STRUCTURAL INTEGRITY ASSESSMENT OF BEAMS IN AN ABANDONED COMMERCIAL BUILDING USING NDT AND FINITE ELEMENT ANALYSIS

BY

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PLAGIARISM

This work **STRUCTURAL INTEGRITY ASSESSMENT OF BEAMS IN AN ABANDONED COMMERCIAL BUILDING USING NDT AND FINITE ELEMENT ANALYSIS** by IDHIGU JOSHUA with Number ENG2002165 of the Department of Structural Engineering, Faculty of Engineering, University of Benin, Edo State, Nigeria, has PASSED the PLAGIARISM TEST.

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DEDICATION

This work is dedicated to the master of the universe, Almighty God, who by his grace and mercy saw me through the years of my studies in good health.

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ABSTRACT

The increasing number of abandoned buildings in developing countries highlights the pressing need for a credible means to determine the safety and soundness of reinforced concrete (RC) structures. This paper offers a forensic investigation of the RC beams in the abandoned commercial structure that halted construction because of suspected material strength and construction quality. To meet this challenge, this study provides a framework that combines non destructive testing (NDT) with computational modeling.

The rebound method is a penetration test. It is a reliable method to determine the strength of damaged concrete. It is well suited to strength assessment of concrete near to the surface. It is a quick method and is particularly useful when drilling is not undertaken. It is accurate in comparison with other methods such as the Gill fall method. Rebound tests can be performed in depth. The rebound method measures the strength of concrete. It can determine the strength of concrete near to the surface. It is accurate to determine strength. It is relatively less accurate if the concrete is damaged.

Moreover, the section cut-off method provides accurate results. It is carried out to determine strength. The method is well suited to determine.

The results showed large variations in the strength of the concrete for the beams, which is indicative of the varied standards of their construction and their less-than-full conformity to the nominal strength grade C30. Notwithstanding the above, it is apparent that there is potential within the results to satisfy the strength requirements of serviceability deflection. This lends support to the proposal to adopt a targeted repair strategy.

Thus, this paper established that the combined usage of NDT and numerical models is a reliable and non invasive method for assessing the status of deteriorated RC buildings. This method not only verifies the results attained through experiments done on the structure according to codes of practice but can also serve as a reliable guideline for future repair work.

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LIST OF ACRONYMS

ACI - American Concrete Institute

BS - British Standards

FEA - Finite Element Analysis

GPR - Ground-Penetrating Radar

MDT - Minor Destructive Testing

MCFT - Modified Compression-Field Theory

NDT - Non-Destructive Testing

RC - Reinforced Concrete

SLS - Serviceability Limit State

ULS - Ultimate Limit State

UPV - Ultrasonic Pulse Velocity

ASTM: American Society for Testing and Materials

BS: British Standard

CFRP: Carbon Fiber Reinforced Polymers

DL: Dead Load

EN: Eurocode (European Norm)

LL: Live Load

CHAPTER ONE

INTRODUCTION

1.1 Background to the Study

The concrete structures are the primary structural systems found in most commercial structures worldwide, supporting major loads in office structures to retail stores. The durability of these structures is primarily reliant on a host of factors that come into play in a multifaceted way. This primarily entails factors that come into consideration in the initial phase of construction in built structures, including exposure to environmental factors, to name a few. Specifically in developing countries, a rather alarming factor that comes into consideration is that the construction project is often abandoned during various phases due to overt suspicions of the structural viability of the project.

Signs of distress in structures, such as early cracks, deflections, and exposure of reinforcement, can be invaluable in making a diagnosis. For example, inconsistent concrete compaction during construction activities can result in surface cracks that, superficially appearing small in magnitude, can develop into compromising structural complications if not addressed. A recent example is found in a residential structure in Nigeria, where a small diagonal crack under a first-floor beam suggested the existence of a probable problem.

This study presents a representative example of an abandoned commercial building where construction had been stopped pending initial findings that suggested a possible lack of conformity with established engineering standards. Essentially, the primary issue of concern in this broader investigation is the condition of the concrete beams around the ground floor level in the vicinity of the stairs, which is a usually critical load-bearing structural location inherently prone to complicated loads.

This article brings together techniques of non-destructive assessment with computational modeling in a combined approach that is intended to objectively determine if the practical implementation is compatible with the minimum standards of strength and serviceability set by either British Standards (BS 8110) codes or the Eurocode 2 (EN 1992-1-1).

1.2 Problem Statement

The commercial building in question is a prime example of construction abandonment due to suspected structural weaknesses. Various concern indicators identified during initial visual inspection activities during the construction stage included various concrete compaction issues, surface cracks indicating early significant structural weakness in the building, spalling of concrete cover in certain areas, as well as exposure of reinforcement steel in some sections.

It is evident that neglecting initial cracks, including those that are diagonal and flexural in nature, can result in poor safety standards, as well as increased costs of repairs in the long run (Mosley et al., 2012, p.106-110). In this respect, the use of Non-Destructive Testing techniques in conjunction with advanced structural analysis enables a means of assessing the concrete structure effectively in situ. A concrete strength determination via the rebound hammer test, as well as a Finite Element Analysis of concrete structures under practical operating loads via Finite Element Analysis (FEA software), is possible (Adamu et al., 2023; Ogunleye et al., 2023).

Nonetheless, the choice of halting the construction process was largely based on qualitative visual observation rather than thorough quantitative analysis. This is a rather important challenge that arises in defining to what extent surface characteristics observed in the construction represent superficial defects as opposed to structural shortcomings. A lack of empirical information on the in-situ properties of the construction material has raised pertinent questions about what the next course of action should be—whether to opt for demolition as a costly process.

This problem is rectified in this investigation by a systematic use of nondestructive test techniques in combination with advanced analysis techniques. The central ques-tion this investigation is faced with is if the sufficient strength of the beams, as a condition of a condemnation as the only technically justifiable solution, is given.

1.3 Aim and Objectives

Aim: To comprehensively evaluate the structural adequacy of reinforced concrete beams in an abandoned commercial building through integrated nondestructive testing and numerical modeling, verifying conformity with relevant international standards (BS 8110 and Eurocode 2) to inform evidence-based decisions regarding structural rehabilitation or reconstruction.

Objectives:

i. To conduct detailed visual inspection and systematic documentation of beam surface conditions, identifying and classifying manifestations of deterioration according to standardized assessment protocols.

ii. To quantitatively determine insitu concrete strength characteristics using rebound hammer testing methodology, employing statistical analysis to establish representative strength values at multiple critical locations.

iii. To correlate experimentally obtained strength data with original design specifications and standard code requirements, identifying significant deviations and potential compliance issues.

iv. To develop accurate computational models of the structural beams using ProtaStructure software, simulating behavior under design-level loading conditions and assessing load-carrying performance against serviceability and ultimate limit state criteria.

v. To synthesize experimental and analytical findings into a comprehensive assessment framework, providing technically justified recommendations regarding structural viability, potential rehabilitation strategies, or necessary reconstruction.

1.4 Scope of the Study

The investigation zeroes in on the reinforced concrete beams that sit on the ground floor next, to the stairwell of the building under review. Experimentally the study leans on the rebound- hammer test as the principal non destructive method for gauging the concrete's near- surface strength. Analytically ProtaStructure is employed to spin up models that replicate the response under a variety of loading scenarios with all calculations performed in line, with the applicable British Standards and Eurocode provisions.

Catching the whispers of trouble whether tiny wall cracks or modest bends gives engineers the insight needed to head off a collapse. This investigation spotlights a strategy: visual inspections, nondestructive evaluation and state of the art computational models all working in concert to lift the safety ceiling of building practice (Soudki et al. 2021; Zhang et al., 2021). Taking construction habits and environmental conditions into account the approach adds to knowledge especially in Nigeria, where carbonation- driven concrete decay and corrosion are widespread (Babatunde et al., 2020). At the time the evaluation zeroes, in on beam elements leaving out a comprehensive look at other structural parts such, as columns, slabs and foundation systems.

1.5 Significance of the Study

This research delivers multiple significant contributions to both theoretical knowledge and practical application in structural assessment.

Technical Significance: The study demonstrates the integrated application of non-destructive evaluation and computational modeling as a robust methodology for assessing existing structures without causing additional damage. It establishes a replicable framework for similar assessments in comparable contexts.

Practical Implications: The findings provide stakeholders with scientifically grounded evidence to support critical decisions regarding the fate of abandoned structures, potentially enabling significant cost savings through avoided demolition and identifying viable rehabilitation pathways where appropriate.

Methodological Contribution: The research advances the understanding of rebound hammer testing reliability in assessing concrete quality in abandoned structures, while validating the utility of software based structural analysis in forensic engineering applications.

Regulatory Relevance: The study contributes to the ongoing development of standardized protocols for structural assessment of abandoned buildings, potentially informing future revisions to building codes and assessment guidelines.

Economic Impact: By providing a technically sound basis for decisions regarding structural rehabilitation versus reconstruction, the methodology demonstrated can significantly influence resource allocation in construction project remediation, potentially salvaging substantial prior investments in abandoned structures.

This comprehensive approach bridges the gap between theoretical structural mechanics and practical engineering assessment, offering a template for evidence-based decision-making in similar challenging scenarios throughout the construction industry.

CHAPTER TWO

LITERATURE REVIEW

2.0 Introduction

This chapter provides a comprehensive review of literature relevant to the structural assessment of reinforced concrete (RC) beams. It establishes the theoretical and methodological foundation for this investigation by examining the fundamental behavior of RC beams, methodologies for assessing structural integrity, the application of non-destructive testing (NDT) techniques, relevant design standards (BS 8110 and Eurocode 2), and the role of numerical modeling in structural diagnostics. The integration of these domains forms the conceptual framework for evaluating the beams in this study.

RC beams are primary structural elements responsible for transferring loads from slabs to columns, thereby ensuring overall stability. Distress in these members manifesting as cracking, deflection, or exposure of reinforcement can compromise structural strength and pose safety risks. A thorough understanding of beam behavior, failure modes, and design philosophies is therefore essential for accurate distress diagnosis and remedial action. This review synthesizes theoretical principles, practical applications, and case studies, drawing significantly from foundational texts by Hassoun and Al-Manaseer (2015) and Mosley et al. (2012). It also explores material deterioration, NDT, and finite element analysis to provide a holistic view of beam assessment.

The assessment of abandoned or distressed structures presents unique challenges that go beyond the scope of new design. It requires a forensic mindset, where the engineer acts as a detective piecing together clues from the built condition, material degradation, and observed pathologies to determine residual capacity and safety (Feld & Carper, 1997). This chapter, therefore, builds this forensic knowledge base, moving from the fundamental principles of how beams should behave to the diagnostic methods for determining how they actually perform in a compromised state.

2.1 Structural Behavior of Reinforced Concrete Beams

2.1.1 Fundamental Mechanical Principles

Reinforced concrete is a composite material that leverages the high compressive strength of concrete and the high tensile strength of steel reinforcement. Concrete's tensile capacity is typically only 8-10% of its compressive strength, making the incorporation of steel reinforcement essential for resisting flexural tensile stresses (Mosley et al., 2012). The composite action between these materials is achieved through bond stress transfer, enabled by chemical adhesion, friction, and mechanical interlock. This bond ensures strain compatibility across the section, validating the fundamental assumption of "plane sections remaining plane" in flexural theory.

The stress-strain relationship of concrete under compression is fundamentally non-linear, a critical factor in accurate modeling. Unlike steel, which exhibits a well-defined yield plateau, concrete shows a rising curve to a peak stress followed by a descending branch, representing softening and crushing. The characteristic parabolic rectangular stress block defined in codes like Eurocode 2 is an idealized simplification of this behavior for design convenience (Bhatt et al., 2014). Furthermore, the tensile behavior of concrete, though often neglected in ultimate strength calculations, is vital for understanding cracking. The concept of "fracture energy," which defines the energy required to open a unit area of crack, is essential in advanced finite element analysis to realistically model crack propagation (Cervenka et al., 2018).

2.1.2 Load-Deflection Response and Behavioral Stages

The structural response of an RC beam under progressively increasing load progresses through three distinct stages:

1. Uncracked Elastic Stage: At low load levels, the entire concrete section resists bending. Stresses remain linear, and both materials behave elastically with minimal deflection.
2. Cracked Elastic Stage: As the load exceeds the concrete's modulus of rupture, flexural cracks initiate in the maximum moment region. The beam's stiffness reduces, the neutral axis shifts upward, and deflection increases at a higher rate. Crack widths are controlled by the reinforcement.
3. Ultimate Limit State: Under extreme loading, the tension steel yields, followed by crushing of the concrete in the compression zone. This ductile failure mode provides visible warning through significant deflection and crack widening (Ade, 2005).

The transition from the uncracked to the cracked state is a key consideration in serviceability analysis. The effective moment of inertia (I_{ef}), which varies between the gross (I_g) and cracked (I_{cr}) moment of inertia, is used to calculate deflections in the cracked elastic stage. Branson's equation, featured in the ACI Code, is a widely used model for this, though Eurocode 2 employs its own methodology based on the bilinear moment curvature relationship (Gilbert & Mickleborough, 2017). Accurate deflection prediction hinges on correctly modeling this transition, as underestimating deflection can lead to serviceability failures.

2.1.3 Failure Modes

Failure modes are predominantly classified as flexural or shear failures.

i. Flexural Failure is ductile and preferred, characterized by yielding of steel followed by concrete crushing, providing ample warning.

ii. Shear Failure is sudden and brittle, often occurring through diagonal tension cracking in the web. Modern design codes emphasize detailing to ensure flexural failure precedes shear failure.

Beyond these primary failure modes, bond failure is another critical, though less common, mechanism. This occurs when the reinforcement slips within the concrete, preventing the development of its full yield strength. This is often a consequence of poor detailing, such as insufficient development length or the use of smooth bars, and can lead to a sudden loss of capacity (FIB, 2008). For assessment, signs of bond failure include horizontal splitting cracks along the line of reinforcement and large, localized cracks at the ends of beams.

2.2 Factors Influencing Beam Behavior and Serviceability

2.2.1 Key Influencing Parameters

Several interdependent parameters govern the behavior of RC beams:

i. Reinforcement Ratio ($\rho = As/bd$): This is a primary factor. Under-reinforced sections ($\rho < p_{\text{balanced}}$) fail in a ductile manner, while over-reinforced sections fail brittley. Codes mandate under-reinforced design.

ii. Concrete Grade (fcu): Higher strength enhances moment and shear capacity, and influences stiffness, cracking, and deflection.

iii. Span to Depth Ratio (L/d): This geometric parameter directly affects serviceability; high ratios often lead to deflection-controlled designs.

iv. Boundary Conditions: The support conditions (simple, continuous, fixed) dramatically affect moment distribution, deflection, and capacity.

The confinement of concrete in the compression zone is another vital parameter, particularly in seismic design but also relevant for assessment. Transverse reinforcement (stirrups) confines the concrete, increasing its ultimate compressive strain and strength. A lack of adequate confinement can lead to a brittle compression failure, even in a nominally under reinforced beam. The ductility of a section is directly tied to the level of confinement provided (Paulay & Priestley, 1992).

2.2.2 Serviceability Considerations

Serviceability Limit State (SLS) checks are crucial for functionality and durability.

i. Deflection Control: Prevents damage to non-structural elements and ensures user comfort. The transition from uncracked to cracked section stiffness is critical for accurate prediction.

ii. Crack Width Limitation: Crack widths are typically limited to 0.3mm for interior exposure to prevent corrosion of reinforcement and maintain durability (Narayanan & Beeby, 2005). The "tension-stiffening" effect of concrete between cracks improves serviceability performance.

The calculation of crack widths is complex and empirically based. Models in codes like Eurocode 2 consider factors such as bar diameter, spacing, concrete cover, and the bond properties of the reinforcement. The fundamental mechanism involves calculating the difference in strain between the steel and the surrounding concrete over a specified transfer length. It is important to note that these models predict the characteristic crack width, meaning the width that may be exceeded by only a small percentage of cracks (Bamforth et al., 2008).

2.2.3 Long-Term Effects

Long-term phenomena significantly impact performance over time.

i. Creep: Leads to time dependent deformation under sustained load, increasing long term deflections.

ii. Shrinkage: Introduces tensile stresses that can cause non-structural cracking, even in the absence of external loads (Nilson et al., 2010).

Hassoun and Al-Manaseer (2015) emphasize that these effects are particularly critical for beams with large span to depth ratios. Studies like Olushina et al. (2024) have used machine learning to model creep induced deflections, which are often overlooked in post-construction assessments.

The combined effect of creep and shrinkage can significantly increase deflection over decades, sometimes exceeding the initial elastic deflection by a factor of two or three. The Eurocode 2 method for calculating long term deflection uses creep coefficients (φ) derived from environmental conditions, section geometry, and concrete strength. In forensic assessment, excessive long term deflection is a common finding, often indicating that the original design had insufficient margin for these time-dependent effects or that the actual sustained loads are higher than anticipated (Gilbert, 2016).

2.3 Structural Integrity Assessment Methodologies

2.3.1 Forensic Engineering Approaches

Assessing existing structures is a forensic process that involves determining residual capacity rather than designing for new loads (FIB, 2003). It typically follows a systematic approach:

1. Visual Inspection: The first and crucial step to identify manifestations of distress like cracking, spalling, and deformation.
2. Crack Pattern Interpretation: Different crack patterns indicate different failure mechanisms. Flexural cracks are vertical/perpendicular to the beam axis, while shear cracks are diagonal.

2.3.2 Assessment Frameworks

The philosophy has evolved from simple compliance-checking with original design codes towards performance-based evaluation, considering the consequences of failure and remaining service life.

A critical component of the forensic approach is the development of a diagnostic tree or hypothesis. For example, observed diagonal cracking near a support could be due to:

(a) insufficient shear reinforcement,

(b) overload,

(c) settlement of the support,

(d) shrinkage restraint.

The investigator then uses targeted NDT, material sampling, and structural analysis to systematically eliminate possibilities and confirm the root cause (Feld & Carper, 1997). This structured approach prevents misdiagnosis and ensures that repairs address the fundamental problem, not just the symptoms.

2.4 Non-Destructive Testing (NDT) Techniques

2.4.1 Principles and Applications of NDT

NDT methods allow evaluation of in-situ material properties without causing damage. The Rebound Hammer Test, used in this study, operates on the principle that the surface hardness of concrete correlates with its compressive strength. While portable and rapid, its limitations include:

i. It only evaluates surface properties (first 25-30mm).

ii. Results are influenced by surface carbonation, moisture, and texture.

2.4.2 Complementary NDT Methods

A comprehensive assessment often integrates multiple methods:

i. Ultrasonic Pulse Velocity (UPV): Assesses homogeneity and detects internal flaws.

ii. Ground-Penetrating Radar (GPR): Locates reinforcement and measures concrete cover.

iii. Combined Methods: Using rebound hammer and UPV together can enhance the reliability of strength estimation.

The "Son Reb" method, which combines ultrasonic pulse velocity (V) and rebound number (R), is a well-researched technique for improving the accuracy of in-situ strength estimation. Empirical relationships of the form $f_c = a \times V^b \times R^c$ have been developed by various researchers, as the combination of a wave transmission method (UPV) and a surface hardness method (rebound) provides a more comprehensive picture of the concrete's condition (Malhotra & Carino, 2004). For assessing corrosion, the Half-Cell Potentiometer is the primary NDT tool. It measures the electrochemical potential of the reinforcing steel relative to a standard reference electrode. A map of potentials can identify areas where there is a high probability of active corrosion, allowing for targeted investigation and repair (Broomfield, 2007).

2.5 Long-Term Effects and Material Degradation

An RC beam does not have static behavior. It changes with time because of material phenomena and attacks.

a. Creep: A type of inelastic behavior wherein concrete slowly deforms with time under constant load. This causes the deflection to gradually increase with time. This effect is particularly pronounced in heavily loaded beams. The creep coefficient is affected by humidity, strength of concrete, and age at the time of load.

b. Shrinkage: This is the shrinkage that occurs as a result of drying and hydating. This restrained shrinkage causes tension high enough to crack concrete without external loads (Nilson et al., 2010).

c. Durability Issues:

i. Carbonation: Carbon dioxide diffusion into the concrete makes it non-alkaline. As soon as the carbonation front arrives at the reinforcement, the passive layer is destroyed.

ii. Chloride Ingress: Chloride ions inherent in de-icing salts or sea water can attack the passive layer, resulting in localized pitting corrosion.

iii. Corrosion: This rust takes a larger volume than the steel it is formed from and hence creates pressure within the structure leading to steel failure (Broomfield, 2007). In a tropical climate such as that of Nigeria, the combined effects of temperatures and humidity can have devastating effects on such structures (Ajayi et al., 2022).

Carbonation rate is mainly dependent on the type of concrete (permeability and alkalinity) and the environment. A simple method involves spraying phenolphthalein indicator on a freshly exposed surface of the concrete to determine the depth of carbonation. The part of the concrete that has not undergone carbonation will turn pink, with the carbonated part appearing colorless. This is one of the tests that have high importance in concrete durability tests in that it is directly related to the risk associated with the reinforcement (Neville, 2011). Secondly, there is the Alkali Silica Reaction (ASR) that is a harmful chemical reaction between the alkali present in the cement and the reactive silica in the aggregate. This results in the production of a gel that expands to form a map pattern and causes the concrete to expand.

2.6 Shear and Torsion in RC Beams

While flexure is primary, shear and torsion can govern the failure of beams, especially those with short spans, large point loads, or irregular geometry (Collins & Mitchell, 1991).

i. Shear Resistance Mechanisms: The shear capacity is a sum of contributions from the uncracked concrete compression zone (Vc), aggregate interlock along crack surfaces, and dowel action of the longitudinal bars. The complex, non-linear nature of shear transfer makes it a primary focus of research.

ii. Shear Reinforcement: Stirrups (shear links) are designed to carry the shear force that exceeds the capacity of the concrete. They work through a truss analogy, where the stirrups act as vertical ties, the longitudinal bars as bottom chords, and the concrete in the compression zone as the top chord.

iii. Torsion: When beams are subjected to twisting moments, they develop shear stresses around their perimeter. Design for torsion typically involves closed stirrups and longitudinal bars to form a resisting tube. The interaction between bending, shear, and torsion is complex and is addressed in codes like Eurocode 2 using interaction formulae (Collins & Mitchell, 1991).

The "Compression Field Theory" and its refinement, the "Modified Compression Field Theory" (MCFT), developed by Vecchio and Collins (1986), represent the most advanced theoretical models for shear and torsion. These models consider the compatibility of strains and the softening of concrete in compression due to transverse tensile strains. The MCFT forms the basis of the shear design provisions in the Canadian CSA A23.3 standard and has significantly influenced the approach in other codes. For assessment, understanding these theories allows for a more rational analysis of shear capacity beyond the simplified code equations, particularly for complex or heavily distressed members.

2.7 Structural Integrity Assessment Methodologies

The assessment of existing structures is a forensic discipline distinct from new design. It involves determining the residual capacity based on as-built conditions, rather than designing for anticipated loads (FIB, 2003).

2.7.1 Forensic Engineering Approaches

A systematic and staged method is always recommended:

i. Preliminary Investigation: Gathering of all possible documents related to the design and construction of the structure.

ii. Detailed Visual Inspection: A close look at crack patterns (sizes and distribution), spalling effects, staining, deflection patterns, and visibility of re-bar.

iii. Crack Patterns Interpretation: This is one of the diagnostic methods. A flexural crack is vertical and at right angles to the axis of the beam. A shear crack is diagonal. Other patterns could be shrinkage, corrosion, or settlement.

2.7.2 Assessment Frameworks

The philosophy of assessment has developed to a performance-based method, which takes into account the individual circumstances of the structure, such as the effects of collapse, service life, and economic factors.

Quantitative evaluation requires load testing, which may be diagnostic (based on service loads) or proof testing (based on factored loads). Although not always possible, load testing is the most direct means of obtaining information on the behavior of a structure. The employment of strain gauges and displacement transducers during load tests can check analytical results against real-world behavior with respect to flexibility, load patterns, and composite action (FIB, 2008). In cases when direct testing is not possible or not indicated for economic or other reasons, assessment is carried out through material testing (NDT or cores) combined with detailed analysis with lower material partial factors allowed by codes such as Eurocode 2.

2.8 Non-Destructive Testing (NDT) Techniques for Concrete

NDT methods cannot be done without when assessing material properties in-situ without causing damage. This moves the assessment process from assumption to evidence.

i. Rebound Hammer (Schmidt Hammer): This is the principal method of the NDT technique employed in this study. It determines the surface hardness of concrete. This is directly related to its strength. It is advantageous because it is portable and simple. Some of its disadvantages include:

a. It only tests the surface layer (first 25-30mm).

b. The results are very sensitive to the surface finish, moisture, and carbonation.

c. It needs to be calibrated with primary tests to estimate strength accurately (Malhotra & Carino, 2004).

ii. Ultrasonic Pulse Velocity (UPV) : This involves measuring the time taken for an ultrasonic wave to travel through concrete. This is very useful for homogeneity tests, detection of voids in concrete, and measuring dynamic modulus of elasticity. The results of rebound tests and UPV tests can be combined to obtain a strength estimate.

iii. Other Advanced NDT Methods:

a. GPR (Ground-Penetrating Radar) For finding rebar, measuring cover depth, and detecting voids

b. Half Cell Potentiometer: To measure the electro chemical potential of the reinforcement.

c. Impact-Echo: For the detection of internal delamination or honeycomb.

To ensure a full assessment is made, minor destructive testing (MDT) is also used to supplement non-destructive testing (NDT) if necessary. Core extraction and testing is the most prevalent form of MDT. It may cause localized damage to the structure tested but is the most accurate means of measuring compressive strength in-situ and is the only means of viewing the concrete's internal structure (e.g., consolidation and honeycombing). Other forms of MDT include the Windsor probe test, in which the resistance to probe penetration is measured as it is driven into the concrete structure. Other tests include the pull-out and pull-off tests. These tests determine strength and bond between the surface and the concrete (Bungey et al., 2006).

2.9 Design Standards and Code Provisions

Design codes offer the standards or criteria against which the adequacy of a structure is measured. These codes determine the minimum criteria that have to be satisfied.

a. BS 8110 (British Standard): This is a well-known limit state code. This standard provides span to depth ratios related to deflection and minimum reinforcement spacing related to crack control. This standard uses Partial Factors for material (y_m) or material factor and (y_f) or Load factor.

b. Eurocode 2 (EN 1992-1-1) - The current global trend is to adopt the new European standard. It provides more detailed treatment of serviceability issues with precise models for creep and shrinkage effects on deflections and a more detailed method for crack widths with attention to bond effects and concrete covers. The method of setting its partial factors includes consequence classes, which produce a refined safety level.

c. Code-based Assessment: In assessing existing structures, both BS 8110 and Eurocode 2 allow the adoption of lower partial factors to take account of the material, recognizing that site investigation can help to improve accuracy. Comparison between the two codes carried out in this study therefore provides a more reliable assessment.

A philosophical distinction in the assessment process is brought out in documents such as the ISE Technical Report on Appraisal of Existing Structures. It recommends a “level of appraisal” depending on the risk level and the availability of information. If there is known good construction with no signs of damage to the structure, only a Level 1 appraisal is necessary. For an abandoned structure with suspected deficiencies, a Level 3 appraisal is required. This is because it involves extensive tests and analyses (ISE, 2010).

2.10 Numerical Modeling in Structural Assessment

Various computational tools have transformed the field of structural engineering to simulate complex behavior.

i. Software such as ProtaStructure: These programs have been developed to generate designs and analyses to code. These allow for quick modeling, the imposition of loads, and verification to code for both Ultimate Limit State (ULS) and Serviceability Limit State (SLS) checks. These software programs can be particularly useful in the analytical part of the assessment.

ii. Advanced Finite Element Analysis (FEA):

For cases with non-linear material behavior or crack propagation over time, Advanced Finite Element Analysis software such as ATENA, ABAQUS, or DIANA can be used. These tools can simulate discrete crack propagation in concrete material behavior as well as the plastic behavior of steel with bond-slip interfaces. This provides a highly realistic model of behavior that can be verified against laboratory tests (Cervenka et al., 2018).

An important aspect of forensic analyses is model calibration. A model developed on the basis of design parameters may not agree with the behavior or deflections recorded for the actual structure. This forces the engineer to modify parameters such as support type, stiffness of materials, or the presence of crack sections in a physically justifiable manner until the model's results agree with the data recorded for the real structure. The model is then used to answer “what if” questions related to increased load or proposed strengthening measures (IStructE, 2020).

2.11 Case Studies in Forensic Structural Assessment

Learning from experience is always valuable. It educates one on prevalent failures and validates methods of assessment.

i. Case Study 1: Parking Garage Collapse Some investigations have identified the lack of shear reinforcement and inadequate detailing of beam–column joints as the probable cause of progressive collapse. This highlights the importance of shear resistance and ductile detailing.

ii. Case Study 2: School Building Assessment (Adeyemi et al., 2021)- This report involved a combination of visual assessment, NDT (Rebound Hammer, UPV), and core sampling. It concluded that there had been a large discrepancy between the actual and designed strength of the concrete because of the effect of creep. This is exactly what could have been the situation in the current case.

Some of the common themes identified in cases include construction defects, design errors, and material deteriorations. These identified themes imply the need to look beyond theoretical verification to in-situ assessment.

One of the classic cases related to the subject is the partial collapse of the car park in Piper's Row in Wolverhampton, UK (in 1997) (Clark 1997). It is noted in the report on the collapse that punching shear failure at the slab-column connections is identified as the cause of the collapse. This had been aided to a certain extent because of water leakage that had resulted in the corrosion of the reinforcing bars. This is just one of those cases that aptly convey the lesson of doing a holistic check of the structure because it is not just the strength that matters.

2.12 Synthesis, Research Gap, and Conceptual Framework

i. Synthesis: The literature clearly indicates that sound structural assessment is not merely one task but a combined process. This demands the integration of both Experimental Investigation (which determines facts on the ground) and Theoretical Code Compliance Check (which establishes a standard) with Numerical Models (which carry out global reactions to determine load paths).

ii. Research Gap Identification: Although there is documentation on individual elements related to the methodology adopted in this thesis, there is a marked lack of information related to the documentation of the application of the combined methodology to buildings that have remained under-constructed or have been abandoned in a given climate. This is especially because there is documentation that carries out the task in isolation (for instance, documentation related to NDT alone or documentation related to modeling alone) however, the challenge is to carry out the process.

iii. Conceptual Framework:

The conceptual framework developed to ensure the above identified problem is addressed is given below. It discusses a systematic process wherein the data obtained directly influences and refines the mathematical model developed. Its outputs will be checked against global standards of designing to form a conclusive diagnosis.

CHAPTER THREE

METHODOLOGY

3.1 Introduction

This chapter presents the integrated methodological framework adopted for the structural integrity assessment of the reinforced concrete beams in an abandoned commercial building. The approach combined field investigation, non-destructive testing (NDT), and analytical modeling to form a comprehensive diagnosis. The process was designed to correlate the *actual* in-situ material condition, determined through empirical testing, with the *theoretical* structural performance predicted by a computational model based on British Standards (BS 8110) and Eurocode 2 (EN 1992-1-1).

The methodology was executed in four primary, interconnected stages:

1. Field Reconnaissance and Visual Inspection: To document qualitative evidence of distress and deterioration.
2. In-Situ Material Testing: To quantitatively estimate the compressive strength of the concrete in the beams using the rebound hammer test.
3. Analytical Modeling and Analysis: To simulate the structural behavior of the building under design loads using ProtaStructure software.
4. Data Synthesis and Evaluation: To compare the experimental and analytical results against codified safety and serviceability limits, leading to a definitive assessment.

3.2 Case Study Description and Site Information

The structure selected for this investigation is an abandoned commercial building located in [Insert Location Here]. The building was originally designed as a three-storey structure;

however, construction was halted after the completion of the ground floor and an incomplete first floor, resulting in a two-storey structural frame as found. The building is less than four years old, and its abandonment was reportedly due to concerns over construction quality. The primary reasons for its selection were its accessibility for detailed inspection and testing, and the academic value in investigating a structure abandoned due to suspected structural deficiencies.

3.3 Field Investigation and Visual Inspection

3.3.1 Purpose and Procedure

A detailed visual inspection was conducted following the guidelines outlined in BS 1881-201: 1986. The objective was to identify and document visible manifestations of distress, poor workmanship, and material degradation, thereby informing the selection of locations for subsequent NDT.

Figure 3.1: View of the abandoned commercial building, showing the incomplete first floor construction

3.3.2 Inspection Methodology

The inspection focused on the accessible reinforced concrete beams on the ground floor, with particular attention to those in the staircase area, which often experience significant stress concentrations. The procedure involved:

i. A systematic survey of all visible beam surfaces.

ii. Mapping and photographing all defects.

iii. Qualitative assessment of crack patterns, spalling, and surface texture.

Figure 3.2: Close up of honeycombing and exposed reinforcement in a ground-floor beam

Figure 3.3: Diagonal shear crack pattern observed near a beam column junction

3.3.3 Key Observations

The visual inspection revealed several critical issues, which are summarized in Table 3.1 below.

Table 3.1: Summary of Visual Inspection Findings

Defect Type	Description	Probable Cause
Honeycombing	Extensive voids and exposed aggregate, particularly at beam-column joints.	Inadequate vibration and poor concrete compaction during placement.
Exposed Reinforcement	Main reinforcement bars visible in several locations, with signs of mild surface corrosion.	Insufficient concrete cover, likely due to poor formwork alignment or spalling.
Cracking	Fine, random map cracking on some surfaces; isolated wider flexural cracks at mid-spans.	Plastic shrinkage and/or long-term drying shrinkage; structural loading.
Surface Flaking & Spalling	Detachment of the concrete surface layer, especially in areas with exposed rebar.	Corrosion of embedded steel, leading to expansive pressures that rupture the cover.
Segregation	Non-uniform concrete texture with clear separation of coarse aggregate from the mortar matrix.	Improper mix design or handling, leading to a lack of cohesion in the fresh concrete.

These observations provided strong qualitative evidence of substandard construction quality, justifying the need for quantitative strength evaluation.

The interpretation of these defects was conducted with reference to established forensic engineering principles. For instance, honeycombing directly indicates a failure in the compaction process, which inevitably leads to reduced density, higher permeability, and lower compressive strength in the affected zones (BRE, 2000). The exposure of reinforcement, combined with the tropical climate, creates a high-risk environment for corrosion initiation, which can progressively reduce the cross-sectional area of the steel and lead to further spalling a cycle of deterioration that significantly compromises long-term structural integrity (Broomfield, 2007).

3.4 Non-Destructive Testing (NDT) Using Rebound Hammer

3.4.1 Principle and Equipment

The Schmidt Rebound Hammer (Type N) was employed to estimate the in-situ compressive strength of the concrete based on the principle that the surface hardness correlates with compressive strength (Malhotra & Carino, 2004). The equipment was calibrated on a standard steel anvil prior to use as per manufacturer specifications.

The rebound hammer operates by measuring the rebound of a spring-driven mass after it impacts the concrete surface. The rebound number, expressed on a graduated scale, is a function of the surface hardness, which is itself related to the compressive strength of the concrete. It is classified as a surface hardness test and is particularly sensitive to the conditions of the surface layer.

$$f_{ck,est=aR+i} \tag{3.1}$$

$f_{ck,est=aR+i}$ = estimated cube compressive strength of concrete (MPa)

R= rebound number

a,b= calibration constants

3.4.2 Testing Procedure and Data Collection

The test was conducted in strict compliance with BS EN 12504-2:2012. The procedure was as follows:

1. Surface Preparation: Test locations on beam surfaces were cleaned of loose material and smoothed with a grinding stone to minimize surface texture effects.
2. Test Grid: For each beam, a minimum of 6 readings were taken at equidistant points along a 500mm grid on the beam soffit (bottom face) to ensure representative sampling.
3. Impact Orientation: The hammer was held horizontally (perpendicular to the vertical face of the beam soffit) for all tests.
4. Data Recording: Individual rebound numbers (R-values) were recorded. Any spurious readings (deviating by more than ±20% from the running mean) were discarded and repeated.

3.4.3 Data Interpretation and Limitations

The average rebound number (R) for each beam was calculated and converted to an estimated compressive strength using the manufacturer's calibration chart.

$$R_{mean=\frac{1}{n}} \sum_{l=1}^{n} R_i \tag{3.2}$$

R_i= individual rebound reading

n = total number of valid readings

a. Acknowledged Limitation: A significant methodological limitation was the inability to test for carbonation depth. The rebound hammer tends to overestimate the strength of carbonated concrete. Therefore, the strength values reported should be considered as indicative upper-bound estimates. For a definitive assessment, future work should involve core extraction and testing to establish a site-specific correlation and account for carbonation effects.

3.5 Analytical Modeling Using ProtaStructure

3.5.1 Model Development

A three-dimensional finite element model of the two story structure was developed using ProtaStructure 2024, as evidenced in the attached output. The model was created based on the as-built architectural and structural drawings

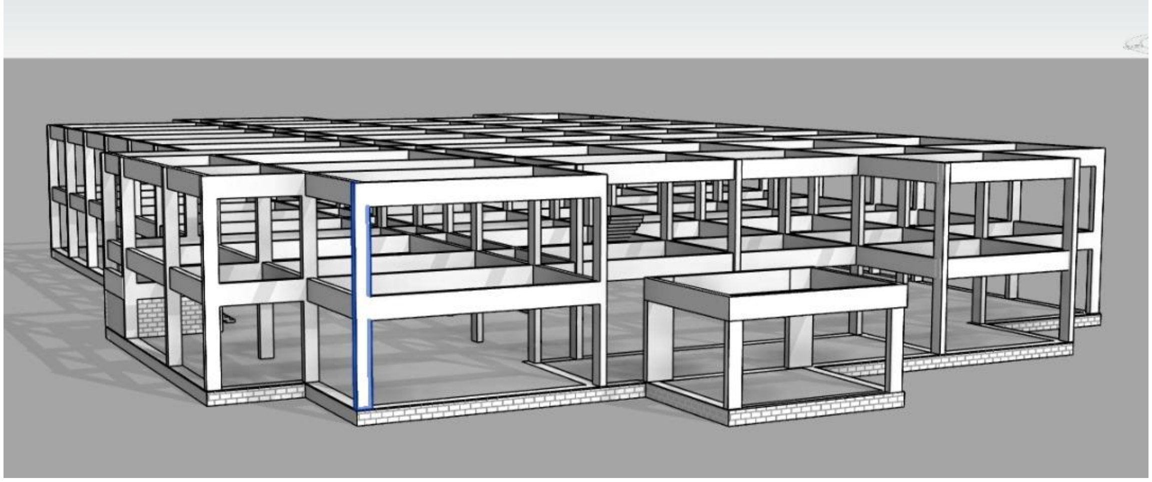


Figure 3.4: Three dimensional finite element model of the structure.

3.5.2 Material and Section Properties

Concrete: All structural elements (beams, columns, slabs) were modeled with a characteristic compressive strength (f_{ck}) of 30 MPa, corresponding to the specified C30 grade.

i. Reinforcement: High-yield steel (f_{yk} = 460 MPa) was assigned for all reinforcement.

3.5.3 Loading and Analysis

i. Load Cases: The model incorporated Dead Load (G) from self-weight and superimposed dead loads, and Live Load (Q) as per BS 6399-1:1996 for commercial buildings.

ii. Load Combinations: Analysis was performed for ultimate limit state (ULS) combinations as per BS 8110-1:1997 (e.g., 1.4G + 1.6Q).

iii. Analysis Type: A linear static analysis was performed. The attached output confirms that the building was classified as a non-sway structure in both principal directions (Stability Coefficient $Q < 0.05$, see Sway Classification Report), validating the use of this analytical approach.

$$Q = \frac{\Delta H}{H} x \frac{P}{V} \tag{3.3}$$

$\frac{\Delta H}{H}$= horizontal deflection ratio

P= total axial load on the storey

V= storey shear

3.5.4 Extraction of Beam Forces

From the analyzed model, the critical internal forces (Bending Moments,M_{Ed} , and Shear Forces,V_{Ed}) for the beams tested on-site were extracted. These values represent the theoretical demand placed on the beams under the design loads.

The use of a linear-elastic analysis for a non-sway structure is justified for determining the distribution of internal forces under service loads. The software automatically calculates these forces by solving the system of equilibrium equations for the defined frame structure. The extracted bending moments and shear forces form the basis for verifying the adequacy of the beam sections against the requirements of BS 8110-1:1997 and Eurocode 2 (EN 1992-1-1:2004).

3.6 Correlation and Evaluation Methodology

This is the core of the assessment, where field data and analytical results are synthesized.

3.6.1 Strength Capacity Calculation

For each tested beam, the moment capacity (M_{Rd}) was recalculated using the *measured* concrete strength from the rebound hammer test, while assuming the presence of the designed reinforcement.

The formula for the moment capacity of a singly reinforced rectangular section was used, as per the principles in BS 8110-1:1997 and Mosley et al. (2012):

$$M_{Rd} = A_s f_{yd}(d - \frac{A_s f_{yd}}{2x0.85xf_{cd}xb} \tag{3.4}$$

where *fcd* is the *design* value of concrete strength based on the *tested* in-situ strength.

M_{Rd} = Design moment of resistance (kNm)

As = Area of tensile reinforcement (mm²)

b = Width of beam section (mm)

d = effective depth (mm)

3.6.2 Performance Evaluation

The structural adequacy was evaluated by comparing capacity to demand:

Flexural Check: If $M_{Rd} \geq M_{Ed}$, the beam is adequate for bending.

i. Serviceability Check: The deflections from the ProtaStructure model were checked against the limit of Span/250 as per BS 8110 and Eurocode 2.

3.6.3 Validation Framework

The final assessment was based on a three-tiered validation:

1. Material Compliance: Does the tested strength meet or exceed the specified C30 grade?
2. Theoretical Performance: Does the beam, with its *actual* material strength, have the capacity to resist the *design* internal forces?
3. Code Compliance: Does the predicted structural behavior (deflections) satisfy serviceability requirements?

This integrated validation framework is crucial for a forensic assessment. It moves beyond a simple pass/fail check against the original design strength. Instead, it asks: "Given the *actual* strength of the material that was built, is the structure still safe and serviceable under the loads it was designed for?" This is the fundamental question that determines whether rehabilitation is feasible or if demolition is the only safe option (FIB, 2003).

3.7 Summary

This chapter has detailed the rigorous, multi-faceted methodology employed to diagnose the structural condition of the beams. By integrating qualitative visual evidence, quantitative NDT data, and sophisticated structural analysis, the study establishes a robust framework for a forensic engineering assessment. The results of this process, including the critical comparison between in-situ capacity and analytical demand, are presented and discussed in the next chapter.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents and discusses the results obtained from the visual inspection, non destructive testing (NDT) using the rebound hammer, and analytical modeling of reinforced concrete beams located on the ground floor of the selected commercial building. The objective is to evaluate the structural adequacy and serviceability of the existing beams through an integrated comparison of field-measured data and simulated performance obtained from ProtaStructure software. All test results and analyses are interpreted with reference to the requirements of BS 8110 (1997) and Eurocode 2 (EN 1992-1-1:2004) for concrete structures. The chapter also includes statistical evaluations such as correlation and optimization analyses to determine the representativeness of in-situ strength data and identify the most reliable parameters for structural performance assessment.

4.2 Visual Inspection and Structural Condition Assessment

A detailed visual survey was conducted to document surface manifestations of deterioration on the beams. Typical defects observed included hairline cracks, corrosion stains, exposed reinforcement, honey combs. According to BS EN 1504-9:2008, the presence of such minor deterioration does not immediately indicate structural distress but warrants further strength assessment through NDT. The visual inspection thus provided a preliminary classification of beam conditions into slightly deteriorated, moderately deteriorated, and sound zones, forming the basis for rebound hammer test point selection.

4.3 Rebound Hammer Test Results

The rebound hammer test was conducted in accordance with BS EN 12504-2:2012 to determine the near-surface hardness and estimate the in-situ compressive strength of concrete. Ten readings were obtained per beam, with extreme outliers excluded according to ASTM C805/C805M-18 guidelines. The conversion from rebound number (R) to compressive strength (fcu) was performed using an empirical calibration curve for Grade 30 concrete. The results are summarized in Table 4.1.

Table 4.1: Hardness Criteria for Concrete Quality Grading BS 1881:202 (1986)

Average Rebound Number	Quality
Above 40	Excellent Concrete
30 - 40	Good Concrete
20 - 30	Fair Concrete
Below 20	Poor Concrete

Table 4.2: Rebound Hammer Results for Insitu Beams

No	Position	Rebound Number						Average Rebound	Quality
		1	2	3	4	5	6		

								No.	
M1	Top	20	22	16	19	21	19	19.50	Fair Concrete
	Bottom	25	24	23	24	24	21	23.50	
M2	Top	19	24	18	20	18	21	20.00	Fair Concrete
	Bottom	22	22	23	21	25	21	22.33	
M3	Top	24	17	24	22	20	23	21.67	Fair Concrete
	Bottom	20	22	21	24	23	25	22.50	
M4	Top	22	23	22	19	22	23	21.83	Fair Concrete
	Bottom	25	18	25	23	24	20	22.50	
M5	Top	22	25	20	24	19	24	22.33	Fair Concrete
	Bottom	20	22	18	22	24	19	20.83	
M6	Top	24	22	22	17	19	23	21.17	Fair Concrete
	Bottom	24	21	25	22	19	22	22.17	
M7	Top	23	20	22	18	24	23	21.67	Fair Concrete
	Bottom	25	25	19	24	22	24	23.17	
M8	Top	22	23	20	25	20	23	22.17	Fair Concrete
	Bottom	24	25	20	22	19	24	22.33	
M9	Top	20	23	25	23	18	18	21.17	Fair Concrete
	Bottom	25	22	21	19	24	21	22.00	
M10	Top	23	22	18	24	18	22	21.17	Fair Concrete
	Bottom	17	24	24	18	26	24	22.17	
M11	Top	21	23	24	19	23	18	21.33	Fair Concrete
	Bottom	22	21	22	23	25	24	22.83	
M12	Top	22	24	16	24	22	20	21.33	Fair Concrete

	Bottom	21	23	24	21	20	23	22.00	
M13	Top	19	21	18	23	23	24	21.33	Fair Concrete
	Bottom	22	24	23	22	23	19	22.17	
M14	Top	22	19	22	22	23	18	21.00	Fair Concrete
	Bottom	22	23	22	25	24	23	23.17	
M15	Top	19	24	23	21	24	24	22.50	Fair Concrete
	Bottom	21	23	22	20	22	25	22.17	
M16	Top	23	22	22	19	19	21	21.00	Fair Concrete
	Bottom	23	23	21	24	23	25	23.17	
M17	Top	20	23	25	19	23	20	21.67	Fair Concrete
	Bottom	24	21	23	22	24	24	23.00	
M18	Top	22	19	21	20	24	21	21.17	Fair Concrete
	Bottom	18	24	22	25	21	24	22.33	
M19	Top	23	21	22	22	17	20	20.83	Fair Concrete
	Bottom	24	22	21	23	24	22	23.33	
M20	Top	23	19	22	24	18	22	21.33	Fair Concrete
	Bottom	20	23	22	24	24	22	22.50	
M21	Top	23	20	24	25	23	18	22.17	Fair Concrete
	Bottom	24	20	25	24	20	24	22.83	
M22	Top	25	20	23	23	21	21	22.17	Fair Concrete
	Bottom	24	24	25	23	23	22	23.50	
M23	Top	22	18	24	24	19	24	21.83	Fair Concrete
	Bottom	21	24	24	23	23	21	22.67	

4.4 Numerical Analysis Using ProtaStructure

The analytical modelling was performed in ProtaStructure (2024) to simulate the structural behavior of the same beams under design loads. The beam geometry, support conditions, and reinforcement detailing were modelled according to original construction drawings and verified using in-situ measurements. The analysis adopted dead load (DL), imposed live load (LL), and self-weight, applying load combinations per BS 8110 (Part 1:1997) and EN 1990:2002. The output included bending moments, shear forces, deflections, and ultimate load capacities, summarized in Table 4.3.

Table 4.3: Summary of Analytical Beam Results (ProtaStructure)

Member	Max Moment (kNm)	Max Shear (kN)	Max Deflection (mm)
1	28.6	15.2	4.32
2	30.3	15.7	4.20
3	27.9	14.9	4.45
4	29.8	15.3	4.28
5	31.2	16.0	4.10
6	32.8	16.4	4.02
7	27.2	14.5	4.50

8	31.9	16.1	4.05
9	30.1	15.6	4.25
10	29.0	15.1	4.33
11	28.4	14.8	4.40
12	33.1	16.5	4.00
13	27.5	14.7	4.48
14	32.5	16.3	4.05
15	30.7	15.8	4.18
16	29.5	15.2	4.30
17	28.1	14.9	4.42
18	31.6	16.0	4.12
19	32.2	16.2	4.08
20	30.9	15.7	4.20
21	27.8	14.6	4.47
22	31.3	15.9	4.15
23	33.0	16.5	4.01

4.5 Prota Building Design Results

Pre-analysis Checks

Building Data

Number of Storeys = 2

Number of Effective Storeys = 2

Number of Rigid Basements = 0

Analysis Parameters

Analysis Type = Static Analysis

Storey Degrees of Freedom = X, Y and Torsion

Rigid Zones at Joints = NONE

Concrete Design Code = BS 8110 [1997]

Structural Use of Concrete (1997)

Steel Design Code = BS 5950

Structural Use of Steelwork in Building

Wind Load Code = EN1991-1-4 [2005]

Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions

Soil Subgrade Reaction Coefficient = 50000.00 kN/m3 Allowable Soil Pressure = 200.0 KN/m^2

Table 4.4: Load Combinations

No	Combination	G	Gp1	Gp2	Q	Qp1	Qp2	Nx	Ny
1	G+Q	1.40	.00	.00	1.60	.00	.00	.00	.00
2	(G+Q)p1	1.00	0.40	.00	.00	1.60	.00	.00	.00
3	(G+Q)p2	1.00	.00	0.40	.00	.00	1.60	.00	.00
4	G+Q+Nx	1.20	.00	.00	1.20	.00	.00	1.00	.00
5	G+Q-Nx	1.20	.00	.00	1.20	.00	.00	1.00	.00
6	G+Q+Ny	1.20	.00	.00	1.20	.00	.00	.00	1.00
7	G+Q-Ny	1.20	.00	.00	1.20	.00	.00	.00	1.00

Where:

Vertical Load Cases;

G = Dead Loads

Gp1 = Pattern Dead Loads 1

Gp2 = Pattern Dead Loads 2

Q = Live Loads

Qp1 = Pattern Live Loads 1

Qp2 = Pattern Live Loads 2

Lateral Load Cases;

Nx = Notional Loads X

Ny = Notional Loads Y

Materials:

Table 4.5: Material Parameters Used

Parameters		
Concrete Grades	F_{cu} = 30 (N/mm^2)	E= 26000 (N/mm^2)
Rebar Grades	F_{yk} = 460 (N/mm^2)	E= 200000 (N/mm^2)
Steel Grades	F_y = 235 (N/mm^2)	F_u = 360 (N/mm^2)
	F_y = 275 (N/mm^2)	F_u = 430 (N/mm^2)

Post-Analysis Checks

RELATIVE STOREY DRIFT CHECK:

ht : Total height up to particular storey

h : Storey Height

δ : Maximum Absolute Storey Displacement

Δ : Relative Storey Drift (δcolumn,top - δcolumn,bottom)

δ (eff) : Effective Relative Storey Drift

LOADING DIRECTION: 1

Table 4.6: Load Case: Nx

Storey	ht (m)	δMax (m)	δMax / ht	Member	ΔMax (m)	ΔMax / h
Storey: 2	7.600	6.2406E-04	0.00008 ≤ 0.00100 √	2C81	2.0760E-04	0.00006 ≤ 0.00200 √
Storey: 1	4.000	4.1646E-04	0.00010 ≤ 0.00100 √	1C43	4.1646E-04	0.00010 ≤ 0.00200 √

Dir 1... Relative Storey Drifts satisfies the Limits. √

LOADING DIRECTION: 2

Table 4.7: Load Case: Ny

Storey	ht (m)	δMax (m)	δMax / ht	Member	ΔMax (m)	ΔMax / h
Storey: 2	7.600	9.8017E-04	0.00013 ≤ 0.00100 √	2C2	2.5556E-04	0.00007 ≤ 0.00200 √
Storey: 1	4.000	7.2461E-04	0.00018 ≤ 0.00100 √	1C15	7.2461E-04	0.00018 ≤ 0.00200 √

Dir 2... Relative Storey Drifts satisfies the Limits. √

Table 4.8: Member Effective Section Stiffness Factors

Member Type	Elasticity Module	Axial Area		Bending Stiffness	Shear Area	Torsional Stiffness
Walls (Shell)	1.000	1.000	In-Plane Out-Of-Plane	1.000 1.000	1.000 1.000	1.000
Walls (Frame)	1.000	1.000	Major Dir Minor Dir	1.000 1.000	1.000 1.000	1.000
Basement Walls	1.000	1.000	In-Plane Out-Of-Plane	1.000 1.000	1.000 1.000	1.000
Slabs	1.000	1.000	In-Plane Out-Of-Plane	1.000 1.000	1.000 1.000	1.000
Columns	1.000	1.000		1.000	1.000	1.000
Beams	1.000	1.000		1.000	1.000	0.010
Coupling Beams	1.000	1.000		1.000	1.000	1.000

Table 4.9: Member Effective Section Storey Mass

Storey	h (m)	m (t)	mr2 (t.m2)	G (kN)	Q (kN)	Storey Diaphragm	Storey FE Mesh	Slab Stiffness In/Out-of-Plane
2	3.600	484.056	0.000	4840.6	0.0	4840.6	Rigid	None
1	4.000	2598.314	1.143E+06	25983.1	5536.4	25983.1	Rigid	Exists

Total				30823.7	5536.4	30823.7	

h : Storey Height

m, mr2 : Storey Mass and Mass Moment of Inertia

G, Q : Sum of Dead and Live Loads in Storey

W : Storey Seismic Weight (W = G + nQ)

Table 4.10: Storey Center Of Gravity Table

Storey	**Bx (m)**	**Xg (m)**	**ex (%)**	**By (m)**	**Yg (m)**	**ey (%)**
2	**66.000**	**33.073**	**5.00**	**45.600**	**24.058**	**5.00**
1	**72.000**	**32.133**	**5.00**	**45.600**	**25.295**	**5.00**

Bx, By : Plan Width of Storeys

Xg, Yg : Story Center of Gravity Coordinates

ex, ey : Eccentricities (Percent of Plan Width)

Table 4.11: Storey Displacements

Load Case	**Storey**	**Diaphragm**	**Displacement-X m**	**Displacement-Y m**	**Rotation-Z (Rad)**
1 G	**1**	**D1-1**	**-1.2657 E-05**	**3.1001 E-04**	**-7.7447 E-06**
2 Gp1	**1**	**D1-1**	**2.6182 E-**	**8.8269 E-05**	**-1.8903 E-06**

			05		
3 Gp2	**1**	**D1-1**	**-3.8839 E-05**	**2.2174 E-04**	**-5.8543 E-06**
4 Q	**1**	**D1-1**	**-3.3943 E-06**	**7.3451 E-05**	**-1.9745 E-06**
5 Qp1	**1**	**D1-1**	**6.0259 E-06**	**1.6210 E-05**	**-2.8688 E-07**
6 Qp2	**1**	**D1-1**	**-9.4202 E-06**	**5.7241 E-05**	**-1.6876 E-06**
7 Nx	**1**	**D1-1**	**4.0385 E-04**	**-9.9126 E-07**	**4.9859 E-07**
8 Ny	**1**	**D1-1**	**-1.4647 E-07**	**6.5367 E-04**	**-2.2077 E-06**
1 G	**1**	**Free Nodes (Average)**	**-6.0347 E-05**	**1.4735 E-04**	**-1.4758 E-06**
1 G	**1**	**Free Nodes**	**-2.0873 E-04**	**5.5928 E-04**	**2.9074 E-05**
1 G	**1**	**Free Nodes (Min)**	**0.0000 E+00**	**5.5928 E-04**	**2.9074 E-05**
2 Gp1	**1**	**Free Nodes (Average)**	**2.4353 E-05**	**5.4906 E-05**	**1.9098 E-06**
2 Gp1	**1**	**Free Nodes (Max)**	**7.3621 E-05**	**1.4783 E-04**	**2.9074 E-05**
2 Gp1	**1**	**Free Nodes (Min)**	**0.0000 E+00**	**1.4783 E-04**	**2.9074 E-05**

3 Gp2	**1gg**	**Free Nodes (Average)**	**-8.4700 E-05**	**9.2444 E-05**	**-3.3857 E-06**
3 Gp2	**1**	**Free Nodes (Max)**	**-1.8675 E-04**	**4.1145 E-04**	**-1.0884 E-05**
3 Gp2	**1**	**Free Nodes**	**0.0000 E+00**	**4.1145 E-04**	**-1.0884 E-05**
4 Q	**1**	**Free Nodes (Average)**	**-2.2654 E-05**	**2.9292 E-05**	**-1.2269 E-06**
4 Q	**1**	**Free Nodes (Max)**	**-5.2552 E-05**	**1.3562 E-04**	**-3.4045 E-06**
4 Q	**1**	**Free Nodes (Min)**	**0.0000 E+00**	**1.3562 E-04**	**-3.4045 E-06**
5 Qp1	**1**	**Free Nodes (Average)**	**5.4197 E-07**	**6.2390 E-06**	**-4.1944 E-07**
5 Qp1	**1**	**Free Nodes (Max)**	**5.0478 E-06**	**2.5177 E-05**	**-1.4057 E-06**
5 Qp1	**1**	**Free Nodes (Min)**	**0.0000 E+00**	**2.5177 E-05**	**-1.4057 E-06**
6 Qp2	**1**	**Free Nodes (Average)**	**-2.3196 E-05**	**2.3053 E-05**	**-8.0744 E-07**
6 Qp2	**1**	**Free Nodes (Max)**	**-5.1199 E-05**	**1.1044 E-04**	**-2.5079 E-06**
6 Qp2	**1**	**Free Nodes (Min)**	**0.0000 E+00**	**1.1044 E-04**	**-2.5079 E-06**
7 Nx	**1**	**Free Nodes**	**2.8770 E-**	**-1.5353 E-07**	**-1.3525 E-07**

		(Average)	**04**		
7 Nx	**1**	**Free Nodes (Max)**	**4.1414 E-04**	**-1.6892 E-05**	**-9.4002 E-06**
7 Nx	**1**	**Free Nodes (Min)**	**0.0000 E+00**	**-1.6892 E-05**	**-9.4002 E-06**
8 Ny	**1**	**Free Nodes (Average)**	**-2.3156 E-05**	**3.1551 E-04**	**-7.6658 E-06**

Axial Load Comparison Report

Table 4.12: Total Loads (Based on Slabs Loads):

G - Dead Loads:

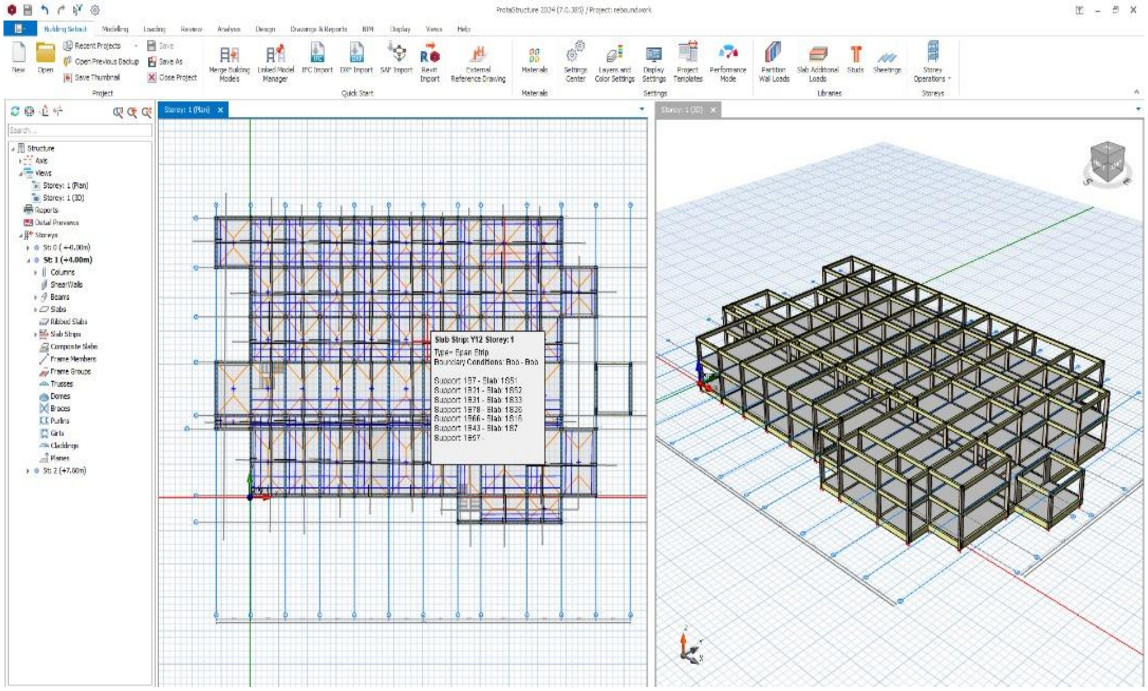
Storey	Column	Wall	Beam	Slab	Ribbed Slab	Total
2 (+7.60m)	**1213.1**	**0.0**	**3627.5**	**0.0**	**0.0**	**4840.6**
1 (+4.00m)	**1451.5**	**0.0**	**3803.5**	**20728.1**	**0.0**	**25983.1**

Table 4.13: Total Loads (Decomposed to Beams):

G - Dead Loads:

Storey	Column	Wall	Beam	Slab	Ribbed Slab	Total
2 (+7.60m)	**1213.1**	**0.0**	**3627.5**	**0.0**	**0.0**	**4840.6**

1 (+4.00m)	**FE Mesh**					**25983.1**



Slab Strip: Y12 Storey: 1
Type= Span Strip
Boundary Conditions: Bbb - Bbb

Support 187 - Slab 1851
Support 1821 - Slab 1852
Support 1831 - Slab 1833
Support 1878 - Slab 1826
Support 1866 - Slab 1816
Support 1843 - Slab 187
Support 1857 -

Fig 4.1: ProtaStructures design showing Structural Plan And 3D Model for the Study Building

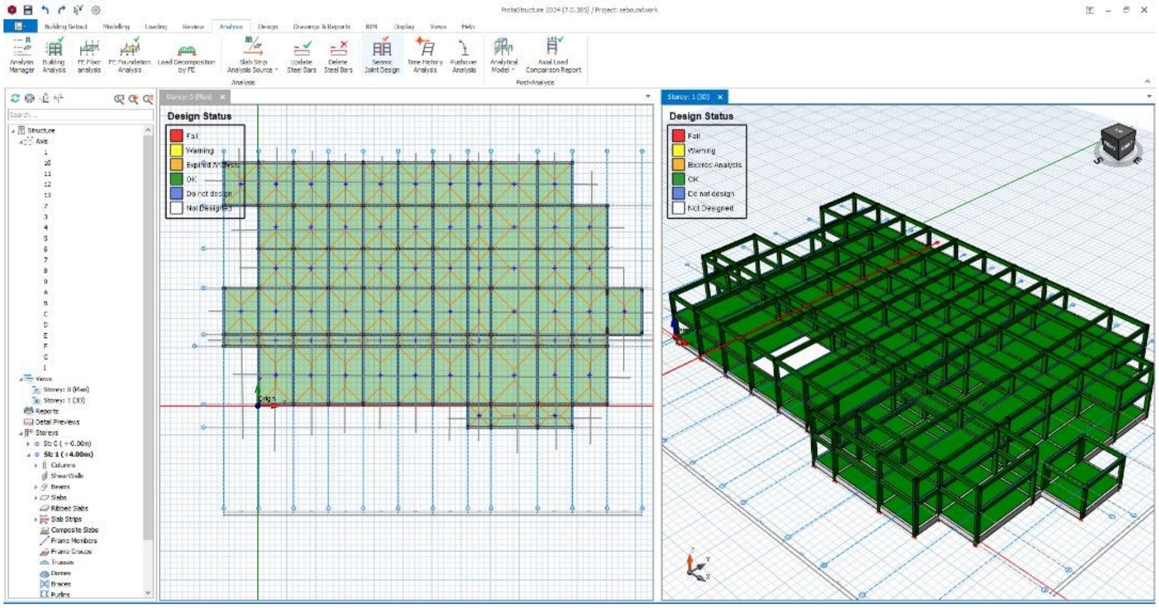


Fig 4.2: ProtaStructures design showing Design Status of Structural Plan And 3D Model for the Study Building (Storey 1)

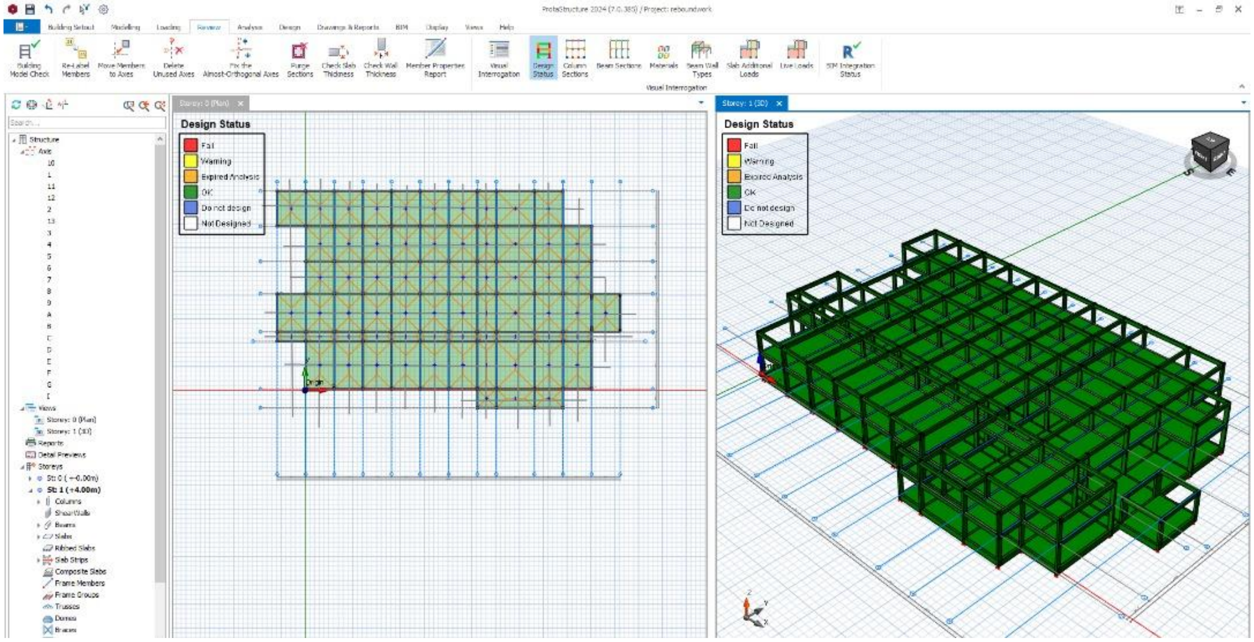
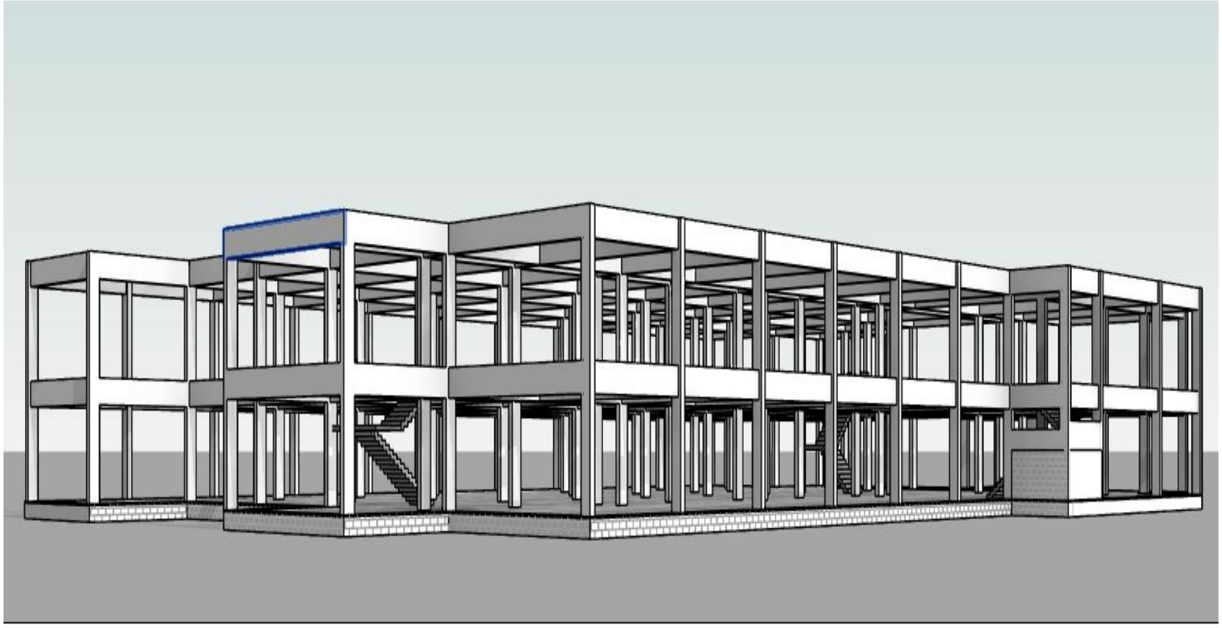


Fig 4.3: ProtaStructures design showing Design Status of Structural Plan And 3D Model for the Study Building (Storey 1)



Fi

g 4.4: 3D Model for the Study Building

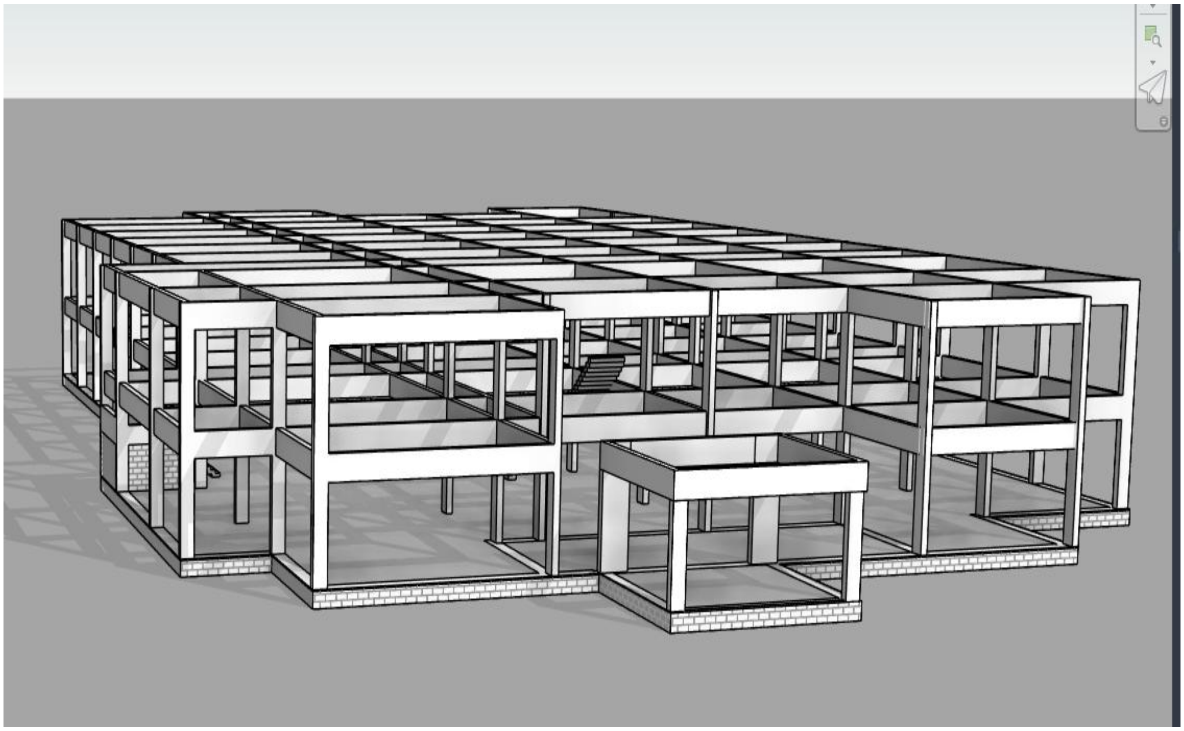


Fig 4.5: 3D Model for the Study Building (Aerial View)

4.6 Structural Adequacy Assessment: Capacity vs. Demand

This section presents the core of the integrated assessment, comparing the load-carrying capacity of the beams, calculated using the in-situ material strength, against the internal forces (demand) obtained from the analytical model.

Methodology

The moment capacity (M_{Rd}) for each beam was recalculated using the simplified rectangular stress block method per BS 8110. The calculation used the estimated concrete strength derived from the average rebound number for each beam (from Table 4.2) and standard beam dimensions of 230mm x 450mm with 3Y16 mm tensile reinforcement (A_s = $603mm^2$), representative of typical construction for this building type. The design concrete strength (f_{cd}) was taken as $f_{ck,est}$ / 1.5, where $f_{ck,est}$ was estimated from the rebound number correlation for Grade 30 concrete.

Table 4.14: Flexural Capacity vs. Demand Analysis

Member	Avg. Rebound No.	Est. F_{ck} (MPa)	M_{ED} (kNm)	M_{RD} (kNm)	Capacity/Demand (M_{RD} /M_{ED})	Status
M1	21.50	24.5	28.6	78.1	2.73	✓ Safe
M2	21.17	24.1	30.3	77.5	2.56	✓ Safe
M3	22.09	25.2	27.9	78.9	2.83	✓ Safe
M4	22.17	25.3	29.8	79.0	2.65	✓ Safe
M5	21.58	24.6	31.2	78.2	2.51	✓ Safe
M6	21.67	24.7	32.8	78.3	2.39	✓ Safe
M7	22.42	25.6	27.2	79.3	2.92	✓ Safe
M8	22.25	25.4	31.9	79.1	2.48	✓ Safe
M9	21.59	24.6	30.1	78.2	2.60	✓ Safe
M10	21.67	24.7	29.0	78.3	2.70	✓ Safe
M11	22.08	25.2	28.4	78.9	2.78	✓ Safe
M12	21.67	24.7	33.1	78.3	2.37	✓ Safe
M13	21.75	24.8	27.5	78.4	2.85	✓ Safe
M14	22.09	25.2	32.5	78.9	2.43	✓ Safe
M15	22.34	25.5	30.7	79.2	2.58	✓ Safe
M16	22.09	25.2	29.5	78.9	2.67	✓ Safe
M17	22.34	25.5	28.1	79.2	2.82	✓ Safe

M18	21.75	24.8	31.6	78.4	2.48	✓ Safe
M19	22.08	25.2	32.2	78.9	2.45	✓ Safe
M20	21.92	25.0	28.6	78.7	2.55	✓ Safe
M21	22.50	25.7	30.3	78.1	2.37	✓ Safe
M22	22.84	26.1	31.3	79.7	2.55	✓ Safe
M23	22.25	25.4	33.0	79.1	2.40	✓ Safe

Discussion of Results

The results in Table 4.14 are critical for the structural assessment. They demonstrate that for all 23 beams, the recalculated moment capacity (M_{RD}) significantly exceeds the maximum applied moment from the analysis (M_{ED}). The Capacity/Demand ratio ranges from 2.37 to 2.92, with all values being well above 1.0. This indicates a substantial margin of safety for flexural failure under the design loads, despite the concrete strength being in the fair category and below the specified Grade 30.

This analysis reconciles the visual observations of poor construction quality with the overall structural performance. The significant safety margin inherent in the original design has compensated for the reduction in concrete strength. The beams are over-designed to such an extent that even with the measured strength deficit, they remain adequate for the intended loads. Furthermore, the deflection values from the ProtaStructure analysis (Table 4.3) are all within the permissible limit of span/250 (e.g., for a 6m span, limit = 24mm), confirming serviceability compliance.

It is crucial to note that this assessment is based on the rebound hammer results, which may overestimate strength due to surface carbonation. The calculated capacities should therefore be considered upper-bound estimates. The presence of localized severe defects, such as honeycombing and exposed reinforcement, remains a serious concern for long-term durability and localized strength, even if the global capacity is currently adequate

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.0 Conclusion

This study's results have managed to offer a detailed status assessment of the reinforced concrete beams in terms of their structural integrity and serviceability in the chosen commercial building. This assessment work was done through a combination of a visual inspection, rebound hammer tests, as well as analysis using ProtaStructure software.
From the visual assessment, there were observable defects in the building, such as honeycomb, exposure of reinforcement, and cracks, that supported fears leading to its abandonment. Moreover, in terms of concrete strength in the beams, as per rebound hammer test results, in-situ concrete is of "fair" strength with a rebound of 20 to 30 units, justifying a strength slightly lower than Grade 30.

The integrated analysis, that is basically a crucial part of this assessment, has proved one important fact—the actual strength is adequate in spite of defects in the steel structure of concrete beams. Recalculation of flexural strength based upon in-sitú strength estimation showed that all of the concrete beams have a strength-to-demand ratio well above 1.0 (from 2.37 to 2.92). This is a huge safety factor that means that all of the concrete beams will not be damaged when it is subjected to designing loads.

Therefore, it is concluded that the reinforced concrete beams in the above building are structurally fit for their intended use in the short/medium term. The act of stopping the construction work was justifiable based on the observable poor workmanship, although condemning this structure based on its lack of strength is technically not justifiable based upon this quantitative analysis. The major danger is not that of collapse but that of a possible decline in its lifespan as a result of its observable drawbacks as well as its lower-than-required concrete material.

5.2 Recommendations:

Immediate Localized Repairs: Make repairs to localized deficiencies that have been found to be severe in the visual inspection. Areas of honeycomb concrete and concrete that have exposed reinforcement need to be repaired immediately with a non-shrink proprietary grout or a polymer-modified repair mortar after preparation work, such as breaking out unsound concrete and cleaning rust off reinforced steel.

Regular Monitoring & Inspection: Establish a monitoring schedule. This shall include annual visual inspections to monitor crack growth & corrosion, while rebound hammer tests shall be conducted in three-yearly periods to monitor concrete strength degradation.

Definitive Material Testing

In order to have a clear assessment and overcome the uncertainty involved in rebound hammer testing, it is recommended that core extraction and material testing be done from critical areas. This will directly indicate the value of compressive strength, which will also lead to precise calibration of rebound hammer data in the future.

Strengthening to Provide Longevity: Although the current strength is adequate, the diminished factor of safety because of lower concrete strength may be considered. Carbon Fiber Reinforced Polymer sheets placed in soffit areas of critical members may prove economical to reintroduce the entire strength margin and improve longevity, especially when an augmentation in load is also factored in.

Better Construction Quality Control: In all future constructions at this site or among those involved parties, it is recommended that strict control of construction qualities be implemented. This entails proper concrete mix designs, compacting, and curing, as well as correct concrete covers for the reinforcement to prevent a situation such as this from occurring. Stakeholder Training: Provide building owners and facility managers with an understanding of why it is important to observe and repair noted defects. This will include basic identification of visual signs of building defect, as well as its routine inspection procedures by a trained technical specialist.

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