\text{Wt. Of dry soil} \times 100\text{m\%}}$

TEST NO	1	2	3	4	5	6	7
Type of test	LL	LL	LL	LL	LL	PL	PL
No of blows	11	17	27	39	47		
Container No	D	L	K	Z	C	A	E
Wt. Of wet soil + Container - gm	49.5	48.5	48	48.5	48.5	42	42
Wt. of dried soil + Container - gm	47	46.5	46	46.5	46.5	41.5	41.5
Wt. Container - gm	39	39	38	38	38	38	38
Wt. of moisture - gm	2.5	2	2	2	2	.5	.5
Wt. of dry soil - gm	8	7.5	8	8.5	8.5	3.5	3.5
Moisture content (m) - %	31	26	25	23	23	14	14



DETERMINATION OF THE LIQUID AND PLASTIC LIMITS OF A SOIL

DATA AND CALCULATION SHEET

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003 **BUILDING SITE** ARAKALE AKURE. **SAMPLE NO:** THREE

Moisture content test to be marked m.

liquid limit test to be marked LL.

Plastic limit test to be marked P.L.

Air-dry moisture content to be marked A.

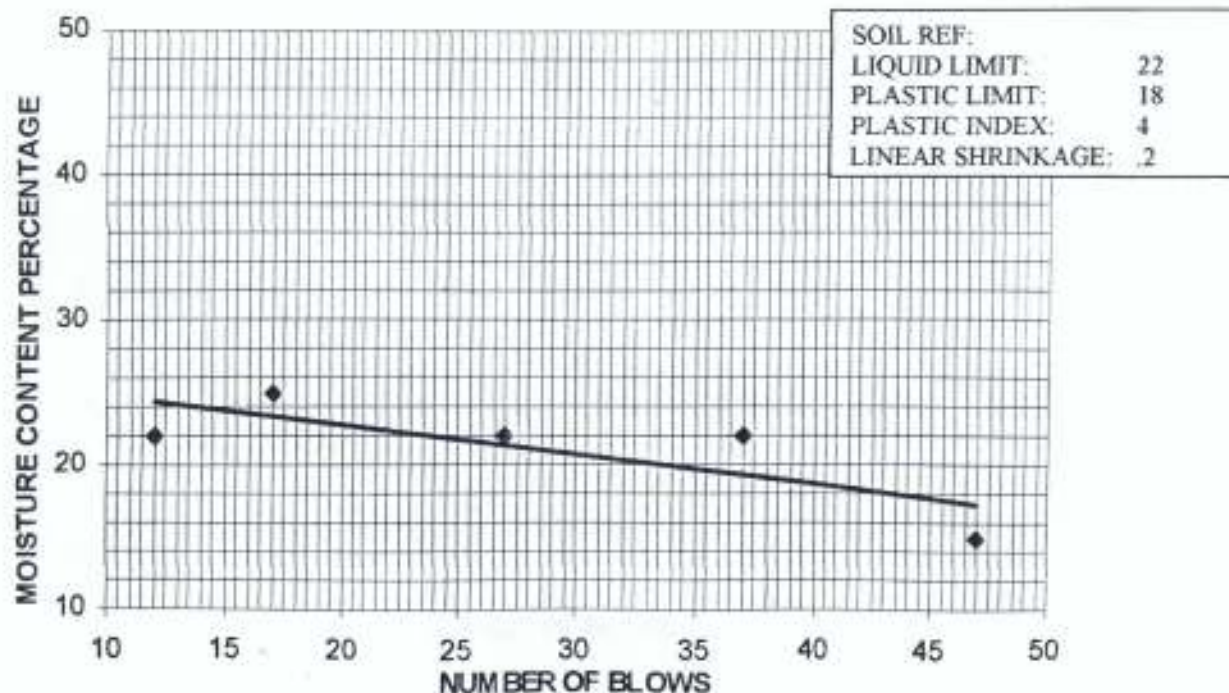
No of blows refers to liquid limit determination

Moisture contents

Wt. Of moisture

Wt. Of dry soil X 100m%

TEST NO	1	2	3	4	5	6	7
Type of test	LL	LL	LL	LL	LL	PL	PL
No of blows	12	17	27	37	47		
Container No	2	Q	61	G	S	17 ^A	8
Wt. Of wet soil + Container - gm	34	35.5	35	36	3.5	30	30
Wt. of dried soil + Container - gm	32	33	33	34	33.5	29	29
Wt. Container - gm	23	23	24	25	24	23	24
Wt. of moisture - gm	2	2.5	2	2	1.5	1	1
Wt. of dry soil - gm	9	10	9	9	9.5	6	5
Moisture content (m) - %	22	25	22	22	15	16	20



DETERMINATION OF THE LIQUID AND PLASTIC LIMITS OF A SOIL

DATA AND CALCULATION SHEET

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003 **BUILDING SITE** ARAKALE AKURE.

SAMPLE NO: FOUR

Moisture content test to be marked m.

liquid limit test to be marked LL.

Plastic limit test to be marked P.L.

Air-dry moisture content to be marked A.

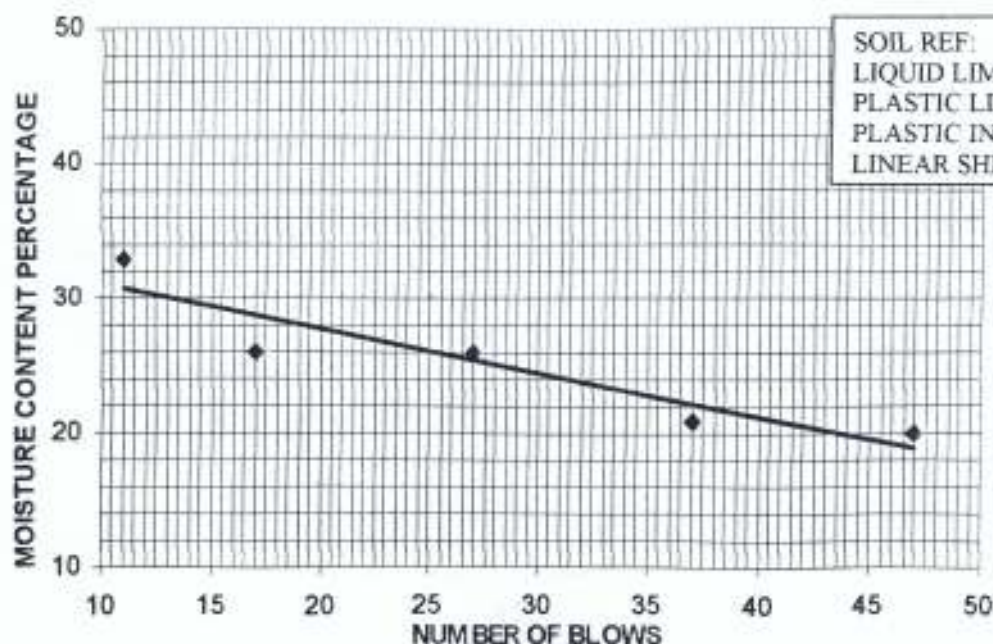
No of blows refers to liquid limit determination

Moisture contents

Wt. Of moisture

Wt. Of dry soil X 100m%

TEST NO	1	2	3	4	5	6	7
Type of test	LL	LL	LL	LL	LL	PL	PL
No of blows	11	17	27	37	47		
Container No	D	K	Z	A	C	L	E
Wt. Of wet soil + Container - gm	52	50	52	53	51	45	44
Wt. of dried soil + Container - gm	49	47.5	49.5	50.5	49	45.5	43.5
Wt. Container - gm	40	38	40	39	39	41	39
Wt. of moisture - gm	3	2.5	2.5	2.5	2	.5	.5
Wt. of dry soil - gm	9	9.5	9.5	11.5	10	4.5	4.5
Moisture content (m) - %	33	26	26	21	20	11	11

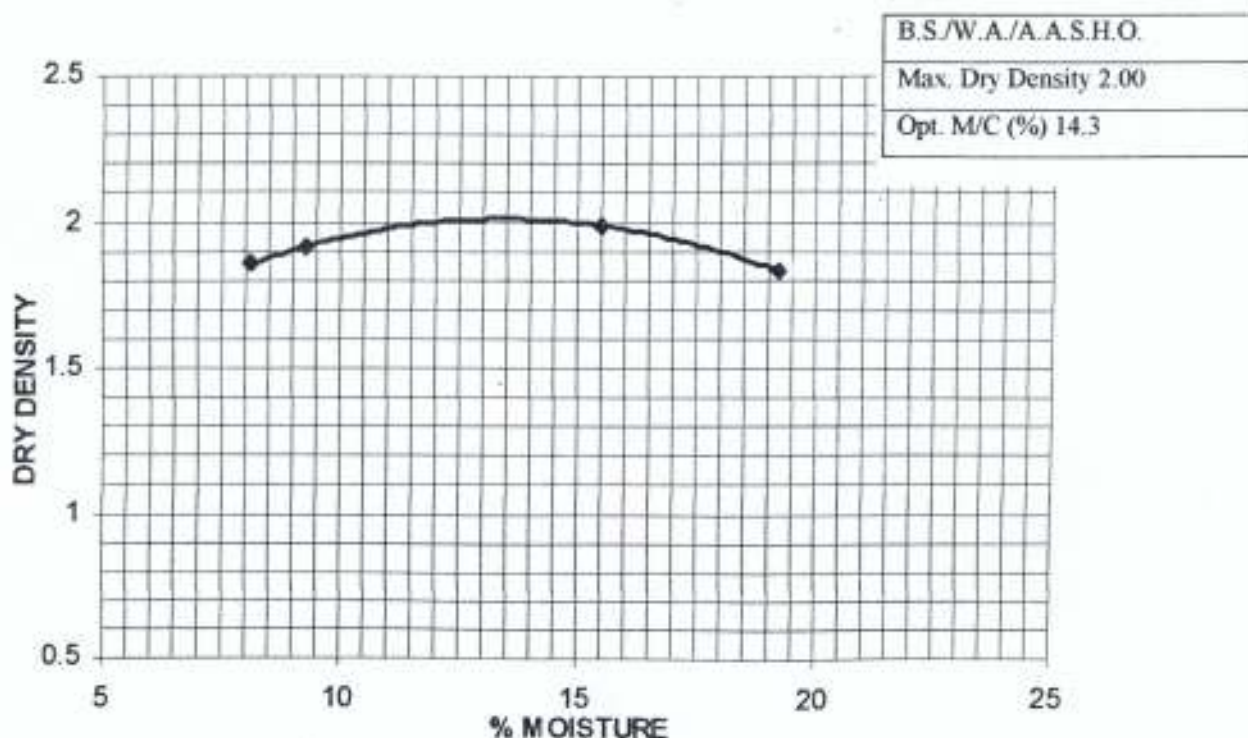


APPENDIX THREE

COMPACTION TEST RESULTS

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOB:** POST GRADUATE PROJECT
DATE: 20/2/2003 **BUILDING SITE** ARAKALE AKURE. **SAMPLE NO:** ONE

WT. CYL. AND WET SAMPLE	9595		9920		10268		9997			
WT. OF CYLINDER	4909		4909		4909		4909			
WT. OF WET SAMPLE	4689		5011		5359		5088			
WET DENSITY	2.01		2.15		2.30		2.18			
CONTAINER NO	61	32	59	25	12	W	3	C		
WT. OF CONT. AND WET SAMPLE	94	96	95	92	95	27	99	97		
WT. OF COUNT. AND DRY SAMPLE	92	91	90	88	88	89	89	88		
WT. OF MOISTURE	4	5	5	4	7	8	10	9		
WT. OF CONTAINER	39	40	40	41	40	40	39	39		
WT. OF DRY SAMPLE	63	51	50	47	48	49	50	49		
MOISTURE CONTENT %	6.3	9.8	10	8.5	14.6	16.3	20	18.4		
AVER. MOISTURE CONTENT %	8.1		9.3		15.5		19.2			
DRY DENSITY	1.80		1.91		1.99		1.83			
C.B.R.										



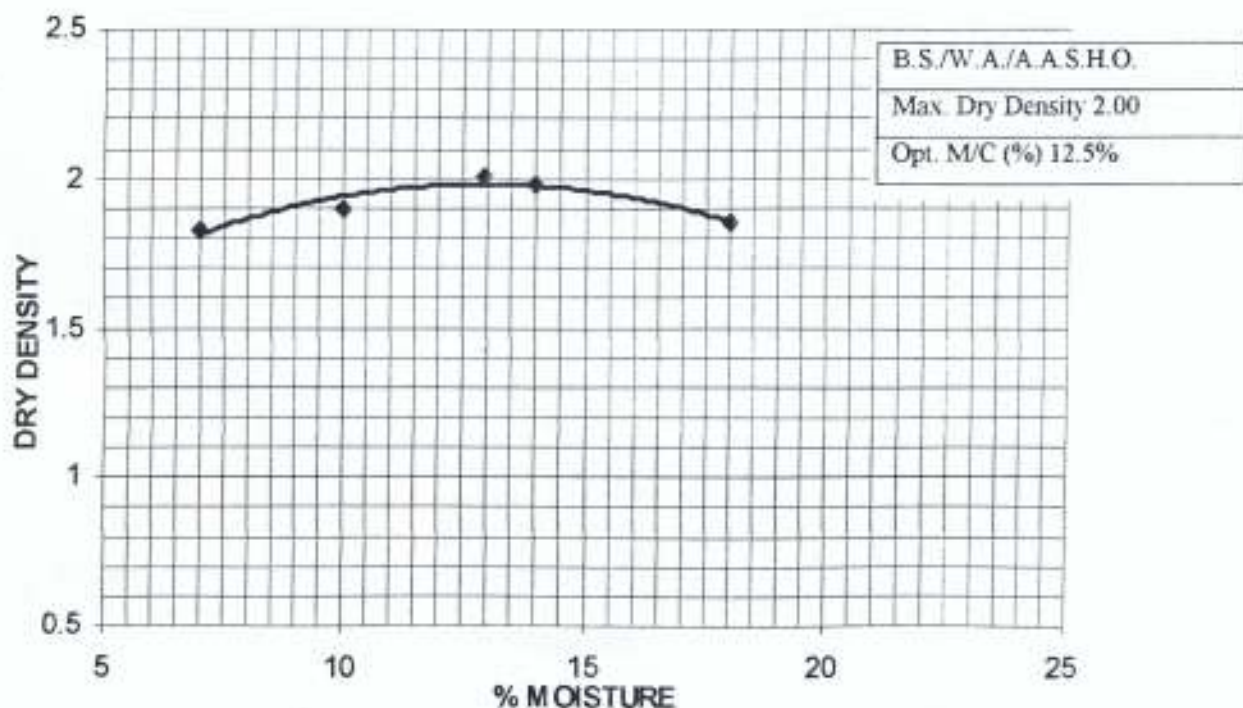
COMPACTION TEST RESULT

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003 **BUILDING SITE** ARAKALE AKURE.

SAMPLE NO: TWO

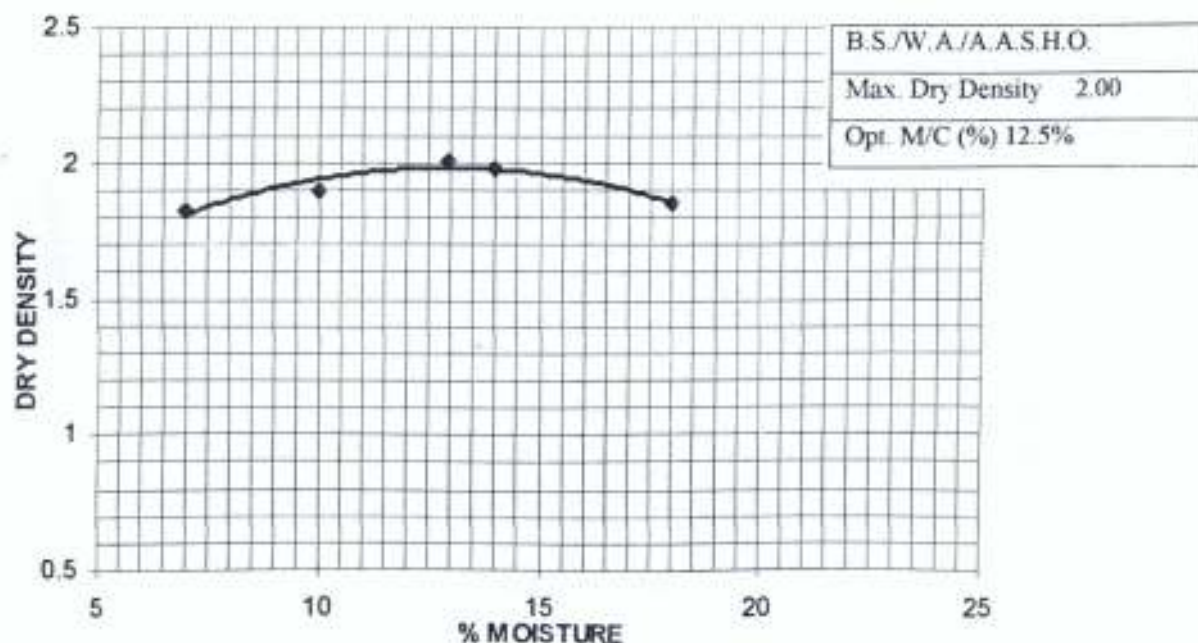
WT. CYL. AND WET SAMPLE	9482		9794		10208		10183		9999			
WT. OF CYLINDER	4909		4909		4909		4909		4909			
WT. OF WET SAMPLE	4573		4885		5299		5274		5090			
WET DENSITY	1.96		2.09		2.27		2.26		2.18			
CONTAINER NO	14	AD	13	16	25	03C	D	32	20	19		
WT. OF CONT. AND WET SAMPLE	84	85	76	99	102	86	91	96	86	88		
WT. OF COUNT. AND DRY SAMPLE	80	80	71	93	95	78	81	87	75	77		
WT. OF MOISTURE	4	5	5	6	7	8	10	9	11	11		
WT. OF CONTAINER	15	15	15	39	41	15	16	16	15	15		
WT. OF DRY SAMPLE	65	65	56	54	54	63	65	71	60	62		
MOISTURE CONTENT %	6.2	7.7	8.9	11.1	13.0	12.7	15.4	12.7	18.3	17.7		
AVER. MOISTURE CONTENT %	7.0		10.0		12.9		14		18			
DRY DENSITY	1.83		1.90		2.01		1.98		1.85			
C.B.R.												



COMPACTION TEST RESULT

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT
DATE: 20/2/2003 BUILDING SITE ARAKALE AKURE. **SAMPLE NO:** THREE

T. CYL. AND WET SAMPLE	9387	9832	10013	9825						
T. OF CYLINDER	4909	4909	4909	4909						
T. OF WET SAMPLE	4478	4923	5104	4916						
ET DENSITY	1.92	2.11	2.19	2.11						
ONTAINER NO	17 ^B	14	2	1	5	4	8	3		
T. OF CONT. AND WET SAMPLE	95	80	94	78	98	95	86	86		
T. OF COUNT. AND DRY SAMPLE	90	74	87	72	9 ^D	85	75	78		
T. OF MOISTURE	5	6	7	6	8	10	11	10		
T. OF CONTAINER	16	16	16	15	16	15	16	15		
T. OF DRY SAMPLE	74	58	71	57	75	70	59	61		
MOISTURE CONTENT %	6.8	10.3	9.9	10.5	9.3	14.3	18.6	16.4		
VER. MOISTURE CONTENT %	8.6	10.2	11.8	17.5						
RY DENSITY	1.77	1.91	1.96	1.80						
B.R.										



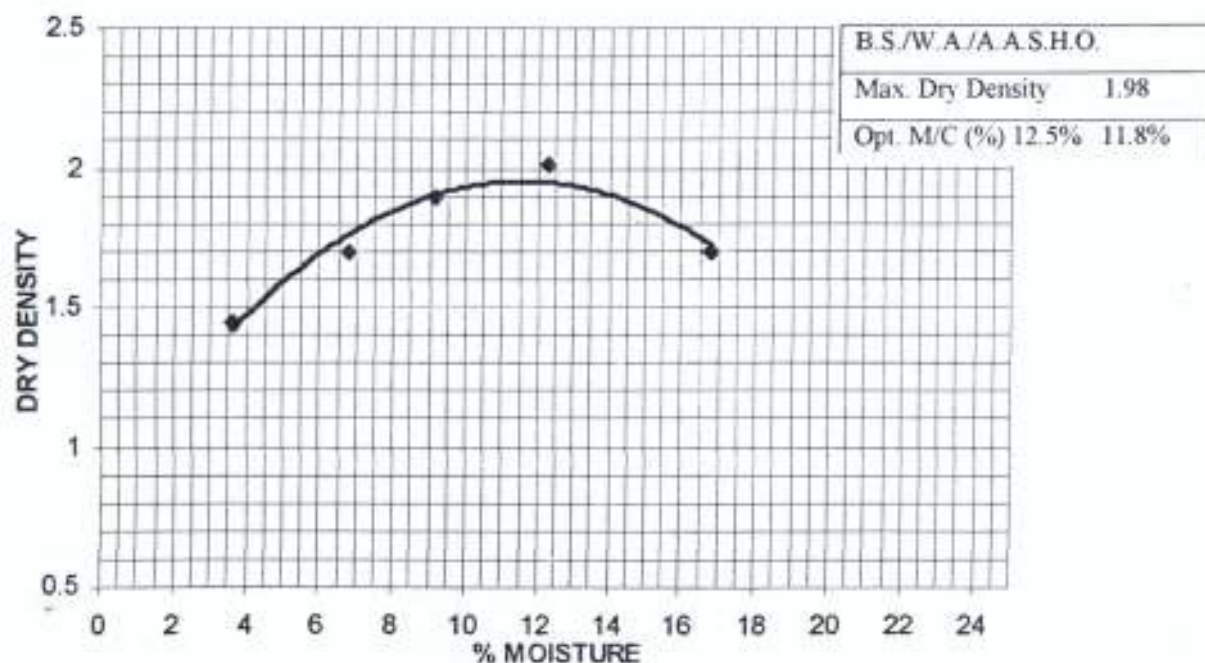
COMPACTION TEST RESULT

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003 BUILDING SITE ARAKALE AKURE.

SAMPLE NO: FOUR

WT. CYL. AND WET SAMPLE	8599	9326	9962	10378	9742		
WT. OF CYLINDER	5098	5098	5098	5098	5098		
WT. OF WET SAMPLE	3501	4228	4864	5280	4644		
WET DENSITY	1.50	1.81	2.07	2.26	1.99		
CONTAINER NO	9	10	15	6	19	16	C K N 21
WT. OF CONT. AND WET SAMPLE	77	65	86	87	108	103	100 105 88 95
WT. OF COUNT. AND DRY SAMPLE	74	64	82	82	101	95	91 95 77 84
WT. OF MOISTURE	3	1	4	5	7	8	9 10 11 11
WT. OF CONTAINER	16	15	15	15	16	16	16 16 15 15
WT. OF DRY SAMPLE	59	45	67	67	85	79	75 79 62 69
MOISTURE CONTENT %	5.2	2.0	6.0	7.5	8.2	10.1	12 12.6 17.7 15.9
AVER. MOISTURE CONTENT %	3.6		6.8		9.2		12.3 16.8
DRY DENSITY	1.45		1.70		1.90		2.01 1.70
C.B.R.							



APPENDIX FOUR

GRAIN SIZE ANALYSIS BY SIEVES

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003 **BUILDING SITE** ARAKALE AKURE **SAMPLE NO:** ONE

WEIGHT OF DRY MATERIAL AND CONTAINER: IN GR.: 751

WEIGHT OF CONTAINER: IN GR.: 219

WEIGHT OF DRY MATERIAL: IN GR.: 532

SIEVE NO.		Sieve Opening IN HM	Retained Material in GR	Passing Material in GR	Passing Material %	
B.S.	ASTM					
3"		75				
2 1/2"		63				
2"		50				
1 1/2"		37.5		532	100%	
1"		25.0	49	483	90.8	
3/4"		19.0	29	454	85.3	
5/8"		16.0				
1/2"		12.5				
3/8"		9.5				
1/4"		6.3	144	310	58.3	
3/16" No. 4		4.75	53	250	47.0	
			Retained in GR	Passing in GR	Passing in (%)	
No. 7	No. 8	2.36	93	164		30.8
No. 8	No. 10	2.00				
No. 14	No. 16	1.18	33	131		24.6
No. 25	No. 30	0.600	100	31		5.8
No. 36	No. 40	0.425	13	18		3.4
No. 52	No. 50	0.300	9	9		1.7
No. 72	No. 70	0.212	5	4		0.8
No. 100		0.150	2	2		0.4
No. 200		0.075	2			

GRAIN SIZE ANALYSIS BY SIEVES

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT
DATE: 20/2/2003 **BUILDING SITE** ARAKALE AKURE. **SAMPLE NO:** TWO

WEIGHT OF DRY MATERIAL AND CONTAINER: **IN GR.:** 491

WEIGHT OF CONTAINER: **IN GR.:** 254

WEIGHT OF DRY MATERIAL: **IN GR.:** 237

SIEVE B.S.	NO. ASTM	Sieve Opening IN HM	Retained Material in GR	Passing Material in GR	Passing Material %	
3"		75				
2 1/2"		63				
2"		50				
1 1/2"		37.5				
1"		25.0				
3/4"		19.0				
5/8"		16.0				
1/2"		12.5				
3/8"		9.5		237	100%	
1/4"		6.3	61	176	73.8	
3/16" No. 4		4.75	24	152	64.1	
			Retained in GR	Passing in GR	Passing in (%)	
No. 7	No. 8	2.36	45	107		45
No. 8	No. 10	2.00				
No. 14	No. 16	1.18	16	91		38
No. 25	No. 30	0.600	67	24		10.1
No. 36	No. 40	0.425	10	14		6.0
No. 52	No. 50	0.300	6	8		3.4
No. 72	No. 70	0.212	3	5		2.1
No. 100		0.150	2	3		1.3
No. 200		0.075	3	-		-

GRAIN SIZE ANALYSIS BY SIEVES

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003 **BUILDING SITE** ARAKALE AKURE. **SAMPLE NO:** THREE

WEIGHT OF DRY MATERIAL AND CONTAINER: IN GR.: 674

WEIGHT OF CONTAINER: IN GR.: 250

WEIGHT OF DRY MATERIAL: IN GR.: 224

SIEVE B.S.	NO. ASTM	Sieve Opening IN MM	Retained Material in GR	Passing Material in GR	Passing Material %	
	3"	75				
	2 1/2"	63				
	2"	50				
	1 1/2"	37.5				
	1"	25.0				
	3/4"	19.0		324	100%	
	5/8"	16.0	9	315	97.2	
	1/2"	12.5				
	3/8"	9.5	32	283	87.3	
	1/4"	6.3	26	257	79.3	
	3/16" No. 4	4.75				
			Retained in GR	Passing in GR	Passing in (%)	
No. 7	No. 8	2.36	29	228		70.4
No. 8	No. 10	2.00				
No. 14	No. 16	1.18	100	128		39.5
No. 25	No. 30	0.600	82	46		14.2
No. 36	No. 40	0.425	16	30		9.3
No. 52	No. 50	0.300	18	12		3.7
No. 72	No. 70	0.212	4	8		2.5
No. 100		0.150	2	6		1.9
No. 200		0.075	4	2		0.6

GRAIN SIZE ANALYSIS BY SIEVES

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003 BUILDING SITE ARAKALE AKURE.

SAMPLE NO: FOUR

WEIGHT OF DRY MATERIAL AND CONTAINER: IN GR.: 1037

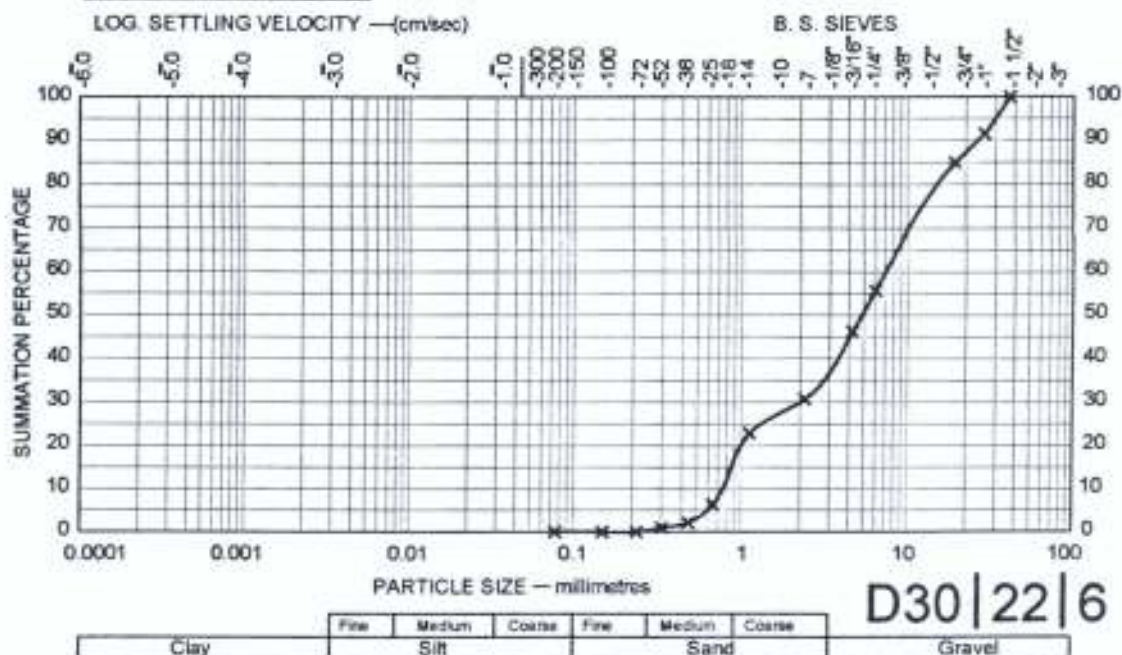
WEIGHT OF CONTAINER: IN GR.: 230

WEIGHT OF DRY MATERIAL: IN GR.: 807

SIEVE B.S.	NO. ASTM	Sieve Opening IN HM	Retained Material in GR	Passing Material in GR	Passing Material %	
	3"	75				
	2 1/2"	63				
	2"	50				
	1 1/2"	37.5				
	1"	25.0				
	3/4"	19.0		807	100%	
	5/8"	16.0	9	798	98.9	
	1/2"	12.5	16	782	96.9	
	3/8"	9.5	28	754	93.4	
	1/4"	6.3	92	662	82.0	
	3/16" No. 4	4.75	35	627	77.7	
			Retained in GR	Passing in GR	Passing in (%)	
No. 7	No. 8	2.36	108	519		64.3
No. 8	No. 10	2.00	210	309		38.4
No. 14	No. 16	1.18	96	213		26.4
No. 25	No. 30	0.600	81	132		16.4
No. 36	No. 40	0.425	72	60		7.4
No. 52	No. 50	0.300	19	41		5.1
No. 72	No. 70	0.212	11	30		3.1
No. 100		0.150	20	10		1.2
No. 200		0.075	8	2		0.2

SUMMATION PERCENTAGE VERSUS PARTICLE SIZE

Sample No : One



REMARK : Gravelly Sandy Soil

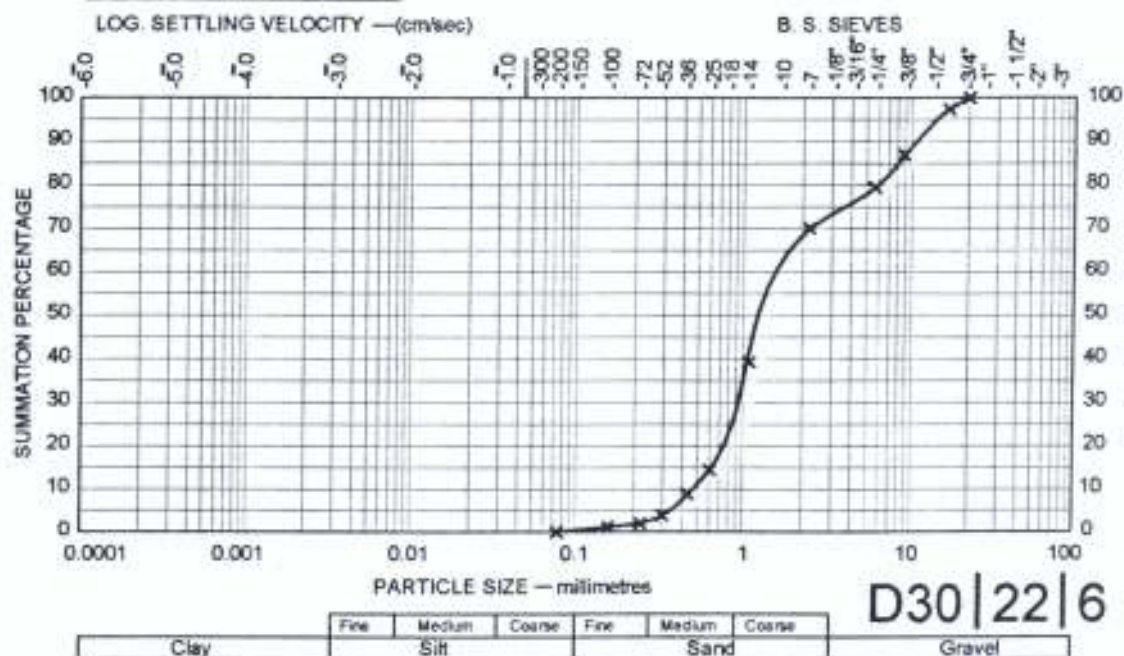
Sample No : Two



REMARK : Gravelly Sandy Soil

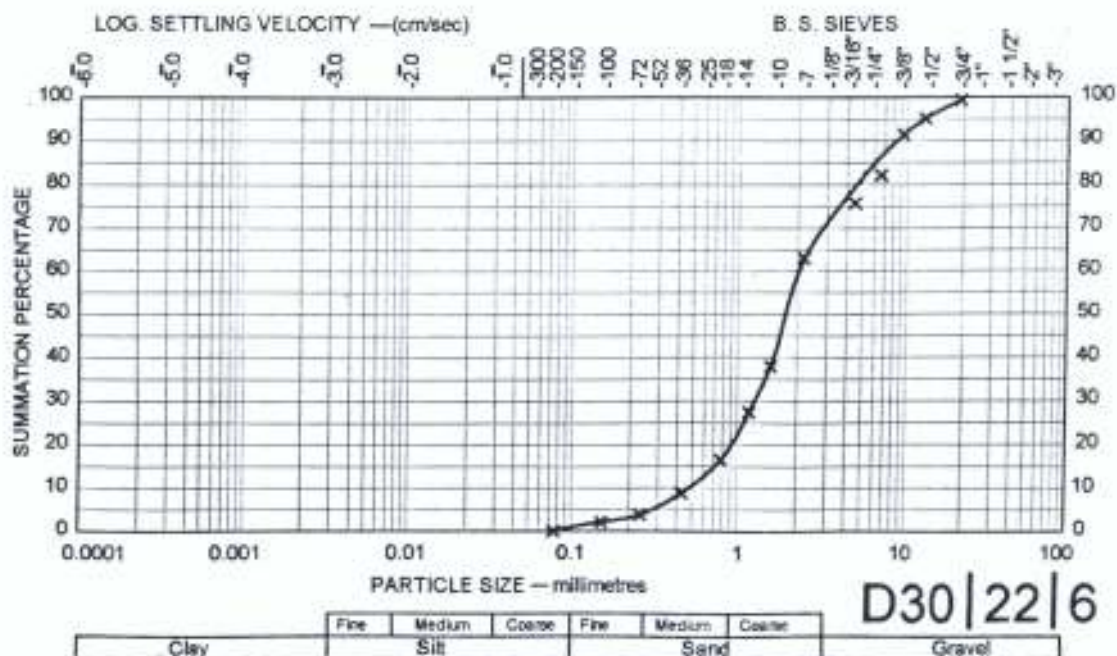
SUMMATION PERCENTAGE VERSUS PARTICLE SIZE

Sample No : Three



REMARK : Gravelly Sandy Soil

Sample No : Four



APPENDIX FIVE

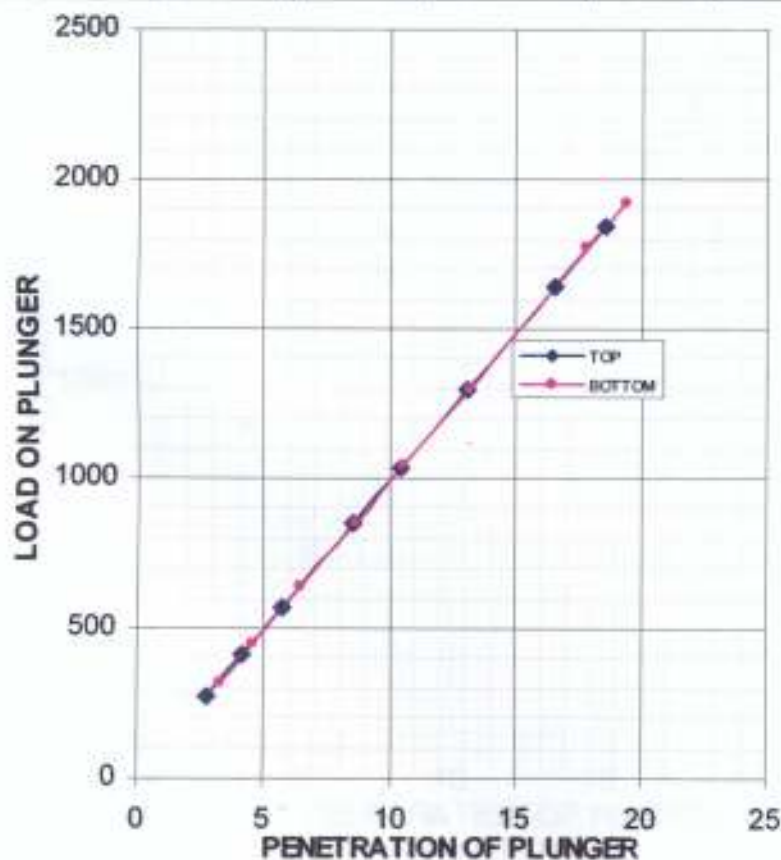
CALIFORNIA BEARING RATIO

(C.B.R. TEST)

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003 **BUILDING SITE** ARAKALE AKURE. **SAMPLE NO:** ONE

Penetration	Prov. Ring	Load	Corrected	C.B.R.	Pro. Ring	Load	Corrected	C.B.R.
0.025"	.0101	275	2.78		.0101	320	3.23	
0.050"	.0101	410	4.14		.0101	450	4.55	
0.075"	.0101	568	5.74		.0101	638	6.44	
0.100"	.0101	849	8.58	62.14	.0101	853	8.62	62.46
0.150"	.0101	1037	10.47		.0101	1048	10.59	
0.200"	.0101	1296	13.09	65.45	.0101	1301	13.14	65.70
0.250"	.0101	1636	16.53		.0101	1769	17.87	
0.300"	.0101	1840	18.58		.0101	1920	19.39	



Prov. Ring Factor
Un-Soak

lbs./kg

Result	
Initial reading	63.80
Final Reading	64.00
Moisture Content	
Dry Density	
C.B.R.	0.1" PEN 62.3
C.B.R.	0.2" PEN 65.58

CALIFORNIA BEARING RATIO

(C.B.R. TEST)

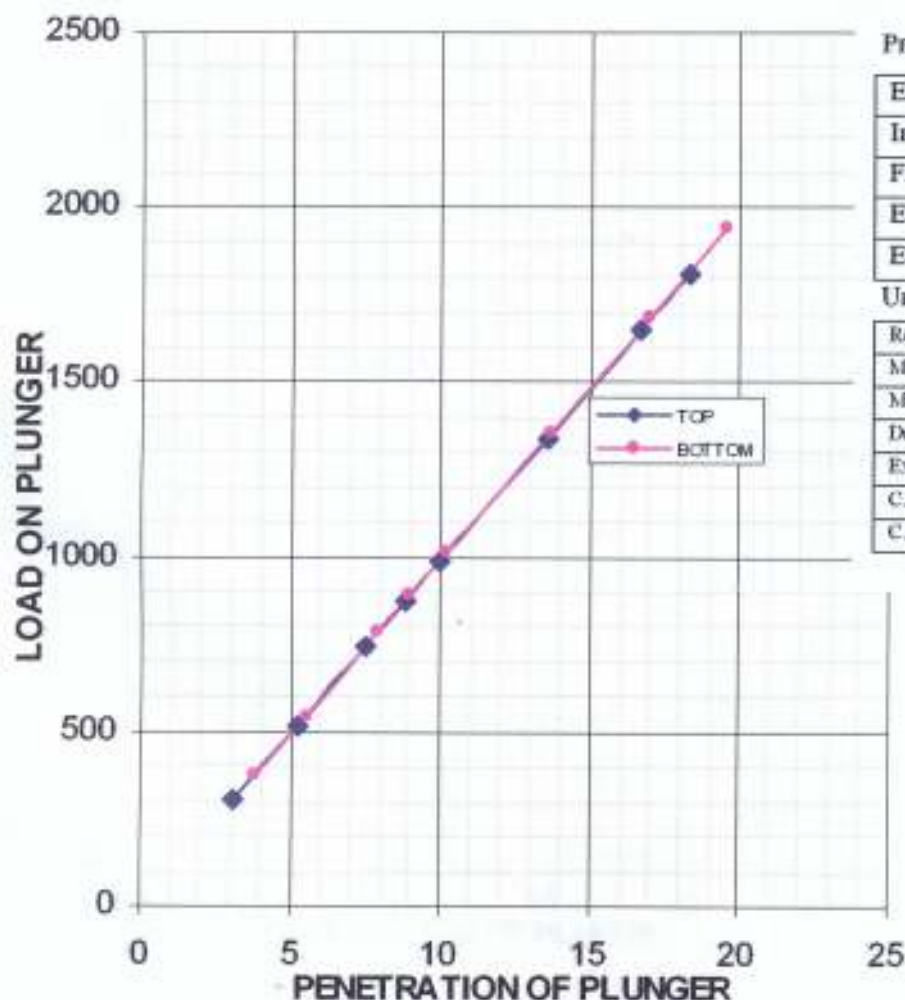
SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003

BUILDING SITE ARAKALE AKURE.

SAMPLE NO: TWO

Penetration	Prov. Ring	Load	Corrected	C.B.R.	Pro. Ring	Load	Corrected	C.B.R.
0.025"	.0101	310	3.13		.0101	380	3.84	
0.050"	.0101	516	5.22		.0101	546	5.52	
0.075"	.0101	744	7.51		.0101	789	7.97	
0.100"	.0101	874	8.83	63.99	.0101	890	8.99	65.15
0.150"	.0101	988	9.98		.0101	1010	10.20	
0.200"	.0101	1340	13.53	67.65	.0101	1351	13.65	68.25
0.250"	.0101	1653	16.70		.0101	1863	16.99	
0.300"	.0101	1810	18.28		.0101	1940	19.59	



Prov. Ring Factor: lbs./kg

Expansion after soak	
Initial reading	
Final reading	
Expansion	inch
Expansion	%

Un-Soak Hrs.

Result	
Moist. Cont. Before soaking	% 64.57
Moist. Cont. After soaking	% 67.95
Dry density	KG m ³
Expansion after soaking	%
C.B.R.	0.1" PEN 64.57
C.B.R.	0.2" PEN 67.95

CALIFORNIA BEARING RATIO

(C.B.R. TEST)

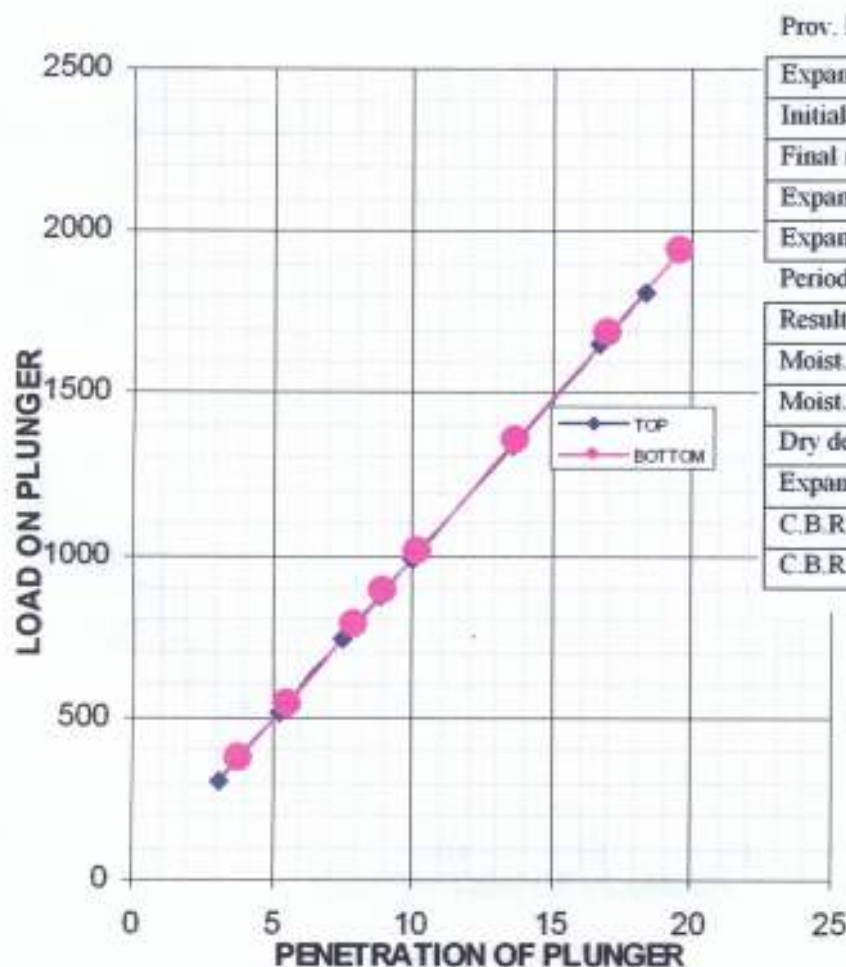
SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

DATE: 20/2/2003

BUILDING SITE ARAKALE AKURE.

SAMPLE NO: THREE

Penetration	Prov. Ring	Load	Corrected	C.B.R.	Pro. Ring	Load	Corrected	C.B.R.
0.025"	.0101	186	1.88		.0101	215	2.17	
0.050"	.0101	300	3.03		.0101	380	3.84	
0.075"	.0101	548	5.54		.0101	624	6.30	
0.100"	.0101	828	8.36	60.58	.0101	892	9.10	65.29
0.150"	.0101	1032	10.42		.0101	1089	11.00	
0.200"	.0101	1316	13.29	66.45	.0101	1326	13.39	66.95
0.250"	.0101	1620	16.36		.0101	1740	17.57	
0.300"	.0101	1848	18.67		.0101	1960	19.80	



Prov. Ring Factor: 0101 lbs./kg

Expansion after soak	
Initial reading	63.52
Final reading	66.12
Expansion	inch
Expansion	%

Period of soaking Hrs.

Result	
Moist. Cont. Before soaking %	
Moist. Cont. After soaking %	
Dry density	KGr/m ³
Expansion after soaking	%
C.B.R.	0.1" PEN 62.94
C.B.R.	0.2" PEN 66.70

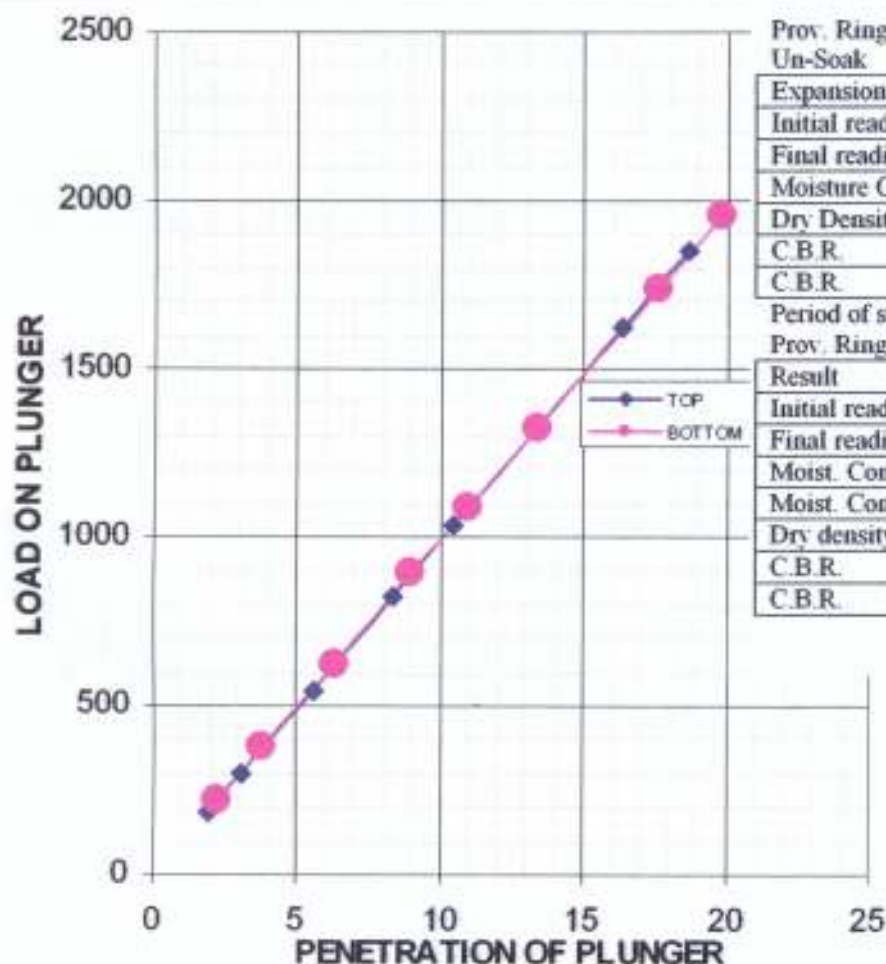
CALIFORNIA BEARING RATIO

(C.B.R. TEST)

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED JOBS: POST GRADUATE PROJECT

DATE: 20/2/2003 BUILDING SITE ARAKALE AKURE. SAMPLE NO: FOUR

Penetration	Prov. Ring	Load	Corrected	C.B.R.	Pro. Ring	Load	Corrected	C.B.R.
0.025"	.0101	98	0.99		.0101	120	1.21	
0.050"	.0101	286	2.89		.0101	310	3.13	
0.075"	.0101	647	6.53		.0101	689	6.96	
0.100"	.0101	998	10.08	73.04	.0101	1100	11.11	80.51
0.150"	.0101	1460	14.75		.0101	1495	15.10	
0.200"	.0101	1489	15.04	75.20	.0101	1512	15.27	76.35
0.250"	.0101	1768	17.86		.0101	1880	18.99	
0.300"	.0101	1977	19.97		.0101	2048	20.68	



Prov. Ring Factor:

Un-Soak

Ibs./kg

Expansion after soak

Initial reading 74.12

Final reading 78.43

Moisture Content

Dry Density

C.B.R. 0.1" PEN 76.77

C.B.R. 0.2" PEN 75.77

Period of soaking

Prov. Ring Factor

Hrs.

Result

Initial reading

Final reading

Moist. Cont. Before soaking %

Moist. Cont. After soaking %

Dry density KGr/m³

C.B.R. 0.1" PEN

C.B.R. 0.2" PEN

APPENDIX SIX

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULT

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT
BUILDING SITE ARAKALE AKURE. **SAMPLE NO:** ONE

DATE: 20/2/2003

1	2	3	4	Cell Pressure							
				31		32			33		
Disp. Dial. Rdg. Div.	Change in length L col. x gauge cost.	Strain $\frac{L}{L_0}$ $\times 10^{-4}$	Corr. Area $A_c \times$ 10 m^2	Load dial Rdg. LDR (Div) 100	Deviator Load $L^D \text{ LRD}$ $\times \text{LRF}$ KN	Deviator Stress D $=LD A_c$ KN/m ²	Load Dial Rrg. LDR Div. 200	Deviator Load L_D LRD $\times$ LRF KN	Deviator Stress D $=LD A_c$ KN/m ²	LDR	L_D
25	0.25	32.90	11.379	170	.340	299	264	.528	464		
50	0.50	65.79	11.416	200	.500	438	395	.780	683		
75	0.75	98.68	11.454	320	.640	559	510	1.020	891		
100	1.00	131.58	11.491	260	.520	452	592	1.184	1030		
125	1.25	164.47	11.531				622	1.244	1078		
150	1.50	197.37	11.573				562	1.124	971		
175	1.75	230.26	11.614								

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULT

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

BUILDING SITE ARAKALE AKURE SAMPLE NO: TWO

DATE: 20/2/2003

[illegible]

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST TEST RESULT

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

BUILDING SITE ARAKALE AKURE **SAMPLE NO:** THREE

DATE: 20/2/2003

1	2	3	4	Cell Pressure								
				31			32			33		
Disp. Dial. Rdg. Div.	Change in length L col. x gauge cost.	Strain $\frac{L}{L_0}$ $\times 10^{-4}$	Corr. Area $A_c \times$ 10 m^2	Load dial Rdg. LDR (Div) 100	Devia tor Load L^D LRD $\times$ LRF KN	Devia tor Stress D $=LD$ A_c KN/m ²	Load Dial Rrg. LDR Div. 200	Devia tor Load L_D LRD $\times$ LRF KN	Devia tor Stress D $=LD$ A_c KN/m ²	LDR 300	L_D	
25	0.25	32.90	11.379	65	.130	114	152	.304	267	210	.420	369
50	0.50	65.79	11.416	163	.326	286	256	.512	448	298	.596	517
75	0.75	98.68	11.454	235	.470	410	336	.660	576	357	.714	623
100	1.00	131.58	11.491	284	.668	581	376	.752	654	398	.796	693
125	1.25	164.47	11.531	310	.620	537	400	.800	694	416	.832	722
150	1.50	197.37	11.573	319	.638	551	406	.812	702	426	.852	736
175	1.75	230.26	11.614	321	.642	553	402	.804	692	434	.868	747
200	2.00	263.16	11.655	321	.642	550				440	.088	755
225	2.25	296.05	11.697							400	.800	686



UNCONSOLIDATED UNDRAINED TRIAXIAL TEST RESULT

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT

BUILDING SITE ARAKALE AKURE. **SAMPLE NO:** FOUR

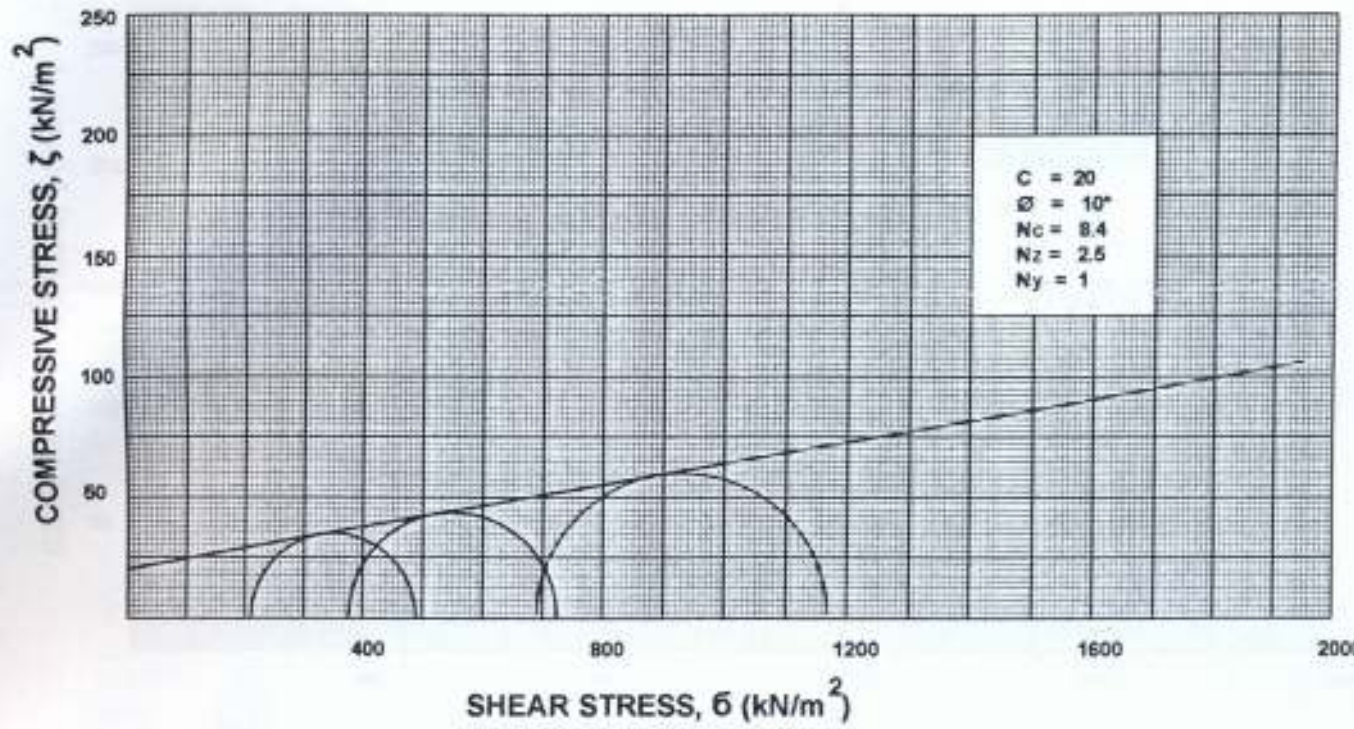
DATE: 20/2/2003

1	2	3	4	Cell Pressure								
				31			32			33		
Disp. Dial. Rdg. Div.	Change in length L col. x gauge const.	Strain L 1.0 x 10 ⁻⁴	Corr. Area Ac x 10 m ²	Load dial Rdg. LDR (Div) 100	Devia tor Load L ^D LRD x LRF KN	Devia tor Stress D =LD Ac KN/m ²	Load Dial Rrg. LDR Div. 200	Devia tor Load L _D LRD x LRF KN	Devia tor Stress D =LD Ac KN/m ²	LDR 300	L _D	
25	0.25	32.90	11.379	84	.168	147.6	134	.268	236	161	.322	283
50	0.50	65.79	11.416	168	.336	294	205	.410	359	230	.460	403
75	0.75	98.68	11.454	212	.424	370	325	.650	568	345	.690	602
100	1.00	131.58	11.491	288	.576	501	365	.730	635	391	.782	680
125	1.25	164.47	11.531	315	.630	546	382	.764	663	410	.820	711
150	1.50	197.37	11.573	324	.648	560	389	.778	672	422	.844	729
175	1.75	230.26	11.614	289	.578	498	389	.778	670	431	.862	742
200	2.00	263.16	11.655				400	.800		443	.886	760
225	2.25	296.05	11.697				390	.780		450	.900	769
250	2.50	328.95	11.739				380			455	.910	775
275	2.75	361.84	11.783							460	.920	781
300	3.00	394.74	11.807							440	.880	745

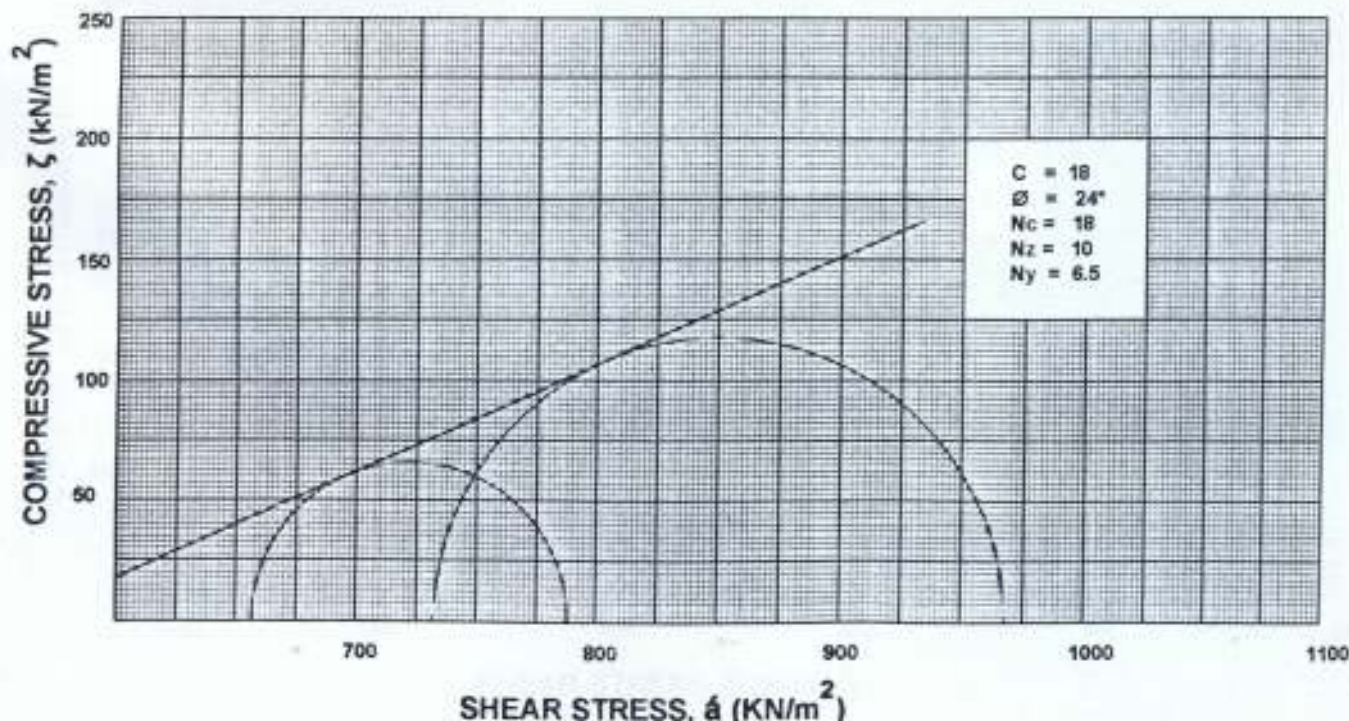
(COMPRESSIVE STRESS AGAINST SHEAR STRESS)

LOCATION : ST THOMAS'S CHURCH, ARAKALE, AKURE.

SAMPLE NO : ONE

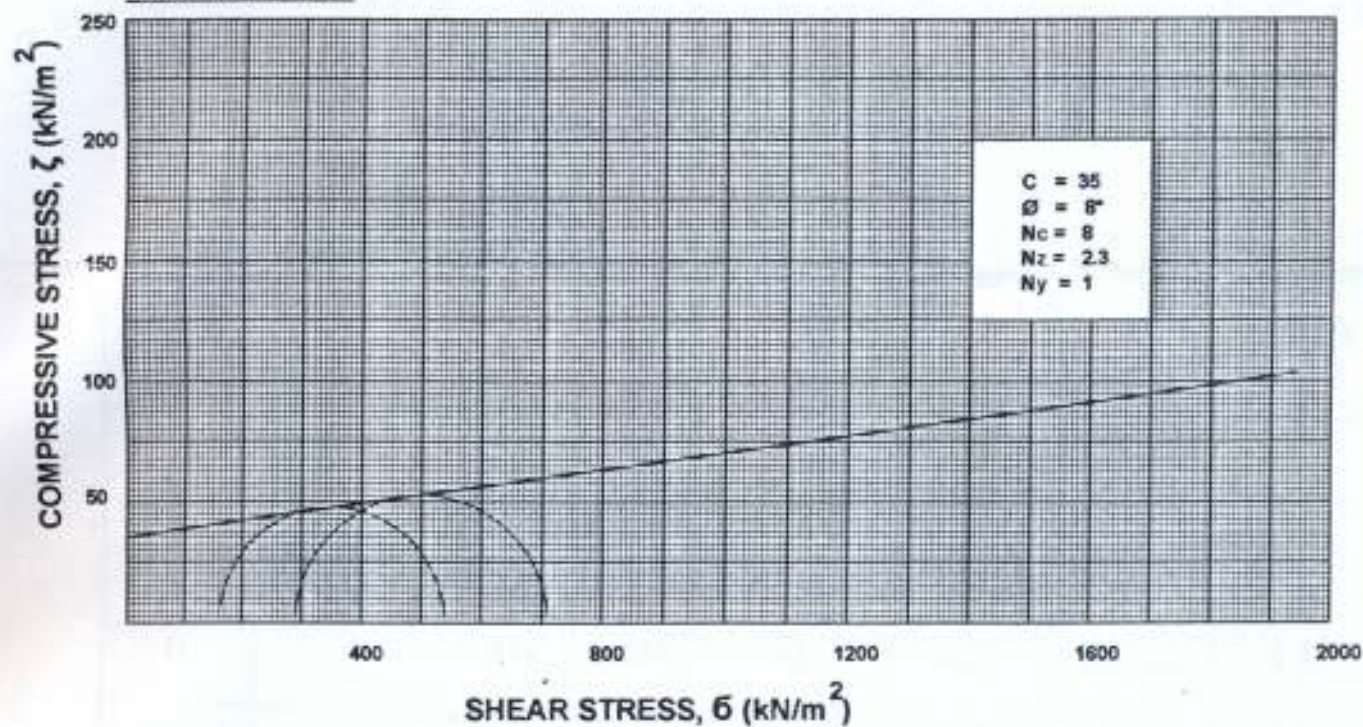


SAMPLE NO : TWO

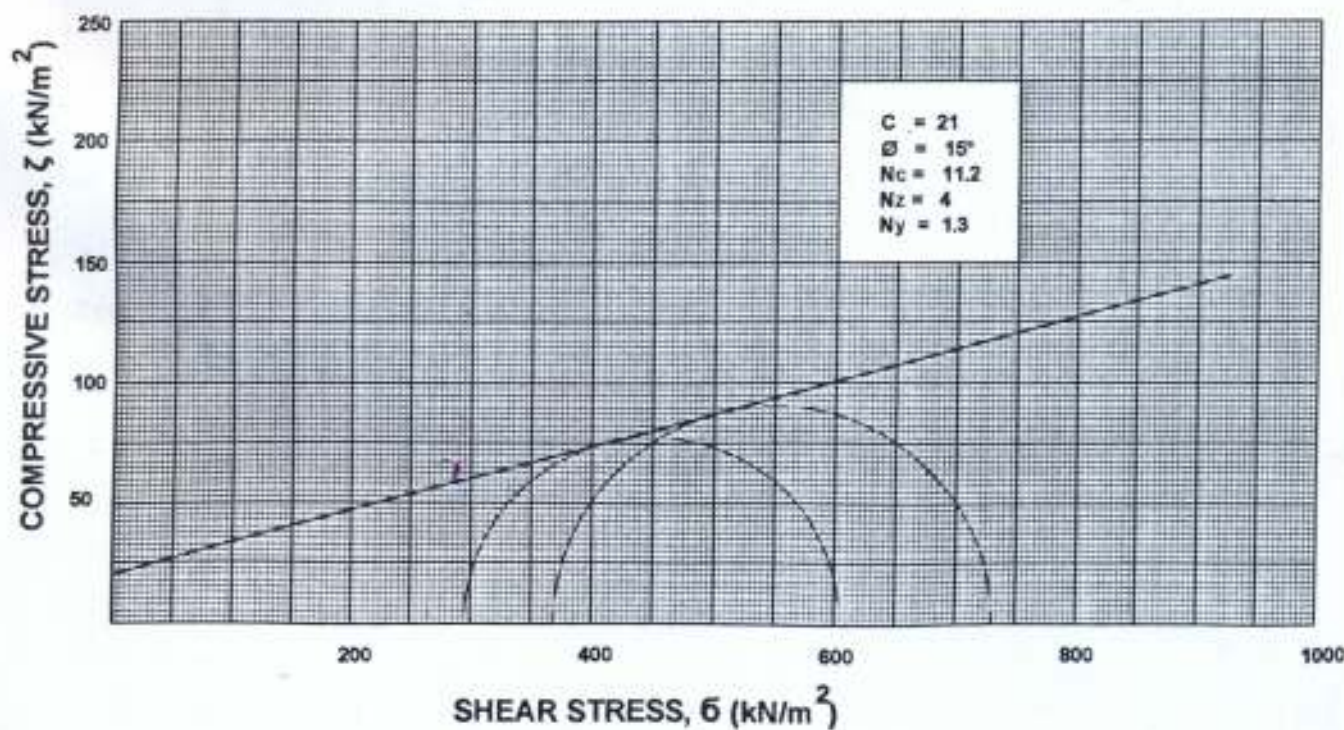


LOCATION : ST THOMAS'S CHURCH, ARAKALE, AKURE.

SAMPLE NO : THREE



SAMPLE NO : FOUR



APPENDIX SEVEN

DETERMINATION OF COMPRESSIVE STRENGTH OF THE CONCRETE RUBBLES

SAMPLE LOCATION: ST THOMAS'S CHURCH COLLAPSED **JOBS:** POST GRADUATE PROJECT
BUILDING SITE ARAKALE AKURE. **SAMPLE NO:** CONCRETE RUBBLES
DATE: 20/2/2003

Cube No.	Date Cast	Age for Testing (days)	Date Tested	Weight of Cube (gm)	Density Kg/m ³	Crushing Load N	Crushing Strength N/mm ²
A		>28days	18/02/03	4958	8471	20,000	5.2
B		>28days	18/02/03	3630	6694	15,000	2.8
C		>28days	18/02/03	4150	6072	20,000	3.6
D		>28days	18/02/03	4170	6043	15,000	3.2
E		>28days	18/02/03	3040	7737	5,000	6.9
F		>28days	18/02/03	6658	4325	15,000	3.8
G		>28days	18/02/03	2795	3864	3,000	6.2
H		>28days	18/02/03	2870	4014	5,000	4.9
I		>28days	18/02/03	3590	4308	3,000	7.0
J		>28days	18/02/03	3504	2354	5,000	5.8
						AV	4.9

Average crushing strength is 4.9N/mm²

Remarks: Below the value expected for ratio 1:2:4 concrete mix design

APPENDIX EIGHT

Federal University of Technology,
Post Graduate Section,
Civil Engineering Department,
Akure,
30th October, 2002.

TO WHOM IT MAY CONCERN

Dear Sir,

QUESTIONNAIRE ON CAUSES OF COLLAPSE OF THE 3 – STOREY BUILDING OF ST. THOMAS'S CHURCH, ARAKALE ROAD, AKURE.

I am a student undergoing a post Graduate course in structural Engineering in the Department of Civil Engineering of the Faculty of Engineering, Federal University of Technology, Akure. I am sending this questionnaire to you as part of addendum for my project.

Please sir, I would be extremely grateful if your answers to the questions are précised, concise and as conscientious as possible. Every answer you gave would be treated as confidential and shall not be used for any purpose other than the project alone.

Thank you.

**QUESTIONNAIRE ON CAUSES OF COLLAPSE OF THE 3 – STOREY
BUILDING OF ST. THOMAS'S CHURCH ARAKALE ROAD, AKURE.**

(a) YOUR PARTICULARS

Please we will like to know this information about you.

You are expected to tick which one is applicable where necessary.

1. Are you a Male? ☐ or Female? ☐
2. Are you married? Yes ☐ No? ☐
3. What is your highest qualification?
PHD ☐ Dr. ☐ B.Sc. ☐ B.A. ☐
HND ☐ OND or ND ☐ GCE/ or Below ☐
4. Year of attaining highest qualification
5. Which institution did you graduate from.....
6. Are you an Engineer ☐ Architect ☐ Clergyman ☐
Doctor ☐ Business man ☐ Others not listed
7. What is your age group
20 – 29 ☐
30 – 39 ☐
40 – 69 ☐
70 years and above ☐
8. How old are you in your profession?
9. Are you a registered professional? Yes ☐ No ☐
10. If No 9 above is Yes, which professional body do you belong to?
11. Are you a Contractor? Yes ☐ No ☐
12. Are you a Civil servant?
13. If No 12 above is Yes, what is your rank in the service?

.....

(B) QUESTIONS ON CAUSES OF COLLAPSE OF THE 3-
STOREY BUILDING OF ST. THOMAS'S CHURCH,
ARAKALE ROAD, AKURE.

THE QUESTIONS

1. What type of structure is the collapsed building?
.....
2. When did the construction of the collapsed building
commence?.....
3. When did the construction of the collapsed building
complete?.....
4. When did the collapse occur?.....
5. Did the building give any sign before it collapsed? If yes, how?
.....
6. How many people (averagely) were working under or on the
building at the time of collapse?.....
7. How many people died?.....
8. How many people got wounded and later recovered?.....
9. Did rain fall prior to the collapse?.....
10. Was the day windy or thundery?.....
11. How many storey was the building designed for?.....
12. Who designed the building?.....
13. Is the designer an Engineer?.....
14. Was the construction carried out according to the design
specification or was there any modification during the
construction to the design? If yes, state the modification.

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15. Was there any test carried out to determine the type of foundation adopted for the structure? If No, why?.....
-
16. If No 15 above is Yes, which test?.....
-
17. Was there any concrete laboratory cube test done to determine the strength of the concrete material of the collapsed building?.....
18. What type of foundation was adopted and why?.....
-
19. What type of supervision was adopted? Adhoc or Consultancy Services? Why?.....
-
20. Were there site meetings held during the construction of the project to determine the problems on site and attend to practical problems?.....
21. Were there daily, weekly and monthly reports written to determine the level of supervision and the progress and quality of the job?.....
22. Making retrospection, what was the site being used for initially?.....
23. Was the site a built-up area, if yes explain.....
-
24. Was there any factory close to the collapsed building site that uses vibrating plants for its operations?.....

25. Was the project contracted to a Contractor? If yes state the type of Tendering whether open, selective or competitive tendering?.....
26. Was there any bill-of-quantities prepared for the project?.....
27. What is the name of the Contractor?.....
.....
28. If Contracted out, what was the contract sum?.....
29. Do you think that the contract sum was under- or over-estimated?.....
30. What in your own opinion should be the contract sum?.....
31. Explain the level that construction reached at the time of the collapse.....
.....
32. What do you think are the causes of the collapse?.....
.....
.....
33. Any other information that you think might be important to this project?.....
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