Assessment of Structural Elements Deteriorated in Semi-Arid Environmental Conditions

مراجعة نقدية في تقييم العناصر الإنشائية المتدهورة في الظروف البيئية القاسية

by

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ABSTRACT

Deterioration of concrete is a prominent occurrence that happens due to several factors surrounding a structure making the place inhabitable with a risk to life and property. These factors and the impact of deterioration depend upon the surrounding environmental conditions of the structure and vary for different geographical locations. The deterioration of concrete in a structure if assessed properly can be stopped from propagating further and economical solutions can be proposed to rehabilitate the structure. This dissertation aims to find the most prominent type of deterioration using different testing methods in semi-arid environmental conditions like the United Arab Emirates. It further aims to assess the data retrieved by statistical analysis to find its effect on the structural elements, such as the slabs, column and beams. The statistical assessment of the data provides significant information about the deterioration and its effect on the structural elements. Furthermore, the dissertation progresses by investigating the effect of deterioration on the design forces by modelling a structure with the acquired data from the site and comparing it with a non-deteriorated structural model of the same. Analysing the design forces helps to assess the effect of deterioration on various structural elements. Moreover, the dissertation also portrays the effect of deterioration on the most effected structural element which are the columns and asses the loss in capacity due to deterioration. The assessment and the interpretation using statistical analysis and modelling of the structure along with design helps to understand the effect of deterioration on structural elements in semi-arid environmental conditions.

نبذة مختصرة

يعد تدهور الخرسانة ظاهرة بارزة تحدث بسبب عدة عوامل تخص الهيكل الإنشائي مما يشكل خطرا على المنشآت الخرسانية . وتعتمد تلك العوامل على الظروف البيئية المحيطة بالهيكل الإنشائي، وتختلف باختلاف المواقع الجغرافية. يمكن التحكم وإيقاف تدهور الخرسانة في الهيكل إذا تم تقييمه بشكل صحيح و اقتراح حلول اقتصادية حيث يمكن إعادة تأهيل المنشأ الخرساني للاستخدام مرة أخرى. تهدف هذه الأطروحة إلى العثور على أبرز أنواع التدهور باستخدام طرق اختبار مختلفة في الظروف البيئية القاسية مثل مناخ دولة الإمارات العربية المتحدة كما تهدف أيضًا إلى تقييم البيانات الخاصة بالهيكل عن طريق التحليل الإحصائي لتحديد تأثيرها على العناصر الهيكلية للاسقف والاعمدة الجسور الخرسانية. يوفر التقييم الإحصائي للبيانات معلومات مهمة حول التدهور وتأثيره على العناصر الانشائية. كما تقدم الأطروحة نموذج لمشروع قائم تم دراسته وتقيمه عن طريق اختبارات عمليه وتم نمذجة الهيكل الانشائي اعتمادا علي تلك الاختبارات العملية ومقارنته بنموذج لنفس المنشأ، غير متدهور. للوقوف على تقييم تأثير التدهور على العناصر الهيكلية المختلفة. خاصة بالعنصر الأكثر تأثراً (الأعمدة الإنشائية) ومقارنه قيم التخفيض في قدرة التحمل الخاصة بالاعمدة. ان تقييم المنشات اعتمادا علي النظريات الإحصائية ونمذجة الهيكل الإنشائي يساعد على فهم تأثير التدهور على العناصر الهيكلية في الظروف البيئية القاسية .

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CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

In the 21st century we come across various sky scrapers and phenomenal buildings, amongst these are hidden structures that were built in early stages of development of the country. Most of these buildings have been built for almost half a decade and has contributed to the development of the country. Amongst these structures most of them have been constructed using reinforced concrete. Concrete alone without reinforcement is a very durable material which can withstand most of the environmental conditions, and these structures mostly involve load transfers to be in compression rather than in tension. This is visible in most of the roman structures such as the colosseum and mosques in the Arabian empire where the structural load transfer mechanism is mainly carried out by arches. Concrete is an element that has very high compressive strength although the major drawback of it is having low resistance to tension. With the incorporation of steel as reinforcement the structure design in the recent times take the concrete to its maximum potential allowing to build massive structure with big spans and heights.

The reinforcement made up of steel within the concrete increases the tensile forces but is also susceptible to corrosion if exposed to external environmental conditions. This usually happens due to the concrete being porous in nature. There can be several reasons for the porosity although a few main contributing factors are improper design, casting imperfections and adverse climatic conditions. In the middle east, where temperature is a major factor for concrete deterioration, it is important to address the sensitivity of the concrete with respect to the temperature and conditions in which it is mixed and furthermore casted on site. In the middle eastern countries, the temperatures tend to reach about 50 °C and concrete while casting tends to produce heat, because of this reasons the casting is usually done in the early hours or later in the night to overcome this temperature factor.

The strength and durability of the existing buildings tend to reduced due to few of the several factors mentioned above, because of which spalling, peeling and cracking of the structural elements can be observed. These buildings are often neglected for the cost of repair and the importance factor compared to structures such as bridges and dams. Although recently with the maintenance of these existing structure erupting in massive scale attention has to be diverted to address the initial cause of deterioration and factors propagating it. Furthermore, assessing the type of deterioration helps to reduce the cost repair and avoid demolishing the existing structure to construct a new one.

Moreover, on assessment of these structures it has been found that deterioration of the structure if detected earlier, it can be stopped from propagating further into the structure and repair can be done via appropriate cost effective methods, which can bring the structure to its full design capacity.

1.2 RESEARCH SIGNIFICANCE

There are several factors that lead to the deterioration of the concrete, which are caused by adverse climatic conditions. And the deterioration of concrete could lead to spalling, peeling and cracking of the concrete which further could contribute to loss of life and property if not properly taken care of. The significance of this research is to narrow down the several factors that lead to the deterioration of the concrete in the hot arid environmental conditions such as the United Arab Emirates, which in turn would help to shorten the time of assessment to know the root cause of the deterioration. Moreover, it could also help in considering the factors while preparing the design mix. With the increase in demand of the maintenance of the existing structures it is essential to know the types of testing that are to be required to assess the deterioration and what possibilities of repair are there depending upon the damage inculcated in the structure.

With today's advancement of technology there are several ways to repair and retrofit the structure. Although there are different ways to repair the structure such as Concrete jacketing, FRP wrapping etc. the main objective is to enhance the structure. Another significance of this research is to understand the effects of deterioration on a structure by comparative modelling of a structure that is deteriorated due to the impact of environmental conditions in an arid environment with a non-deteriorated structure using ETABS, moreover the data is used to further analyse the loss in capacity of an element in the structure using a software called PROKON. The comparative modelling could help to assess the loss of capacity and make it easy to understand what type of repair works is required for which element of the structure and its location in the structure.

1.3 RESEARCH OBJECTIVE

Concrete is a simple material that brings a lot of strength and durability to the structure, it gets mostly affected by external factors that leads to ingress of foreign agents into the concrete mainly due to high porosity and reduce the strength and durability of concrete. The primary objective of this research is to analyse the data obtained by performing tests on existing buildings, in which there are durability related issues which has caused deterioration in the concrete that are visible and may lead to loss of property and life, if not paid attention to. The analysis of the data obtained by non-destructive testing helps to understand the root cause of deterioration and will help to reduce the time for investigation and repair methodologies ascertained for similar type of deterioration in the United Arab Emirates or regions that have the same environmental conditions.

Furthermore, the research also focuses on modelling of the structure with the acquired data to assess the condition of the building in terms of design. This type of analysis helps to understand the extent of damage concrete deterioration has done to the structure and why durability needs to be given high priority to in the future design and constructions. Moreover, the research also compares the deteriorated model with the full strength design model in order to compare the loss of strength capacity and moment resistance of different elements in the structure. The main purpose of comparing the Models is to understand the behaviour of the deteriorated structure making it easier to decide the type of repair and retrofitting methods and requirements.

1.4 RESEARCH METHODOLY

The study of deterioration of concrete in the United Arab Emirates involves initially identifying the structures that has issues related to concrete durability. A further in depth assessment of the structure is done by conducting tests that are non-destructive in nature. The results from this test helps us to understand the type of deterioration that is usually encountered in United Arab Emirates, this in turn could assist in taking precautions in design and construction methods to avoid this type of deterioration in the future. Furthermore, the research progresses towards analysis of the structure with the acquired data to understand the overall effect of deterioration on the structure.

Furthermore, the structure is modelled, using commercial software like ETABS, for the acquired compressive strengths to know the effects of deterioration on the overall structure and compare it to another model of the same structure without the deterioration. This helps us to understand the loss of capacity and understand the type of repair required. Further it also assists to understand the cost of the repair works depending on the time and also the cost. This would play a major role in planning, cost estimation and analysis. Studying these elements would also help to understand weather the structure needs to be demolished or weather its life span can be prolonged further ahead.

1.5 RESEARCH GAP

There are several researches available exhibiting different methods of retrofitting a structure under seismic action as the structures built earlier where not designed to the full potential to resist seismic forces. Although very little attention had been bought to retrofitting of structures due to adverse environmental effects specially in hot climatic conditions. This research helps to understand the effect of hot climatic conditions on existing structures.

1.6 RESEARCH CHALLENGES

There are several research and studies on the deterioration of concrete in cold environmental conditions but only a few studies have been made in deterioration of concrete in semi-arid climatic conditions like the United Arab Emirates. This is mainly due to the recent growth and development of countries in this type of environmental conditions in comparison to the already developed countries like United Kingdom and The United States of America. Furthermore, there are several studies that exhibit different type of methods for retrofitting but mainly for seismic as buildings earlier were designed for only gravity loads and seismic actions were neglected.

Most of the structures that are facing issues with concrete deterioration have been occupied by people and analysis involves taking sample and performing non-destructive tests with in the presence of residence which makes it difficult to procure the sample. Furthermore, the structure that are being assessed are very old and data of the structure is tough to acquire and requires a lot of time and effort to accumulate data in case of non-availability of structural drawings.

1.7 DISSERTATION OUTLINE

The study initially makes us understand the history of concrete and how it has been developed and the current stage where it is right now. Further the study discusses about the components of reinforced cement concrete (RCC) and their roles in achieving the structural integrity of the structure. The components are studied at a macro level as well as in a micro level.

Further the study discusses about the type of deterioration encountered in RCC. The types of deteriorations are discussed along with their cause and their assessment methods. Further the climatic conditions are described in the semi-arid environment and how it would effect and propagate deterioration in reinforced cement concrete.

Later in the study different methods of testing reinforced cement concrete are discussed along with their methodology, limitations and advantages. the study later exhibits the sample collected from deteriorated structure and the data analysis is done for same. Few hypotheses were made and data was analysed to assess the hypothesis.

The final stage of the study involves a comparative model analysis of a deteriorated structure alongside of a non-deteriorated structure. The analysis helps to understand the intensity at which a structure gets effected due to deterioration. Further the data from analysing and comparative structural modelling is used to design the columns and find the loss in capacity for columns in the deteriorated structure. Later conclusions and recommendations are suggested on the basis of the study.

CHAPTER 2: CONSTITUENTS OF REINFORCED CEMENT CONCRETE

2.1 EVOLUTION OF CONCRETE

Concrete is generally a mixture of cement, or any other binding material, along with aggregates mixed together in a liquid mostly water. It is characterised by having very high compressive strengths and is usually brittle in nature, furthermore it is one of the most widely used man made material. The use of cement can be tracked earlier towards 3000 B.C used in the pyramids.(Cowan & J. 1977). The blend of mud along with straws were used to create bricks and laid down in an astonishing manner and held together with a form of mortar made from lime and gypsum (Malinowski & Garfinkel 1991). Later the use of concrete can be traced to the roman era lasting between 300BC to almost nearly towards 475 AD. The roman architecture and structures are remarkably well known such as the colosseum and the pantheon. These are considered to be prehistoric marvels that are lasting till today. The cement used in the roman era has a very high resemblance to the cement used today. Furthermore, the use of admixtures in the cement can be found in the era where in which animal products were incorporated as admixtures (Brandon et al. 2014). Although the earliest form of concrete can be found at the site of Yiftah'el in the region of southern Galilee and it can be traced to back around 7000 BC (Hanso 2016). to the contrary of this there was a major decline in the usage of mortar as the quality had reduced after the roman period. Later in the middle ages in around 14th century the use of mortar had risen to satisfy the demands, furthermore as the use of it was widely increased it was the 18th century where in which there were major advances in the use of it where in which (Hanso 2016).

The major recognizable development was in the year 1824 where in which Joseph Aspdin invented a material that was hard as a rock, and named it after a rock quarry eventually giving rise to Portland cement. further testing of this material was done in the country of Germany in year 1836 to get more insight on the material properties (Christopher 1976).

The further study into this material gave more confidence to the construction industry to use the materials where in construction of roads, bridges, buildings, dams etc. the evolution of this didn't stop and later in the year 1913 concrete was made easier to handle and casting by the use of ready mix. As the years went by the concrete got more evolved with its colour and other additives getting introduced in it such as air entraining agents, plasticizers to increase its workability making it more feasible for usage. Furthermore, till date the research on this material is still going on making it one of the most versatile man made materials (Hanso 2016)(Christopher 1976).

Reinforced Cement Concrete is used to improve the tensile strength of concrete. As we know, concrete is strong in compression but weak in tension. To improve its tensile strength, steel reinforcements are added which helps concrete become strong in tension. In this report, we are going to understand the major components of Reinforced Cement Concrete. Concrete is the second most used substance in the world after water (Crow & Mitchell 2008)

The components can be divided into two as macro and micro. The macro components mainly consist of cement, water, and aggregates and the mix of these the properties of concrete. In the case of Reinforced Cement Concrete, steel reinforcements are also added to enhance the tensile strength of concrete. Apart from this, concrete is available in many different forms. This can be attributed to the fact that no two environments in this world are same. Concrete is assumed to be designed for ideal conditions. But in reality, concrete is exposed to many environmental factors that are unaccounted for while concrete preparation. Owing to these environmental conditions, concrete is modified in its properties to meet the requirements of the surroundings. Different kinds of concrete used frequently are also discussed within this section.

2.2 CEMENT

Cement is one of the most important elements in reinforced cement concrete RCC. It has truly little usage on its own, but it acts as binder that sets, hardens, and glues to combine sand and gravel together (aggregates). Fine aggregates which are mixed in cement produces mortar which has day to day usage in masonry works and the same cement produces concrete when it is mixed with gravel and sand. Cement is produced in a two-step process. Primitive materials required for the preparation of cement are typically sand, clay, and limestone which are mixed properly at elevated temperatures. These raw materials are then grinded at elevated temperatures. Limestone is a calcareous material whereas clay is an argillaceous material. These raw materials are grinded in a kiln where temperatures go as high 1300^{o}C to 1500^o C. Then cement is obtained from kilns in the form of cement clinkers. This is cooled and mixed with 3-5% gypsum. Second step revolves around the blending, hydration, and setting of cement powder resulting into a final cementitious product (Maclaren & White 2003). Based on the mixing of materials conducted with or without the mixing of water, manufacturing of cement is done in two different methods called as ‘wet’ and ‘dry’ processes. In the Dry process, the materials are mixed in no moisture condition without adding any water, and with the use of compressed air. The consumption of fuel to run the kiln in dry process is much less compares to the other process. In the Wet process, limestone is crushed into finer particles, then it is brought to a fine consistency slurry by mixing with sale or clay with water in a tube. This slurry contains water content of about 30-35 per cent, and then this slurry is evaluated for the correct composition of chemicals. This tested slurry is passed through the rotatory kilns. As the moisture content in the wet process is considerably high in wet process due to the formation of slurry, it required more heat of combustion, and thus more fuel is consumed. This slurry is then obtained as clinkers at the end of the kilns.

Cement has the flexibility to improve its performance by few modifications. Because of this, many forms of cement are produced based on the requirements. Low heat cement, sulphate resisting cement, quick setting cement, air entraining cement, rapid hardening cement, Portland pozzolana cement, etc. are distinct kinds of cement produced by altering the chemical reactions taking place or by altering the quantities of raw materials while manufacturing. Looking at a macro scale, cement seems to be working just as a binder but, many chemical reactions take place when it is mixed with water which determines the strength of concrete to be produced.

2.2.1 TYPES OF CEMENT

To solve the demands arising in the construction industry, cement is modified in its properties by adding various materials or by varying the composition of chemicals or even by the usage of additives to cater specific purposes. There are several types of cements, a few of the Diverse kinds of cement are described.

Ordinary Portland Cement:

The most commonly used and one of the most important type of cement is the ordinary Portland cement. It is classified based on its grade, which means that the strength of the cement is not less than its grade value. (McLeod 2005)

Rapid Hardening Cement:

This kind of cement develops strength rapidly and is similar to ordinary Portland cement except that it has a higher alite and lower belite content, which leads to having relatively large early strength in cement concrete. The cement particles also have a higher fineness of grinding, which exposes the particles to a greater surface area to interact with water. It has its applications in prefabricated concrete construction, repair of road works or where there is requirement of early removal of formwork. (Popovics, Rajendran & Penko 1987)

Sulphate Resisting Cement:

Ordinary Portland cement is prone to sulphate attacks. To resist the cement to such attacks, the content of C_3A is lowered. Low C_3A content and low C_4AF content achieved in cement by modifying its reactions is known as sulphate resisting cement. This is used in marine constructions, concrete in foundations and basement commonly. (Prasad, Jain & Ahuja 2006)

Portland Pozzolana Cement:

This variety of cement is produced by inter grinding of OPC clinker with 10- 25 per cent pozzolanic material. A siliceous material or also an aluminous material which does not have cementitious properties in itself, but when divided finely and allowed to react with the Ca $(OH)_2$ generated as a by-product in process pertaining to mixing of water with cement, also called hydration, results in compounds with cementitious properties is called as a pozzolanic material. Examples of pozzolanic materials are calcined clay and fly ash (Dinakar, Reddy & Sharma 2013). The equation A shows the pozzolanic action

$Ca(OH)_2$ + pozzolana + water $\rightarrow$ C S H -------(A)

PPC enormously enhances the cement properties and even produces less heat of hydration. The early strength of concrete is not affected by the inclusion of the PPC.

Other prominent kinds of cement include air entraining cement, expansive cement, hydrophobic cement, high alumina cement, etc.

2.2 WATER

Water is the most essential element for human survival and also is the most essential element to create concrete. The utmost important ingredient in making concrete that effectively takes participation in the chemical reaction with cement is water. This process is often referred to as hydration. It is always advised to use pure water for making concrete. Organic matter should be absent in the water used for hydration and pH should be between 6-8. Cement needs water to function as a binder for aggregates. Cement gets its adhesive property only after mixing with water. Cement turns into a C-S-H gel (carbon silicate hydrate gel) only after hydration. There are many chemical reactions involved in the process of hydration which will be studied in detail under micro level section. Water used must not cause any harmful reactions to the cement mix. Sometimes, due to the presence of chlorides, sulphates, or salts in water, they cause unwanted reactions which may deteriorate the strength of concrete. It can also cause corrosion in the reinforcement.

Another important term is the water to cement ratio. Excessive amount of water reduces strength of cement mortar but insufficient water content results in poor workability (Singh, Munjal & Thammishetti 2015). Figure 1 exhibits the functionality between compressive strength of cementitious sample and water cement ratio.

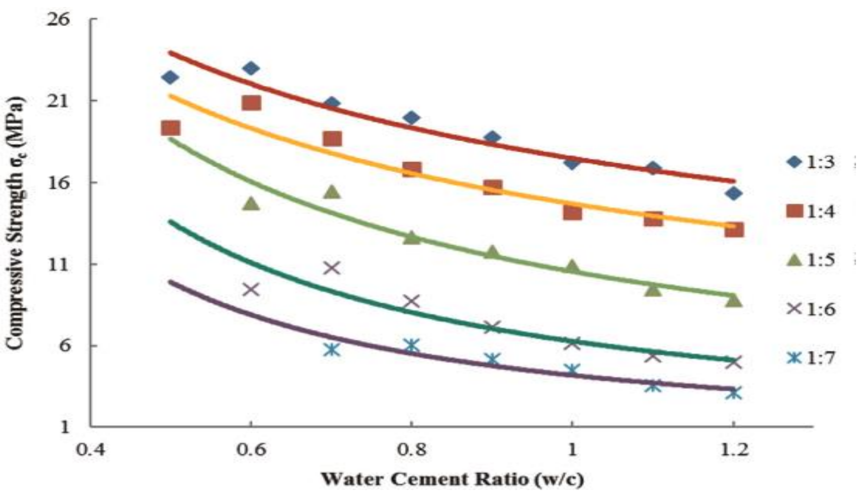


Figure 1 Relation of compressive strength of cementitious mortar and water cement ratio (Singh, Munjal & Thammishetti 2015)

From the relationship of water and cement exhibited in figure 1, we can observe that for the cement mortar sample having a 1:3 water to cement ratio has the highest compressive strength values of all. As this ratio of water cement increases, that is, as the weight of cement increases for each unit weight of water, the compressive strength values are decreasing. From the figure, as water cement ratio changes from 1:3 to 1:7, the average compressive strength value has changes from around 21 MPa to 6 MPa. This behaviour can be understood as the availability of water in decreasing due to the increase in water to cement ratio, more amount of cement is left without hydration or it is difficult for the water to reach all the cement particles and hydrate (Popovics 1990).

2.3 AGGREGATES

Aggregates are mainly crushed rocks, gravel and sand and assist to shape and harden the concrete. They reduce shrinkage and help to make the concrete economical in cost. Aggregates can be classified based on many factors. Popularly, they are characterised based on their size. It can be separated as fine aggregates and coarse aggregates based on size. Aggregate with size more than 4.75mm is said to be coarse and aggregate less than 4.75mm in size is considered as fine aggregate. The classification is done on the basis of size using sieve analysis and are used in concrete in a definite proportion. Grading curves are obtained after performing sieve analysis and based on these curves and the requirement at field, a mixed proportion of aggregate sizes are used. From the curves obtained, aggregates mixture can be classified as gap graded, well graded, and uniformly graded. Apart from size, aggregates are also classified based on shape, strength, texture, etc… Aggregates to be used in concrete must not be flaky or elongated. Flakiness and Elongation test is performed for this purpose. There are many tests to determine the strength and durability of aggregates. Los Angeles Abrasion resistance test, Aggregate Impact Value test, and Aggregate Crushing Value test are performed to evaluate the aggregates.

The quality of aggregate has major influence on the future aspects of concrete in its fresh and hardened state including the resistance to cracking and long-term durability (Mehta & Monteiro 2006). Aggregates must also be inert in nature. They must not cause any unwanted chemical reaction during hydration that may cause adverse effects on the strength. Soundness test are usually performed to know the chemical inertness in aggregates.

Interfacial transition zone is a region between aggregates and mortar, 15-30 μm in thickness, having high porosity and low cement particle content due to wall effect (Scrivener & Technik 2004). Aggregate and hardened cement paste has a relatively higher difference in stiffness values. Due to this, interfacial zone becomes the hub of stress concentrations around the particles of cement and aggregate. That is why the compressive strength is highly influenced by the bond strength that controls the stress variation at the interfacial zone (Aulia et al. 2022).

2.4 REINFORCEMENT

Generally steel reinforcements are used in concrete. The sole purpose of reinforcements is to increase the tensile strength of concrete. Reinforcements are added regions of concrete to protect it from cracks and structural failure. Reinforcements can be longitudinal, shear or sometimes fibres are used as reinforcements and such concrete is called Fibre Reinforced Concrete. In Reinforced Cement Concrete, reinforcements are particularly shear and longitudinal. Concrete tends to be alkaline in nature and this nature of concrete protects the steel reinforcements from corrosion. Passive film, also known as the protective oxide film, protects the steel in the form of reinforcements in concrete from corrosion. This film is a combined result of the high alkalinity in the pore water of concrete and the concrete cover which serves as a physical barrier (Scrivener & Technik 2004).

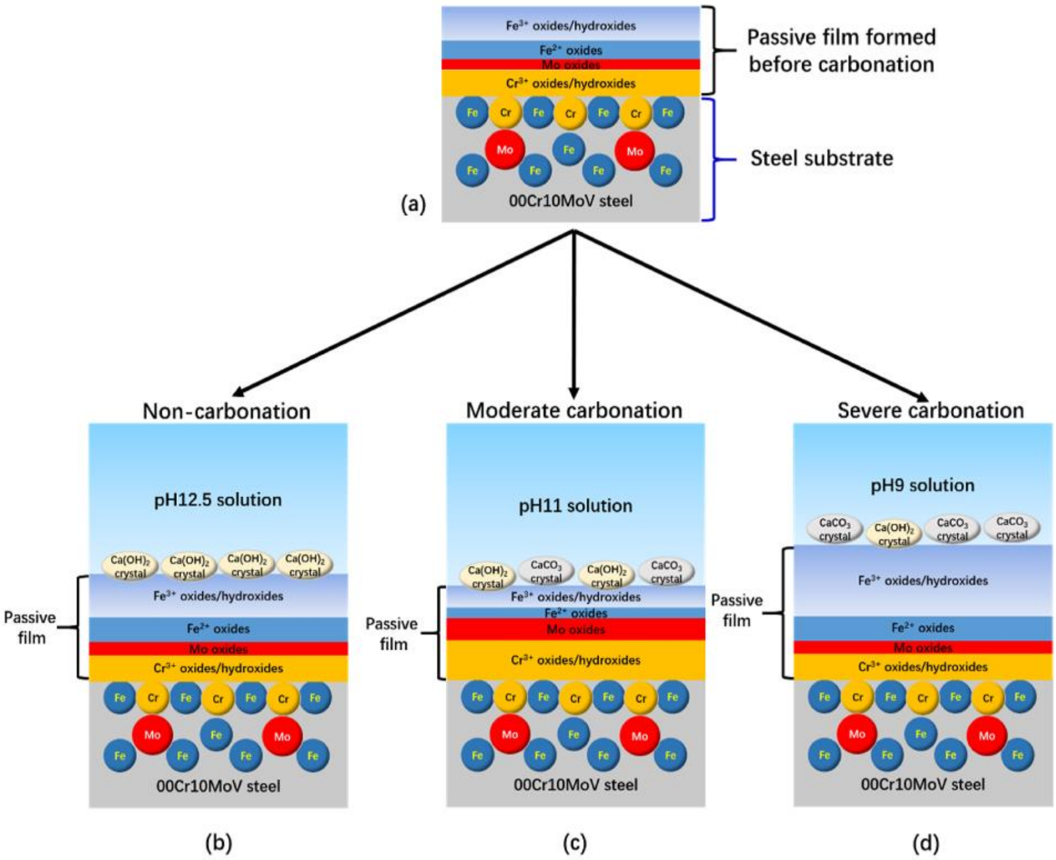


Figure 2 Schematic diagrams of passive film formation and the effect of various carbonation levels (Ming, Wu & Shi 2021)

The Figure 2 portrays the detailed mechanism of passive film and its deterioration by carbonation. 2 (A) shows the effect of pore water solution resulting in the formation of passive film. Further diagrams represent the effect of decreasing pH on the passive film. In figure 2 (b), we can see the effect of further carbonation as pH reduces to 12.5. 2 (c) shows moderate carbonation at pH11 and at pH9 as shown in 2 (d), extreme carbonation is observed.

2.5 MICRO COMPONENTS

Lime, silica, alumina, and iron oxide are the materials used for preparation of cement. These ordinary oxides, during the formation of the cement clinkers at high temperature in a rotatory kiln, react with one another and form compounds much complex in nature. These complex substances are often referred to as Bogue's compounds. These compounds are tricalcium silicate $3CaO.SiO_2$ (C_3S), Dicalcium Silicate $2CaO.SiO_2$ (C_2S), tricalcium aluminate $3CaO.Al_2O_3$ (C_3A), and tetra calcium alumina ferrite $4CaO.Al_2O_3.Fe_2O_3$ (C_4AF). Among these compounds, tricalcium silicate and dicalcium silicate are mostly responsible for the strength of concrete. Together they constitute about 70-80% strength of concrete.

The hydration reaction is cement is exothermic, meaning energy is released when cement reacts with water. This heat liberated from the reactions is known as heat of hydration. Hydration of C_3S is the primary cause in the contribution to the early heat of hydration. The other Bogue's compounds also release heat of hydration but in different quantity and rates. The Hydration process is never complete fully and cannot be estimated, it is a lifetime process (Locher 1976). It starts at a much rapid phase but can take years to achieve 100% completion. The most important result of hydration is calcium silicate hydrate gel which is written as the C-S-H gel. This attributes to almost 60% of volume of cement. Calcium Hydroxide is also released during hydration. Equations (1) & (2) show the formation of C-S-H gel. This reaction is further illustrated in the figure 03.

$$2C_3S + 6H \rightarrow C_3S_2H_3 + 3Ca\,(OH)_2 \quad \text{---------(1)}$$

$$2C_2S + 4H \rightarrow C_3S_2H_3 + Ca\,(OH)_2 \quad \text{------------(2)}$$

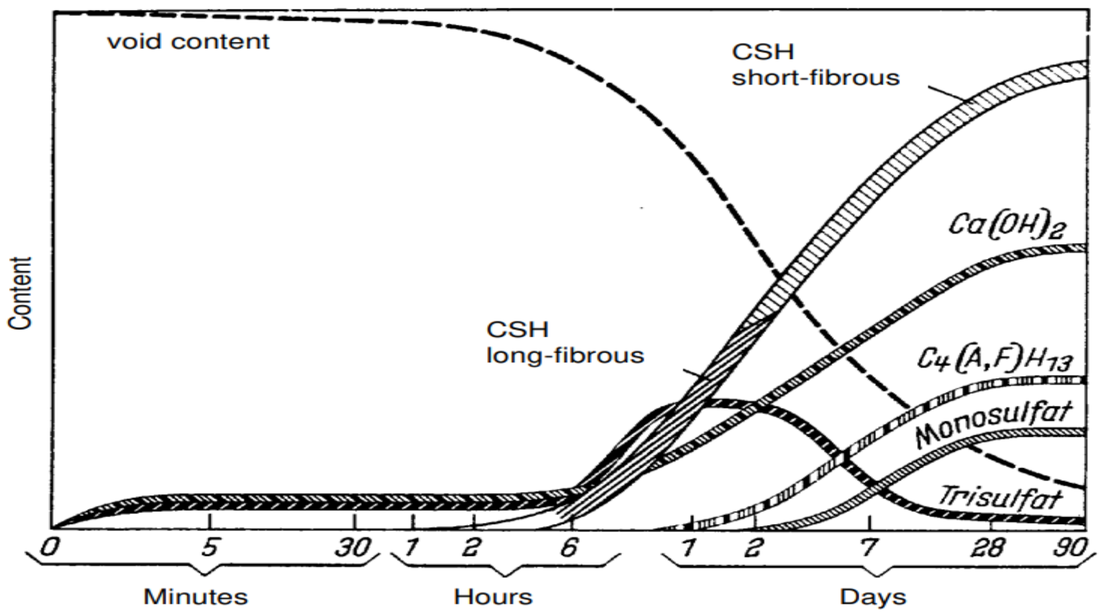


Figure 3 Formation of hydrate phases and the structure development during cement hydration (Locher 1976)

Tri calcium silicate (alite) is responsible for the early strength of concrete whereas dicalcium silicate (belite) is reason the later strength of concrete. Calcium Hydroxide produced is largely of no use to concrete but it accounts for almost up to 25% volume of hydrated paste. In presence of sulphur, calcium hydroxide reacts with it to form calcium sulphate which causes deterioration in concrete by further reacting with alite. This is known to be as sulphate attack. The ill effects of Ca $(OH)_2$ could be countered with the inclusion of admixtures like fly ash, silica fumes and other pozzolanic materials. The only good thing of Ca $(OH)_2$ is that being alkaline in nature as it protects the reinforcements from corrosion.

$$3\ C_3A + 3\ CaSO_4 + 26\ H_2O \rightarrow 3CaO.Al_2O_3.3CaSO_4.32H_2O \quad \text{-------(3)}$$

Primary ettringite formation can be is illustrated and explained from the equation (3). Tricalcium aluminate, C_3A reacts very rapidly with water. This even causes flash setting. To eliminate these effects, gypsum is added while manufacturing of cement which prevents flash setting. The products formed from the hydration of C_3A and C_4AF contribute truly little to the strength of concrete and form similar products in the presence of gypsum. Based upon the extent of aluminate and sulphate ions in the solution, the result of the chemical reaction is either calcium aluminate tri sulphate hydrate or calcium aluminate mono sulphate hydrate. calcium aluminate tri sulphate hydrate is also called as ettringite. This is the most stable compoundas the Ettringite forms hexagonal prismatic crystals (Day 1992).

CHAPTER 3: DETERIORATION OF REINFORCED CEMENT CONCRETE

Reinforced cement concrete (RCC) indeed is an exceptionally durable material. But this may not be true when it is subjected to overly aggressive environments. The surroundings concrete gets exposed to is not always ideal such as highly polluted areas, marine ecosystems, and many other critical conditions. On examining these challenges in recent years, the notion that RCC is an exceptionally durable material is at a fallacy. Now, RCC faces deterioration of many forms and it mostly depends on the environmental factors it is subjected to. Amongst the several factors, a few have been classified below which are often encountered.

3.1 CARBONATION

Carbonation is a neutralizing process which involves the chemical reaction between $Ca\ (OH)_2$ and calcium silicate hydrate with CO_2 to form $CaCO_3$ and water and takes places in the pores of concrete. This is one of many processes that happens and may affect the service life of concrete. CO_2 by itself is not reactive. Concrete is actually threatened by carbonic acid which is formed by the CO_2 with moisture, and it reduces the basic nature of concrete. Carbonation happens to have two major consequences. First is the drop of pH value, which is the drop of concentration of hydroxide ions which can destroy the passive layer of embedded reinforcement bars. Second is the changes in the permeability because of the micro cracking and volume changes. Increase in permeability is observed in concrete containing fly ash or decrease in permeability in case of OPC-concrete. The outcome of numerous steps through which calcium carbonate is formed can be explained through the given irreversible reaction in equation (4) (Johannesson & Utgenannt 2001).

$$CO_{2\,(aq)} + Ca^{2+}_{(aq)} + 2OH^{-}_{(aq)} \rightarrow CaCO_{3\,(s)} + H_2O \text{ ------(4)}$$

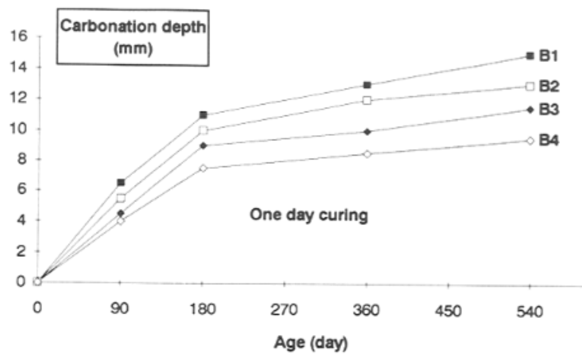
Traditionally, carbonation is determined by its depth measured by sprinkling phenolphthalein indicator onto the freshly split surface of a concrete prism. This will turn the surface into purple when the pH falls past 9. This may not be uniform but can have patches of pink colour showing carbonated zones (Parrott 1987).

The carbonation rate depends on many factors some of which are humidity, concrete grade, protection provided to concrete, concrete permeability, concrete cover, and time. Carbonation is at its highest at an average of 60 per cent humidity. Strong concrete or the higher graded concretes tend to have more rate of carbonation. This can be attributed to the fact that stronger concrete is denser with less water cement ratio. These can be observed from the graph shown in figure (4).

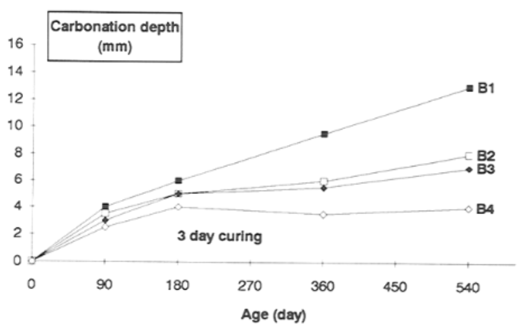
Concrete	Cement	C (kg/m^3)	W/C	G/S	R28 (MPa)
B1	CPJ45	300	0.65	1.00	25.1
B2	CPJ45	340	0.61	1.13	32.6
B3	CPJ45	380	0.53	1.13	37.8
B4	CPJ45	420	0.48	1.15	43.5
–	CPA55	250	0.73	0.92	26.4
–	CPA55	280	0.65	0.94	30.0
–	CPA55	300	0.59	0.96	35.0
–	CPA55	350	0.54	1.04	41.8
–	CLK45	340	0.61	1.17	24.9
–	CLK45	400	0.50	1.15	31.9

C: cement content
G/S: gravel on sand ratio
R28: 28 day strength (after storage in water at 20°C, three samples tested)

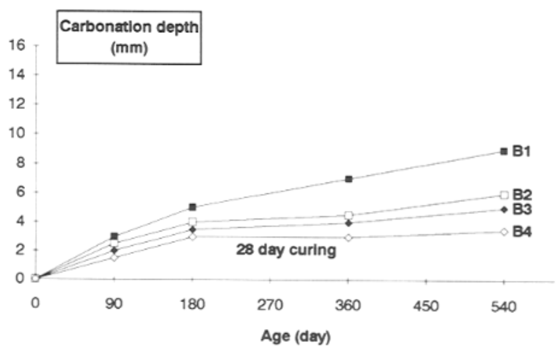
A) concrete specimen's constituency



B) carbonation depth upon one-day curing



C) carbonation depth upon three days curing



D) carbonation depth upon twenty-eight days curing

Figure 4 Carbonation Ingress In Concrete (Grandet 1995)

Figure 4 depicts the ingress of carbonation with the progress in time. Figure 4 (A) explains the concrete used for the observation. Ingress of carbonation is measured for different intervals of curing. The graphs clearly convey that the carbonation is spreading rapidly for low compressive strength concrete which also has higher water cement ratio compared to others. It is also observed that carbonation decreases for properly cured concrete as 28 days curing carbonation values are significantly lesser that the 1-day curing carbonation values. Carbonation is less for concrete with lesser water cement ratio and higher gravel sand ratio.

Carbonation is a diffusion process, and its length can be approximated with the equation (1) (Ho & Lewis 1987).

$L = L_o + c*t^{0.5}$ -------------(1)

Where, L = depth up to which carbon dioxide has attacked

t = time up to which it has progresses

c = change in carbonation with respect to time, and

L_o = depth of the cover already reacted with carbon dioxide

The carbonation may also affect the CSH gel according to the following reaction (5) (Chinchón-payá, Andrade & Chinchón 2016).

$C\text{-}S\text{-}H + 3CO_2 \rightarrow 3CaCO_3 \, . \, 2SiO_2 \, . \, 3H_2O$ -------------------(5)

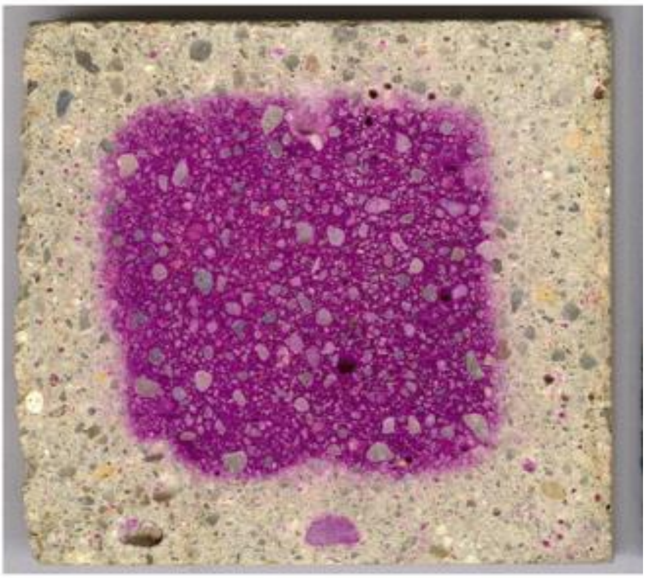


Figure 5 Concrete Section treated with phenolphthalein solution (Chinchón-payá, Andrade & Chinchón 2016)

The test for carbonation is most popularly performed by using phenolphthalein solution as indicator. Phenolphthalein solution contains 1 per cent phenolphthalein in a mixture of 70 per cent ethanol and 30 per cent water. Phenolphthalein is wide used as a pH indicator. It changes its colour to pink when it comes in contact with the base, typically pH values above 10.5 and remains colourless when the pH is below 9. The formula for this organic compound is $C_{20}H_{14}O_4$. When this is sprayed on concrete, owing to the alkaline nature of concrete, it must turn into pink colour. As observed in figure 5, the boundary portions of the concrete specimens tend to be colourless signalling the absence of alkalinity which can be attributed to the occurrence of carbonation since carbonation tends to drop the pH below 9. This distance is measured and reported as carbonation value.

3.2 CHLORIDE ATTACK

Chloride attack primarily causes corrosion of reinforcements. the steel reinforcements have a protective oxide film due to high alkalinity of concrete and this due to carbonation can be lost. But the loss of this can also be caused by the chloride occurrence in oxygen and water. This results in the area of steel loss, load carrying capacity of structure drastically reduced, and loss of durability (Guoping, Fangjian & Yongxian 2011). The chloride in concrete can be in soluble and insoluble forms. The soluble ones are responsible for corroding reinforcements. The insoluble ones are chloroaluminates. Corrosion of steel in concrete is an electro chemical process.

Chlorides can enter concrete from the basic ingredients such as cement, water, aggregates and from admixtures. Chloride can also diffuse into the specimens via diffusion from surroundings. The pH value of pore water in concrete is partly responsible for the initiation of chloride attack. Corrosion may occur without the need of chlorides below a pH of 11.5 but above this pH, good amount of chloride is needed. Corrosion won't happen if concrete is dry, or the relative humidity of concrete is below 60 percent as not enough water to initiate corrosion is present. Also, corrosion won't happen when concrete is fully immersed in water as corrosion is an electro chemical process and it requires oxygen to create potential difference. 70-80 percent of relative humidity needs to be present for the corrosion to occur.

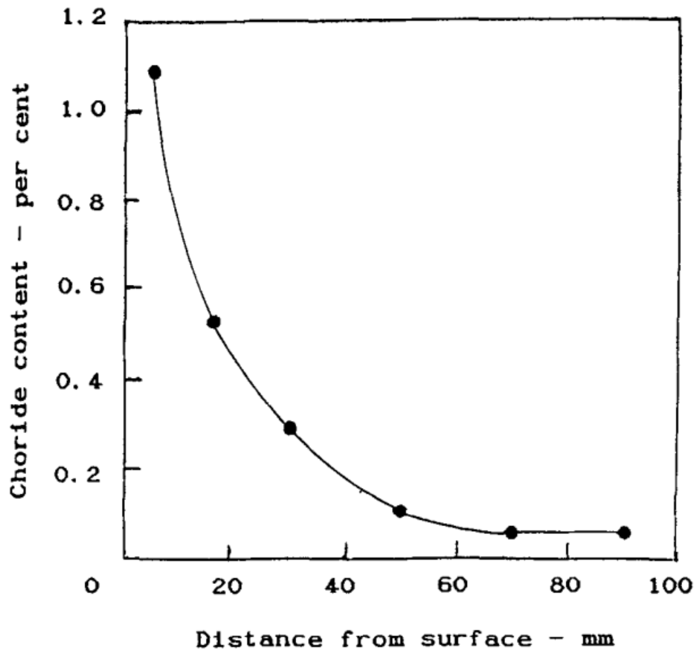


Figure 6 Concentration of chloride content with respect to the distance from surface (Neville 1995)

Figure 6 exhibits the chloride concentration in concrete from the surface. It is observed that chloride content decreases as we move away from concrete surface. This also tells that concrete cover plays an important role to prevent chloride attack. The more the concrete cover, the more time chloride ions take to reach the surface of reinforcements and crosses threshold value. Therefore, concrete must have low permeability (good quality concrete) and sufficient cover to avoid chloride attack (Neville 1995) .

3.3 ALKALI AGGREGATE REACTION

The chemical reaction that involves the anionic part of water in the pores of concrete and the rock minerals that are present as a part of the aggregates is popularly known as the Alkali Aggregate Reaction (AAR). Conventionally, aggregates are assumed to be inert, and they do not cause any reactions in the concrete mix. In some instances, it is also called as alkali-silica reaction. This reaction occurs only where the pH of pore water is considerably high which can be inferred as high concentration of hydroxyl ions in the concrete pore water. Generally, the alkali content of cement governs the pH value of the water in the concrete pores. This governing factor is so much so that 10 times increase of the hydroxyl ionic concentration happens by an increase in pH by 1 time. Hence, a cement which has low alkali content, and therefore has a low hydroxyl ions concentration which makes the pH in the pore water of concrete less is suitable to counteract the attacks of potentially reactive aggregates.

The crucial factors that affect alkali aggregate reaction are alkalis, moisture, temperature, and the pozzolanic conditions. Feldspars, micas and clay minerals contained in the limestone and clayey material which are often used as raw material in manufacture of cement are the source of the alkaline nature that forms the basic property the concrete. The progress of AAR is dependent on the availability of moisture. This chemical reaction is greatly reduced in the absence of water. This reaction, therefore, is more on the surface of concrete where water is easily available. By the application of water proofing agents on concrete which prevents additional penetration of water into the structure can enormously reduce the alkali silicate reaction. Expansion due to reaction between bases and aggregates may be prevented or reduced by replacing 20-30 percent cement by a suitable pozzolan as it is scientifically proved that the pozzolans have a huge influence on some of the reactive aggregate and can also prevent the expansion caused by the alkali silica reaction in the aggregates (Gillott 1975).

Figure 7 Cracking due to alkali aggregate reaction in Concrete (Brueckner & Lambert 2013)

The figure 7 portrays an image of cracking due to alkali aggregate reaction. The mechanism of the Alkali Aggregate Reaction and its contribution in the deterioration of cementitious concrete can be explained as follows. First of all, strongly caustic solutions are formed from the mixing of water owing to the alkalies from the cement. Alkali silica gel is formed by the invasion of this caustic liquid on the reactive silica. The process is more rapid for highly reactive substances. This silica gel thus formed grows in size with the correct temperature and the continuous supply of alkaline water. At some stage, this causes cracking patterns due the pressure exerted which turns out to be Osmotic. This happens in thinner sections of concrete like pavements. Mass concrete does not experience the major effects of such osmotic pressure as it very little for such big cross sections. These cracks to subsequent loss in strength and elasticity. These so formed cracks can lead to other kinds of deterioration of concrete.

3.4 SULPHATE ATTACK

The occurrence of sulphate attacks is common in the natural and in those environments where industries are present. Calcium, sodium, potassium, and magnesium are most of the forms that sulphates tend to exist in moist soils. Frequently present in agricultural soil and water is ammonium sulphate as a direct result of the of use of fertilizers. Solid sulphates, that is, sulphates present in solid form, do not deteriorate concrete severely. But as they form a solution or get converted to solution, they attack concrete by entering into the porous portions of concrete and reacting with the products of hydration. Sulphate attack on concrete is often characterized by a white colour appearance on concrete. This indicates that sulphate has deteriorated concrete. This is often called as external sulphate attack. For this to happen, the concrete must have high permeability, the environment should be sulphate rich, and there must be presence of water. The most damage of all the sulphates is known to be caused by magnesium sulphate.

Ettringite formation is known to be the cause of most the expansion caused as a result of sulphate invasion in deterioration of concrete structures. But that does not mean every sulphate attack is a result of the formation of ettringite. The ettringite formed in a few hours after hydration is often referred as primary ettringite and it does not have any adverse effects on concrete structures. However, the same ettringite, when gets delayed in its formation forms after several months because of external surroundings factors is known to be as delayed ettringite formation. This late expansion in the rigid hardened concrete is heterogeneous in nature and can bring along cracking and spalling. The expansion is non uniform and subjected only to the areas of concrete when formation of ettringite happens, that is, it is localized. This is associated with damage due to sulphate attack (Collepardi 2003).

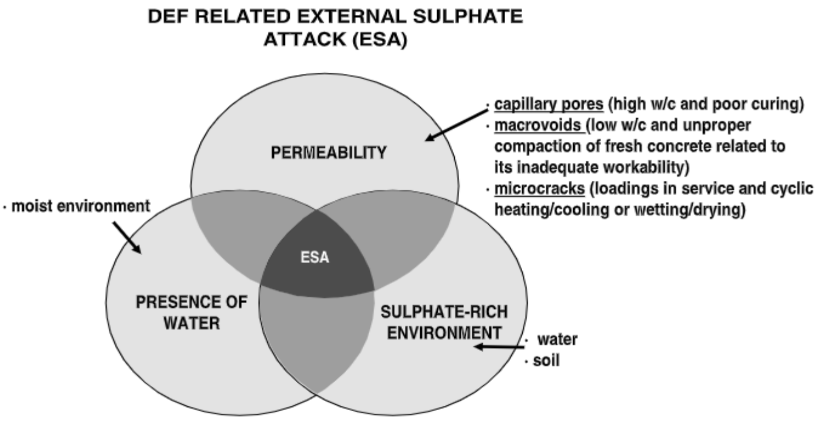


Fig. 8 (A) Delayed Ettringite Formation, DEF, related External Sulphate Attack, ESA

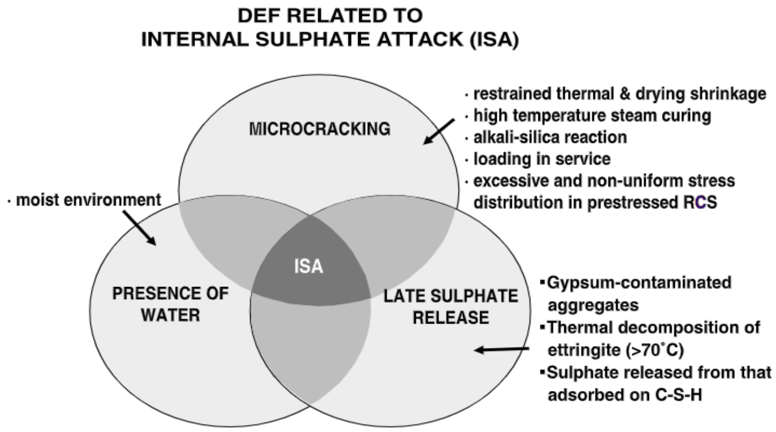


Fig. 8 (B) Delayed Ettringite Formation, DEF related to Internal Sulphate Attack, ISA

Figure 8 External sulphate attack and Internal sulphate attack resulting in Delayed Ettringite Formation (Collepardi 2003)

Figure 8 exhibits the ternary representation of the external sulphate attack (E S A) and internal sulphate attack (I S A). Water presence is crucial for both the processes. It is evident from figure 8 (A) that for an external sulphate attack to take place, it must be in a sulphate rich environment and the concrete specimen in such specimen shall be permeable. Permeability in concrete can result from the high water cement ratio and poor curing or due to macro voids resulting from low water cement ratio and improper curing which makes the concrete less workable or due to microcracks which can occur due to loading or atmospheric changes. All these can promote external sulphate attack and cause the delay in ettringite formation.

Furthermore, from the figure 8 (B) we can observe Venn diagram for the delayed ettringite formation due to internal sulphate attack. Unlike external sulphate attack, the internal sulphate attack is triggered by microcracking and late release of sulphate coming from the hydration reactions in concrete. The micro cracks in concrete are developed due to the various factors like temperature stresses, service loads, alkali silica reactions as discussed earlier, thermal restrains and drying shrinkage.

Other prominent reason for internal sulphate attack is the late sulphur release. This can occur when the sulphate released from the gypsum contaminated aggregates does not react with calcium sulphate and thus does not contribute in the early ettringite formation. This sulphur then later participates in the delayed ettringite formation. Other sources of internal sulphate release are the decomposition of primary ettringite thermally, and the ions of sulphate that are absorbed in the high temperature steam cured concrete (Collepardi 2003).

3.5 FREEZE –THAW

Water is present in considerable amount in freshly prepared concrete. This water turns into ice when the temperature drops subjecting it to freezing. During this process of freezing, expansion of water occurs by 9 percent in volume. The formation of ice disrupts the newly prepared concrete and causes permanent damage. The fresh concrete also loses its structural integrity due to freezing and thawing. Concrete deterioration due to freezing and thawing can be explained as the damage to concrete resulting directly from the disturbance that is created when the space available is not suffice to accommodate the additional solids created when the water freely available in concrete turns into ice by the action of freezing (Cai & Liu 1998).

3.6 ABRASION

Abrasion can be explained as the resistance of a substance to borne the action of friction caused by the rubbing and keep itself held together without being worn out. Paste hardness, aggregate hardness, and aggregate paste bond are some of the factors that have an influence on the abrasion resisting strength of concrete. Concrete's hardness, which is also related to its strength determines the resistance towards abrasion (Profile 2016). Abrasion causes significant deterioration in concrete because of wear and tear effect that friction has on it (Mufid Al-Samarai 2015). Four ways are defined by American Concrete Institute (ACI) by which abrasion can happen on the surface of any concrete specimen. First one being spoiling on floors of concrete due to heavy traffic caused by human activity. Wearing caused by the tires of vehicles studding which happens due to the vehicular traffic. Structures submerged in water partly or fully like spillways, and high water velocities producing cavities at the surfaces of concrete are prone to abrasion by abrasive things in water.

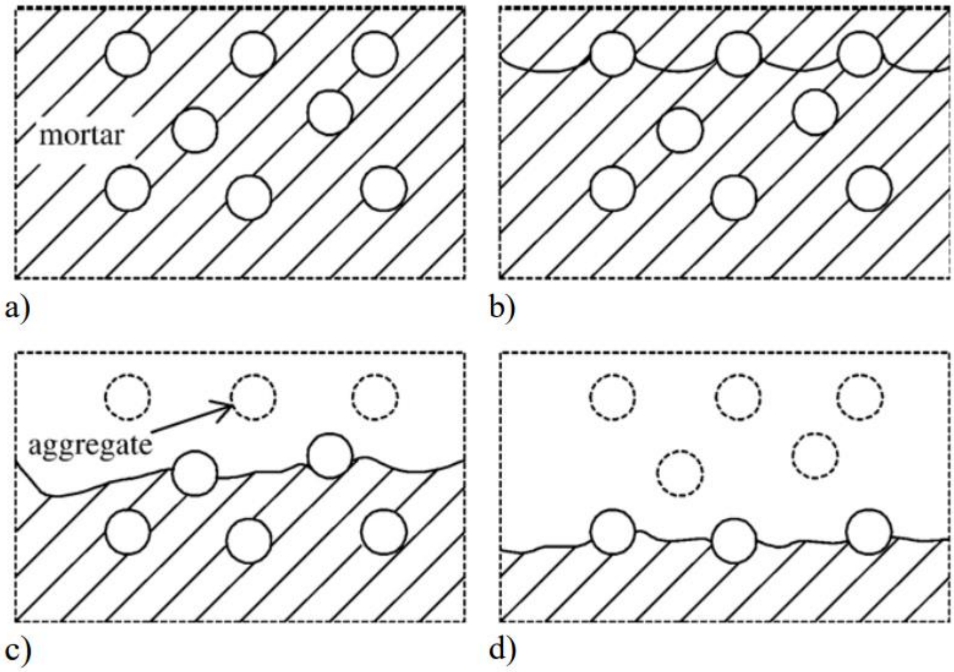


Figure 9 Abrasion in surface of concrete in hydraulic structures (Scott & Safiuddin 2015)

The Figure 9 exhibits the mechanism of abrasion in hydraulic structure. (a) portrays the initial condition of concrete mortar free from abrasion. In (b), it is observed that the abrasion action initiates as a consequence to hydraulic action. At (c) abrasion results in the loss of mortar and coarse aggregates from the surface of concrete structure. At (d) we can see that as the abrasion continues, further aggregate and mortar is lost from the concrete surface.

3.7 ADVERSE HEAT

Concrete, when it is subjected to heat in the atmosphere, causes the moisture in the chemical bond, the water that is freely present in the pores of concrete structure, and moisture in CSH and sulpho aluminate to evaporate from the specimen. This results in the reduction in volume of concrete specimens starting at around 300 degrees centigrade. The resistance of concrete cover is reduced due to the steam pressure formed and the water loss caused. In case of any fire breakout, the hot gases coming out of fire are directly in contact with the reinforcements if the concrete cover weakens. At temperatures above 400 degrees centigrade, a number of reactions occur in the cement paste as the pore system receives heat and dries up. Because of this, the products of hydration are dispatched and CSH gel breaks into fragments. Further at 530^{o} C, calcium hydroxide transforms to anhydrous lime and as a consequence, surface of concrete becomes whitish, and it breaks into small fragments (Topc 2007) .

The major concern when concrete is subjected to fire is Spalling. The inner layers of specimens are exposed to higher temperatures which also increases the probability of the passage of this heat into the deeper layers of specimen and eventually the reinforcements as the consequence of concrete lost as a result of occurrence of spalling. One of the primary reasons that leads to spalling is low water to cement ratio (M. Maslehuddin1, O. S. B. Al-Amoudi2 & 1 2014).

The semi-arid environment is characterized by high heat reaching up to 60 degrees Celsius. This effects concrete in many aspects. For freshly poured concrete the head of hydration is high when in combination with high heat due to environmental conditions this leads to shrinkage cracking which is cause by high rate of evaporation of fresh concrete. This tends to cause cracks at micro levels reducing the durability of concrete and exposing the steel underneath making it prone to corrosion. Hence it’s a common practice to have casting in the early hours of the day or at the night to avoid high rate of evaporation. Further for the reinforced cement concrete which is already casted there is a high tendency of spalling due to adverse heat in contact with the surface. Moreover, semiarid regions encounter high levels of humidity sometimes reaching highs of 90 percentages. The moisture tends to ingress the cracks within concrete causing corrosion and deteriorating the structure. (M. Maslehuddin1, O. S. B. Al-Amoudi2 & 1 2014).

CHAPTER 4: TESTING OF REINFORCED CEMENT CONCRETE

Quality of concrete works can be confirmed and controlled by performing tests on concrete specimens or samples. Systematic testing of raw materials, fresh concrete, and hardened concrete are a part of the quality checks for concrete. By this, strength and durability of concrete as a measure of its performance are ensured to have a higher efficiency and assurance. The main objective of tests performed is to find its strength. With the results obtained from these tests, we can get an idea about the quality of concrete and decide if any adjustments are required. The tests that are performed on concrete samples or specimens. These are either cast from the concrete in a particular shape say cube and/or cylinders or by obtaining core specimens from actual concrete structures. It is to be noted that these tests only give us information about the potential concrete strength, and it is not to be misunderstood with the concrete strength in structures. Two kinds of testing are done on concrete broadly:

1) Non-Destructive Testing.
2) Destructive Testing.

4.1 NON-DESTRUCTIVE TESTING

Non-Destructive Testing, (NDT) is explained as the testing or analysing the strength of structures, materials or even parts of structures by not destroying them and not affecting the service life of the structure (Workman & Moore 2012). The intention behind Non Destructive is to gather information on the durability, strength and quality of the structures such that the structures or the testing specimen's regular functions does not get affected. That said, the testing methods are not completely harmless, they might cause very minute damage to the structural component being tested and such tests are considered to be invasive but it is to be noted that they still do not affect the function of the structure. Non Destructive Testing methods do not reach the failures of the structural components and yet explore their properties. The need for detection and prevention of structural damage by Non Destructive Testing methods have materialized. The use of Non Destructive Testing is extensively carried forward by economics and safety (Helal, Sofi & Mendis 2015). With Non Destructive Testing, we can also find the voids present, elastic modulus and even aging of concrete. Below are some important Non-Destructive Tests:

4.1.1 PENETRATION METHODS

This method of Non Destructive Testing that finds the concrete strength by establishing a correlation first and then using it on the present sample is invasive by nature, that is, it can cause some minor damage to the structural part being tested but does not affect its day tp day activities. Probes are driven into concrete samples with force applied such that it stays uniform throughout the testing. The depth of penetration of the probes conveys an idea on the compressive strength of concrete with the help of already established correlations. Although it disturbs the structural integrity of concrete, it is considered non-destructive due to insignificant effect of penetration.

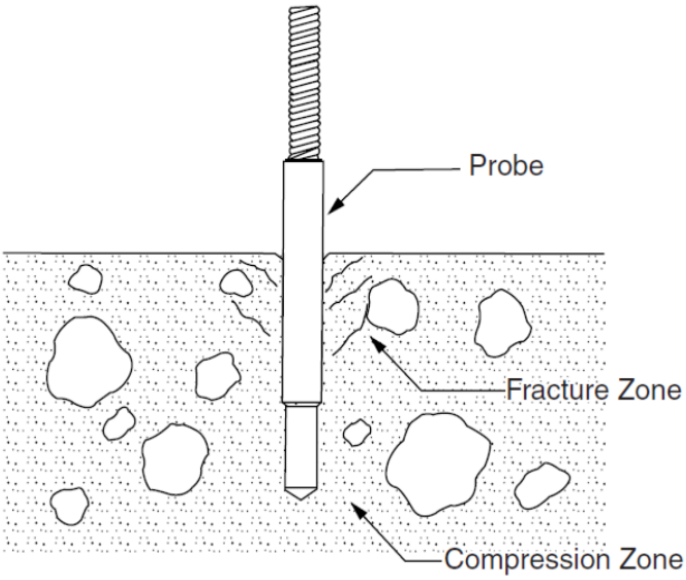


Figure 10 Schematic Presentation of Penetration Test (Malhotra & Carino 2003)

Figure 10 portrays a schematic diagram of penetration testing. The extent to which probe is penetrated and the strength of concrete follows inverse relations. The less penetrating depth, the more compressive strength of concrete and vice-versa. We can also identify uniformity in concrete as well as deterioration in concrete. Furthermore, it is also possible to identify cracks and flaws in concrete through this method (Ramezanianpour et al. 2011).

The penetration test has a few limitations. It fails to yield absolute value of strength. It causes minor damages to the structure. This tests only on through thin layers of concrete surface. Despite these limitations, this method is apt for on-site estimation of concrete strength.

4.1.2 SCHMIDT HARDNESS TEST

Schmidt Hardness Test is performed on site so as to get the compressive strength of concrete in the structure without invoking any damage to it. This test requires no specimen, no penetration, just a rebound hammer, a device used to perform this test, also called as Schmidt Hammer. This device works on the principles of wave propagation. The Schmidt Hammer, when used on a surface to be tested shows a rebound number which is recorded with an impact on the surface. This rebound number is used to find the strength in concrete specimen by using it with an already calibrated empirical relations between the strength of concrete and rebound number.

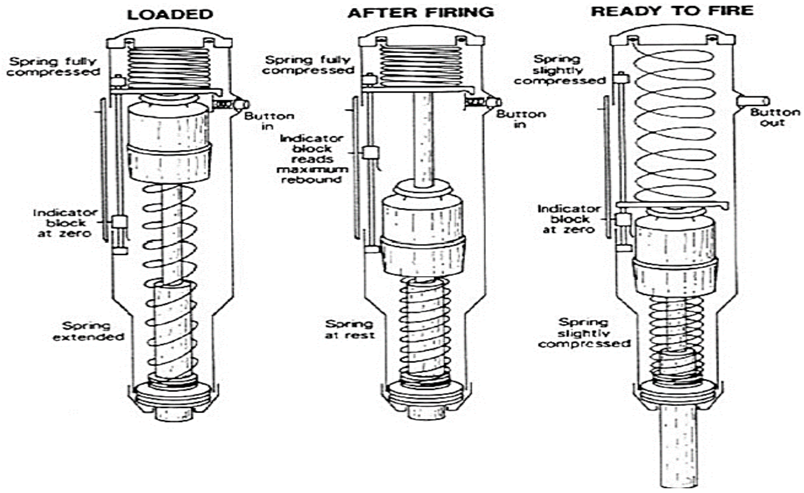


Figure 11 Operation Mechanic Of Schmidt Hammer (Shariati et al. 2011)

The Schmidt hammer has a calibration curve provided by the manufacturer which helps to calibrate the reading first. This device must first be calibrated to develop a correlation between the rebound number and the concrete strength. Then it can be used as a reliable source of testing to test the surface strengths of concrete structures and to check their homogeneity also. As the number of specimens tested increases, the accuracy of the calibration curve increases. This must be developed and tested for only the specific concrete project on which it is calibrated. For other tests, separate calibration must be performed, and the new curve correlating the strength and rebound number is to be developed.

Factors that cause a change in surface hardness, like water content, age of specimen, carbonation, smoothness of surface and temperature have a direct or indirect influence on the results of the test. Achieving the actual strength is difficult due to all these influences. Concrete being exposed to the kind of environment also affects results of rebound hammer test (Sanchez & Tarranza 2014).

4.1.3 PULL OUT TEST

Pull out test can also be classified as a Non-Destructive Test which is invasive in nature, i.e., it causes damage to the structure when tested, but not very serious. It measures the force required to pull a disc of a special shape which is made of steel and had been cast into hardened concrete. The pull out cases a cone of hardened concrete with surface slope to vertical being 45^o to come out owing to the shape of the steel rod disc. This force can be almost equal to the compressive strength in concrete. This pull out rod has a disc of steel 1 inch in diameter. From the concrete surface, this rod is held at 1 inch with the help of a moving shaft. Using an adjustable quarter inch diameter screw, this shaft is attached to the formwork. Most important result from performing this pull out test is to get an idea about the safe time to remove the formwork and also to determine the time required to know post tensioning takes place at the earliest (Vijayan et al. 2019).

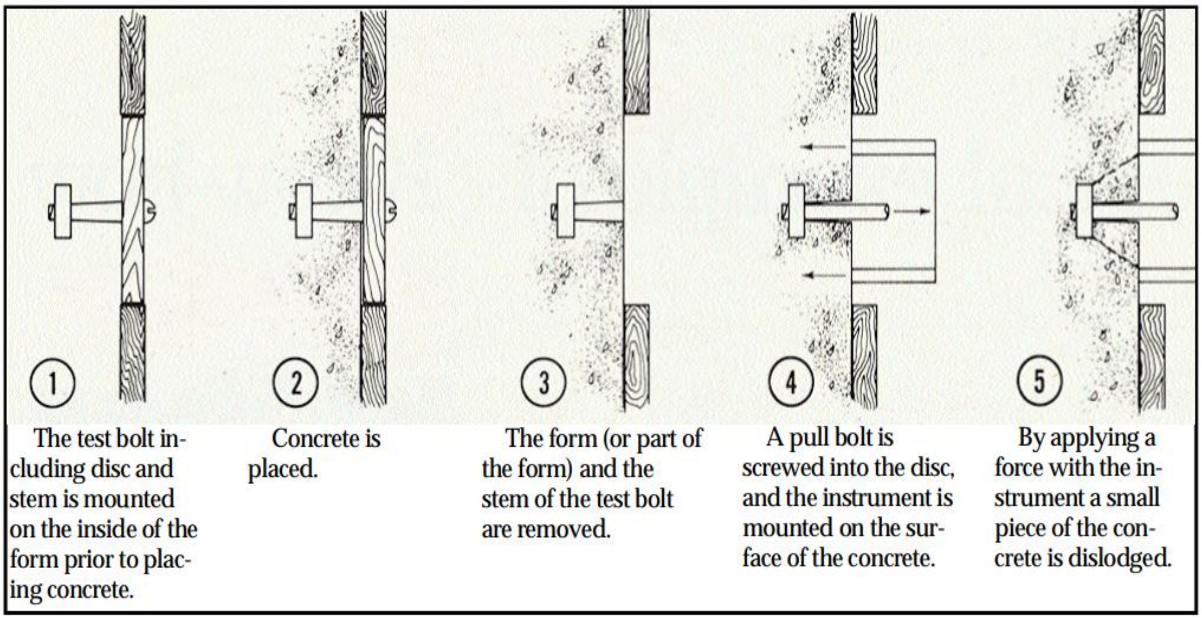


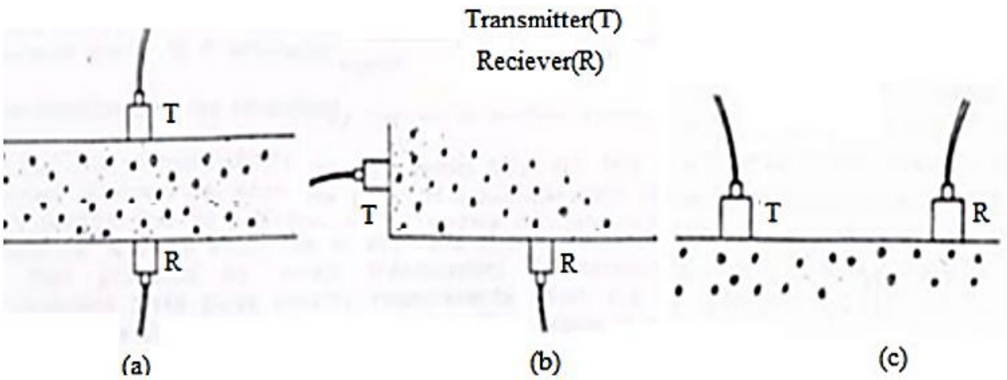
Figure 12 Schematic diagram of pull-out test method (Vijayan et al. 2019)

Figure 12 shows the procedure of pull-out method. First the disc and the stem are assembled into the formwork prior to pouring of concrete. The concrete is poured, and the pull out assembly is set. The instrument is arranged on to the surface at which a part of the formwork is removed and with the help of a pull bolt which is screwed into the steel rod disc. On application of force on the instrument, a small piece of concrete is dislodged.

This test is absolute best to determine when the formwork can be removed safely. It is also useful to determine the on-site compressive strength of concrete. Concrete sections which are undergoing repair can also be tested can also be tested with this method. Depending on the results obtained from these tests, post tensioning operations can be carried out by ascertaining the concrete strength. However, the major disadvantage of this test is that this test should be planned and the concrete must be poured after the assembly of the pull-out has been set into the formwork (Shen et al. 2016).

4.1.4 ULTRASONIC PULSE VELOCITY

This method involves sending of waves. There is a sending point and a receiving point. The waves which are ultrasonic in nature travel from on point to another and the time taken by these waves is measured with a special device. This has many applications and can characterize the material composition of concrete structure, geometry, density, elastic properties, etc… This method does not damage the concrete at all and hence it is non-invasive method. It can be used to find the defects in the structure also. The test set-up includes an electronic timing device, amplifier, pair of transducers, and electric pulse generator. An ultrasonic pulse is transmitted through the specimen medium by transmitting transducer which is placed on concrete surface. It is received at the other transducer placed at a known distance. The time is measured with an electronic timing device and the velocities are calculated. The detailed report on the specimen under testing is obtained from the velocity of the ultrasonic pulse (Shariati et al. 2011).



a) Direct transmission b) Semi-direct transmission c) Indirect transmission

Figure 13 Arrangement of Transmitter & Receiver (Shariati et al. 2011)

The placement of transducers is shown in figure 13. However, direct transmission of ultrasonic waves is preferred for better results. The Velocity of the waves can be calculated by dividing the length of travel, which can be manually measured, with the time taken by the waves. The velocity is related to concrete modulus of elasticity as shown in equation 2.

$$V = \sqrt{\frac{E}{\rho}} \quad \text{.........} \ (2)$$

E is the modulus of elasticity and ρ is the density of concrete. USPV is independent of the geometry of the material. Higher velocity means better concrete strength and vice versa. This test gives an idea about the concrete continuity. It can detect the presence of cracks, voids, and imperfections in concrete. We can also get an idea about the density of concrete based on the velocity we obtain.

The table 01 indicates the relation amongst the longitudinal pulse velocities and the approximate strength of concrete they indicate. It can be observed that pulse velocities recorded the value of 4.5 km/hr and above indicate concrete of excellent quality, with compressive strength up to 40 N/mm^2. Fairly good concrete quality is understood when the pulse velocities are 3-3.5 km/hr with the compressive strengths up to 10N/mm^2. Below 3km/hr pulse velocity indicates concrete of poor quality and should be discarded.

Longitudinal pulse velocity (km/sec.)	Approximate compressive strength (N/mm^2)	Quality of concrete
Below 2.0	---	Very poor
2.0 to 3.0	4.0	Poor
3.0 to 3.5	Upto 10	Fairly good
3.5 to 4.0	Upto 25	Good
4.0 to 4.5	Upto 40	Very good
Above 4.5	Upto 40	Excellent

Table 1 Relation between Pulse Velocity & Quality of Concrete (Velu 2014)

The ultrasonic pulse velocity test is affected by many factors and hence it is not a complete measure of compressive strength. The velocity obtained is influenced by the irregularities present in between the transducers. The presence of reinforcements disturbs the wave propagation. Further, it is influenced by the concrete mix, temperature, and stress level of concrete (Brandon et al. 2014).

4.1.5 RADIOACTIVE METHODS

One among the Non Destructive methods that helps in collecting the results on concrete quality and the defects within the structure are Radioactive Methods. It is a reliable technique to locate internal cracks, voids and variations in density of concrete. Radioactive methods are broadly classified into two methods which uses different rays from the electro-magnetic spectrum. One method uses the x rays and the other uses gamma rays. These methods work on the principle that the photons emitted from the generator, which is radioactive, are transformed in the visible spectrum with the use of a flu metallic converter which in turn attains maximum energy. This phenomenon helps generate photographs of concrete specimens which can be used to analyse the reinforcements defects, presence of any voids or cracks etc (Velu 2014).

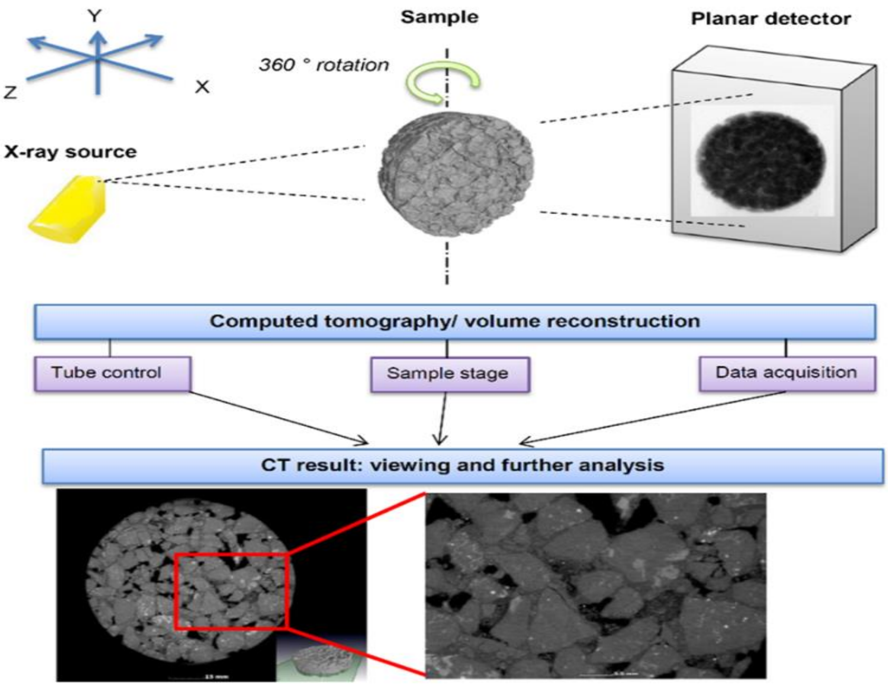


Figure 14 Working Mechanism of CT Scan (Boshoff 2019)

These rays can penetrate concrete. They are electromagnetic in nature, and they move along in a straight line. They can penetrate all materials owing to their minute wavelengths, so small such that they can transmit anywhere with some absorption and some scattering. Attenuation of rays depend on the nature of concrete structure, its density and the travel path of waves which in many cases is thickness. The intensity is calculated from the following equation 3 which is shown below:

$$I = I_o * e^{(-\int_0^L \mu(x,y,z)dL)} \text{---------------(3)}$$

Here, I_o means the incident beam (rays) intensity, attenuation coefficient of the material which varies with the space is $\mu(x, y, z)$ and the path length in which the beams/ rays travel inside material is L (Bliylikzt 1998).

Computed Tomography, CT, is also one of the x-ray methods adopted to study the properties of concrete. It gives even finer results and high-resolution images both 2D and 3D of concrete specimen. Porosity analysis is one among the wide applications of micro CT. In this, we can identify material voids and pore spaces, qualitatively visualising the spatial variation of porosity and quantitatively analysing it. Other applications include phase identification and density measurement, permeability and pore network, fibre reinforced concrete, etc. This method can also be used to detect cracks and analyse level of damage in concrete. A simple CT slice image of a concrete core of 200mm high and 60mm diameter is shown in figure 15.

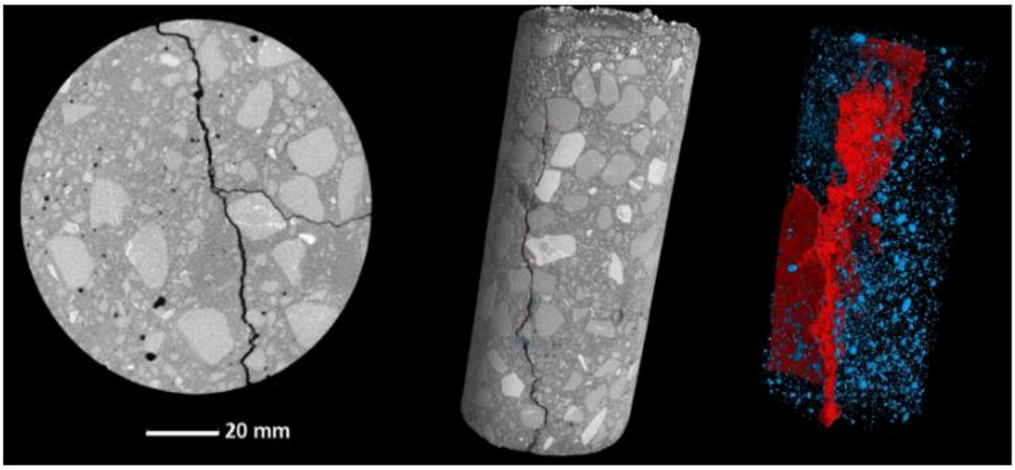


Figure 15 Cracking of Concrete Sample (first & second), 3D Cracks & Porosity View (right) (Boshoff 2019)

This method, though very useful has many limitations. It requires access to two side of specimen material, precautions for safety and so much time is required due to long exposing of material to the rays. This method has the need of skilled labour, costly and large equipment and the access to two parallel side of the structure.

4.2 DESTRUCTIVE TESTING

The destructive test, as the name suggests is the testing of concrete specimens where they are destroyed by loading. This helps us to comprehend the mechanism and quality by destroying the test sample at required loads. For the specimens created at large scale, this method proves to be most suitable, and it turn out to be economically advantageous. The primary intentions behind destructive tests are to find the serviceability life and find the defects in design that won’t appear under normal circumstances. Mechanical properties like hardness and strength are found out by the breaking the specimens of concrete. These tests are simple and not so hard to carry out, interpret, and yields more information. These tests depend on the concrete quality, water-cement ratio, mix proportions etc. With the results obtained from these tests, one can get an idea on the strength of concrete, quality, uniformity of concrete, cracking patterns, stress distributions, behaviour of concrete under various loadings, etc. Mainly there are three types of destructive tests, they are explained below.

4.2.1 COMPRESSIVE TEST

Concrete compressive strength is ability to withstand loads without any cracking or deformation. Compressive strength test conducted on hardened concrete is most common, reason being that most of the desirable properties depend on the compressive strength and because it is easy to perform. The specimens on which this test is performed are mostly cubical or cylindrical in shape. The cube specimen size should be 15x15x15 cm and the largest nominal aggregate size should not exceed 20mm. Cylindrical specimens are 15cm diameter and 30cm long. These specimens, either casted or collected from field using core cutting, should be tested under a loading machine. The centreline of the specimen should be matching with the axis of thrust. The loading shall be applied with no inclusion of jerks, i.e., without any sudden loads, and the loading shall be increased continuously at a pace of 140 kg per cm^2 per minute until the specimen fails. The max load imparted on the specimen is noted and the looks of concrete in view of the kind of failure should be observed.

The compressive strength of concrete can now be known as the loading at which the sample fails divided by the surface area of the concrete sample. This test, if performed on casted specimens, should be repeated for seven, fourteen, and twenty-eight days of curing and on three specimens which are not same each time. The compressive strength at 28 days of curing is usually considered. The specimen after loading, if it has same amount of cracking on all exposed faces with minimal damage on the above and below faces is considered as usual cracking. Sometimes the concrete cubes may fail by developing cracks on one side or they may get crushed on one side, such type of failure is unusual (Del Viso, Carmona & Ruiz 2008). The figure 16 portrays an image of hydraulic compressive testing machine.

Figure 16 Compression Testing Of Concrete Core

4.2.2 SPLIT TENSILE TEST

The tensile strength is one of the fundamental characteristics. Since the concrete is brittle, it is weak in tension and can incur cracks. Split Tensile strength measure the strength of concrete in tension. This test also provides information on the use of sand and aggregate. We can determine uniform stress distribution with this test and study the behaviour of concrete. The apparatus used in this test mainly comprises of loading machine, supplementary bearing bar, tamping rod and the concrete specimen. Concrete specimen can be cylindrical or cube (Raphael 1984).

Before testing, the specimens shall be kept in water for 48 hours. It must be tested immediately after removing from water. Centre lines must be plotted on two parallel faces of the specimen to ensure the loading axis and axis of specimen coincide. The load shall be applied without any jerks and increased continuously at a pace within 1.2-2.4 N/mm^2/min. This load must be given until failure occurs, and the load should be recorded along with the type of failure.

The split tensile strength f_c can be calculated from the given formulas in equations 4 and 5. Equation 4 is for cylinder specimen and equation 5 is for cube specimen.

$$f_c = \frac{2p}{\pi * l * d} \text{ -----------------(4)}$$

$$f_c = \frac{p}{2 * l^2} \text{ ----------------------(5)}$$

Where, p = max load the specimen can withstand in Newtons (N)

l= cylinder length or cube side, in milli metre (mm)

d = diameter of cylindrical specimen in milli metre (mm)

4.2.3 FLEXURAL STRENGTH TEST

Concrete is weak in tension and to supplement it, we provide reinforcements to the concrete. Drying shrinkage, temperature gradient, rusting of steel reinforcements, and other reasons can cause the stresses in tension to develop in concrete. Hence, Flexural Strength test must be conducted.

This test is conducted on concrete beams with dimension 15x15x70 cm. The testing machine bed shall have the provision of two steel rollers 38 mm in diameter and the distance between their centre to centre is 60 cm. The load is applied on specimen through same kinds of rollers placed with 20cm centre to centre spacing such that load is always applied axially without any torsional stresses. The testing arrangements are shown in figure 17. The axis of loading should be matching with the centreline of specimen. The load should be applied without jerks at a continuous pace of 400 kg per minute or such that the stresses increase in the fibres at extreme at approximately 7 kg per cm^2 per minute. This load should be continuously given until the specimen attains failure and load at which it fails is noted with the failure pattern.

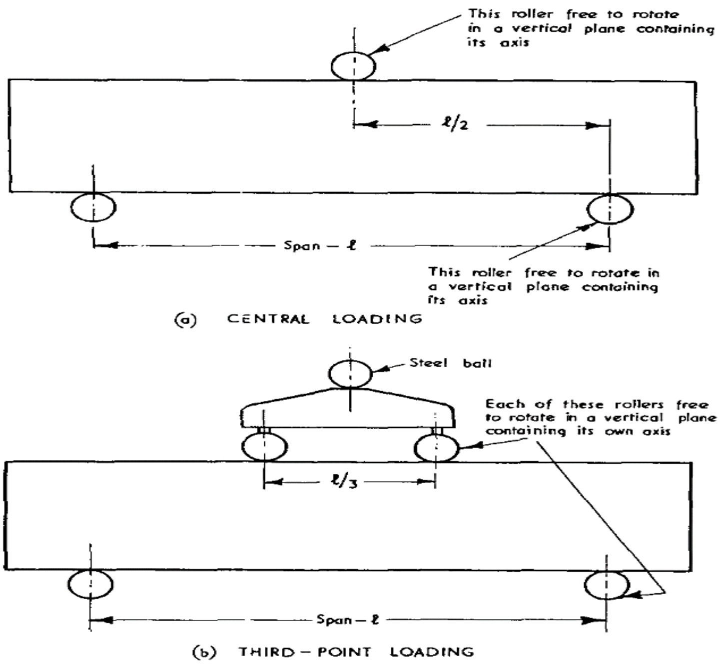


Figure 17 Types Of Loading for Split Tensile Test. (Wright & Garwood 1952)

Modulus of rupture f_b is used to express the flexural strength of specimen as shown in equations 6 and 7.

$$f_b = \frac{p*l}{b*d^2} \text{------------------(6)}$$

$$f_b = \frac{3p*a}{b*d^2} \text{----------------------(7)}$$

Where, a = length from the fracture line and nearby support

b = measures width of specimen

d = measured depth of specimen at point of failure

l = measures span on which specimen is supported

p = maximum load in kg applied to specimen

Note that all the dimensions here are discussed in centi-meters. Equation 6 is used when the value of a is greater than 20cm and equation 7 is used when a is less than 20cm but greater than 17cm while calculating flexure strength. If a is less than 17cm, then the results shall be discarded.

For all the tests observed in the destructive testing, we either cast the concrete specimen or obtain it from the site as core samples. Casting is done by taking a representative sample from the mix and mould into the required specimen shape. This is particularly useful before or during the construction of structures. We cannot prepare these sample specimens again after casting as the mix might differ along with various other factors. Hence, core samples are drilled from the existing structures for testing. The core specimens are usually cylindrical in shape. These are obtained by drilling with special drills into the concrete with a hollow tube called a core drill. Core hole is the hole made for extraction of a core sample. Generally, core tests are done for projects to assess the condition of the concrete. In the United Arab Emirates, the authority has put forward a rule that any building resuming constructing after a pause longer than 5 years must provide core test result in order to examine the current state of the structure.

CHAPTER 5: DATA ANALYSIS AND INTERPRETATION

In the United Arab Emirates, the building is inspected on a regular basis to ensure the safety of the residence in the country. The data is obtained by performing various non-destructive test on the deteriorated buildings. The data for compressive strength test is collected by performing compressive strength test on concrete core obtained by drilling in the deteriorated structures. The figure 18 (A) illustrates an image of obtained cylindrical core via drilling and the figure 18 (B) exhibits the core tested under compressive testing machine

Fig. 18 (A) Cylindrical Core Specimen

Fig. 18 (B) Core Tested Under Compressive Testing Machine

Figure 18 Cylindrical Core & Its Testing Under Compression

From the tests the results are obtained. The results not only contain information of the compressive strength but also provides us with further information about the presence of carbonation and its depth. The tests are conducted for concrete core obtained from every floor for a multi storey structure for all the essential structural elements like the beam, columns and the slab. This data contains detailed analysis of four projects namely P054, P056, P068 and P069. It compares the statistical similarities and differences of the statistical measures namely compressive strength, ultrasonic pulse velocity and carbonation values across the structural elements, i.e., beams, columns, and slabs.

5.1 STATISTICAL ANALYSIS

Statistical analysis is an essential part of the study helping to understand the nature of concrete deterioration and its pattern. The figure 19 compares the means of the statistical measures across beams, columns, and slabs of the four given projects. We can observe that slabs have the highest compressive strength among the structural elements. It reflects that a higher-grade concrete was used to construct slabs of these projects. The compressive strength of beams and columns are almost similar. Interestingly, slabs have lesser carbonation compared to beams and columns. They are least affected by carbon dioxide. This reflects that slab elements, as they are having high strength concrete and hence less porous. The effects of carbonation are studied later in this report as well. The ultrasonic pulse velocity has almost the same values across all the structural elements. This could be due to good quality of concrete with no of minute honeycombing.

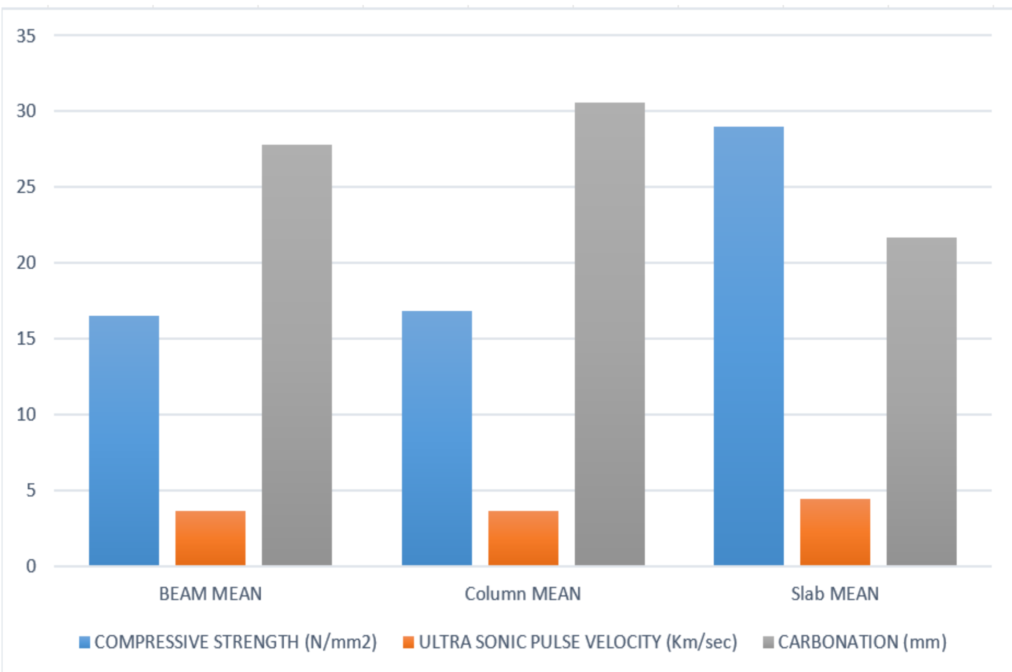


Figure 19 Mean of Statistical Measures across Structural Components

Furthermore, ahead in the study, we can observe that the ultra-sonic pulse velocity of concrete across the structural elements have less deviation from its mean which also reflects the good quality of concrete.

The Table 02 illustrates the statistical report of the data obtained from four projects. A total of 207 samples were collected from the beams, columns, and slabs of four projects undertaken. We can observe that the strength of concrete has a mean strength of 20.9 N/mm^2 (compressive) across structural elements with a standard deviation of 17.6 N/mm^2. The standard error in its measurement is about 1.22 N/mm^2. The median compressive strength is 18.7 N/mm^2 most of the structural elements has a strength of 22.8 N/mm^2.

COMPRESSIVE STRENGTH (N/mm2)		*ULTRA SONIC PULSE VELOCITY (Km/sec)*		*CARBONATION (mm)*	
Mean	20.93285024	Mean	3.907729469	Mean	26.58454106
Standard Error	1.223011511	Standard Error	0.043131391	Standard Error	0.69522129
Median	18.7	Median	3.8	Median	26
Mode	22.8	Mode	3.2	Mode	25
Standard Deviation	17.59607147	Standard Deviation	0.620552654	Standard Deviation	10.00249254
Sample Variance	309.6217312	Sample Variance	0.385085596	Sample Variance	100.0498569
Kurtosis	138.100204	Kurtosis	-0.987209054	Kurtosis	0.4088021
Skewness	10.67125826	Skewness	0.138466382	Skewness	0.449010565
Range	249	Range	2.7	Range	56
Minimum	0	Minimum	2.6	Minimum	2
Maximum	249	Maximum	5.3	Maximum	58
Sum	4333.1	Sum	808.9	Sum	5503
Count	207	Count	207	Count	207

Table 2 Statistical Evaluation of Different Parameters

These samples were also measured for carbonation and the mean carbonation among 207 sample elements comes out to be 26.58 mm. These values tend to deviate from its mean by 10mm. The median carbonation is 26mm with the most repeating carbonation value being 25mm. Carbonation is studied in detail in the next section. The ultra-sonic pulse velocity has a mean value of 3.9 km/s. It has little deviation of 0.62 km/s from its mean value. Standard error in measurement of ultrasonic pulse velocity is very less. 3.8 km/s is the median ultra-sonic pulse velocity.

5.2 ANALYSIS OF CARBONATION

Carbonation of concrete is the chemical reaction that mainly produces carbonates by the reaction of carbon dioxide in the atmosphere with the hydrated calcium silicate and/ or the calcium hydroxide available in the concrete. This also reduces the alkalinity of concrete causing the reinforcements inside concrete to corrode. Carbonation is measured in milli meters (mm) with the aid of a chemical named as phenolphthalein solution. The histograms shown below in figure 20 depict carbonation values in mm across all the structural elements. It can be observed that the carbonation is least observed in the middle storeys and is more dominant in the top and bottom storeys.

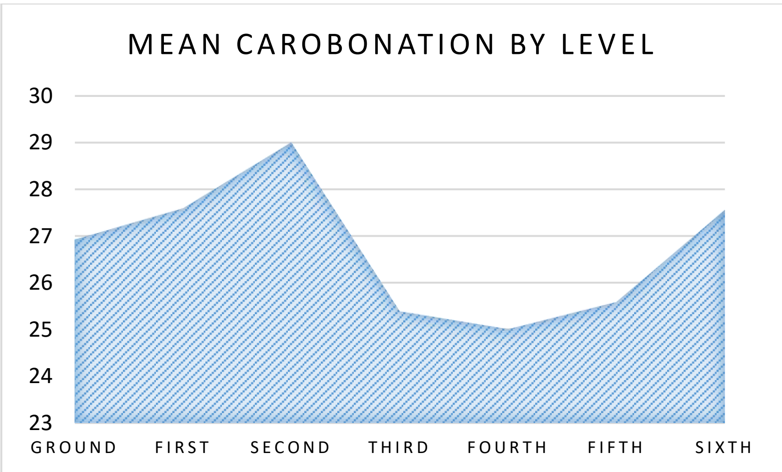


Figure 20 Mean Values Carbonation by Level across all structural components

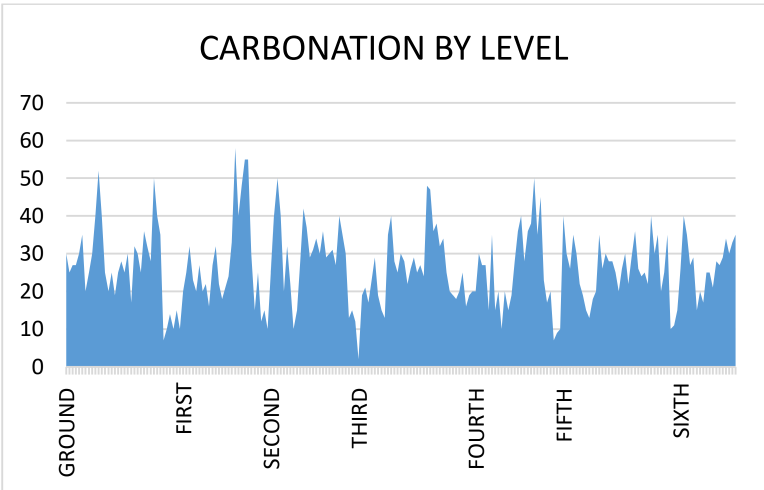
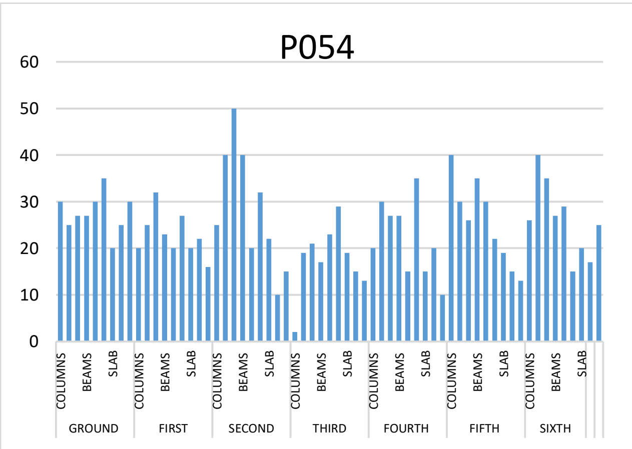
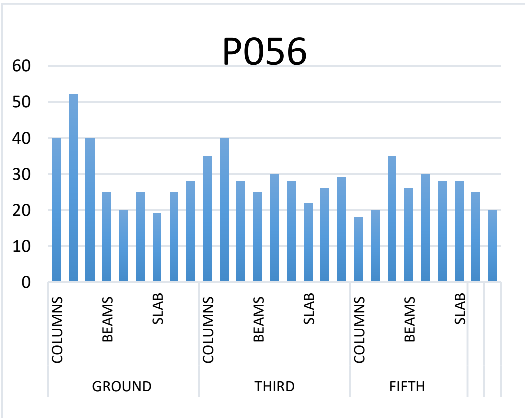


Figure 21 Carbonation of Concrete with respect to Different Components at Different Storey Height

Further the figure 21 portrays a figurative plotting of carbonation levels amongst the different components of a structure in respect to the floors. The observation made is that there are some elements that have a high carbonation value whereas a few others have low for the same storey. Further in the study this deviation is studied.

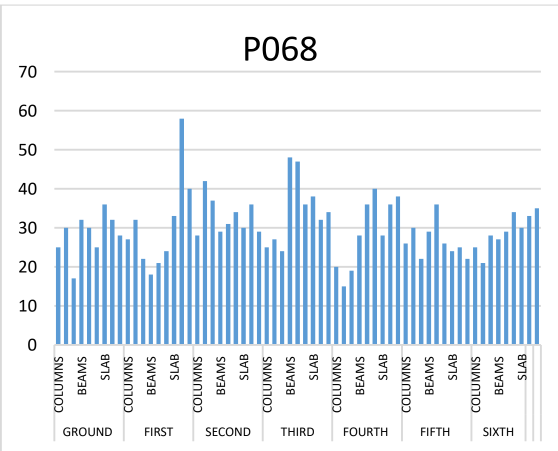
Figure 22 shows the varies carbonation values throughout the projects assessed. It can be observed from the graphs that the carbonation values tend to rise to a certain elevation then they fall. The peak carbonation values occur at first and second floors for almost all the projects. This peak carbonation at a particular level can be due to higher carbon dioxide concentrations at those levels or It can also be attributed to higher relative humidity at those levels.

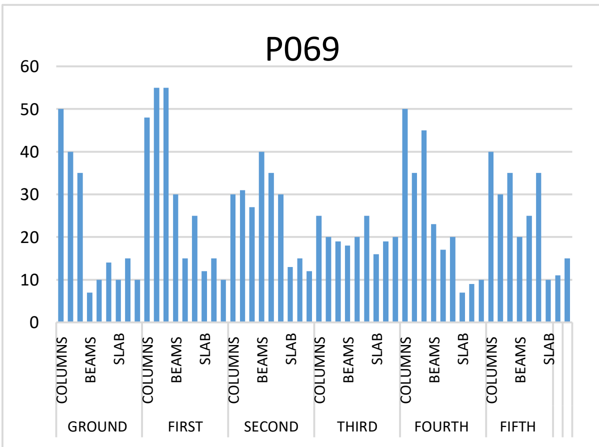




(A)

Carbonation Values by level in P054 *(B) Carbonation Values by level in P056*





(C) Carbonation Values by level in P068 *(D) Carbonation Values by level in P069*

Figure 22 Carbonation Values at different Storey Height for different projects.

5.3 T - TEST

In this part of the study we perform statistical analysis of carbonation, compressive strength and ultrasonic pulse velocity obtained from core test. The statistical analysis performed is a T test. This test helps to understand and compare the means of two groups. Initially a null hypothesis is defined and the test result are used to interpret if the hypothesis is valid or not.

5.3.1 PROJECT P054

The structural elements like the columns, beams and slab are assessed for carbonation Initially, for the project P054. Later the structural elements for Ultrasonic pulse velocity and finally for compressive strengths.

1.) Null Hypothesis: There is no significant difference in the rates of carbonation between columns and beams.

t-Test: Paired Two Sample for Means		
Carbonation		Columns and Beams
	30	*27*
Mean	28	26.55
Variance	103.1578947	50.05
Observations	20	20
Pearson Correlation	0.238054548	
Hypothesized Mean Difference	0	
df	19	
t Stat	0.594448618	
P(T<=t) one-tail	0.279612471	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.559224942	0.05
t Critical two-tail	2.093024054	

Table 3: T test Results for Carbonation between Columns & Beams P054

The results from the t-test for carbonation for the project P054 for columns and beams are shown in table 3. It is observed from the results that the p value is greater than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is true. There is no significant difference in the rate of carbonation.

2.) Null Hypothesis: There is no significant difference in the rates of carbonation between columns and slabs.

t-Test: Paired Two Sample for Means		
Carbonation		columns and slabs
	30	*20*
Mean	28	18.05
Variance	103.1578947	26.68157895
Observations	20	20
Pearson Correlation	-0.122390929	
Hypothesized Mean Difference	0	
df	19	
t Stat	3.725238865	
P(T<=t) one-tail	0.00071738	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.001434761	
t Critical two-tail	2.093024054	

Table 4 T test Results for Carbonation between Columns & Slabs P054

The results from the t-test for carbonation for the project P054 for columns and slabs are shown in table 4. It can be observed from the results that the P value is less than the standard significance level of 0.05. Hence, the null hypothesis is false. There is a significant difference in the rates of carbonation between the columns and slabs.

3.) Null Hypothesis: There is no significant difference in the rates of carbonation between beams and slabs.

t-Test: Paired Two Sample for Means		
Carbonation		beams and slab
	27	*20*
Mean	26.55	18.05
Variance	50.05	26.68157895
Observations	20	20
Pearson Correlation	0.023692151	
Hypothesized Mean Difference	0	
df	19	
t Stat	4.389381126	
P(T<=t) one-tail	0.000157621	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.000315242	
t Critical two-tail	2.093024054	

Table 5 T test Results for Carbonation between Beams & Slabs P054

The results from the t-test for carbonation for the project P054 for beams and slabs are shown in table 5. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is false. There is a significant difference in the rates of carbonation between beams and slabs.

4.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between columns and beams after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic pulse velocity		Columns and Beams
	3.8	*3.3*
Mean	3.62	3.625
Variance	0.244842105	0.214605263
Observations	20	20
Pearson Correlation	0.241086261	
Hypothesized Mean Difference	0	
df	19	
t Stat	-0.03785484	
P(T<=t) one-tail	0.485099167	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.970198334	
t Critical two-tail	2.093024054	

Table 6: T test Results for Ultrasonic Pulse Velocity between Columns & Beams P054

The results from the t-test for Ultrasonic pulse velocity for the project P054 for columns and beams are shown in table 6. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, the null hypothesis is accepted. There is no significant difference in the ultrasonic pulse velocity values between columns and beams after deterioration.

5.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between columns and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic pulse velocity		Columns and Slabs
	3.8	*4.5*
Mean	3.62	4.58
Variance	0.244842105	0.145894737
Observations	20	20
Pearson Correlation	0.057922443	
Hypothesized Mean Difference	0	
df	19	
t Stat	-7.069130268	
P(T<=t) one-tail	4.99988E-07	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	9.99976E-07	
t Critical two-tail	2.093024054	

Table 7: T test Results for Ultrasonic Pulse Velocity between Columns & Slabs P054

The results from the t-test for Ultrasonic pulse velocity for the project P054 for Columns & slabs are shown in table 7. It can be observed from the results that the p value is much lesser than the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the ultrasonic pulse velocity values between columns and slabs after deterioration.

6.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between beams and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic pulse velocity		Beams and Slabs
	3.3	*4.5*
Mean	3.625	4.58
Variance	0.214605263	0.145894737
Observations	20	20
Pearson Correlation	-1.32092E-16	
Hypothesized Mean Difference	0	
df	19	
t Stat	-7.113211712	
P(T<=t) one-tail	4.58227E-07	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	9.16455E-07	
t Critical two-tail	2.093024054	

Table 8: T test Results for Ultrasonic Pulse Velocity between Columns & Slabs P054

The results from the t-test for Ultrasonic pulse velocity for the project P054 for beams & slabs are shown in table 8. It can be observed from the results that the p value is lesser than the standard significance level of 0.05. Hence, the null hypothesis is omitted. There is a significant difference in the ultrasonic pulse velocity values between beams and slabs after deterioration.

7.) Null Hypothesis: There is no significant difference in the compressive strength between columns and beams after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Columns and Beams
	18.5	*13.4*
Mean	17.25	16.715
Variance	30.72894737	29.27397368
Observations	20	20
Pearson Correlation	0.134831329	
Hypothesized Mean Difference	0	
df	19	
t Stat	0.332064341	
P(T<=t) one-tail	0.371737964	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.743475929	
t Critical two-tail	2.093024054	

Table 9: T test Results for Compressive Strength between Columns & Beams P054

The results from the t-test for Compressive strength for the project P054 for Columns & beams are shown in table 9. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, the null hypothesis is accepted. There is no significant difference in the compressive strength between columns and beams after deterioration.

8.) Null Hypothesis: There is no significant difference in the compressive strength between columns and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Columns and Slab
	18.5	*29.5*
Mean	17.25	28.86
Variance	30.72894737	28.60673684
Observations	20	20
Pearson Correlation	0.033373077	
Hypothesized Mean Difference	0	
df	19	
t Stat	-6.855750435	
P(T<=t) one-tail	7.65468E-07	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	1.53094E-06	
t Critical two-tail	2.093024054	

Table 10: T test Results for Compressive Strength between Columns & Slab P054

The results from the t-test for Compressive strength for the project P054 for Columns & slabs are shown in table 10. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the compressive strength between columns and slabs after deterioration.

9.) Null Hypothesis: There is no significant difference in the compressive strength between beams and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Beams and Slab
	13.4	*29.5*
Mean	16.715	28.86
Variance	29.27397368	28.60673684
Observations	20	20
Pearson Correlation	-0.007544147	
Hypothesized Mean Difference	0	
df	19	
t Stat	-7.112358773	
P(T<=t) one-tail	4.59E-07	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	9.18E-07	
t Critical two-tail	2.093024054	

Table 11: T test Results for Compressive Strength between Beams & Slab P054

The results from the t-test for Compressive strength for the project P054 for beams & slab are shown in table 11. It can be observed from the results that the p value is lesser than the standard significance level of 0.05. Hence, the null hypothesis is not accepted. There is a significant difference in the compressive strength between beams and slabs after deterioration.

5.3.2 PROJECT P056

The structural elements like the columns, beams and slab are assessed for compressive strength Initially, for the project P056. Later the structural elements for Ultrasonic pulse velocity and finally for carbonation.

1.) Null Hypothesis: There is no significant difference in the compressive strength between columns and beams after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Columns and Beams
	9.3	*16*
Mean	13.75	22.1625
Variance	29.85428571	15.25696429
Observations	8	8
Pearson Correlation	0.662439441	
Hypothesized Mean Difference	0	
df	7	
t Stat	-5.799046625	
P(T<=t) one-tail	0.000332094	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.000664187	
t Critical two-tail	2.364624252	

Table 12: T test Results for Compressive strength between Columns & Beams P056

The results from the t-test for compressive strength for the project P056 for columns and beams are shown in table 12. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the compressive strength between columns and beams after deterioration.

2.) Null Hypothesis: There is no significant difference in the compressive strength between columns and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Columns and Slabs
	9.3	*19.3*
Mean	13.75	22.2375
Variance	29.85428571	8.036964286
Observations	8	8
Pearson Correlation	-0.159596808	
Hypothesized Mean Difference	0	
df	7	
t Stat	-3.667943856	
P(T<=t) one-tail	0.003992965	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.007985931	
t Critical two-tail	2.364624252	

Table 13: T test Results for Compressive strength between Columns & Slabs P056

The results from the t-test for Compressive strength for the project P056 for columns and slabs are shown in table 13. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is not accepted. There is a significant difference in the compressive strength between columns and slabs after deterioration.

3.) Null Hypothesis: There is no significant difference in the compressive strength between beams and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Beams and Slabs
	16	*19.3*
Mean	22.1625	22.2375
Variance	15.25696429	8.036964286
Observations	8	8
Pearson Correlation	0.35234122	
Hypothesized Mean Difference	0	
df	7	
t Stat	-0.053897738	
P(T<=t) one-tail	0.479261306	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.958522613	0.05
t Critical two-tail	2.364624252	

Table 14: T test Results for Compressive strength between Beams & Slabs P056

The results from the t-test for compressive strength for the project P056 for beams and slabs are shown in table 14. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, the null hypothesis is accepted. There is no significant difference in the compressive strength between beams and slabs after deterioration.

4.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between columns and beams after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Columns and Beams
	3	*3.4*
Mean	3.2875	4.15
Variance	0.338392857	0.16
Observations	8	8
Pearson Correlation	0.580180574	
Hypothesized Mean Difference	0	
df	7	
t Stat	-5.104613192	
P(T<=t) one-tail	0.000696172	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.001392344	
t Critical two-tail	2.364624252	

Table 15: T test Results for Ultrasonic Pulse Velocity between Columns & Beams P056

The results from the t-test for Ultrasonic pulse velocity for the project P056 for columns and beams are shown in table 15. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is omitted. There is a significant difference in the ultrasonic pulse velocity values between columns and beams after deterioration.

5.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between columns and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Columns and Slabs
	3	*4*
Mean	3.2875	4.2375
Variance	0.338392857	0.05125
Observations	8	8
Pearson Correlation	0.090850903	
Hypothesized Mean Difference	0	
df	7	
t Stat	-4.443218147	
P(T<=t) one-tail	0.001498023	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.002996046	
t Critical two-tail	2.364624252	

Table 16: T test Results for Ultrasonic Pulse Velocity between Columns & Slabs P056

The results from the t-test for Ultrasonic pulse velocity for the project P056 for columns and slabs are shown in table 16. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the ultrasonic pulse velocity values between columns and slabs after deterioration.

6.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between beams and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Beams and Slabs
	3.4	*4*
Mean	4.15	4.2375
Variance	0.16	0.05125
Observations	8	8
Pearson Correlation	0.433838138	
Hypothesized Mean Difference	0	
df	7	
t Stat	-0.679442413	
P(T<=t) one-tail	0.259342947	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.518685894	
t Critical two-tail	2.364624252	

Table 17: T test Results for Ultrasonic Pulse Velocity between Beams & Slab P056

The results from the t-test for Ultrasonic pulse velocity for the project P056 for beams and slabs are shown in table 17. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, the null hypothesis is accepted. There is no significant difference in the ultrasonic pulse velocity values between beams and slabs after deterioration.

7.) Null Hypothesis: There is no significant difference in the rates of carbonation between columns and beams.

t-Test: Paired Two Sample for Means		
Carbonation		Columns and Beams
	40	*25*
Mean	33.5	26.5
Variance	126.2857143	10.85714286
Observations	8	8
Pearson Correlation	-0.597996675	
Hypothesized Mean Difference	0	
df	7	
t Stat	1.469908134	
P(T<=t) one-tail	0.092523607	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.185047214	
t Critical two-tail	2.364624252	

Table 18: T test Results for Carbonation between Columns & Beams P056

The results from the t-test for Carbonation for the project P056 for columns & beams are shown in table 18. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is true. There is no significant difference in the rate of carbonation between columns and beams.

8.) Null Hypothesis: There is no significant difference in the rates of carbonation between columns and slabs.

t-Test: Paired Two Sample for Means		
Carbonation		Columns and Slabs
	40	*19*
Mean	33.5	25.375
Variance	126.2857143	9.696428571
Observations	8	8
Pearson Correlation	-0.222492537	
Hypothesized Mean Difference	0	
df	7	
t Stat	1.866746198	
P(T<=t) one-tail	0.052087224	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.104174449	
t Critical two-tail	2.364624252	

Table 19: T test Results for Carbonation between Columns & Slabs P056

The results from the t-test for Carbonation for the project P056 for columns and slabs are shown in table 18. It can be seen from the results that the p value is greater than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is true. There is no significant difference in the rate of carbonation.

9.) Null Hypothesis: There is no significant difference in the rates of carbonation between beams and slabs.

t-Test: Paired Two Sample for Means		
Carbonation		Beams and Slabs
	25	*19*
Mean	26.5	25.375
Variance	10.85714286	9.696428571
Observations	8	8
Pearson Correlation	0.006961588	
Hypothesized Mean Difference	0	
df	7	
t Stat	0.704317394	
P(T<=t) one-tail	0.25199277	
t Critical one-tail	1.894578605	
P(T<=t) two-tail	0.503985541	
t Critical two-tail	2.364624252	

Table 20: T test Results for Carbonation between Beams & Slabs P056

The results from the t-test for Carbonation for the project P056 for beams and slabs are shown in table 20. It can be seen from the results that the p value is greater than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is true. There is no significant difference in the rate of carbonation.

5.3.3 PROJECT P068

The structural elements like the columns, beams and slab are assessed for compressive strength Initially, for the project P068. Later the structural elements for Ultrasonic pulse velocity and finally for carbonation.

1.) Null Hypothesis: There is no significant difference in the compressive strength between columns and beams after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Columns and Beams
	10.4	*12.7*
Mean	19.17	14.665
Variance	30.38115789	13.32871053
Observations	20	20
Pearson Correlation	0.469239319	
Hypothesized Mean Difference	0	
df	19	
t Stat	4.043595426	
P(T<=t) one-tail	0.000346786	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.000693572	
t Critical two-tail	2.093024054	

Table 21: T test Results for Compressive strength between Columns & Beams P068

The results from the t-test for compressive strength for the project P068 for columns and beams are shown in table 21. It can be seen from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the compressive strength between columns and beams after deterioration.

2.) Null Hypothesis: There is no significant difference in the compressive strength between columns and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Columns and Slabs
	10.4	*35.1*
Mean	19.17	38.38
Variance	30.38115789	2485.132211
Observations	20	20
Pearson Correlation	0.262279466	
Hypothesized Mean Difference	0	
df	19	
t Stat	-1.764176321	
P(T<=t) one-tail	0.046887286	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.093774573	
t Critical two-tail	2.093024054	

Table 22: T test Results for Compressive strength between Columns & Slabs P068

The results from the t-test for compressive strengths for the project P068 for columns & beams are shown in table 22. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, the null hypothesis is accepted. There is no significant difference in the compressive strength between columns and slabs after deterioration.

3.) Null Hypothesis: There is no significant difference in the compressive strength between beams and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Beams and Slabs
	12.7	*35.1*
Mean	14.665	38.38
Variance	13.32871053	2485.132211
Observations	20	20
Pearson Correlation	0.004293258	
Hypothesized Mean Difference	0	
df	19	
t Stat	-2.122451183	
P(T<=t) one-tail	0.023582658	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.047165316	
t Critical two-tail	2.093024054	

Table 23: T test Results for Compressive strength between Beams & Slabs P068

The results from the t-test for compressive strengths for the project P068 for beams & slabs are shown in table 23. It can be observed from the results that the p value is just less than the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the compressive strength between beams and slabs after deterioration.

4.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between columns and beams after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Columns and Beams
	3.2	*3.2*
Mean	3.9	3.425
Variance	0.296842105	0.081973684
Observations	20	20
Pearson Correlation	0.479109481	
Hypothesized Mean Difference	0	
df	19	
t Stat	4.435746413	
P(T<=t) one-tail	0.000141851	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.000283702	
t Critical two-tail	2.093024054	

Table 24: T test Results for Ultrasonic Pulse Velocity between Columns & Beams P068

The results from the t-test for Ultrasonic pulse velocity for the project P068 for columns & beams are shown in table 24. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is omitted. There is a significant difference in the ultrasonic pulse velocity values between columns and beams after deterioration.

5.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between columns and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Columns and Slabs
	3.2	*4.9*
Mean	3.9	4.61
Variance	0.296842105	0.092526316
Observations	20	20
Pearson Correlation	0.40332462	
Hypothesized Mean Difference	0	
df	19	
t Stat	-6.279446892	
P(T<=t) one-tail	2.49312E-06	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	4.98624E-06	
t Critical two-tail	2.093024054	

Table 25: T test Results for Ultrasonic Pulse Velocity between Columns & Slabs P068

The results from the t-test for Ultrasonic pulse velocity for the project P068 for columns & slabs are shown in table 25. It can be observed from the results that the p value is very less compared to the standard significance level of 0.05. Hence, the null hypothesis is not accepted. There is a significant difference in the ultrasonic pulse velocity values between columns and slabs after deterioration.

6.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between beams and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Beams and Slab
	3.2	*4.9*
Mean	3.425	4.61
Variance	0.081973684	0.092526316
Observations	20	20
Pearson Correlation	0.166191662	
Hypothesized Mean Difference	0	
df	19	
t Stat	-13.89067394	
P(T<=t) one-tail	1.05469E-11	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	2.10939E-11	
t Critical two-tail	2.093024054	

Table 26: T test Results for Ultrasonic Pulse Velocity between Beams & Slabs P068

The results from the t-test for Ultrasonic pulse velocity for the project P068 for beams & slabs are shown in table 26. It can be observed from the results that the p value is very less compared to the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the ultrasonic pulse velocity values between beams and slabs after deterioration.

7.) Null Hypothesis: There is no significant difference in the rates of carbonation between beams and columns.

t-Test: Paired Two Sample for Means		
Carbonation		Beams and Columns
	25	*32*
Mean	25.85	31.4
Variance	42.34473684	59.62105263
Observations	20	20
Pearson Correlation	-0.016550227	
Hypothesized Mean Difference	0	
df	19	
t Stat	-2.438189393	
P(T<=t) one-tail	0.012376526	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.024753052	
t Critical two-tail	2.093024054	

Table 27: T test Results for Carbonation between Columns & Beams P068

The results from the t-test for carbonation for the project P068 for columns & beams are shown in table 27. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is not true. There is a significant difference in the rate of carbonation between beams and columns.

8.) Null Hypothesis: There is no significant difference in the rates of carbonation between columns and slabs.

t-Test: Paired Two Sample for Means		
Carbonation		Columns and Slabs
	25	*36*
Mean	25.85	33.05
Variance	42.34473684	57.83947368
Observations	20	20
Pearson Correlation	0.142667546	
Hypothesized Mean Difference	0	
df	19	
t Stat	-3.470874059	
P(T<=t) one-tail	0.001279473	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.002558945	
t Critical two-tail	2.093024054	

Table 28: T test Results for Carbonation between Columns & Slabs P068

The results from the t-test for carbonation for the project P068 for columns & slabs are shown in table 28. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is false. There is a significant difference in the rate of carbonation between columns and slabs.

9.) Null Hypothesis: There is no significant difference in the rates of carbonation between beams and slabs.

t-Test: Paired Two Sample for Means		
Carbonation		Beams and Slabs
	32	*36*
Mean	31.4	33.05
Variance	59.62105263	57.83947368
Observations	20	20
Pearson Correlation	-0.066681805	
Hypothesized Mean Difference	0	
df	19	
t Stat	-0.659230191	
P(T<=t) one-tail	0.258829453	
t Critical one-tail	1.729132812	
P(T<=t) two-tail	0.517658906	
t Critical two-tail	2.093024054	

Table 29: T test Results for Carbonation between Beams & Slabs P068

The results from the t-test for carbonation for the project P068 for beams & slabs are shown in table 29. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is true. There is no significant difference in the rate of carbonation between beams and slabs.

5.3.4 PROJECT P069

The structural elements like the columns, beams and slab are assessed for compressive strength Initially, for the project P069. Later the structural elements for Ultrasonic pulse velocity and finally for carbonation.

1.) Null Hypothesis: There is no significant difference in the compressive strength between columns and beams after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Columns and Beams
	7.3	*16*
Mean	16.21176471	16.05882353
Variance	37.76485294	15.07882353
Observations	17	17
Pearson Correlation	-0.292898783	
Hypothesized Mean Difference	0	
df	16	
t Stat	0.077141317	
P(T<=t) one-tail	0.469733775	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	0.939467551	
t Critical two-tail	2.119905299	

Table 30: T test Results for Compressive strength between Columns & Beams P069

The results from the t-test for compressive strength for the project P069 for columns and beams are shown in table 30. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, the null hypothesis is accepted. There is no significant difference in the compressive strength between columns and beams after deterioration.

2.) Null Hypothesis: There is no significant difference in the compressive strength between columns and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Compressive Strength		Columns and Slabs
	7.3	*41*
Mean	16.21176471	22.78235294
Variance	37.76485294	60.77779412
Observations	17	17
Pearson Correlation	-0.331092414	
Hypothesized Mean Difference	0	
df	16	
t Stat	-2.37361984	
P(T<=t) one-tail	0.015237845	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	0.03047569	
t Critical two-tail	2.119905299	

Table 31: T test Results for Compressive strength between Columns & Slabs P069

The results from the t-test for compressive strengths for the project P069 for columns & beams are shown in table 31. It can be observed from the results that the p value is just less than the standard significance level of 0.05. Hence, the null hypothesis is not accepted. There is a significant difference in the compressive strength between columns and slabs after deterioration.

3.) Null Hypothesis: There is no significant difference in the compressive strength between beams and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Comresive Strength		Beams and Slab
	16	*41*
Mean	16.05882353	22.78235294
Variance	15.07882353	60.77779412
Observations	17	17
Pearson Correlation	0.609448079	
Hypothesized Mean Difference	0	
df	16	
t Stat	-4.441498388	
P(T<=t) one-tail	0.000205174	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	0.000410349	
t Critical two-tail	2.119905299	

Table 32: T test Results for Compressive strength between Beams & Slabs P069

The results from the t-test for compressive strengths for the project P069 for beams & slabs are shown in table 32. It can be seen from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the compressive strength between beams and slabs after deterioration.

4.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between beams and columns after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Beams and Columns
	2.7	*3.6*
Mean	3.670588235	3.6
Variance	0.254705882	0.16625
Observations	17	17
Pearson Correlation	-0.185271893	
Hypothesized Mean Difference	0	
df	16	
t Stat	0.412751496	
P(T<=t) one-tail	0.342633373	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	0.685266746	
t Critical two-tail	2.119905299	

Table 33: T test Results for Ultrasonic Pulse Velocity between Columns & Beams P069

The results from the t-test for Ultrasonic pulse velocity for the project P069 for columns & beams are shown in table 33. It can be observed from the results that the p value is greater than the standard significance level of 0.05. Hence, the null hypothesis is true. There is no significant difference in the ultrasonic pulse velocity values between beams and columns after deterioration.

5.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between columns and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Columns and Slabs
	2.7	*5.3*
Mean	3.670588235	4.141176471
Variance	0.254705882	0.368823529
Observations	17	17
Pearson Correlation	-0.291479873	
Hypothesized Mean Difference	0	
df	16	
t Stat	-2.16631998	
P(T<=t) one-tail	0.022865464	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	0.045730928	
t Critical two-tail	2.119905299	

Table 34: T test Results for Ultrasonic Pulse Velocity between Columns & Slabs P069

The results from the t-test for Ultrasonic pulse velocity for the project P069 for columns & slabs are shown in table 34. It can be observed from the results that the p value is just less than the standard significance level of 0.05. Hence, the null hypothesis is rejected. There is a significant difference in the ultrasonic pulse velocity values between columns and slabs after deterioration.

6.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values between beams and slabs after deterioration.

t-Test: Paired Two Sample for Means		
Ultrasonic Pulse Velocity		Beams and Slab
	3.6	*5.3*
Mean	3.6	4.141176471
Variance	0.16625	0.368823529
Observations	17	17
Pearson Correlation	0.421508514	
Hypothesized Mean Difference	0	
df	16	
t Stat	-3.906062355	
P(T<=t) one-tail	0.000628804	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	0.001257609	
t Critical two-tail	2.119905299	

Table 35: T test Results for Ultrasonic Pulse Velocity between Beams & Slabs P069

The results from the t-test for Ultrasonic pulse velocity for the project P069 for beams & slabs are shown in table 35. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, the null hypothesis is omitted. There is a significant difference in the ultrasonic pulse velocity values between beams and slabs after deterioration.

7.) Null Hypothesis: There is no significant difference in the rates of carbonation between columns and beams.

t-Test: Paired Two Sample for Means		
Carbonation		Columns and Beams
	50	*25.2*
Mean	36.47058824	25.05882353
Variance	126.1397059	50.05882353
Observations	17	17
Pearson Correlation	-0.185990569	
Hypothesized Mean Difference	0	
df	16	
t Stat	3.280194433	
P(T<=t) one-tail	0.002355904	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	0.004711809	
t Critical two-tail	2.119905299	

Table 36: T test Results for Carbonation between Columns & Beams P069

The results from the t-test for carbonation for the project P069 for columns & beams are shown in table 36. It can be observed from the results that the p value is less than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is not true. There is a significant difference in the rate of carbonation between columns and beams.

8.) Null Hypothesis: There is no significant difference in the rates of carbonation between columns and slabs.

t-Test: Paired Two Sample for Means		
Carbonation		Columns and Slabs
	50	*10*
Mean	36.47058824	12.88235294
Variance	126.1397059	12.73529412
Observations	17	17
Pearson Correlation	-0.587974821	
Hypothesized Mean Difference	0	
df	16	
t Stat	7.131072564	
P(T<=t) one-tail	1.19176E-06	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	2.38352E-06	
t Critical two-tail	2.119905299	

Table 37: T test Results for Carbonation between Columns & Slabs P069

The results from the t-test for carbonation for the project P069 for columns & slabs are shown in table 37. It can be observed from the results that the p value is much lesser than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is false. There is a significant difference in the rate of carbonation between columns and slabs.

9.) Null Hypothesis: There is no significant difference in the rates of carbonation between beams and slabs.

t-Test: Paired Two Sample for Means		
Carbonation		Beams and Slabs
	25.2	*10*
Mean	25.05882353	12.88235294
Variance	50.05882353	12.73529412
Observations	17	17
Pearson Correlation	0.133959684	
Hypothesized Mean Difference	0	
df	16	
t Stat	6.707147439	
P(T<=t) one-tail	2.5179E-06	
t Critical one-tail	1.745883676	
P(T<=t) two-tail	5.03581E-06	
t Critical two-tail	2.119905299	

Table 38: T test Results for Carbonation between Beams & Slabs P069

The results from the t-test for carbonation for the project P068 for beams & slabs are shown in table 38. It can be observed from the results that the p value is much lesser than the standard significance level of 0.05. Hence, it can be inferred that the null hypothesis is rejected. There is a significant difference in the rate of carbonation between beams and slabs.

		Compressive Strength		
		Columns	Beams	Slabs
	Columns			
	Beams	Insignificant		
P054/BIN	Slabs	Significant	Significant	

		Ultrasonic Pulse Velocity		
		Columns	Beams	Slabs
	Columns			
	Beams	Insignificant		
P054/BIN	Slabs	Significant	Significant	

		Rate of Carbonation		
		Columns	Beams	Slabs
	Columns			
	Beams	Insignificant		
P054/BIN	Slabs	Significant	Significant	

Table 39: P054 T Test Summary

		Compressive Strength		
		Columns	Beams	Slabs
	Columns			
	Beams	Significant		
P056	Slabs	Significant	Insignificant	

		Ultrasonic Pulse Velocity		
		Columns	Beams	Slabs
	Columns			
	Beams	Significant		
P056	Slabs	Significant	Insignificant	

		Rate of Carbonation		
		Columns	Beams	Slabs
	Columns			
	Beams	Insignificant		
P056	Slabs	Insignificant	Insignificant	

Table 40: P056 T Test Summary

		Compressive Strength		
		Columns	Beams	Slabs
	Columns			
	Beams	Significant		
P068	Slabs	Insignificant	Significant	

		Ultrasonic Pulse Velocity		
		Columns	Beams	Slabs
	Columns			
	Beams	Significant		
P068	Slabs	Significant	Significant	

		Rate of Carbonation		
		Columns	Beams	Slabs
	Columns			
	Beams	Significant		
P068	Slabs	Significant	Insignificant	

Table 41: P068 T Test Summary

		Compressive Strength		
		Columns	Beams	Slabs
	Columns			
	Beams	Insignificant		
P069	Slabs	Significant	Significant	

		Ultrasonic Pulse Velocity		
		Columns	Beams	Slabs
	Columns			
	Beams	Insignificant		
P069	Slabs	Significant	Significant	

		Rate of Carbonation		
		Columns	Beams	Slabs
	Columns			
	Beams	Significant		
P069	Slabs	Significant	Significant	

Table 42: P069 T Test Summary

The Tables 39-42 portrayed above exhibit the summary of T-test conducted. It is observed that the columns and beams exhibit similarities as compared to beams with slab and column with slab. Furthermore, on observing the data it is inferred that most of the projects have different behaviour although carbonation and loss of compressive strength is more in beams and columns than in slabs. The similar is observed for rate of carbonation after deterioration and ultrasonic pulse velocity after deterioration.

5.4 ONE WAY ANOVA

This test is done to simultaneously compare all the data for the structural elements that are the beams, columns and slab together for the compressive strength after deterioration, ultrasonic pulse velocity after deterioration and rate of carbonation. As in the previous chapter the anova test is done separately for each project as compiling them and performing the test for the larger data base does not provide accurate results and interpretations.

5.4.1 PROJECT P054

1.) Null Hypothesis: There is no significant difference in the compressive strengths of columns, beams, and slabs after deterioration

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	21	363.5	17.30952381	29.26690476		
Beams	21	347.7	16.55714286	28.33357143		
Slabs	21	606.7	28.89047619	27.19590476		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	2007.569524	2	1003.784762	35.51276896	6.6659E-11	3.150411311
Within Groups	1695.927619	60	28.26546032			
Total	3703.497143	62				

Table 43: one-way anova test for compressive strength amongst the statistical elements P054

From table 43, we can clearly observe that the p value is less than the standard significant level and hence, the null hypothesis is to be rejected. There is a significant difference between the statistical elements in compressive strength after deterioration.

2.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values of columns, beams, and slabs after deterioration

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	21	76.2	3.628571429	0.234142857		
Beams	21	75.8	3.60952381	0.208904762		
Slabs	21	96.1	4.576190476	0.138904762		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	12.82952381	2	6.414761905	33.06848867	2.08576E-10	3.150411311
Within Groups	11.63904762	60	0.193984127			
Total	24.46857143	62				

Table 44: one-way anova test for ultrasonic pulse velocity amongst the statistical elements P054

From table 44, we can clearly see that the p value is less than the standard significant level of 0.05. Hence, the null hypothesis is not accepted. This means that there is a significant difference between the statistical elements in ultrasonic pulse velocity after deterioration.

3.) Null Hypothesis: There is no significant difference in the rates of carbonation of statistical elements.

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	21	590	28.0952381	98.19047619		
Beams	21	558	26.57142857	47.55714286		
Slabs	21	381	18.14285714	25.52857143		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	1206.888889	2	603.4444444	10.56967304	0.00011684	3.150411311
Within Groups	3425.52381	60	57.09206349			
Total	4632.412698	62				

Table 45: one-way anova test for carbonation amongst the statistical elements P054

From table 45, we can clearly observe that the p value is less than the standard significant level and hence, the null hypothesis is omitted. There is a significant difference between the statistical elements in the rate of carbonation.

5.4.2 PROJECT P056

1.) Null Hypothesis: There is no significant difference in the compressive strength of columns, beams, and slabs after deterioration

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	9	119.3	13.25555556	28.32277778		
Beams	9	193.3	21.47777778	17.56944444		
Slabs	9	197.2	21.91111111	7.991111111		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	428.1340741	2	214.067037	11.91836272	0.00025433	3.402826105
Within Groups	431.0666667	24	17.96111111			
Total	859.2007407	26				

Table 46: one-way anova test for compressive strength amongst the statistical elements P056

From table 46, we can clearly observe that the p value is less than the standard significant level and therefore, the null hypothesis is to be rejected. There is a significant difference between the statistical elements in compressive strength after deterioration.

2.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values of columns, beams, and slabs after deterioration

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	9	29.3	3.255555556	0.305277778		
Beams	9	36.6	4.066666667	0.2025		
Slabs	9	37.9	4.211111111	0.051111111		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	4.775555556	2	2.387777778	12.81709742	0.000163367	3.402826105
Within Groups	4.471111111	24	0.186296296			
Total	9.246666667	26				

Table 47: one-way anova test for ultrasonic pulse velocity amongst the statistical elements P056

From table 47, we can clearly see that the p value is less than the standard significant level of 0.05 and the null hypothesis is rejected. There is a significant difference between the statistical elements in ultrasonic pulse velocity after deterioration.

3.) Null Hypothesis: There is no significant difference in the rates of carbonation of columns, beams, and slabs.

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	9	308	34.22222222	115.1944444		
Beams	9	237	26.33333333	9.75		
Slabs	9	222	24.66666667	13		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	468.962963	2	234.4814815	5.09947644	0.014268594	3.402826105
Within Groups	1103.555556	24	45.98148148			
Total	1572.518519	26				

Table 48: one-way anova test for carbonation amongst the statistical elements P056

From table 48, we can clearly observe that the p value is less than the standard significant level and hence, the null hypothesis is omitted. There is a significant difference between the statistical elements in the rate of carbonation.

5.4.3 PROJECT P068

1.) Null Hypothesis: There is no significant difference in the compressive strength of columns, beams, and slabs after deterioration

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	21	393.8	18.75238095	32.52461905		
Beams	21	306	14.57142857	12.84614286		
Slabs	21	802.7	38.22380952	2361.387905		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	6692.364127	2	3346.182063	4.170981632	0.020133255	3.150411311
Within Groups	48135.17333	60	802.2528889			
Total	54827.53746	62				

Table 49: one-way anova test for compressive strength amongst the statistical elements P068

From table 49, we can clearly observe that the p value is less than the standard significant level. The null hypothesis is therefore rejected. There is a significant difference between the statistical elements in compressive strength after deterioration.

2.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values of columns, beams, and slabs after deterioration

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	21	81.2	3.866666667	0.305333333		
Beams	21	71.7	3.414285714	0.080285714		
Slabs	21	97.1	4.623809524	0.091904762		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	15.68603175	2	7.843015873	49.2730355	2.18712E-13	3.150411311
Within Groups	9.55047619	60	0.159174603			
Total	25.23650794	62				

Table 50: one-way anova test for ultrasonic pulse velocity amongst the statistical elements P068

From table 50, we can clearly see that the p value is less than the standard significant level of 0.05 and hence, the null hypothesis is rejected. There is a significant difference between the statistical elements in ultrasonic pulse velocity after deterioration.

3.) Null Hypothesis: There is no significant difference in the rates of carbonation of columns, beams, and slabs.

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	21	542	25.80952381	40.26190476		
Beams	21	660	31.42857143	56.65714286		
Slabs	21	697	33.19047619	55.36190476		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	624.0952381	2	312.047619	6.147471778	0.003726489	3.150411311
Within Groups	3045.619048	60	50.76031746			
Total	3669.714286	62				

Table 51: one-way anova test for carbonation amongst the statistical elements P068

From table 51, we can clearly observe that the p value is less than the standard significant. The null hypothesis is omitted. There is a significant difference between the statistical elements in the rate of carbonation.

5.4.3 PROJECT P069

1.) Null Hypothesis: There is no significant difference in the compressive strength of columns, beams, and slabs after deterioration

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	18	282.9	15.71666667	39.95558824		
Beams	18	289	16.05555556	14.19202614		
Slabs	18	428.3	23.79444444	75.64055556		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	751.5344444	2	375.7672222	8.685704308	0.000567204	3.178799292
Within Groups	2206.398889	51	43.26272331			
Total	2957.933333	53				

Table 52: one-way anova test for compressive strength amongst the statistical elements P069

From table 52, we can clearly observe that the p value is less than the standard significant level of 0.05 and the null hypothesis is rejected. There is a significant difference between the statistical elements in compressive strength after deterioration.

2.) Null Hypothesis: There is no significant difference in the ultrasonic pulse velocity values of columns, beams, and slabs after deterioration

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	18	65.1	3.616666667	0.292058824		
Beams	18	64.8	3.6	0.156470588		
Slabs	18	75.7	4.205555556	0.421732026		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	4.282592593	2	2.141296296	7.381562148	0.001529281	3.178799292
Within Groups	14.79444444	51	0.290087146			
Total	19.07703704	53				

Table 53: one-way anova test for ultrasonic pulse velocity amongst the statistical elements P069

From table 53, we can clearly see that the p value is less than the standard significant level of 0.05 and hence, the null hypothesis is rejected. There is a significant difference between the statistical elements in ultrasonic pulse velocity after deterioration.

3.) Null Hypothesis: There is no significant difference in the rates of carbonation of columns, beams, and slabs.

Anova: Single Factor						
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
Columns	18	670	37.22222222	128.8888889		
Beams	18	451.2	25.06666667	47.11529412		
Slabs	18	229	12.72222222	12.44771242		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	5402.357037	2	2701.178519	43.00055214	1.13914E-11	3.178799292
Within Groups	3203.682222	51	62.81729847			
Total	8606.039259	53				

Table 54: one-way anova test for carbonation amongst the statistical elements P069

From table 54, we can clearly observe that the p value is less than the standard significant level. We can reject the null hypothesis. There is a significant difference between the statistical elements in the rate of carbonation.

From the anova test we can observe that the rate of carbonation, the compressive strength after deterioration and the ultrasonic pulse velocity after deterioration are not similar for columns, beams and slab. They vary accordingly. Hence we can say that occurrence of deterioration and reduction in the compressive strength varies for all the elements and is not constant throughout.

5.5 NORMAL DISTRIBUTION CURVE

The normal distribution of beams, columns and slabs are plotted to understand the spread of compressive strength after deterioration, rate of carbonation and Ultra sonic pulse velocity after deterioration.

1.) CARBONATION

The Figure 23 depicts the bell curve for carbonation values after deterioration of structural elements which are beams, column and slab. It is observed that slabs have the least amount of carbonation with the peak of normal distribution with the highest frequency at 21.6mm. Furthermore, the beams are the second most carbonated elements the peak of normal distribution with the highest frequency at 27.8mm. and finally the columns exhibit the highest normal distribution for the maximum frequency at 32mm. Froom this we can understand that the columns have more tendency to be exposed to carbonation than beams and slabs.

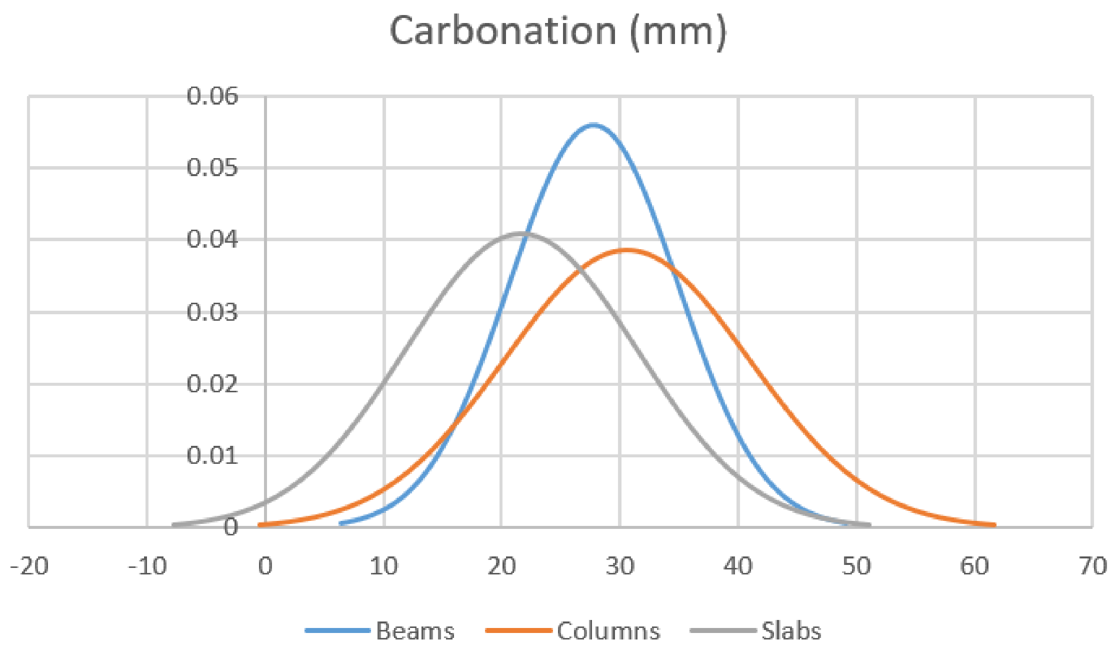


Figure 23 Normal Distribution Curve for Carbonation of different structural elements

2.) ULTRASONIC PULSE VELOCITY

This part of the study does statistical analysis of Ultrasonic pulse velocity amongst all structural elements. Figure 24 depicts the bell curve for Ultrasonic pulse velocity after deterioration for the structural elements, which are the beams, columns and slabs. We observe that for the highest probability frequency the slabs have an ultrasonic pulse velocity of 4.40 Km/s. whereas the beams have 3.54 Km/s and the columns have 3.6 Km/s. this indicates that the slabs have a good structural integrity as the speed is the fastest in slabs, then followed by beams and columns. It is observed that although beams have the least Ultrasonic pulse velocity there is only a slight difference in comparison to the columns. Thus we can say that the beams and columns show similar effect after deterioration of the structure.

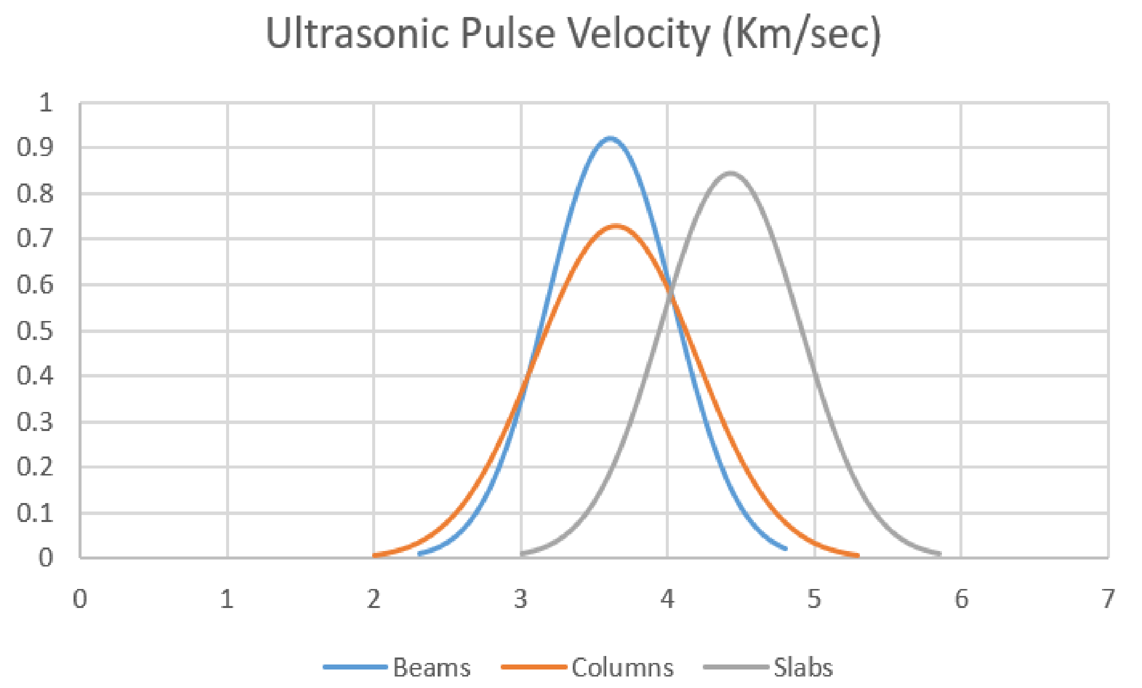


Figure 24: Normal Distribution Curve for Ultrasonic Pulse Velocity of different structural elements

3.) COMPRESSIVE STRENGTH

The normal distribution curve for compressive strengths is portrayed in the Figure 25. It is observed from the normal distribution curve that for the highest probability frequency the slabs exhibit the maximum compressive strength of 25N/Sq. mm. and then followed by columns and beams. The columns and beams have a very small difference where in which the columns exhibit a value of 19N/Sq. mm, whereas the beams exhibit a value of 18.89 N/Sq. mm. Moreover, we can understand that the beams and columns exhibit a similar behaviour like that ultrasonic pulse velocity. Thus we can tell that columns and beams are more exposed to deterioration than slabs, and exhibit a similar behaviour in deteriorating.

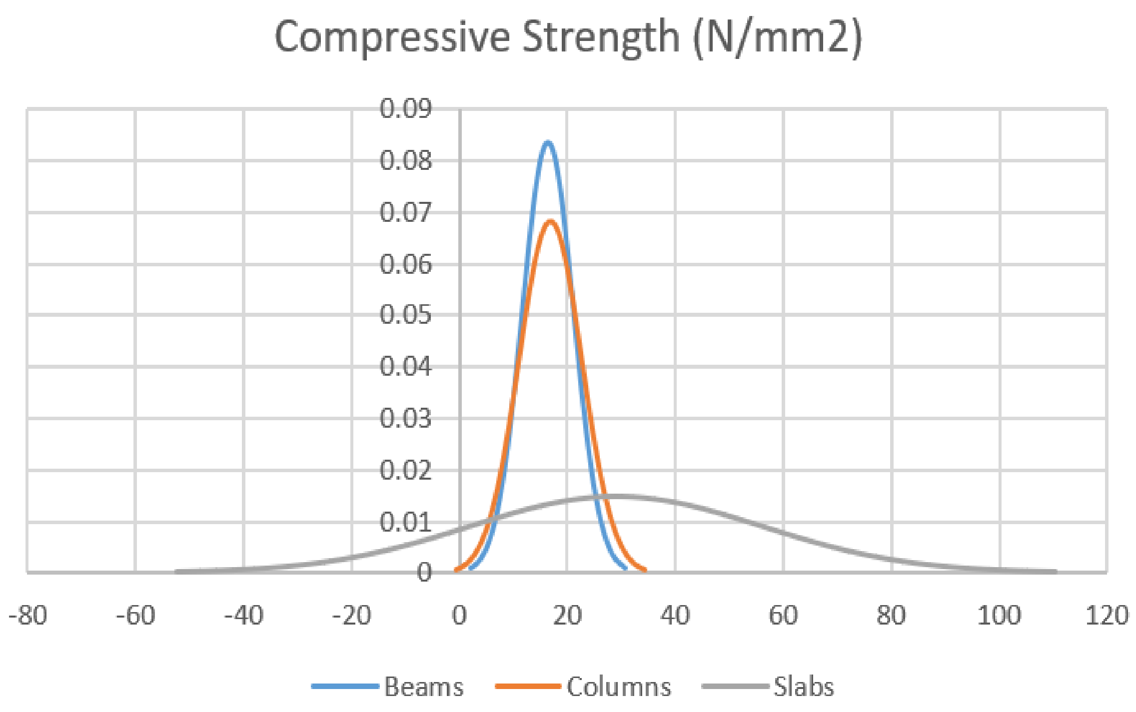


Figure 25 Normal Distribution Curve for Compressive strength of different structural elements

5.4 CO-RELATIONS

In this part of the study, correlation curves are plotted between the statistical measurements. The Table 55 shows the Pearson co-relation table between compressive strength, ultrasonic pulse velocity, and carbonation.

	COMPRESSIVE STRENGTH (N/mm^2)	ULTRA SONIC PULSE VELOCITY (Km/sec)	CARBONATION (mm)
COMPRESSIVE STRENGTH (N/mm^2)	1		
ULTRA SONIC PULSE VELOCITY (Km/sec)	0.465999728	1	
CARBONATION (mm)	-0.098926569	-0.424300828	1

Table 55: Pearson's Correlation Table

We can observe that compressive strength and ultrasonic pulse velocity have a positive correlation with each other while the other two pairs, i.e., compressive strength – carbonation and ultrasonic pulse velocity – carbonation have a negative co-relation with each other

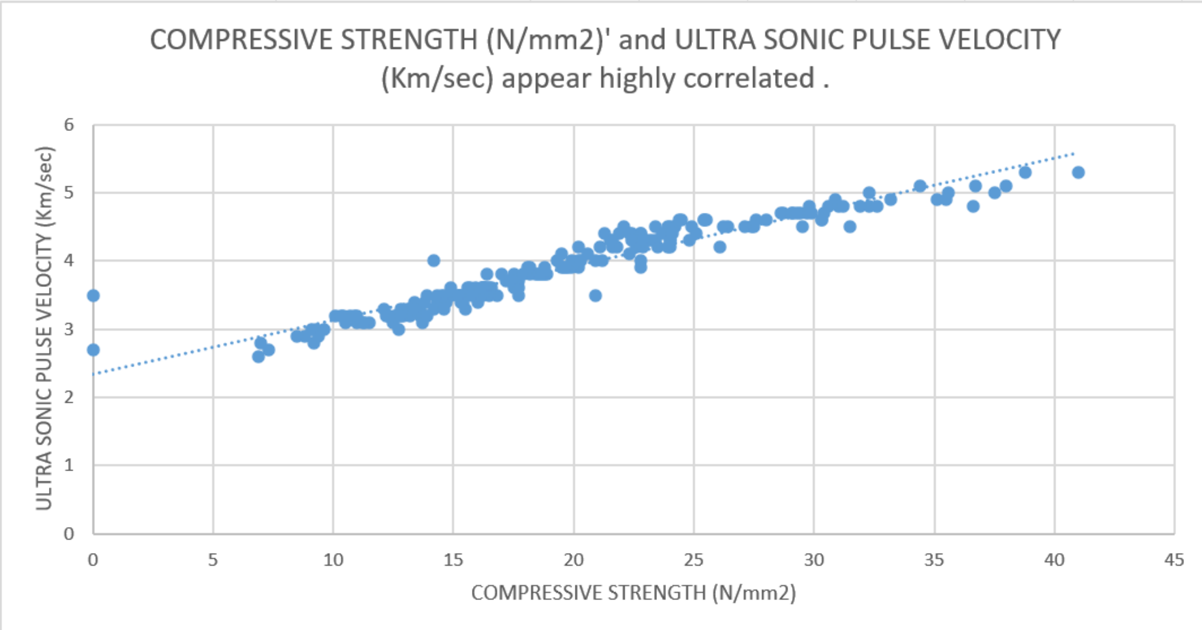


Figure 26 Correlation Graph of Compressive Strength and Ultra Sonic Pulse Velocity

On observing the figure 26 we understand the correlation graph plotted is a linear graph indicating a positive correlation. This depicts that the relation is proportional to each other that is with higher ultra-sonic pulse velocity values the higher is the compressive strength.

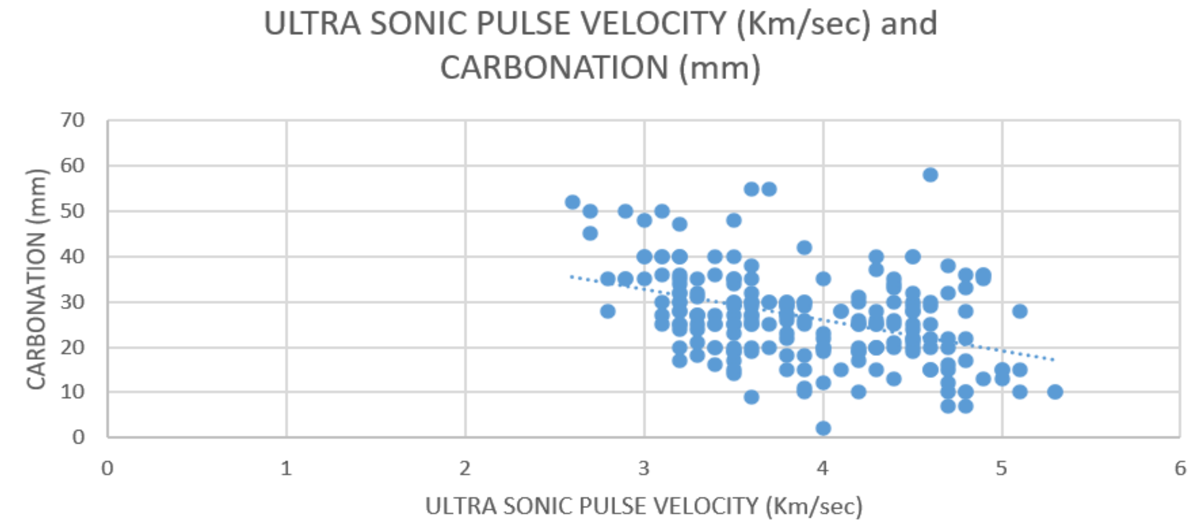


Figure 27: Correlation Graph Of Ultrasonic Pulse Velocity and Carbonation

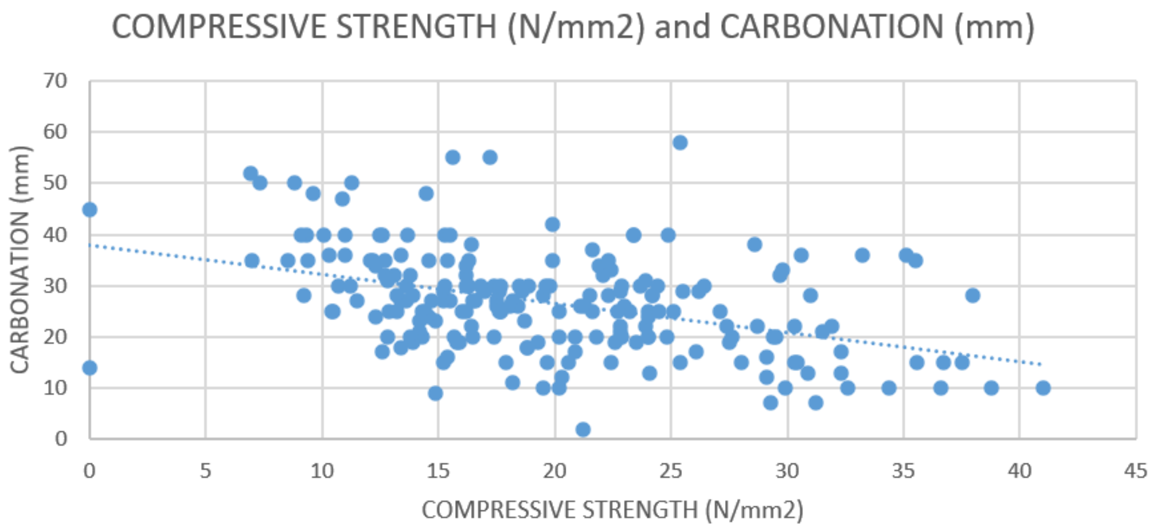


Figure 28 Correlation Graph of Compressive strength and Carbonation.

On observing the correlation graph between ultrasonic pulse velocity and carbonation portrayed in figure 27, it is observed that the parameters are inversely proportional to each other. As the carbonation value increases the value of ultrasonic pulse velocity decreases.

Further on observing the figure 28, which portrays the correlation between compressive strength and carbonation, it is observed that they are inversely proportional to each other. This indicates that the compressive strength decreases with increase in carbonation. In some instances, we can see that there are high compressive strength values for carbonation, these plots portray the points of initial carbonation stage where the strength of concrete increases due to formation of calcium carbonate. In the later stage of carbonation, i.e. when the reinforcement is effected the compressive strength value decreases.

CHAPTER 6: STRUCTURAL ANAYLSIS

Analysing the structure with the data obtained by core test results exhibits that the study had progressed to understand the relation between different parameters discussed in the previous chapter. Further in this chapter the analysis of the structure is done by modelling of the structure using software called ETABS. Amongst the four projects, the modelling had been done for the project P054 as it was larger in span amongst the other and analysing this would provide us with a wider prospect as compared to others.

With the data obtained from core test, two models were made. The first model was made with parameters such that the structure was not deteriorated and the other model was created using the compressive strengths acquired from the core test. The main purpose of modelling is to study if there are any major differences in transfer of moments and shear forces in the columns and beams due to deterioration of the structure.

6.1 MODELLING

Modelling is an essential part of the structural analysis, as the acquired data needs to be precise and accurate for designing. Improper modelling could lead to obtaining inaccurate values for forces that need to be further designed. For this project one of the main challenge was modelling as there was inadequate structural data as the building was constructed in the late 1980's.

6.1.1 DATA ACQUISITION

The unavailability of data is often seen in the projects that are really old, and makes it harder for further analysis and design. The drawings available for the existing structure had the layout and the data of the columns missing, the only available data for the structure was beam and slab reinforcements provided for typical floors. Using this data, the gap to unavailable data was bridged. Using the beams layout another as built layout for the beams were made to confirm the available data Figures 30-32. After confirming the as built data of the beams the column location was determined and confirmed using a hammer, by its hardness as compared to the surrounding finishing. The length of the columns was determined by the same procedure using the hammer whereas the width was kept same as the wall thickness. In some floors the width was more and was comparatively easier to determine as it would protrude outside the finishing. The buildings symmetrical nature had helped to replicate the date and reconfirm if there was any data missing.

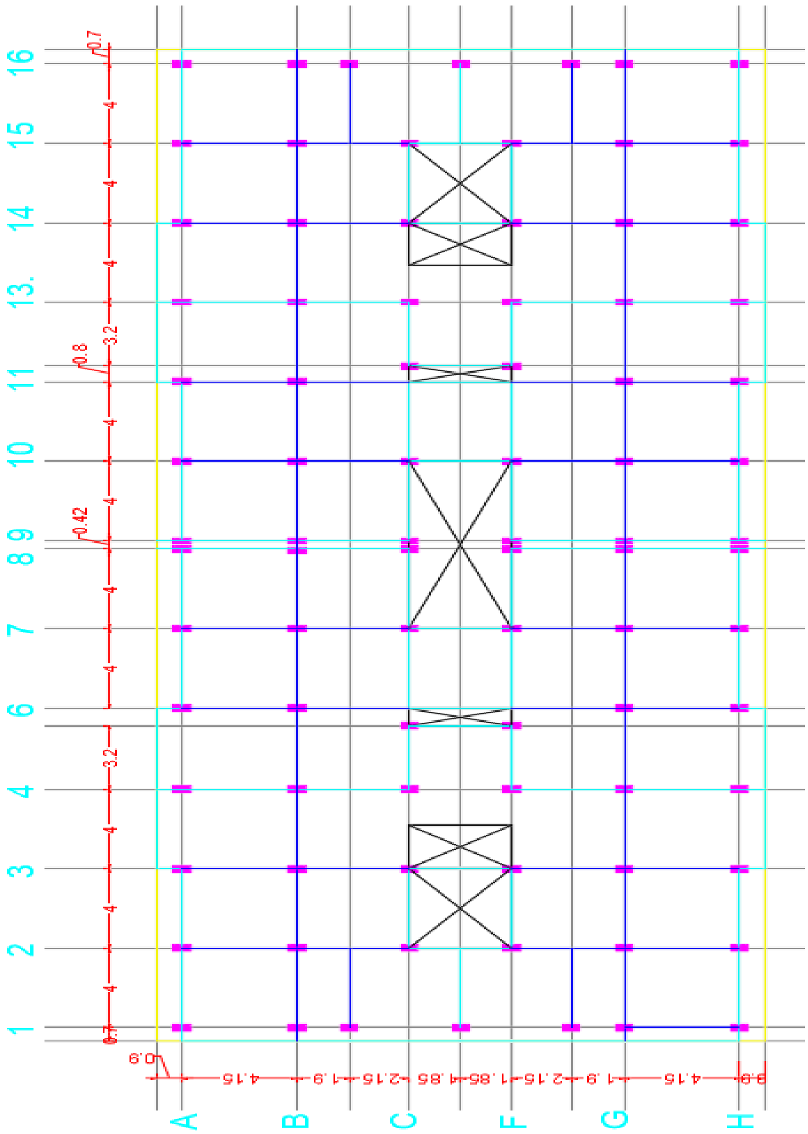


Figure 29 Columns Layout as per site conditions

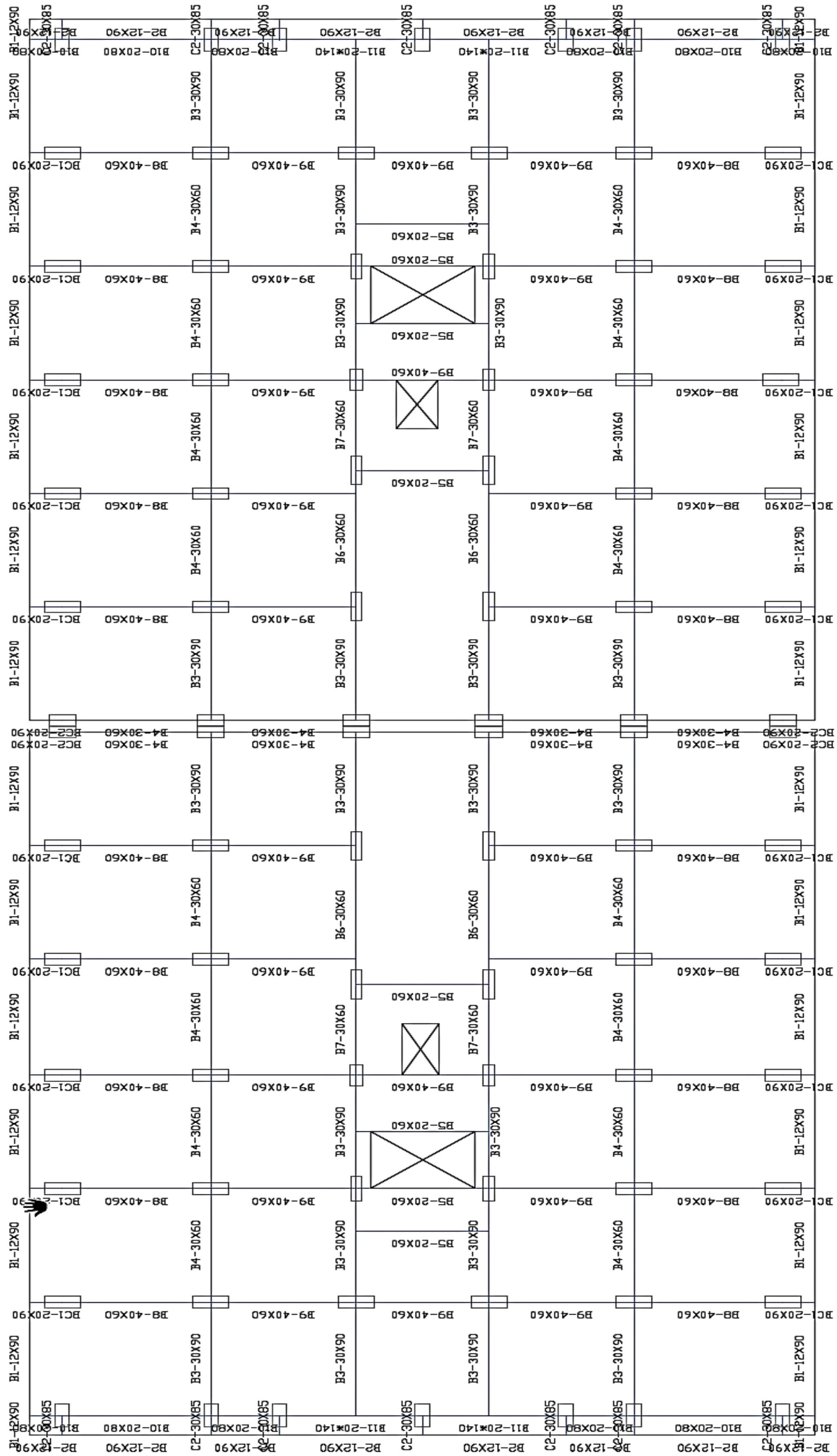


Figure 30 First Slab Layout as per site condition

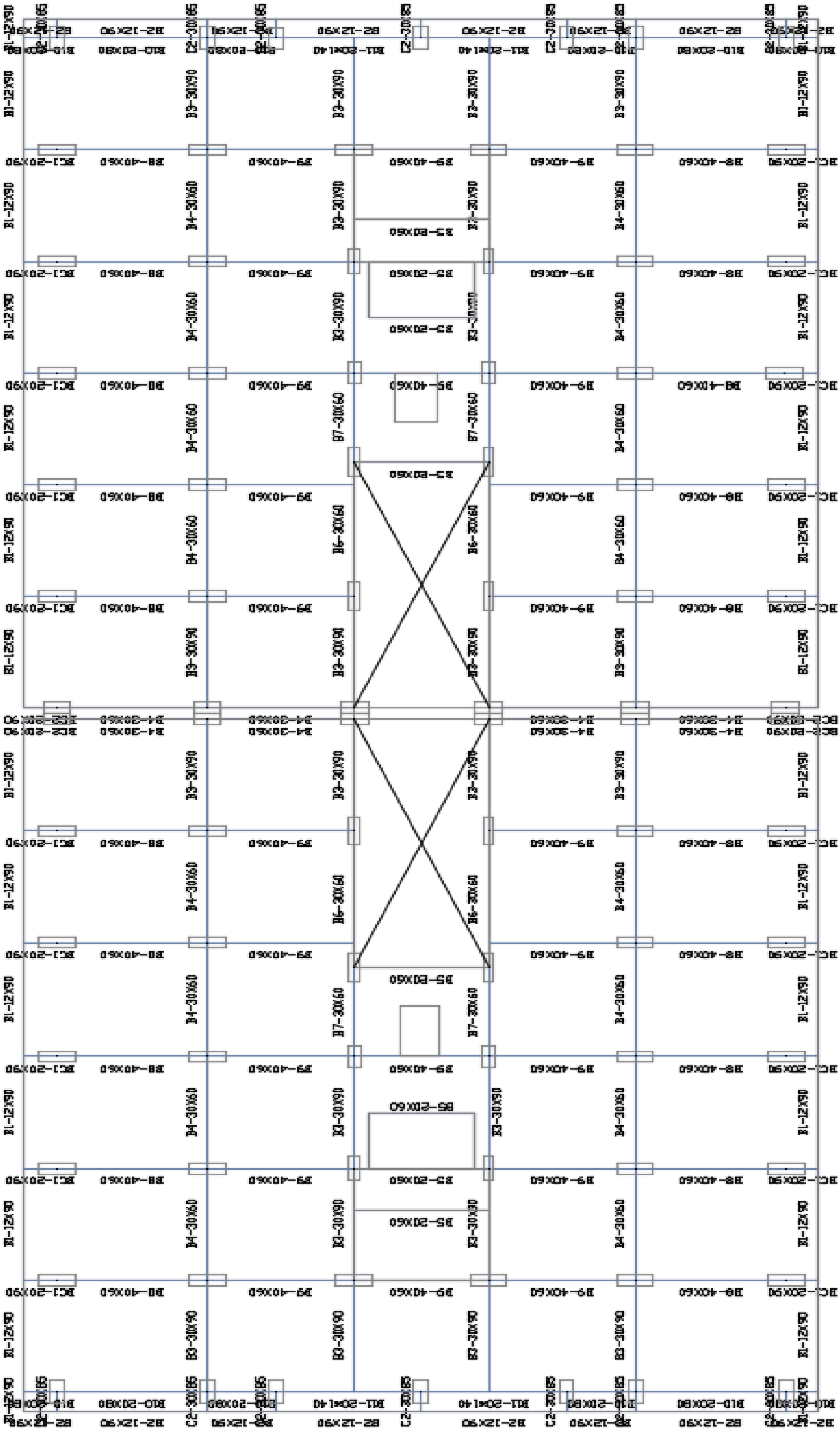


Figure 31 Typical Slab Layout as per site conditions.

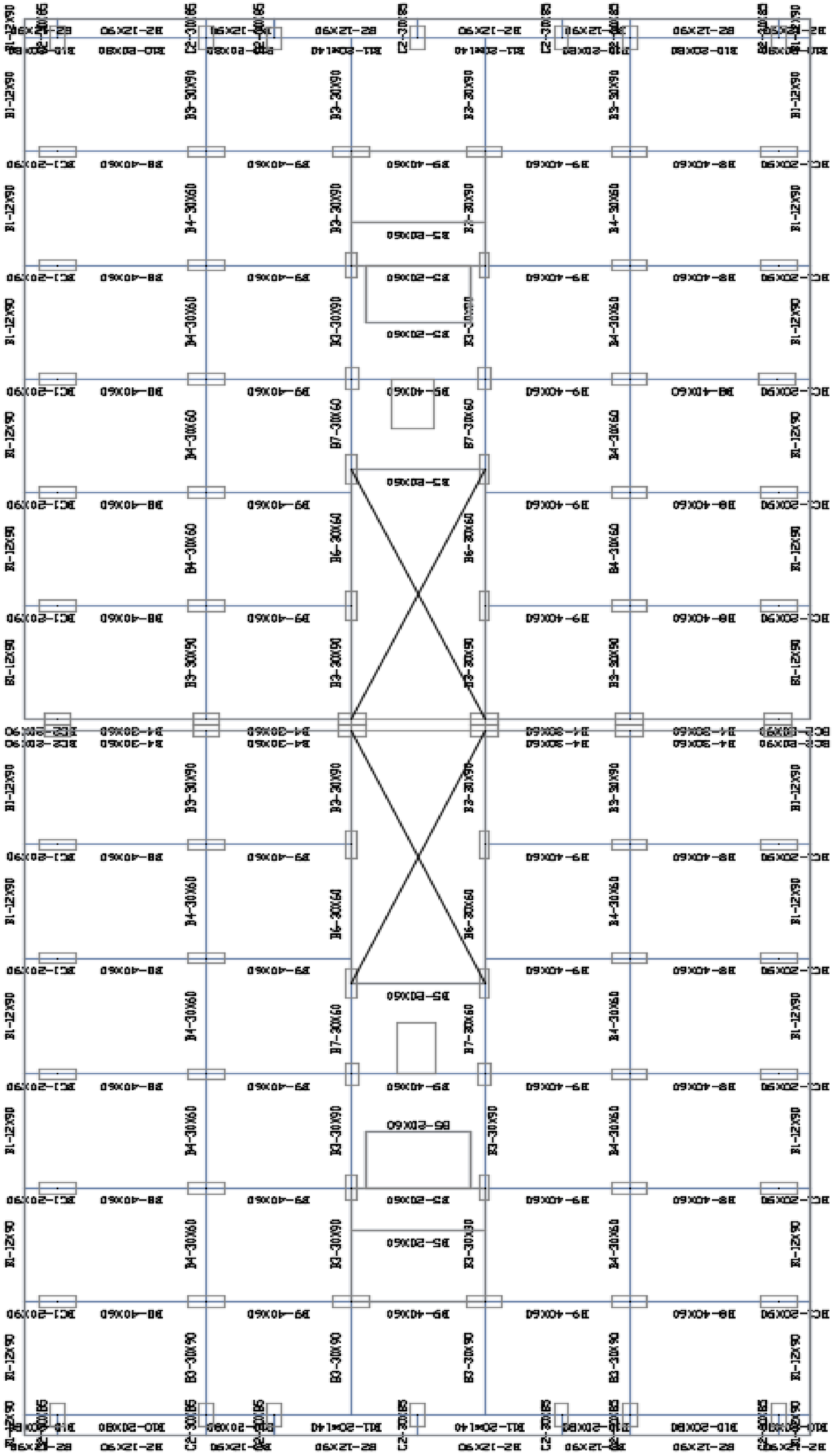
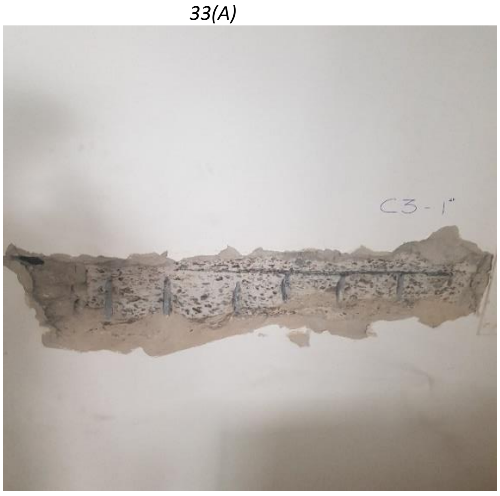


Figure 32 Roof slab Layout as per site conditions.

After acquiring the initial size of the structural elements the reinforcement data was acquired for the columns, the reinforcement data was available for the beams and the slabs. To acquire the data of reinforcement for the columns the cover of the column was removed to visible see the reinforcement provided and then was immediately covered up using micro concrete to avoid further deterioration. another way to procure this data was by ultra-sonic scanning. This method of removing the concrete cover was chosen as it was comparatively cheaper. The columns were classified into groups and few elements of this group were chosen to acquire the data. The following images exhibit on the reinforcement data acquisition of the columns. The Figure 33 exhibits image of retrieving columns data by chipping of cover for Ground floor.

33(A)

33(B)



33(C)

33(D)

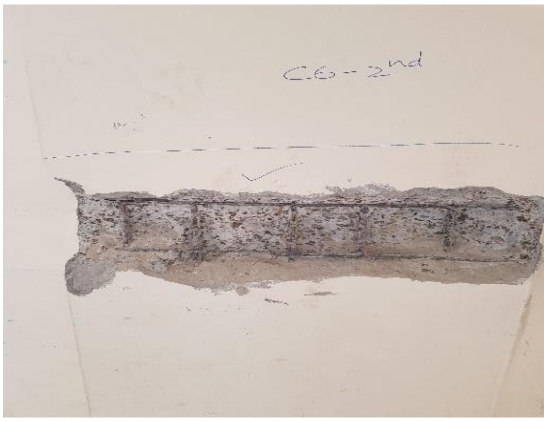
Figure 33 Reinforcement for columns in ground & first floor.(A.) column C1, (B.) column C3,(C.) column C3 middle, (D). Columns C6

The images shown below (figure 34) exhibit the chipping of the cover of the reinforced cement concrete column in the second floor in order to visually see the reinforcement provided in the columns

34(A)

34(B)

34(C)



34(D)

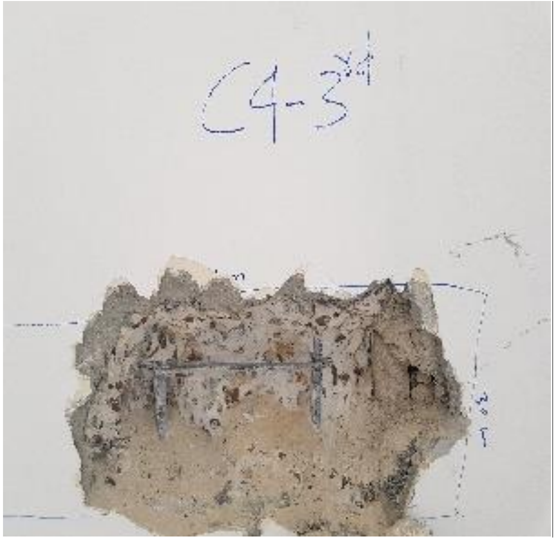
34(E)

Figure 34 Reinforcement for columns in second floor. (A.) column C1, (B.) column C3, (C.) column c5, (D.) Columns C6 and (E.) column C4

The images shown below (figure 35) exhibit the chipping of the cover of the reinforced cement concrete column in the third floor in order to visually see the reinforcement provided in the columns

35(A)

35(B)



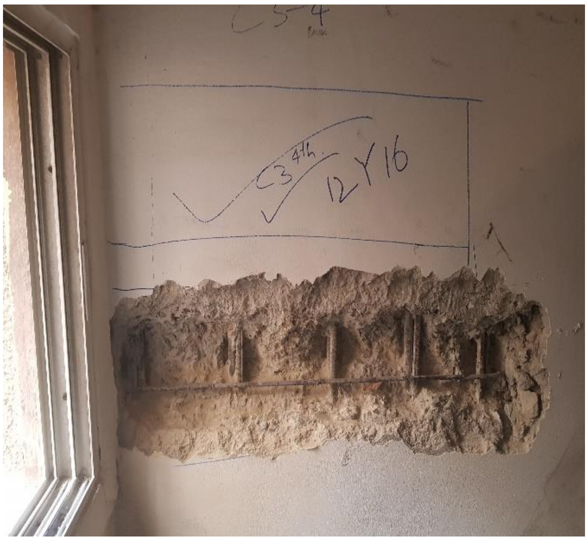
35(C)

35(D)

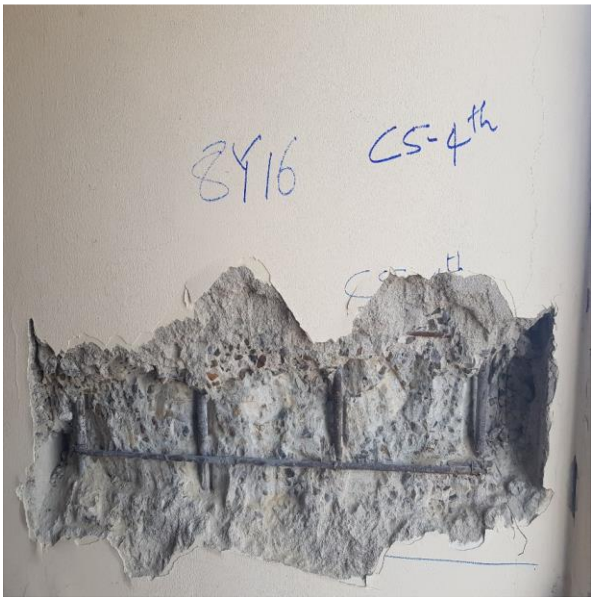
Figure 35 Reinforcement for columns in third floor. (A.) column C1, (B.) column C3, (C.) column C4, (D.) Columns C6

The images shown below (figure 36) exhibit the chipping of the cover of the reinforced cement concrete column in the fourth floor in order to visually see the reinforcement provided in the columns

36(A)



36(B)



36(C)

36(D))

Figure 36Reinforcement for columns in fourth floor. (A.) column C1, (B.) column C3, (C.) column C5, (D.) Columns C6

The images shown below (figure 37) exhibit the chipping of the cover of the reinforced cement concrete column in the fifth floor in order to visually see the reinforcement provided in the columns

37(A) 37(B)

Figure 37 Reinforcement for columns in fifth floor. (A.) column C1, (B.) column C3

The images shown below (figure 38) exhibit the chipping of the cover of the reinforced cement concrete column in the fifth floor in order to visually see the reinforcement provided in the columns

38(A) 38(B)

Figure 38 Reinforcement for columns in sixth floor. (A.) column C1, (B.) column C5

After investigating and collecting the data of columns from site the data was used to model the structure in Etabs.

6.1.2 MATERIAL DEFINATION

The behaviour of the building depends upon the materials used and their integrity. In the first model the materials were defined such that the structure has not encountered any deterioration. the modelling and the material definition is as per ACI 318 as its widely accepted by the local authorities in the United Arab Emirates.

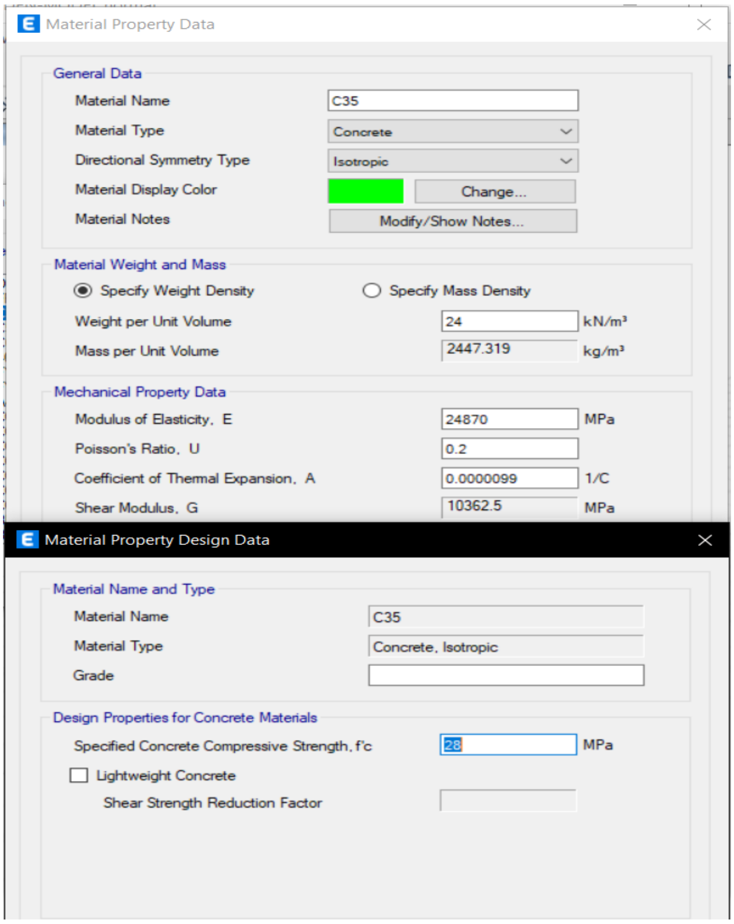


Figure 39 Material Property Definition for Non Deteriorated Model

The structure was modelled for C35 grade of concrete. The specified compressive strength of which is 0.8 times the compressive strength and the modulus of elasticity if 4700 times square root of the specified compressive strength. The figure 39 exhibits the property definition of the model

The second structure is modelled with respect to the data received from the core test reports and the parametric changes associated with it. The preliminary data on which the other material properties depend upon is the compressive strength. Further the parametric change in the material property related to compressive strength is the modulus of elasticity (E). The modulus of elasticity is related as 4700 times square root of fc'. Moreover, the other parametric change in the material property is the shear modulus (G). This is related to the modulus of elasticity with the equation (a).

$$E = 2G(1+\mu) \ldots\ldots\ldots\ldots \text{(a)}$$

Where in which
E = Young's Modulus Of Elasticity
G = Shear Modulus Of Elasticity
μ = Poisson's Ratio

The compressive strength and the parameters associated with it used to model the deteriorated model are exhibited in the table 56.

LEVEL	ELEMENT	COMPRESSIVE STRENGTH fc' (MPa)	MODULUS OF ELASTICITY E (MPa)	SHEAR MODULUS G (MPa)
GROUND	COLUMNS	15.3	18384	7660
	BEAMS	9.3	14333	5972.08
	SLABS	26.4	24149	10062.08
FIRST	COLUMNS	13.5	17269	7195.42
	BEAMS	19.9	20966	8735.83
	SLABS	32	26587	11077.92
SECOND	COLUMNS	11.7	16076	6698.33
	BEAMS	16.7	19207	8002.92
	SLABS	36.5	28395	11831.25
THIRD	COLUMNS	12.5	16617	6923.75
	BEAMS	18.9	20433	8513.75
	SLABS	30.1	25786	10744.17
FOURTH	COLUMNS	18.1	19996	8331.67
	BEAMS	16.3	18975	7906.25
	SLABS	36.8	26511	11879.58
FIFTH	COLUMNS	7.2	12611	5254.58
	BEAMS	16.1	18859	7857.92
	SLABS	36.7	28472	11863.33
SIXTH	COLUMNS	14.7	18384	7660
	BEAMS	18.1	19996	8331.67

	SLABS	26.9	24376	10156.67

Table 56: Material Property Parameters for Deteriorated Structure Model

On further observation of the data we can observe a pattern which indicates that the parameters fluctuate in a similar manner as the compressive strength with respect to other elements on different levels. Its further observed that slabs are least effected due to deterioration and columns on an overall are the most effected due to deterioration. This is observed from the figures 40 & 41. Furthermore, these figure exhibit graphically the data input in the structural modelling

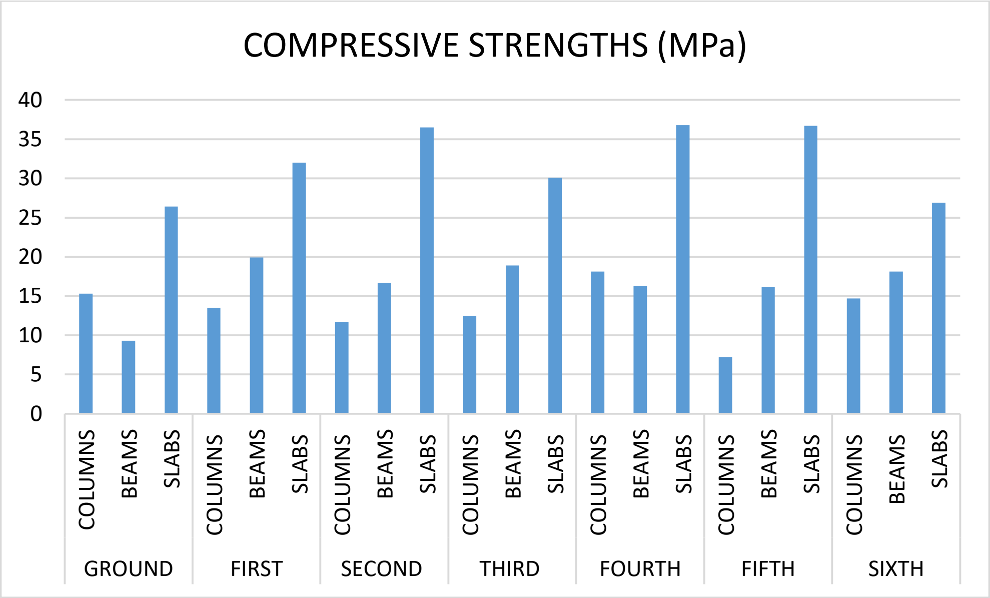


Figure 40: Compressive Strength used for structural modelling

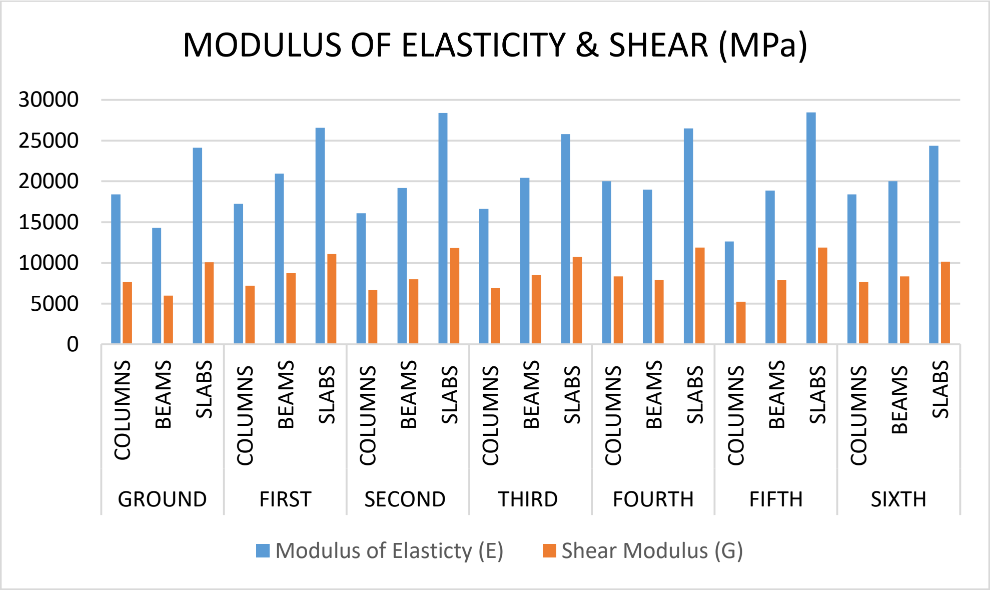
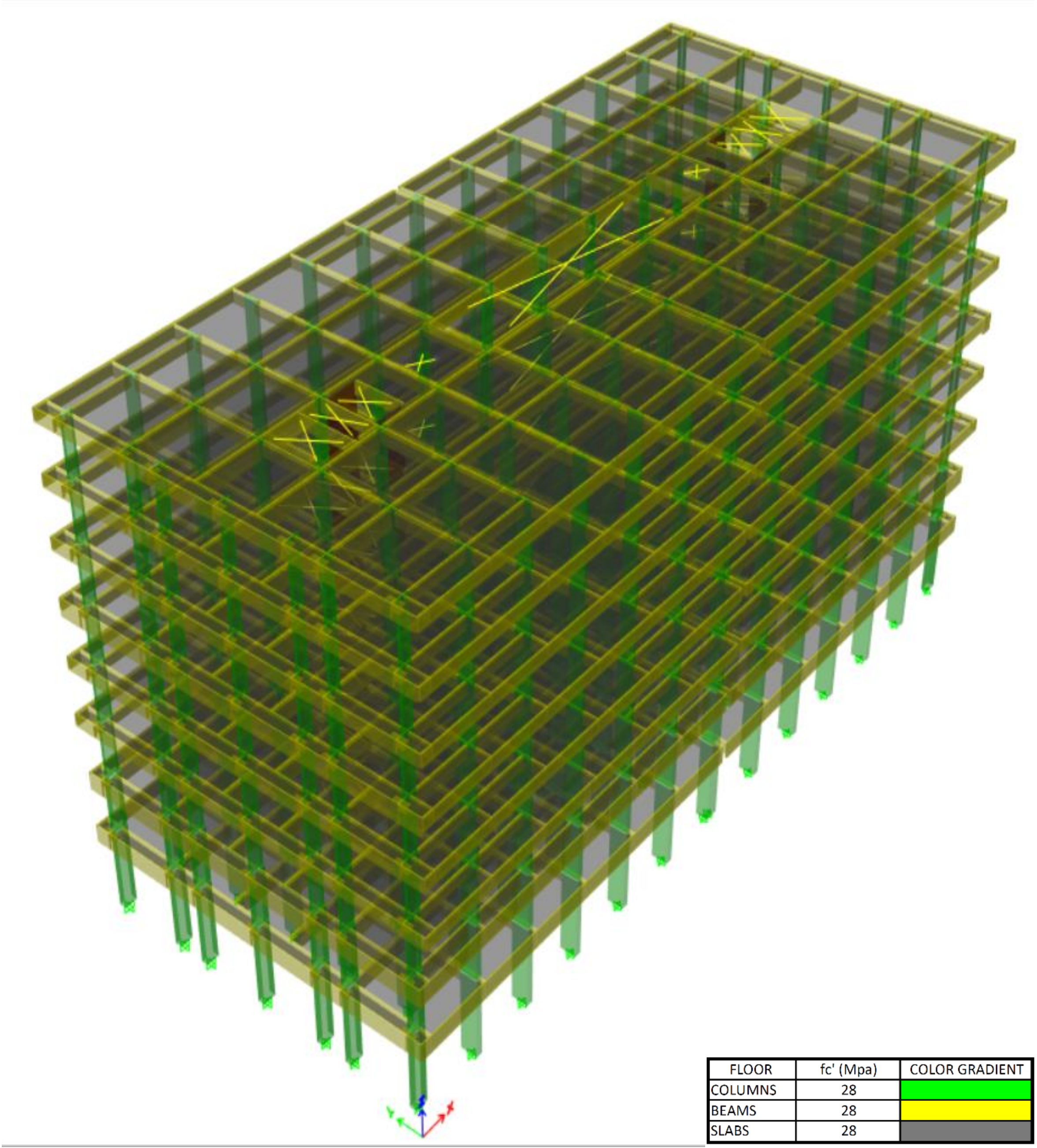


Figure 41: Modulus Of Elasticity & Shear Used for structural Modelling

6.1.3 STRUCTURE MODELLING

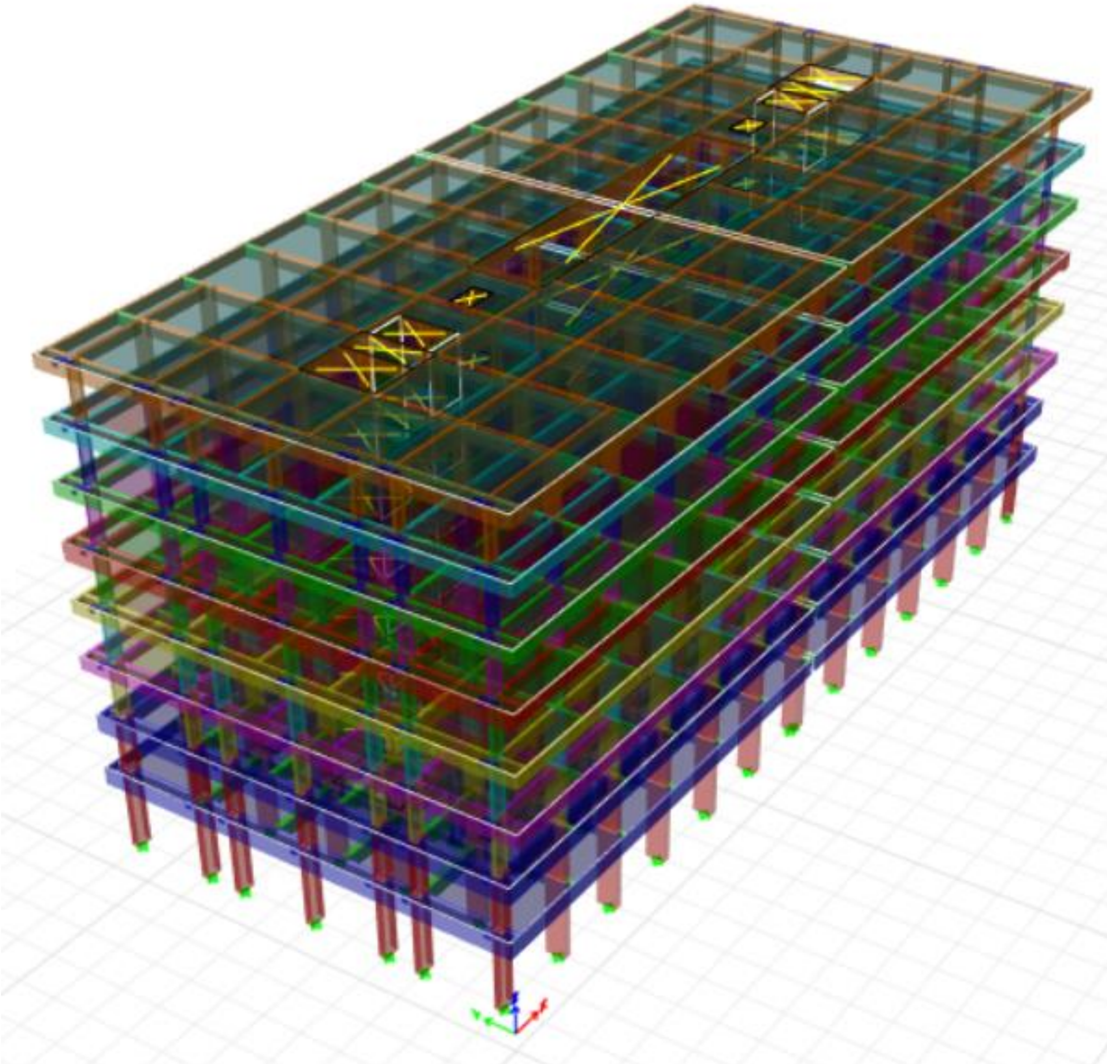
Two models are made where in which the one model is made with parameters such that the reinforced cement concrete has not been corroded. The figure 42 exhibits the structure modelled with full strength parameters.



FLOOR	fc' (Mpa)	COLOR GRADIENT
COLUMNS	28	
BEAMS	28	
SLABS	28	

Figure 42 Structural Model – Non deteriorated Structure

Further with the acquired data from site conditions and the defined properties mentioned previously the second structure is modelled exhibited in figure 43. Each element of the floor is defined with their respective core test values properties associated with it, as indicated.



COLUMNS		
FLOOR	fc' (Mpa)	COLOR GRADIENT
FIFTH	7.2	
SECOND	11.7	
THIRD	12.5	
FIRST	13.5	
SIXTH	14.7	
GROUND	15.3	
FOURTH	18.1	

BEAMS		
FLOOR	fc' (Mpa)	COLOR GRADIENT
GROUND	26.4	
SIXTH	26.9	
THIRD	30.1	
FIRST	32	
SECOND	36.5	
FIFTH	36.7	
FOURTH	36.8	

BEAMS		
FLOOR	fc' (Mpa)	COLOR GRADIENT
GROUND	9.3	
FIFTH	16.1	
FOURTH	16.3	
SECOND	16.7	
SIXTH	18.1	
THIRD	18.9	
FIRST	19.9	

Figure 43 Structural Model – Deteriorated Structure

The purpose of having two models is to have a datum to compare the deteriorated model with. this helps us to study the difference in design forces and the required area of steel along with the capacity ratios for the deteriorated structure.

6.1.4 LOAD APPLICATION

The Load applied for all the slabs are uniform and have been kept same to simply calculations. The slab loading is as per the figure 44.

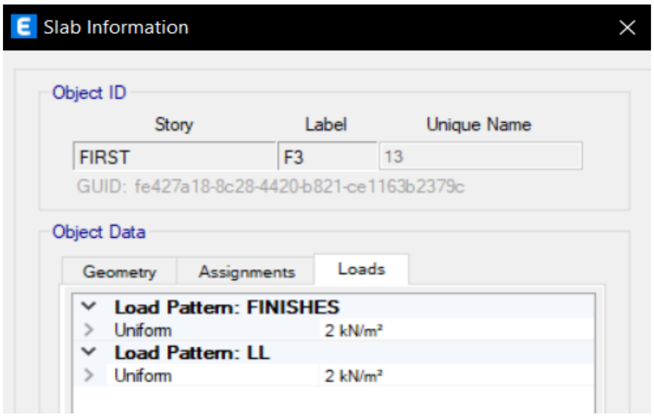


Figure 44 Slab Loading Data

Further the walls loads are applied as per architectural layering and the wall load depends upon the width of the wall and the height of the respective storey. For 20cm wide walls the applied load is 10Kn/sq. and for the 10cm wide wall the applied load is 7Kn/Sq. The wall loads application can be seen in the figure 45.

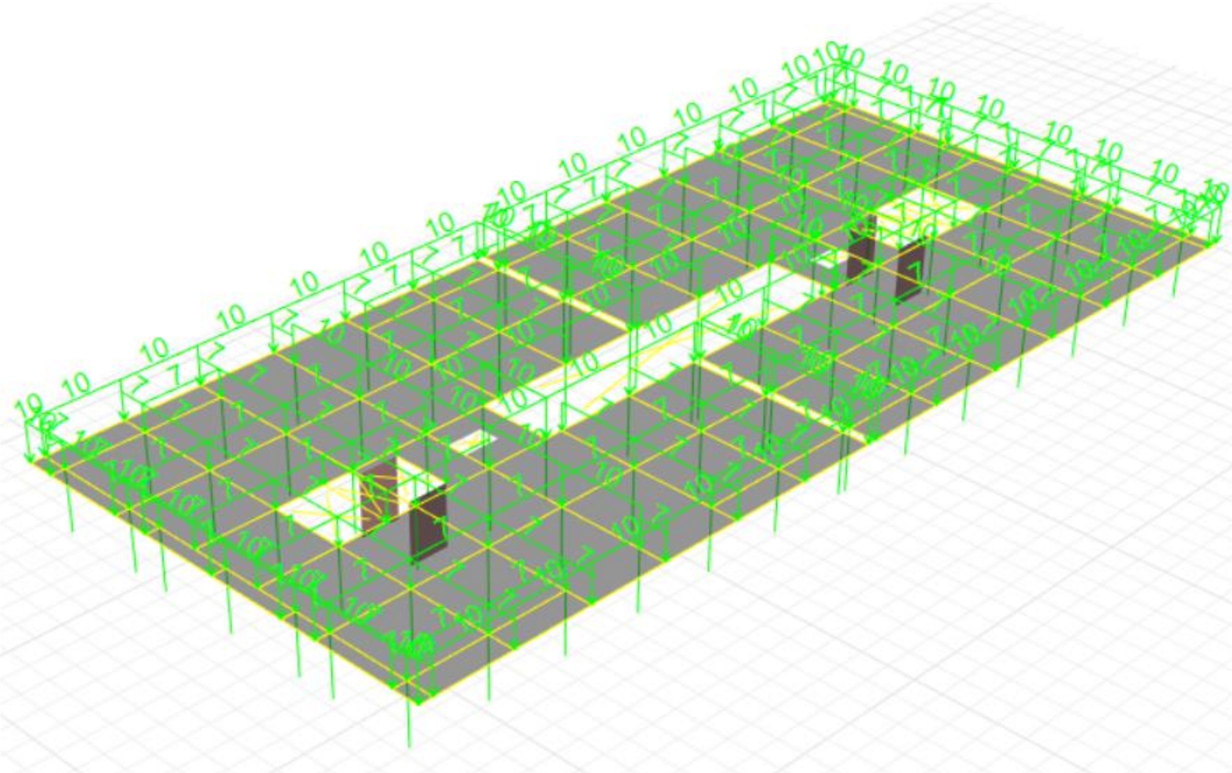


Figure 45 Loading Diagram of Beams.

6.1.5 MODAL ANALYSIS

The two models were analysed and the design forces were extracted to compare for any difference. The analysis is made for the models with the same loading criteria as mentioned, in the previous model with the only difference of material properties acquired from core test results. The figures 46-47 exhibit the retrieved design forces for columns.

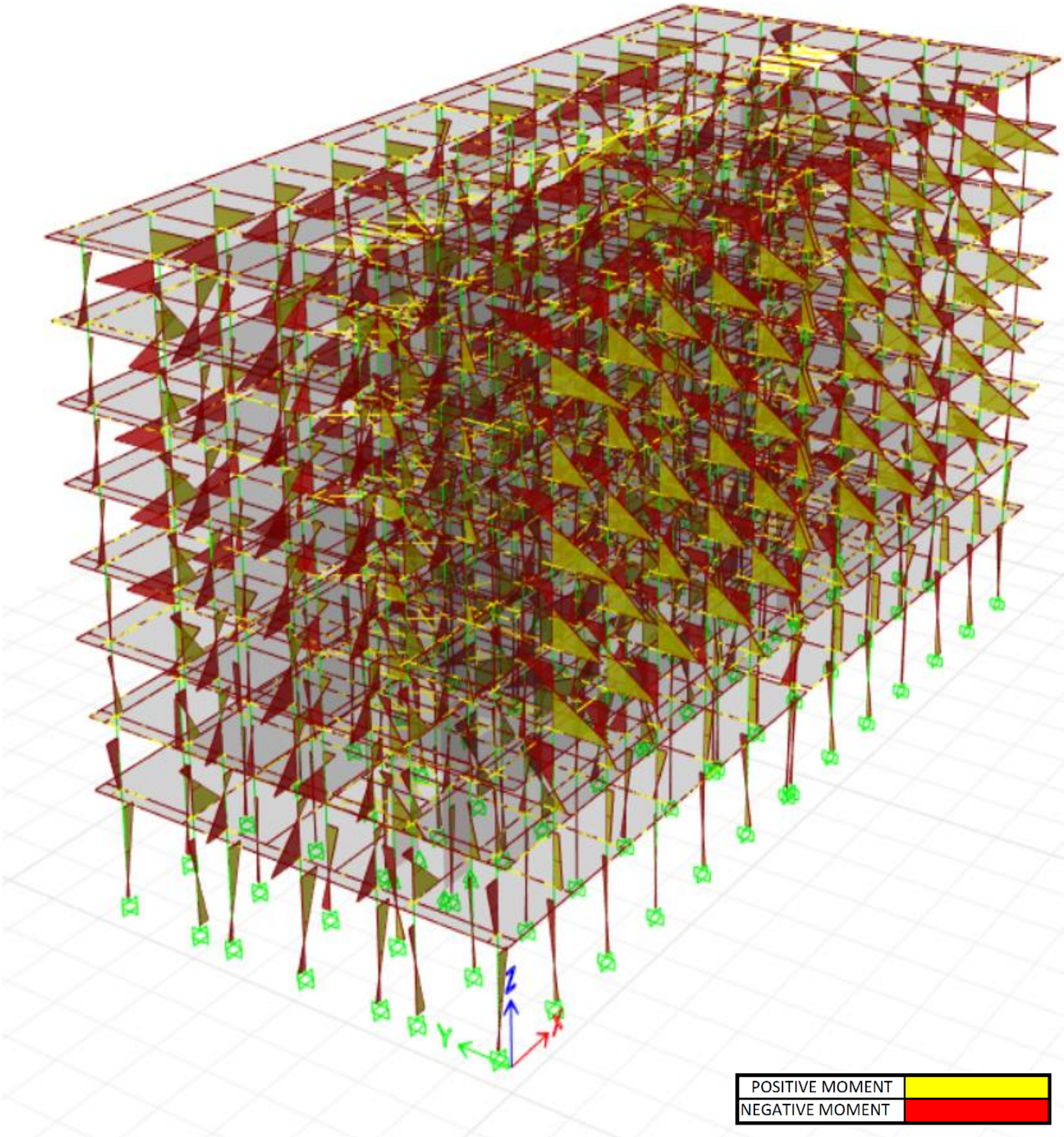


Figure 46 Moments Diagram for Columns.

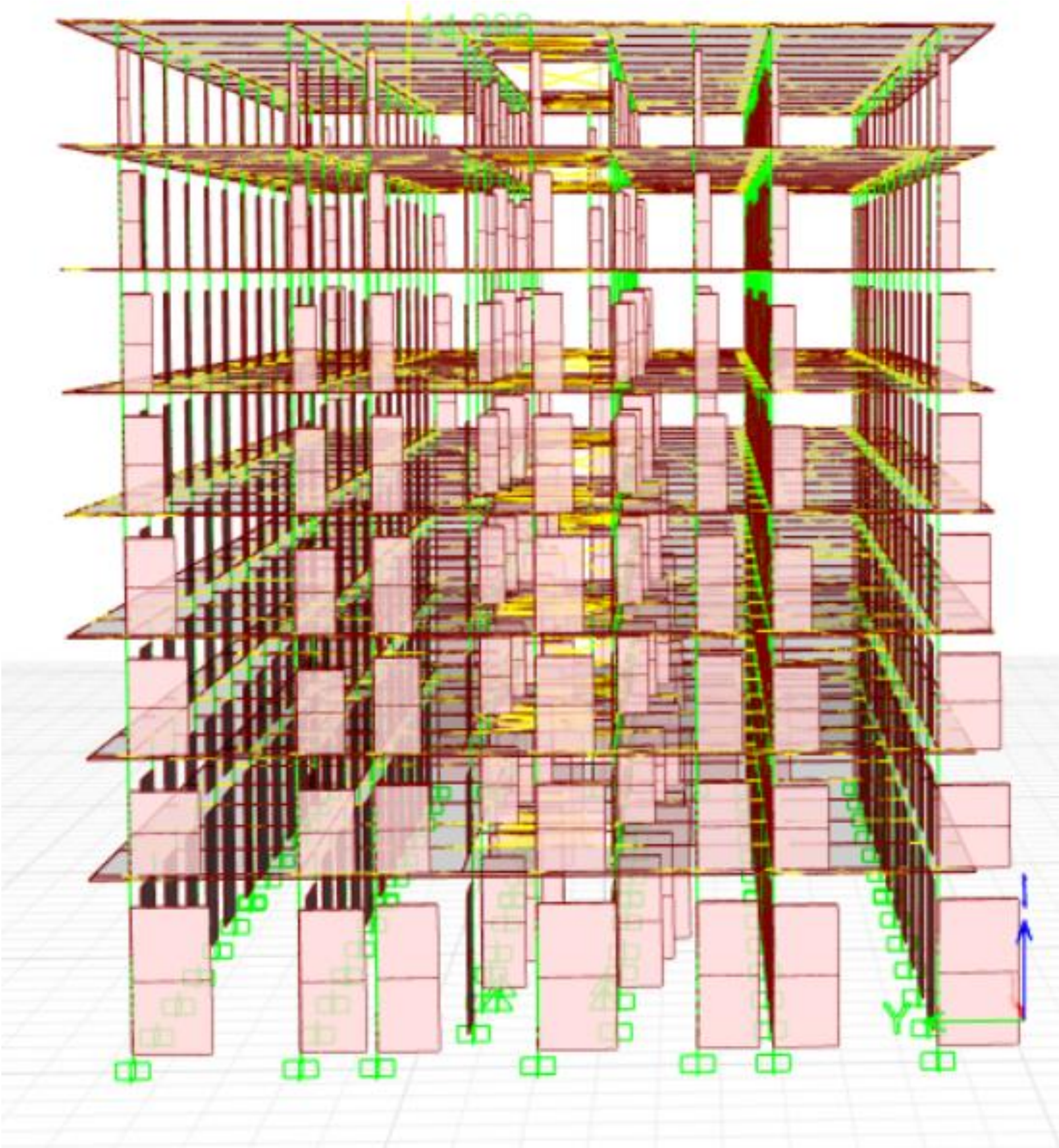
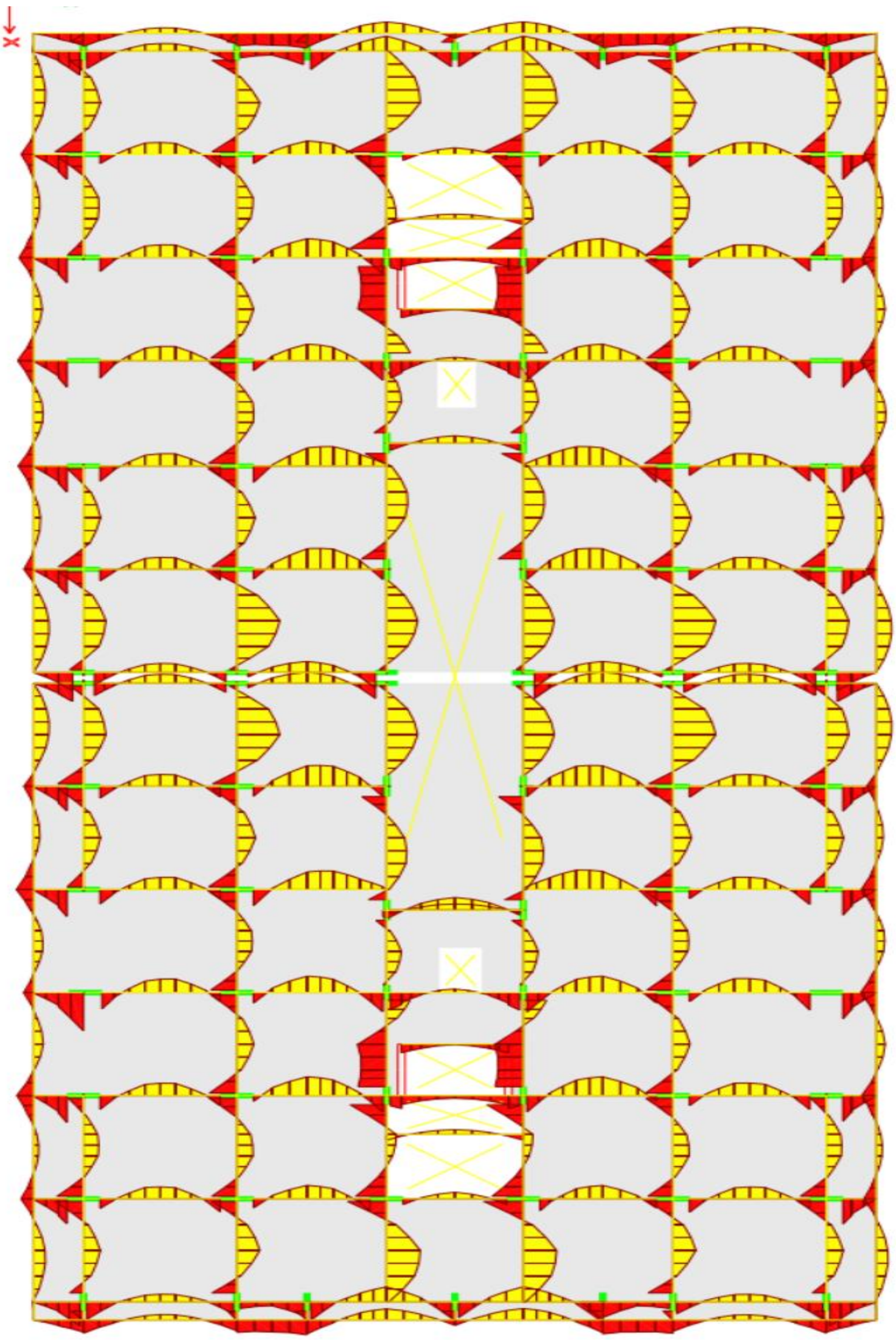


Figure 47 Axial Force Diagram for Columns.

Upon observing the Shear forces and the Bending moment for columns for both the models, the columns can be classified into three categories. The first being corner columns, these are the column that has only two faces connected to the slab. Second are the intermediate columns, these columns have three faces restrained by beams or slabs and have one face free. The third type of columns can be classified as middle columns where in which all four faces of the columns are restrained by beams or slabs. Similarly, the forces were taken out for beams in the structure.



POSITIVE MOMENT	
NEGATIVE MOMENT	

Figure 48 Bending Moment Diagram for Beams.

The above figure 48 shows the bending moment diagrams obtained for the beams on analysing the structure. It can be observed that the bending moment is comparatively less in the edge beams as compared to the middle beams.

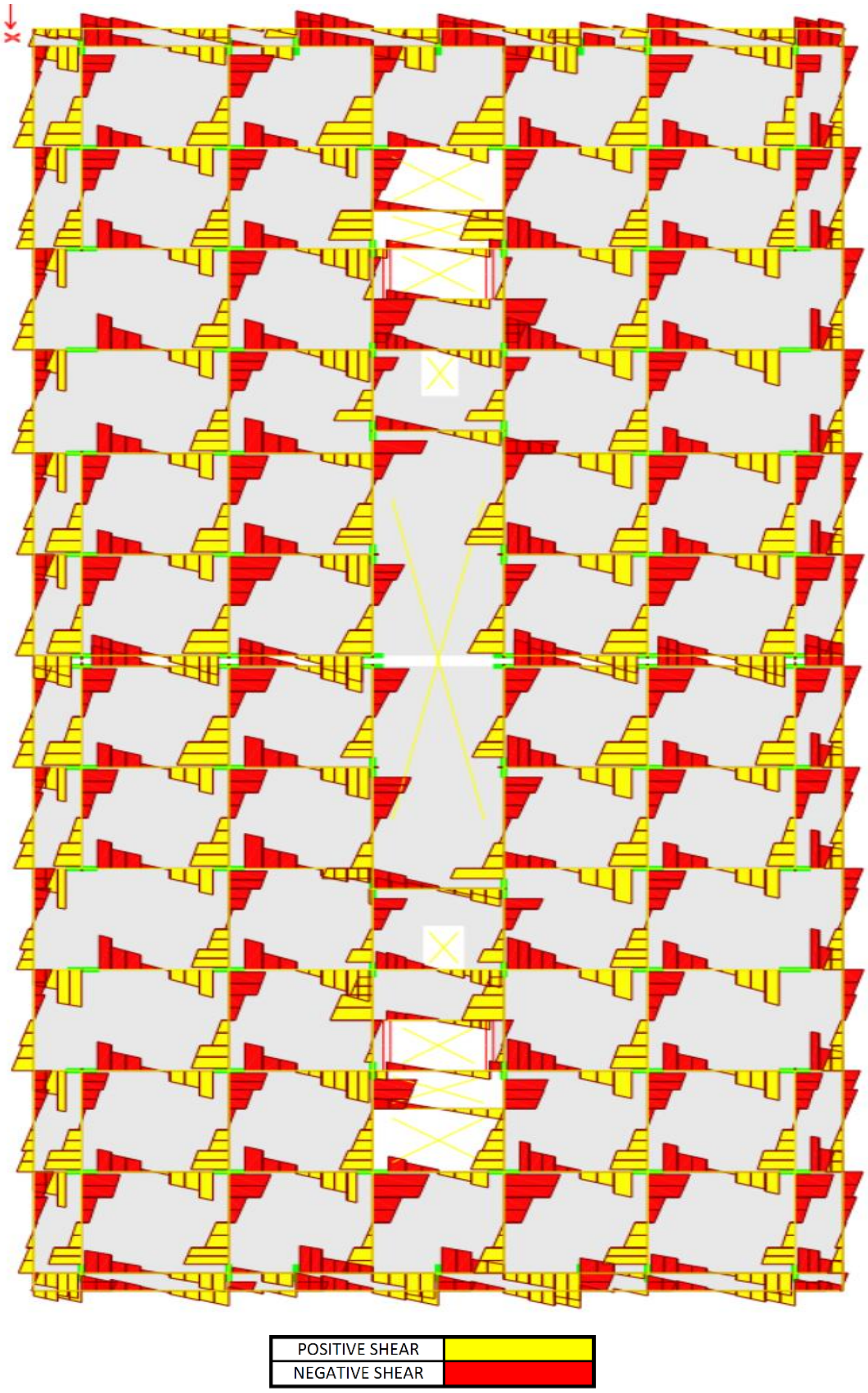


Figure 49 Shear force Diagram for Beams.

The figure 49 exhibits the shear forces attained after analysing the structure. A similar observation is made in beams shear force. The edge beams have lesser shear force values in comparison to middle beams. The analysis of the slabs was overlooked as observed the statistical analysis that the deterioration is less in slabs compared to beams and columns.

6.1.6 DESIGN FORCES

On the basis of the observations made the columns and beams were classified in categories. For the columns they have been categories as corner columns, intermediate columns and middle corners. This classification was done on the basis of its location and the number of faces of columns constrained in each direction. Similarly, the beams were classified into two categories as edge beams and middle beams. This classification was on the basis of the location and number of face of the beam that is constrained and free.

6.1.6.1 COLUMNS

With the acquired design forces and the classification made the design forces of the two models were compared for maximum values of an element in the classification at each storey.

Design Forces - Corner Columns										
Story	Column	UniqueNam	Combo	Station	P	V2	V3	T	M2	M3
				m	kN	kN	kN	kN-m	kN-m	kN-m
SIXTH	C1	1486	U2-1	0	-243.246	-4.7498	-4.2781	0.0207	-5.4469	-6.9489
SIXTH	C1	1486	U2-1	2.4	-234.952	-4.7498	-4.2781	0.0207	4.822	4.5103
SIXTH	C1	1486	U2-1	0	-244.637	-4.7085	-4.1426	0.0203	-5.0002	-6.8092
SIXTH	C1	1486	U2-1	2.4	-236.342	-4.7085	-4.1426	0.0203	4.9318	4.5367
FIFTH	C1	1487	U2-1	0	-484.974	-2.7934	-2.2386	0.0008	-3.1889	-4.1274
FIFTH	C1	1487	U2-1	2.4	-476.68	-2.7934	-2.2386	0.0008	2.2031	2.6789
FIFTH	C1	1487	U2-1	0	-487.257	-4.0177	-2.5532	0.0039	-3.7073	-5.97
FIFTH	C1	1487	U2-1	2.4	-478.963	-4.0177	-2.5532	0.0039	2.4032	3.7441
FOURTH	C1	1488	U2-1	0	-728.765	-4.6588	-2.7592	0.0126	-3.9911	-6.9001
FOURTH	C1	1488	U2-1	2.4	-719.088	-4.6588	-2.7592	0.0126	2.6253	4.4168
FOURTH	C1	1488	U2-1	0	-731.775	-4.3521	-2.8856	0.009	-4.0779	-6.4669
FOURTH	C1	1488	U2-1	2.4	-722.098	-4.3521	-2.8856	0.009	2.8158	4.0776
THIRD	C1	1489	U2-1	0	-969.935	-3.4952	-2.7122	0.0043	-4.1449	-5.1935
THIRD	C1	1489	U2-1	2.4	-960.258	-3.4952	-2.7122	0.0043	2.3691	3.3588
THIRD	C1	1489	U2-1	0	-974.162	-3.9438	-3.0309	0.005	-4.1704	-5.8208
THIRD	C1	1489	U2-1	2.4	-964.485	-3.9438	-3.0309	0.005	3.084	3.7686
SECOND	C1	1490	U2-1	0	-1216.51	-3.4759	-0.8376	0.0081	0.06	-5.148
SECOND	C1	1490	U2-1	2.4	-1205.45	-3.4759	-0.8376	0.0081	2.0596	3.3728
SECOND	C1	1490	U2-1	0	-1221.01	-4.0913	-1.0739	0.0109	0.3826	-6.0965
SECOND	C1	1490	U2-1	2.4	-1209.95	-4.0913	-1.0739	0.0109	2.9549	3.8509
FIRST	C1	1491	U2-1	0	-1484.16	-1.4885	2.5635	0.0005	3.948	-1.362
FIRST	C1	1491	U2-1	2.4	-1473.1	-1.4885	2.5635	0.0005	-2.2643	2.4496
FIRST	C1	1491	U2-1	0	-1488.5	-1.7885	1.7422	-0.0003	3.2186	-1.7032
FIRST	C1	1491	U2-1	2.4	-1477.44	-1.7885	1.7422	-0.0003	-0.9504	2.7732
MEZZ	C1	1485	U2-1	0	-1731.57	3.4301	2.4375	-0.0228	-1.6546	4.6522
MEZZ	C1	1485	U2-1	2.1	-1712.22	3.4301	2.4375	-0.0228	-6.9489	-2.5745
MEZZ	C1	1485	U2-1	0	-1735.49	2.1759	2.3448	-0.0305	-2.0052	2.8852
MEZZ	C1	1485	U2-1	2.1	-1716.13	2.1759	2.3448	-0.0305	-7.0105	-1.6949
GROUND	C1	2087	U2-1	0	-1969.75	0.5774	-2.0986	0.0029	-1.3785	0.8611
GROUND	C1	2087	U2-1	3.7	-1935.65	0.5774	-2.0986	0.0029	6.3252	-1.3065
GROUND	C1	2087	U2-1	0	-1974.02	0.3374	-2.1064	0.004	-2.33	0.4899
GROUND	C1	2087	U2-1	3.7	-1939.92	0.3374	-2.1064	0.004	5.4571	-0.7702

	Non deteriorated Structure
	Deteriorated Structure

Table 57: Design forces Corner Column C1

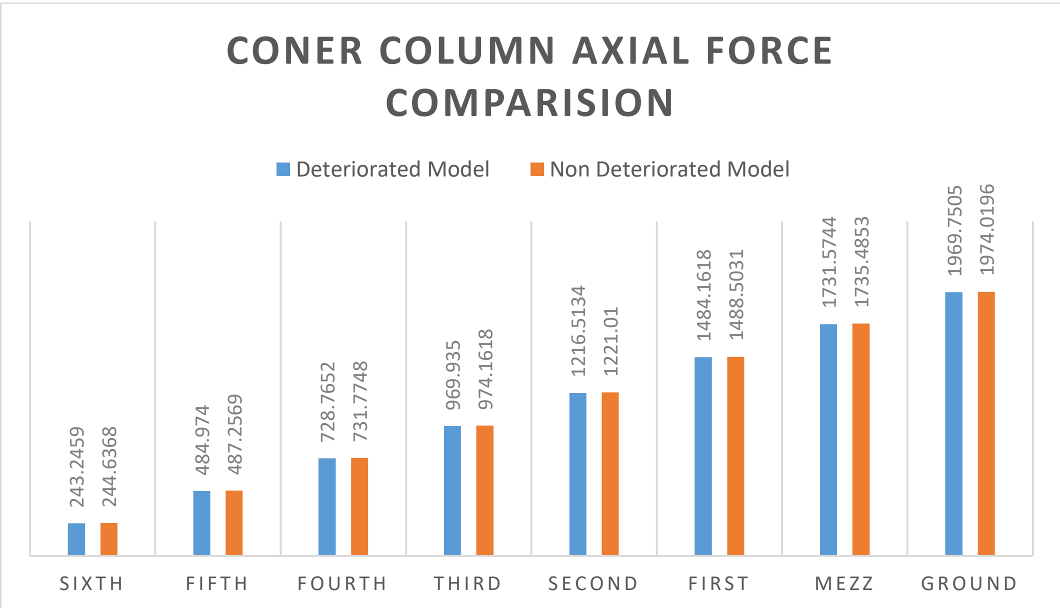


Figure 50 Corner Column Axial Forces Comparison

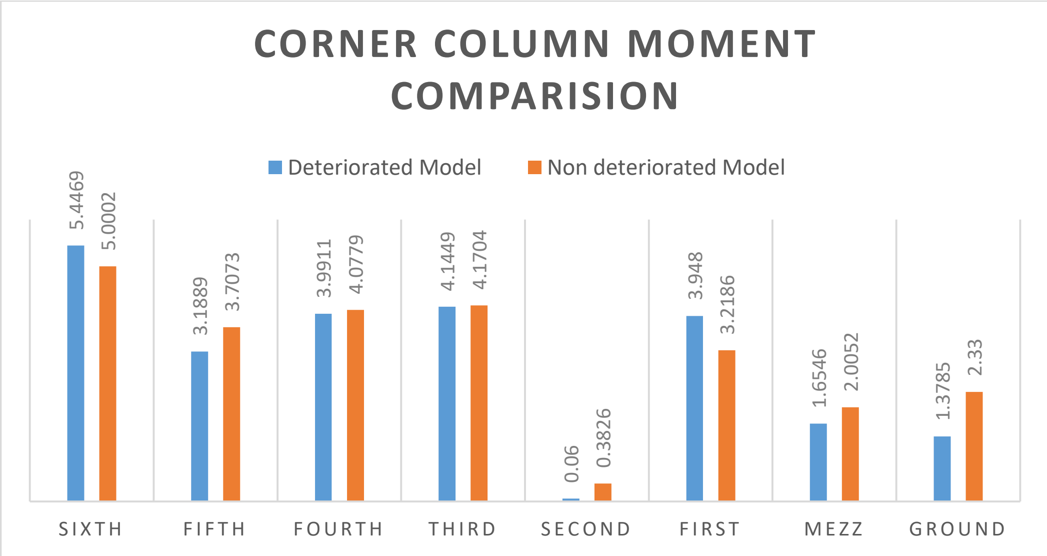


Figure 51 Corner Column Moment Comparison

The Table 57 exhibits the design forces for the two models made for the corner column. It has been observed from the figure 50 that there is not much significant difference in the axial forces. Whereas from the figure 51 its can be observed that there is a significant difference in the moments acquired. in most cases the deteriorated model has lower moment in comparison to the non-deteriorated model.

Design Forces - Intermediate Columns										
Story	Column	UniqueName	Combo	Station	P	V2	V3	T	M2	M3
				m	kN	kN	kN	kN-m	kN-m	kN-m
SIXTH	C69	1920	U2-1	0	-268.283	-1.2127	26.3102	0.0218	34.6255	-1.7764
SIXTH	C69	1920	U2-1	2.4	-257.224	-1.2127	26.3102	0.0218	-28.5397	1.1402
SIXTH	C69	1920	U2-1	0	-267.978	-0.7169	24.0459	0.022	29.9594	-1.0388
SIXTH	C69	1920	U2-1	2.4	-256.919	-0.7169	24.0459	0.022	-27.767	0.6706
FIFTH	C69	1921	U2-1	0	-569.743	-0.8396	12.3532	-0.0004	18.2626	-1.2409
FIFTH	C69	1921	U2-1	2.4	-558.684	-0.8396	12.3532	-0.0004	-11.4159	0.8179
FIFTH	C69	1921	U2-1	0	-568.048	-0.7312	14.6029	0.0032	22.1645	-1.0471
FIFTH	C69	1921	U2-1	2.4	-556.989	-0.7312	14.6029	0.0032	-12.9012	0.6988
FOURTH	C69	1922	U2-1	0	-871.511	-1.1178	21.2433	0.0085	29.9143	-1.6353
FOURTH	C69	1922	U2-1	2.4	-859.069	-1.1178	21.2433	0.0085	-21.1015	1.0734
FOURTH	C69	1922	U2-1	0	-868.61	-0.6252	18.2933	0.0066	26.5261	-0.8783
FOURTH	C69	1922	U2-1	2.4	-856.169	-0.6252	18.2933	0.0066	-17.4011	0.6004
THIRD	C69	1923	U2-1	0	-1172.52	-0.7698	15.4403	0.0024	23.2289	-1.1074
THIRD	C69	1923	U2-1	2.4	-1160.08	-0.7698	15.4403	0.0024	-13.8654	0.787
THIRD	C69	1923	U2-1	0	-1169.12	-0.5472	15.624	0.0041	22.6153	-0.7492
THIRD	C69	1923	U2-1	2.4	-1156.68	-0.5472	15.624	0.0041	-14.9137	0.559
SECOND	C69	1924	U2-1	0	-1474.34	-0.4443	18.8094	0.0066	28.0016	-0.5866
SECOND	C69	1924	U2-1	2.4	-1460.52	-0.4443	18.8094	0.0066	-17.1756	0.49
SECOND	C69	1924	U2-1	0	-1469.97	-0.3444	18.5029	0.0099	28.1569	-0.4233
SECOND	C69	1924	U2-1	2.4	-1456.14	-0.3444	18.5029	0.0099	-16.2721	0.4039
FIRST	C69	1925	U2-1	0	-1777.04	-0.2799	8.9109	0.004	6.2454	-0.4134
FIRST	C69	1925	U2-1	2.4	-1763.21	-0.2799	8.9109	0.004	-15.2168	0.1922
FIRST	C69	1925	U2-1	0	-1771.68	-0.369	8.457	0.0045	5.5047	-0.6124
FIRST	C69	1925	U2-1	2.4	-1757.86	-0.369	8.457	0.0045	-14.85	0.2827
MEZZ	C69	1919	U2-1	0	-2063.81	0.0826	0.908	0.0201	4.4649	0.5482
MEZZ	C69	1919	U2-1	2.1	-2039.62	0.0826	0.908	0.0201	2.534	0.2025
MEZZ	C69	1919	U2-1	0	-2057.81	-0.2627	0.1324	0.0216	3.3257	0.2428
MEZZ	C69	1919	U2-1	2.1	-2033.62	-0.2627	0.1324	0.0216	3.0266	0.7287
GROUND	C69	2149	U2-1	0	-2323.1	-0.0363	1.334	0.0188	2.6902	0.5555
GROUND	C69	2149	U2-1	3.9	-2278.18	-0.0363	1.334	0.0188	-2.5511	0.5644
GROUND	C69	2149	U2-1	0	-2315.45	0.0271	1.0107	0.0212	2.2391	0.3424
GROUND	C69	2149	U2-1	3.9	-2270.52	0.0271	1.0107	0.0212	-1.7297	0.1864

	Non deteriorated Structure
	Deteriorated Structure

Table 58 Design forces Intermediate Column C69

The table 58 exhibits the design forces for the two models made for the intermediate column. It has been observed that there is a difference in the design force values. On calculating the difference, it was observed that it is lesser than 1% in axial forces and no difference in the shear force values thereby considered insignificant. For design purposes the value higher amongst the two have been considered for evaluation of columns in the intermediate location.

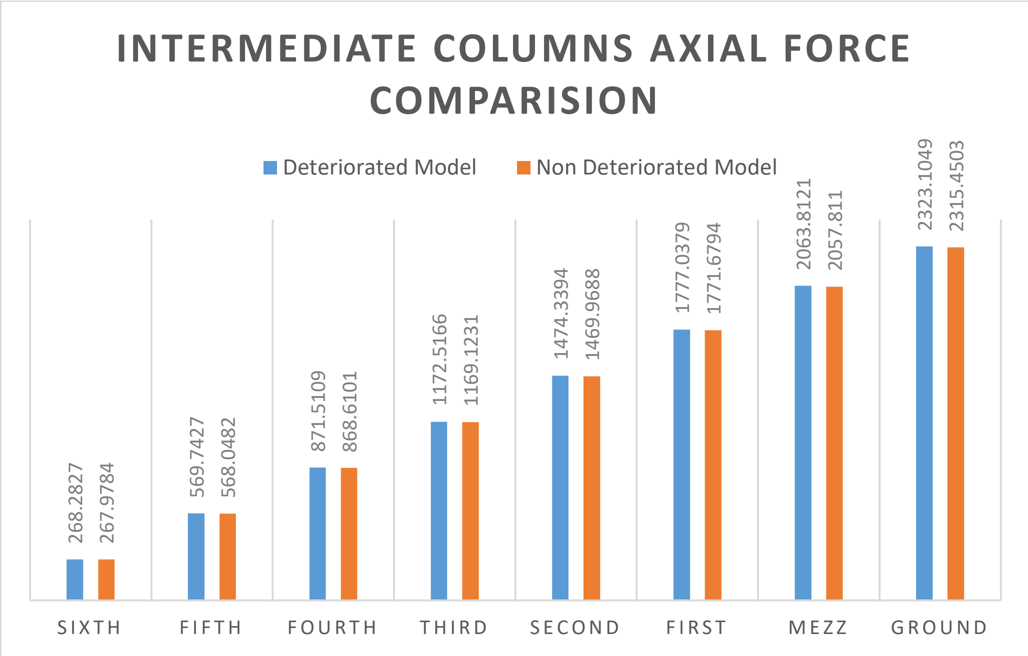


Figure 52 Intermediate Column Axial Forces Comparison

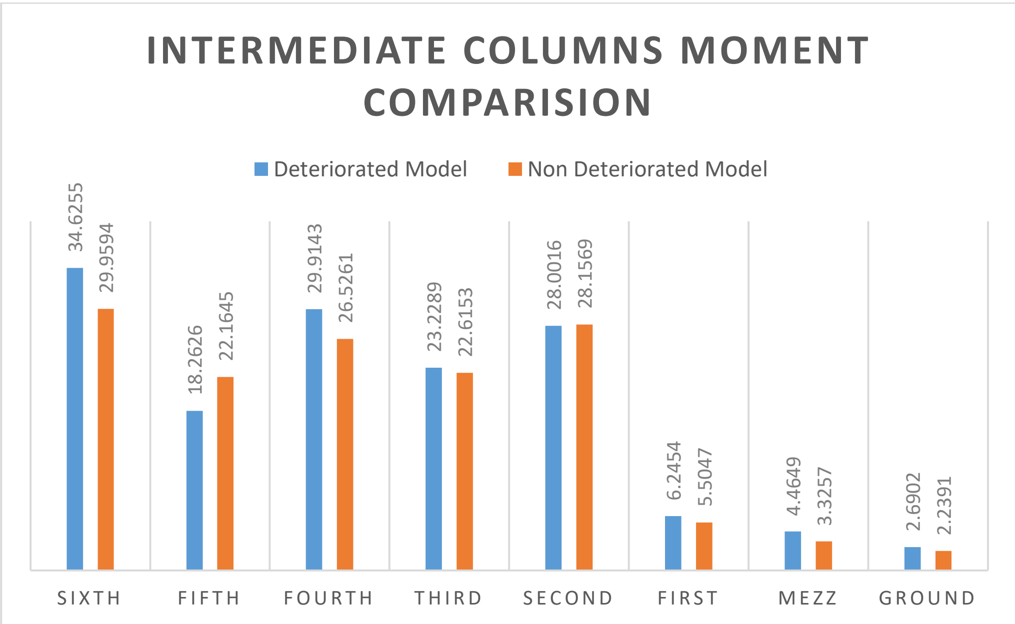


Figure 53 Intermediate Column Moment Comparison

The figure 52 shows the comparative axial forces for the deteriorated and non-deteriorated model. It can be observed that there is no significant difference in the axial forces of both the models. Although figure 53 helps to understand that there is a difference in moment where in which the moments in the deteriorated model are more with respect to non-deteriorated model.

Design Forces - Middle Columns

Story	Column	UniqueNam	Combo	Station	P	V2	V3	T	M2	M3
				m	kN	kN	kN	kN-m	kN-m	kN-m
SIXTH	C82	2011	U2-1	0	-228.964	-2.2784	6.6254	-0.0416	9.3291	-3.3457
SIXTH	C82	2011	U2-1	2.4	-220.67	-2.2784	6.6254	-0.0416	-6.5668	2.149
SIXTH	C82	2011	U2-1	0	-229.663	-2.6912	5.7838	-0.0418	7.8276	-3.8877
SIXTH	C82	2011	U2-1	2.4	-221.368	-2.6912	5.7838	-0.0418	-6.0611	2.5947
FIFTH	C82	2012	U2-1	0	-454.576	-1.2152	3.5643	0.0161	4.886	-1.74
FIFTH	C82	2012	U2-1	2.4	-446.282	-1.2152	3.5643	0.0161	-3.6713	1.213
FIFTH	C82	2012	U2-1	0	-455.609	-2.2489	4.4985	0.0108	6.39	-3.3386
FIFTH	C82	2012	U2-1	2.4	-447.315	-2.2489	4.4985	0.0108	-4.4296	2.0894
FOURTH	C82	2013	U2-1	0	-680.353	-2.199	8.0666	-0.0099	11.1001	-3.2732
FOURTH	C82	2013	U2-1	2.4	-669.294	-2.199	8.0666	-0.0099	-8.2576	2.051
FOURTH	C82	2013	U2-1	0	-680.814	-2.8278	7.4467	0.0009	10.7755	-4.2036
FOURTH	C82	2013	U2-1	2.4	-669.754	-2.8278	7.4467	0.0009	-7.1236	2.6276
THIRD	C82	2014	U2-1	0	-905.669	-1.8661	5.8701	0.0114	8.8606	-2.8593
THIRD	C82	2014	U2-1	2.4	-894.61	-1.8661	5.8701	0.0114	-5.2265	1.6678
THIRD	C82	2014	U2-1	0	-905.957	-2.6435	6.1153	0.0091	9.4083	-3.9269
THIRD	C82	2014	U2-1	2.4	-894.898	-2.6435	6.1153	0.0091	-5.2892	2.4719
SECOND	C82	2015	U2-1	0	-1130.21	-2.4127	8.067	-0.0061	10.0281	-3.616
SECOND	C82	2015	U2-1	2.4	-1116.39	-2.4127	8.067	-0.0061	-9.3471	2.2534
SECOND	C82	2015	U2-1	0	-1130.85	-3.2964	7.6993	-0.0112	10.6623	-4.9833
SECOND	C82	2015	U2-1	2.4	-1117.02	-3.2964	7.6993	-0.0112	-7.8209	2.9939
FIRST	C82	2016	U2-1	0	-1352.43	-1.9133	12.573	0.0199	20.8544	-2.5814
FIRST	C82	2016	U2-1	2.4	-1338.61	-1.9133	12.573	0.0199	-9.3825	2.1094
FIRST	C82	2016	U2-1	0	-1353.35	-2.5077	11.6535	0.026	20.7194	-3.3727
FIRST	C82	2016	U2-1	2.4	-1339.53	-2.5077	11.6535	0.026	-7.2469	2.7348
MEZZ	C82	2010	U2-1	0	-1635.31	-4.6837	33.5065	-0.0327	50.8097	-7.0612
MEZZ	C82	2010	U2-1	2.4	-1614.58	-4.6837	33.5065	-0.0327	-29.7776	4.2284
MEZZ	C82	2010	U2-1	0	-1633.86	-5.5236	32.5325	-0.0395	48.8051	-8.2194
MEZZ	C82	2010	U2-1	2.4	-1613.12	-5.5236	32.5325	-0.0395	-29.3459	5.0821
GROUND	C82	2162	U2-1	0	-1931.09	-1.0692	7.6376	0.0078	13.1485	-1.6738
GROUND	C82	2162	U2-1	3.9	-1897.4	-1.0692	7.6376	0.0078	-16.793	2.5997
GROUND	C82	2162	U2-1	0	-1927	-1.2185	7.461	0.0109	11.5726	-1.8866
GROUND	C82	2162	U2-1	3.9	-1893.31	-1.2185	7.461	0.0109	-17.5995	2.952

	Non deteriorated Structure
	Deteriorated Structure

Table 59 Design forces Middle Column C82

The Table 59 exhibits the design forces for the two models made for the middle column. It has been observed that there is a difference in the design force values, although very minor and can be considered negligible for axial forces. For design purposes the value higher amongst the two have been considered for evaluation of columns in the intermediate location.

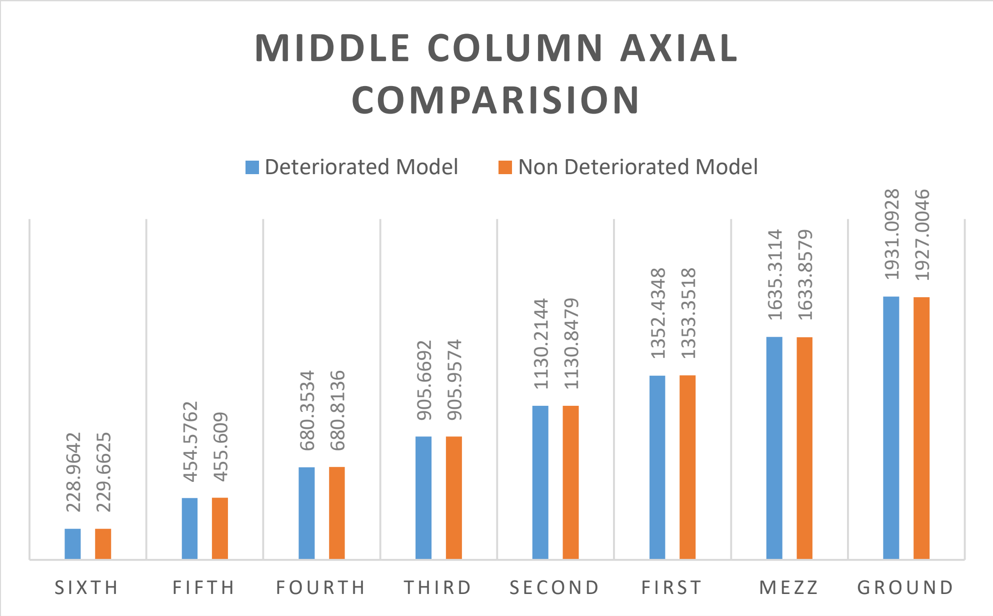


Figure 54 Middle Column Axial Forces Comparison

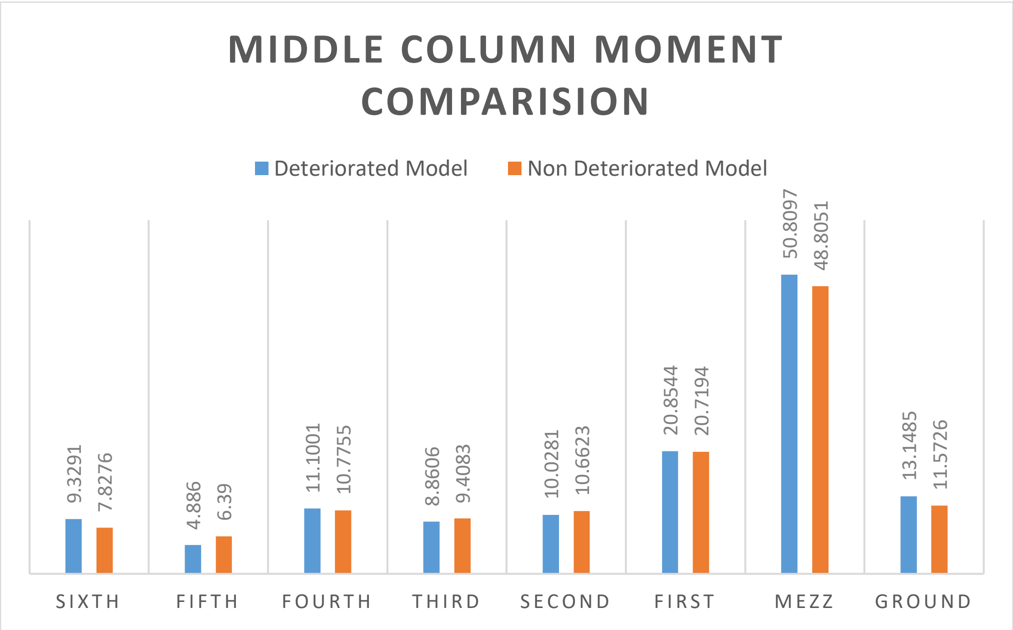


Figure 55 Middle Column Moment Comparison

The figure 54 exhibits the axial forces for the two models made for the middle column. It has been observed that there is a difference in the design force values, although very minor and can be considered negligible for axial forces. Similarly, there is a very minor difference in the design moment values which is exhibited in the figure 55. As there is a very minor difference it can be neglected as it does not have a major impact on the design.

6.1.6.2 BEAMS

This part of the study exhibits the design forces acquired from the two Etabs model for deteriorated and non-deteriorated structure. The study focussed on comparing the design forces for the two different categories of beam namely the edge beam and the middle beam for the highest design force values.

Story	Beam	UniqueNam	Combo	P	V2	V3	T	M2	M3 TOP	M3 BOT
				kN	kN	kN	kN-m	kN-m	kN-m	kN-m
SIXTH	B343	513	U2-1	-3.4769	30.0158	0.1251	0.0003	0.053	20.0505	17.5303
SIXTH	B343	513	U2-1	-3.8094	30.325	0.1587	0.0003	0.0681	19.5371	17.1381
FIFTH	B343	513	U2-1	-1.2148	30.702	0.0531	0.0003	0.0212	20.9552	17.6253
FIFTH	B343	513	U2-1	-1.7634	31.596	0.0622	0.0003	0.0242	21.4096	17.3364
FOURTH	B343	513	U2-1	-0.228	29.0368	0.0352	0.0002	0.0151	19.6683	16.6037
FOURTH	B343	513	U2-1	0.104	29.8696	0.0275	0.0003	0.0118	19.7914	16.0845
THIRD	B343	513	U2-1	-0.1966	35.3051	0.0049	0.0001	0.0024	26.8294	21.8447
THIRD	B343	513	U2-1	-0.109	35.6358	0.0026	0.0001	0.0016	25.793	21.0142
SECOND	B343	513	U2-1	0.5652	29.3826	0.0049	0.0002	0.0087	18.7551	16.5397
SECOND	B343	513	U2-1	0.5161	30.6119	0.0055	0.0003	0.0111	19.3667	16.565
FIRST	B343	513	U2-1	0.527	28.6295	0.0141	0.0002	0.0062	18.5716	16.036
FIRST	B343	513	U2-1	0.2773	30.0604	0.0205	0.0003	0.0086	20.6501	16.2582
MEZZ	B343	513	U2-1	0.1577	24.9074	0.0091	4.57E-05	0.0044	15.8364	12.7451
MEZZ	B343	513	U2-1	0.4302	26.0456	0.0084	0.0001	0.0042	16.7151	12.1311
GROUND	B343	513	U2-1	-1.0649	24.3479	0.0102	4.77E-05	0.0064	16.5619	12.25
GROUND	B343	513	U2-1	-1.2745	24.9417	0.0162	0.0001	0.0101	14.925	11.5835

	Non deteriorated Structure
	Deteriorated Structure

Table 60 Design forces Edge Beam 343

The table 60 exhibits the design forces for edge beam. It is observed that there is very little difference in the design forces amongst the two models.

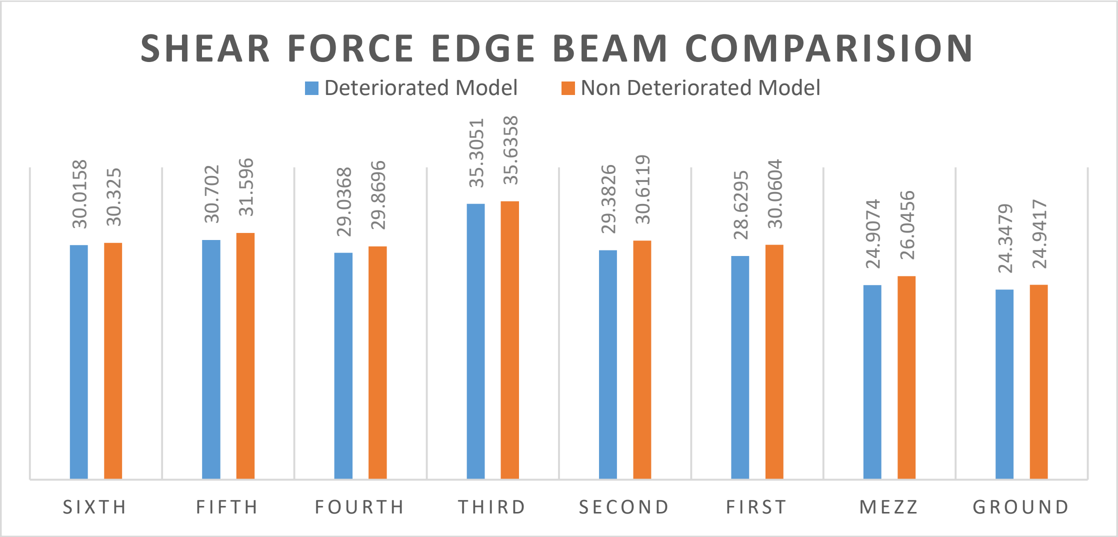


Figure 56 Shear force Comparison Edge Beam

The figure 56 exhibits the design shear forces for edge beam. It is observed that there is a minor difference between the two models. The percentage of difference in with 1% and hence can be neglected.

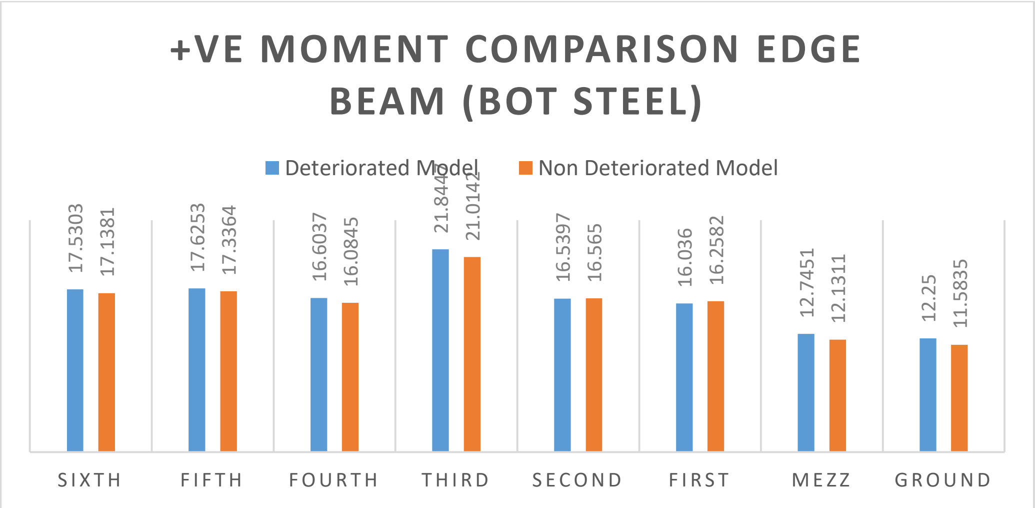


Figure 57 Positive Moment Comparison Edge Beam

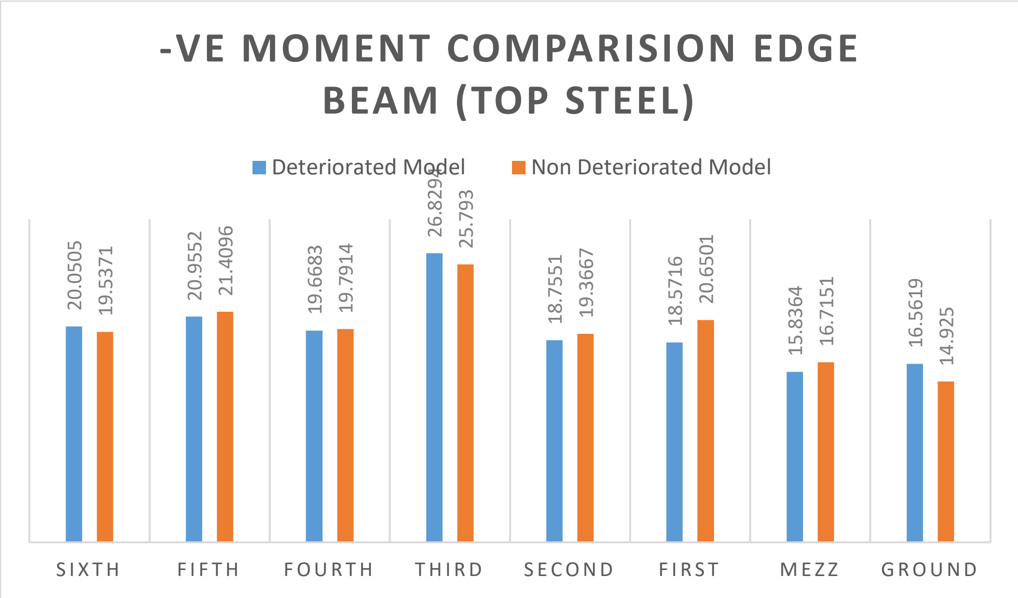


Figure 58 Negative Moment Comparison Edge Beam

The figure 57 & 58 exhibits the design moment values used to design the beam for bottom and top reinforcement respectively. It is observed that there is a marginal difference in between the two models generated. The difference is so small that it can be neglected. Although it is observed that the non-deteriorated model has a higher moment values whereas the deteriorated model has a slightly higher shear force design values.

Story	Beam	UniqueNam	Combo	P	V2	V3	T	M2	M3 TOP	M3 BOT
				kN	kN	kN	kN-m	kN-m	kN-m	kN-m
SIXTH	B169	2279	U2-1	9.2044	73.7245	-0.1285	4.28E-05	0.08	40.7286	26.4952
SIXTH	B169	2279	U2-1	9.6116	74.2711	-0.121	4.8E-05	0.0755	40.9464	26.7238
FIFTH	B169	2279	U2-1	-4.3052	71.8842	0.0986	0.0001	0.0364	37.0237	26.9548
FIFTH	B169	2279	U2-1	-2.7814	73.2743	0.0482	0.0002	0.0166	37.6644	27.8915
FOURTH	B169	2279	U2-1	2.5022	67.9312	-0.0119	2.93E-05	0.011	32.0013	25.9718
FOURTH	B169	2279	U2-1	1.2675	71.7451	0.0056	4.62E-05	0.0012	33.5805	27.6624
THIRD	B169	2279	U2-1	-1.9001	68.1101	0.0718	3.27E-05	0.028	31.6167	26.3847
THIRD	B169	2279	U2-1	-1.1021	71.1307	0.0476	0.0001	0.0174	32.6135	27.5935
SECOND	B169	2279	U2-1	0.8372	67.1708	0.0228	3E-05	0.0095	27.1891	26.2484
SECOND	B169	2279	U2-1	0.7875	69.4145	0.0166	4.04E-05	0.0059	27.9409	27.4325
FIRST	B169	2279	U2-1	-4.0017	65.6175	0.1073	2.88E-05	0.0413	26.4655	25.6993
FIRST	B169	2279	U2-1	-4.9112	68.5736	0.1033	3.48E-05	0.0368	27.0347	27.2035
MEZZ	B169	2279	U2-1	-3.1843	53.626	0.0634	4.27E-05	0.0185	19.8964	25.0361
MEZZ	B169	2279	U2-1	-4.6042	56.03	0.0656	0.0001	0.0201	21.8378	27.1617
GROUND	B169	2279	U2-1	-0.4923	68.669	0.1251	0.0001	0.0487	21.7361	27.3798
GROUND	B169	2279	U2-1	-0.3704	72.8765	0.1514	0.0002	0.0562	23.5979	29.4843

	Non deteriorated Structure
	Deteriorated Structure

Table 61: Design forces Middle Beam 169

The table 61 exhibits the design forces for middle beam. It is observed that there is very little difference in the design forces amongst the two models. Although in comparison to the edge beams the variations in the middle beams are more.

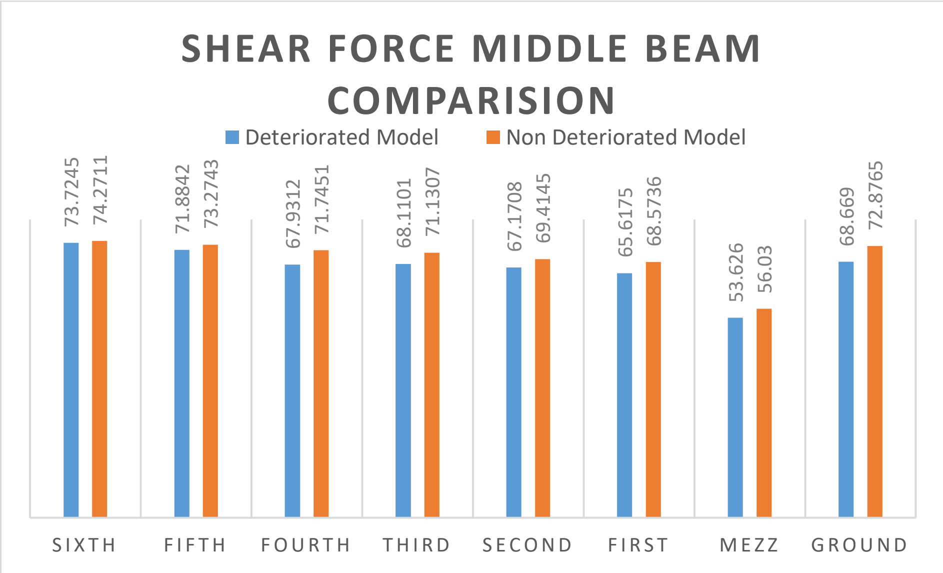


Figure 59 Shear force Comparison Middle Beam

The figure 59 exhibits the design shear forces for middle beam. it is observed that the shear force values for the non-deteriorated model is more in comparison to the deteriorated model, although the difference is very small and can be deemed insignificant.

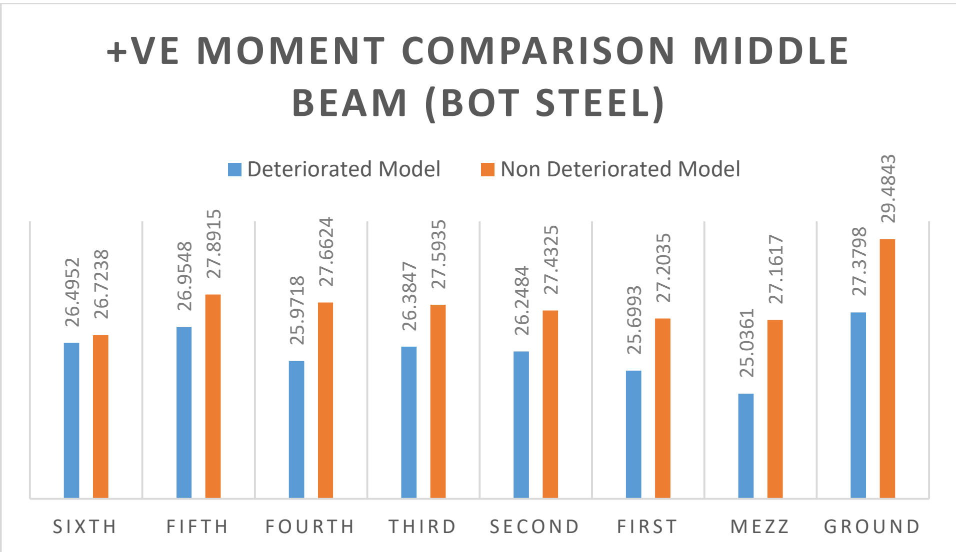


Figure 60 Positive Moment Comparison Middle Beam

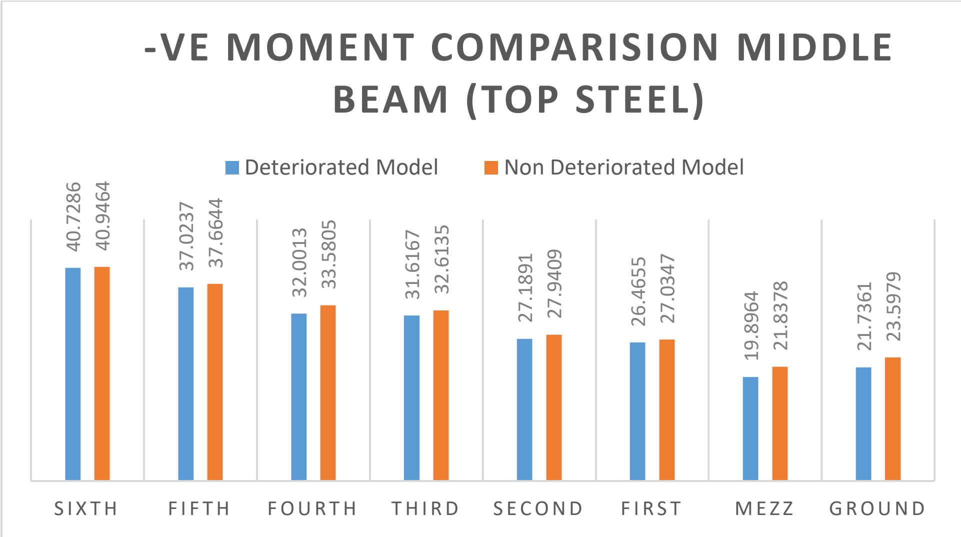
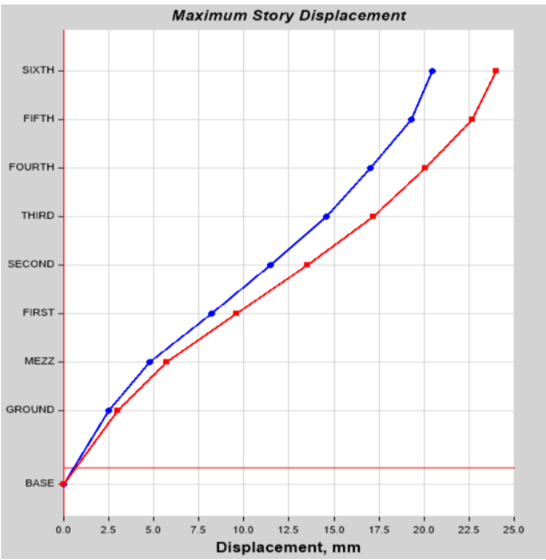
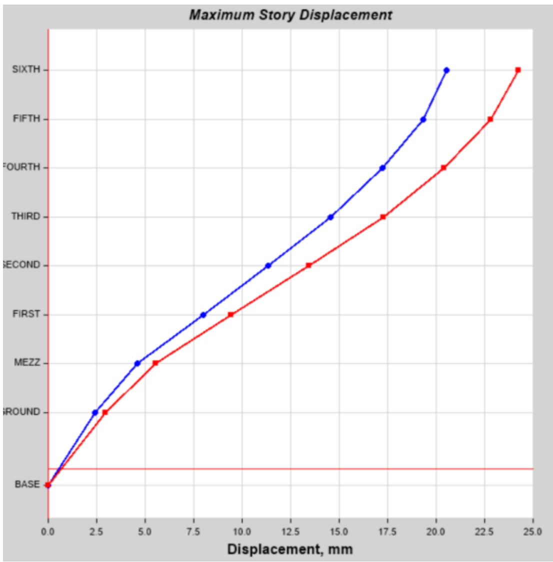


Figure 61 Negative Moment Comparison Middle Beam

The figure 60 & 61 exhibits the design moment values used to design the beam for bottom and top reinforcement respectively. Figure 65 shows slight variations of a difference of 2 Kn/m^2. This difference is not much significant in terms of having a major impact in design changes. Further for the negative moment seen in figure 66 the difference is quite marginal and can be considered insignificant to have an impact on design. In this study we can understand that with the change in material properties there is a small difference in the design forces, although the difference is very small and has no significant impact in the design of the structural elements and thus the difference found in the forces can be considered negligible.

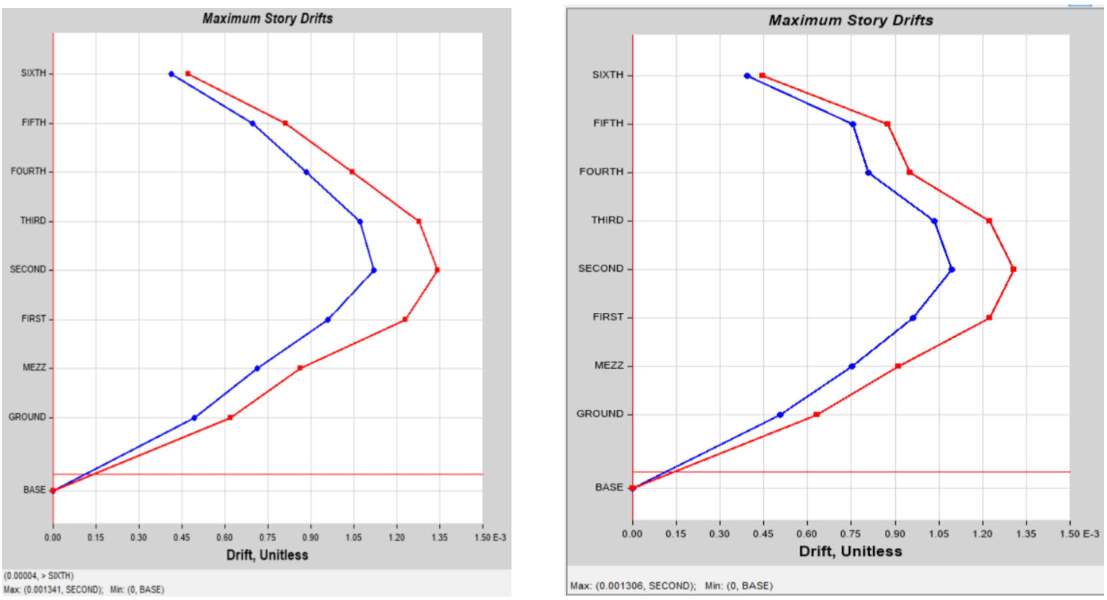




(A.) Max. Story Displacement Deteriorated Model *(B.) Max Story Displacement Normal Model*

Figure 62 Maximum Story Displacement Comparison

The figure 62 exhibits the comparative view of story displacement for the deteriorated model and non-deteriorated model. The max story displacement has no significant difference due to the change in property parameters. Although the figure 67 exhibits that the drift for non-deteriorated model is lesser than the deteriorated model. Thus exhibiting a small difference in drift values due to change in property parameters while modelling. A smaller drift indicates a better rigidity for the structure.



(A.) Max. Story Drift for Deteriorated Model *(B.) Max Story Drift for Normal Model*

Figure 63 Maximum Story Drift Comparison

CHAPTER 7: STRUCTURAL DESIGN ASSESMENT OF DETERIORATED COLUMNS

Deterioration of structural element not only spoils the aesthetics but reduces the structural capacity of the building. Not all deteriorated structures require strengthening and small repair works could be sufficient to restore the building to its full adequacy. This chapter helps to understand the capacity lost due to the deterioration of the structural columns. The forces retrieved from modelling in the previous chapter are used to design the structural columns with different strength acquired from core test results and are compared to a full strength and capacity column. The design comparison for the structural element was done using a Design Software named PROKON, where in which the design forces along with the compressive strength of the element and its sizes are input in the software to give the capacity and reinforcement of the element to be designed. Side by side comparison of the elements using this software helps to understand the amount of capacity lost and whether or not an element has to be retrofitted. The structural element assessed are the columns, as it is observed in the previous chapter that the force acting on it is more as compared to the beams making it more critical to assess. Furthermore, the columns are an integral part of the structure transferring the loads from horizontal members at different floor levels to the foundation where in which the load gets dissipated in the soil, thus any failure would affect the entire structure. Whereas for beams failure could be localised.

Initially in this chapter the columns are compared to analyse the capacity lost in its design. The columns were classified in three categories in the previous chapter as: corner columns, intermediate columns and middle columns depending on the number of side constrained. Similarly, the comparative design results are portrayed for each floor

7.1 CORNER COLUMNS.

The sizes of the columns and the reinforcement assessed for the corner columns as per the existing site conditions are as follows.

FLOOR	COLUMNS SIZE (cm)	REINFORCEMENT
GROUND	40x80	8T20 + 6T16
FIRST	20x80	4T20 + 6T16
SECOND	20x80	4T20 + 6T16
THRID	20x70	8T16
FOURTH	20x70	8T16
FIFTH	20x60	8T16
SIXTH	20x60	8T16

Table 62: Corner Columns Size and Reinforcement

A. GROUND FLOOR CORNER COLUMNS

Firstly, The Column element C1 has been compared for ground floor for deteriorated and non-deteriorated structure. The figure 64 is an interaction diagram for corner column at ground floor for the compressive strength of 15.3 Mpa. It can be observed that it has a maximum moment capacity of 434.7 KNm at 431 KN axial force. Further the maximum axial force capacity is 3000 KN for 0 moment. It is also observed that the applied moment and axial force are within the curve thus indicating it is structurally safe.

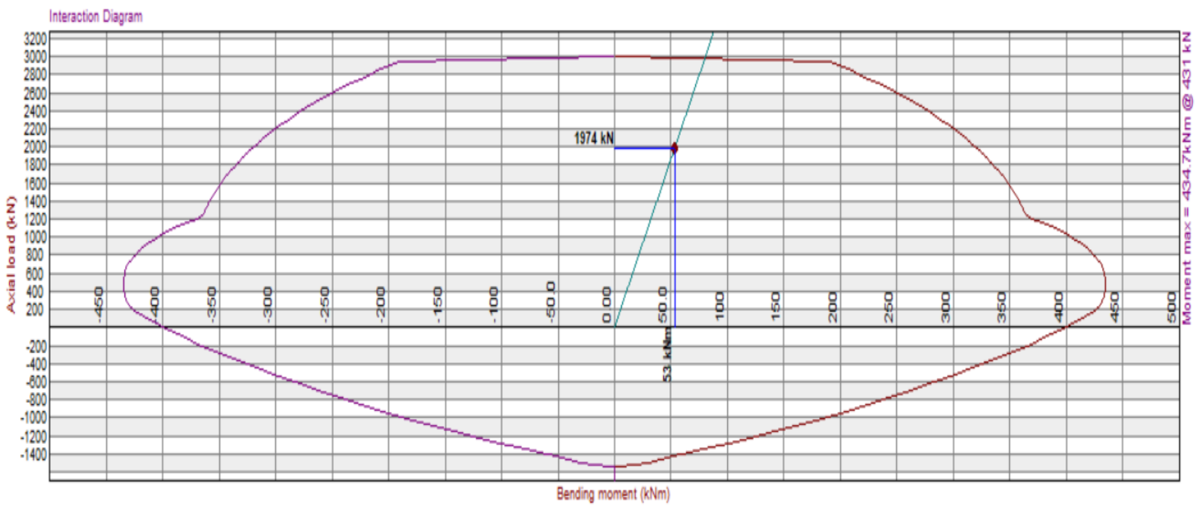


Figure 64 Interaction Diagram of Deteriorated Corner Column Ground Floor

The figure 65 is an interaction diagram for corner column at ground floor for the compressive strength of 28 Mpa. It can be observed that it has a maximum moment capacity of 670 KNm at 1355 KN axial force. Further the maximum axial force capacity is 4758 KN for 0 moment. It is also observed that the applied moment and axial force are within the curve thus indicating it is structurally safe.

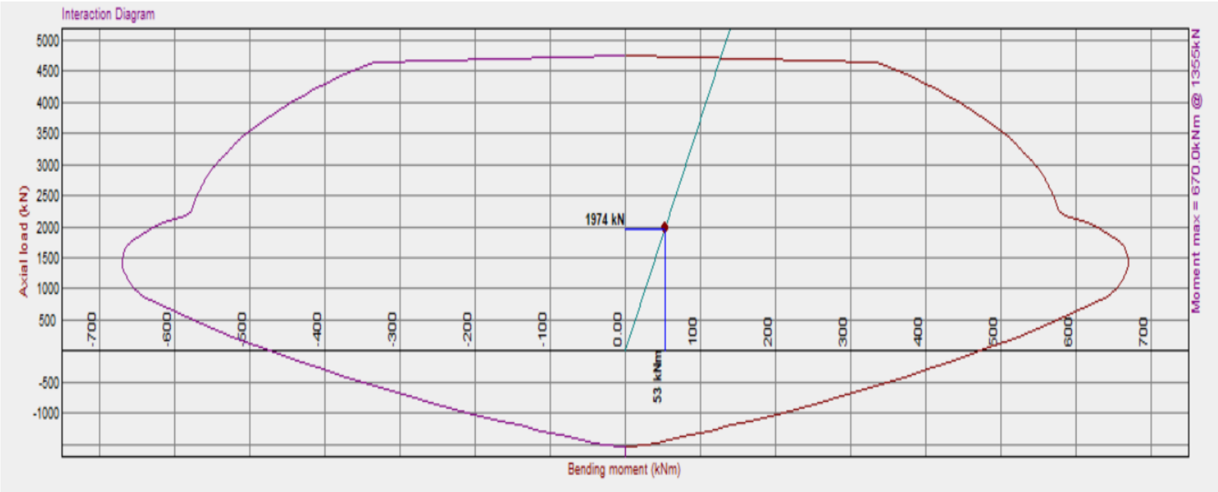


Figure 65 Interaction Diagram of Non Deteriorated Corner Column Ground Floor

On comparing the interactive diagrams for the corner columns of the ground floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 36% and the axial force capacity lost is 37%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

B. FIRST FLOOR CORNER COLUMNS

The Column element C1 has been compared of first floor for deteriorated and non-deteriorated structure. The figure 66 is an interaction diagram for corner column at first floor for the compressive strength of 13.5 Mpa. It can be observed that it has a maximum moment capacity of 72.8 KNm at 162 KN axial force. Further the maximum axial force capacity is 1490 KN for

0 moment. For the applied design forces the point lies outside the interaction diagram indicating that the columns size and reinforcement provided is inadequate.

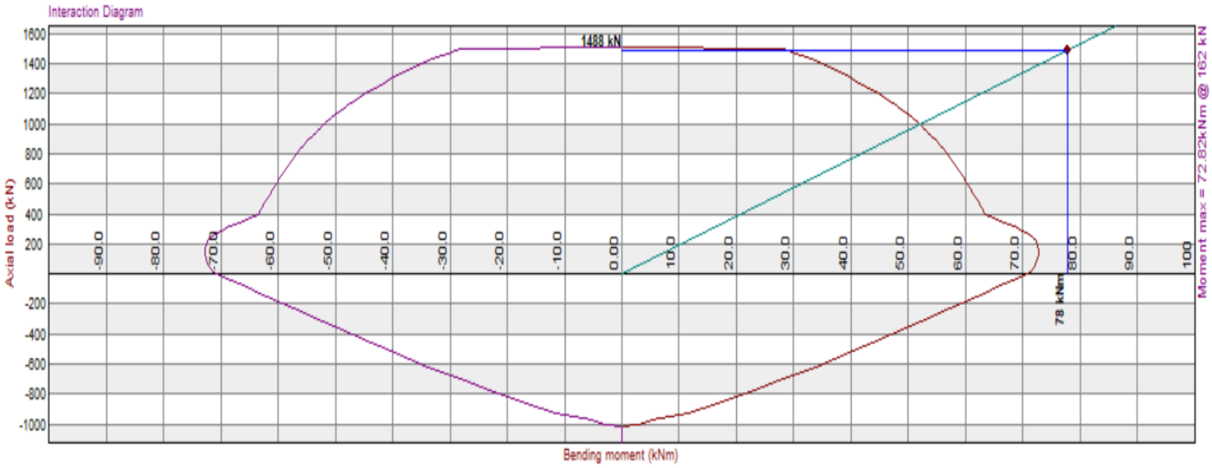


Figure 66 Interaction Diagram of Deteriorated Corner Column First Floor

The figure 67 is an interaction diagram for corner column at first floor for the compressive strength of 28 Mpa. It can be observed that it has a maximum moment capacity of 103.3 KNm at 557 KN axial force. Further the maximum axial force capacity is 2575 KN for 0 moment.

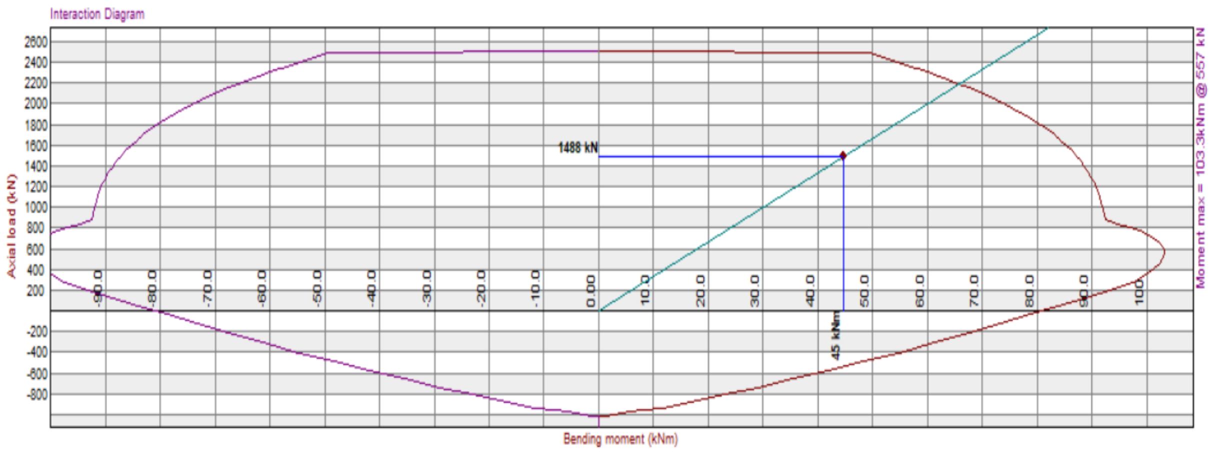


Figure 67 Interaction Diagram of Non Deteriorated Corner Column First Floor

On comparing the interactive diagrams for the corner columns of the first floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 29.5% and the axial force capacity lost is 42%. The reduction in capacity of the column due to deterioration is to the extent that the column requires retrofitting and repair works to bring the column in to a condition to withstand the existing loads.

C. SECOND FLOOR CORNER COLUMNS

The Column element C1 has been compared of second floor for deteriorated and non-deteriorated structure. The figure 68 is an interaction diagram for corner column at second floor for the compressive strength of 11.7 Mpa. It can be observed that it has a maximum moment capacity of 67.17 KNm at 52.3 KN axial force. Further the maximum axial force capacity is 1400 KN for 0 moment. For the applied design forces the columns size and reinforcement provided is inadequate.

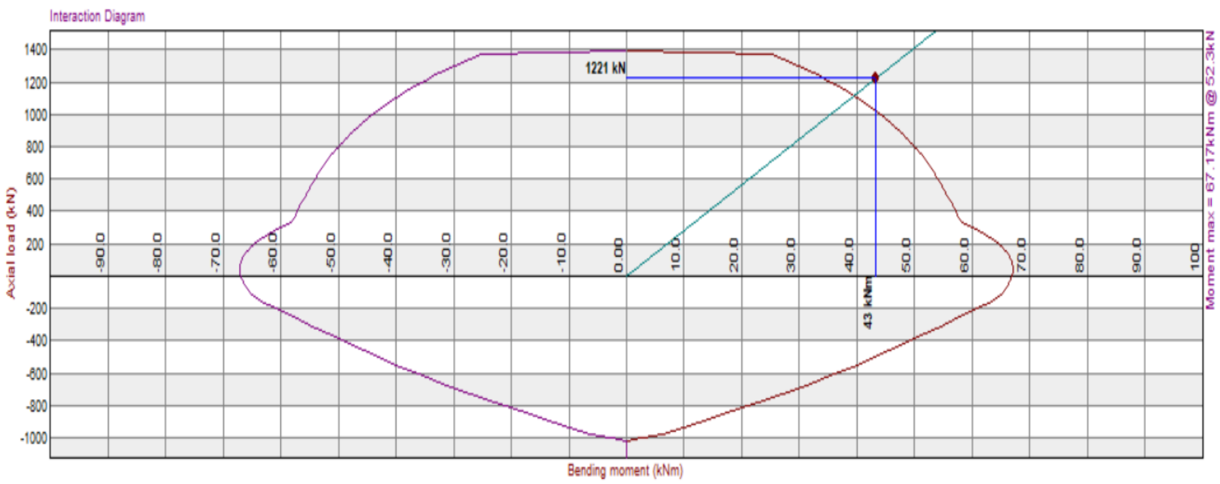


Figure 68 Interaction Diagram of Deteriorated Corner Column Second Floor

Further in the interaction diagram for corner column at second floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 107.5 KNm at 642 KN axial force. Further the maximum axial force capacity is 2500 KN for 0 moment. The figure E-1 exhibits the interaction diagram for non-deteriorated corner column of second floor.

On comparing the interactive diagrams for the corner columns of the second floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 37.5% and the axial force capacity lost is 44%. The reduction in capacity of the column due to deterioration is to the extent that the column requires retrofitting and repair works to bring the column in to a condition to withstand the existing loads.

D. THIRD FLOOR CORNER COLUMNS

The Column element C1 has been compared of third floor for deteriorated and non-deteriorated structure. The figure 69 is an interaction diagram for corner column at third floor for the compressive strength of 12.5 Mpa, it is observed that it has a maximum moment capacity of 51.37 KNm at 132 KN axial force. Further the maximum axial force capacity is 1125 KN for 0 moment. For the applied design forces the point lies outside the interaction diagram indicating that the columns size and reinforcement provided is inadequate.

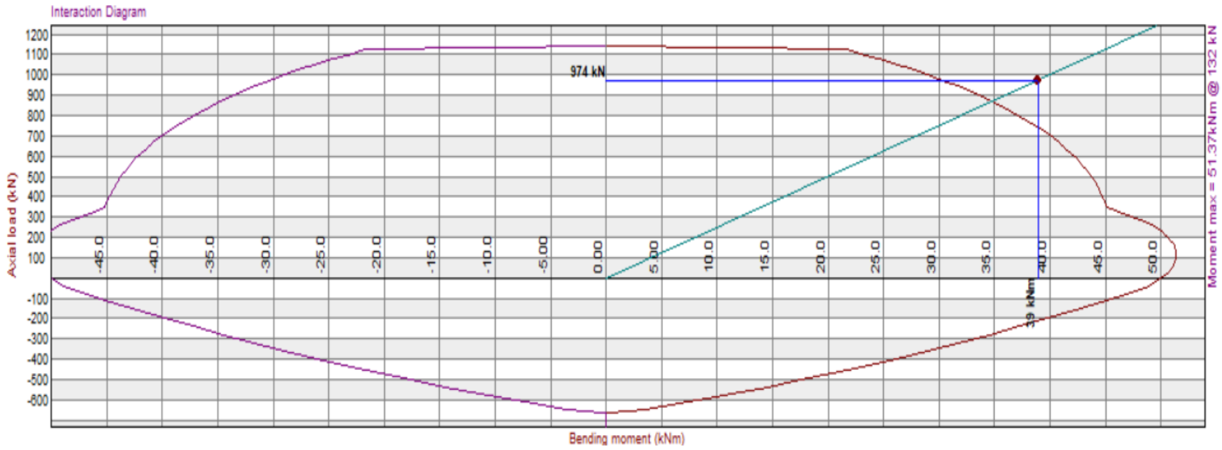


Figure 69 Interaction Diagram of Deteriorated Corner Column Third Floor

Further in the interaction diagram for corner column at third floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 82.62 KNm at 575 KN axial force. Further the maximum axial force capacity is 2080 KN for 0 moment. The Figure E-2 exhibits the interaction diagram for non-deteriorated corner columns of third floor.

On comparing the interactive diagrams for the corner columns of the third floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 37.8% and the axial force capacity lost is 46%. The reduction in capacity of the column due to deterioration is to the extent that the column requires retrofitting and repair works to bring the column in to a condition to withstand the existing loads.

E. FOURTH FLOOR CORNER COLUMNS

The Column element C1 has been compared of Fourth floor for deteriorated and non-deteriorated structure. On designing an obtaining the interaction diagram for corner column at fourth floor for the compressive strength of 18.1Mpa, it is observed that it has a maximum moment capacity of 68.13 KNm at 271 KN axial force. Further the maximum axial force capacity is 1380 KN for 0 moment. The point of the applied design forces lies within the interaction diagram indicating the columns size and reinforcement provided is adequate. The interaction diagram of the deteriorated corner column in the fourth floor is exhibited in figure E-3.

Furthermore, in the interaction diagram for corner column at Fourth floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 87.46 KNm at 544 KN axial force. Further the maximum axial force capacity is 2050 KN for 0 moment. The interaction diagram for the non-deteriorated corner column of the fourth floor is exhibited in figure E-4.

On comparing the interactive diagrams for the corner columns of the Fourth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 22.1% and the axial force capacity lost is 32.68%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

F. FIFTH FLOOR CORNER COLUMNS

The Column element C1 has been compared of Fifth floor for deteriorated and non-deteriorated structure. For the specified design forces the interaction diagram for corner column at fifth floor for the compressive strength of 7.20Mpa, it is observed that it has a maximum moment capacity of 49.9 KNm at 49.60 KN axial force. Further the maximum axial force capacity is 900 KN for 0 moment. For the applied design forces the columns size and reinforcement provided is adequate as the design forces lie within the interaction diagram. The interaction diagram for deteriorated corner column of the fifth floor is portrayed in figure E-5.

In the interaction diagram for corner column at Fifth floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 84.16 KNm at 395 KN axial force. Further the maximum axial force capacity is 1820 KN for 0 moment. The interaction diagram for the non-deteriorated corner columns of fifth floor is exhibited in figure E-6.

On comparing the interactive diagrams for the corner columns of the Fifth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 40.71% and the axial force capacity lost is 50.5%. Although the deteriorated column does not require retrofitting and can withstand the applied load its recommended to retrofit as the compressive strength is less than 10Mpa, along with repair to avoid propagation of deterioration.

G. SIXTH FLOOR CORNER COLUMNS

The Column element C1 has been compared of Sixth floor for deteriorated and non-deteriorated structure. The figure 70 is an interaction diagram for corner column at sixth floor for the compressive strength of 7.20Mpa. It can be observed that it has a maximum moment capacity of 90.64 KNm at 140 KN axial force. Further the maximum axial force capacity is 1120 KN for 0 moment. For the applied design forces the columns size and reinforcement provided is adequate. Further it can be observed from the interaction diagram that the applied forces are within the one third of the interaction diagram indicating that the size provided is more than required initially.

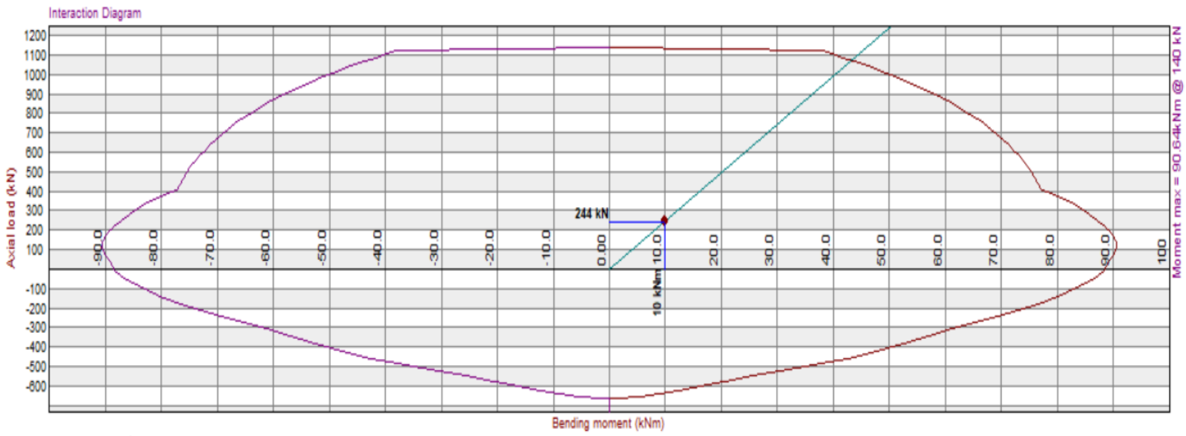


Figure 70 Interaction Diagram of Deteriorated Corner Column Sixth Floor

In interaction diagram for corner column at Sixth floor for the compressive strength of 28 Mpa., it is observed that it has a maximum moment capacity of 131 KNm at 421 KN axial force. Further the maximum axial force capacity is 1820 KN for 0 moment. The interaction diagram for the sixth floor corner column is portrayed in figure E-7.

On comparing the interactive diagrams for the corner columns of the Sixth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 30.8% and the axial force capacity lost is 38%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

7.2 INTERMEDIATE COLUMNS

In this part of the study the intermediate column i.e. column that are constrained in three face and free at one face are assessed for the loss of capacity for moment and axial force. The sizes of the columns and the reinforcement assessed for the intermediate columns as per the existing site conditions are as follows.

FLOOR	COLUMNS SIZE (cm)	REINFORCEMENT
GROUND	40x100	8T25 + 12T16
FIRST	20x100	4T25 + 12T16
SECOND	20x100	4T25 + 12T16
THRID	20x90	12T16
FOURTH	20x90	12T16
FIFTH	20x80	8T16
SIXTH	20x80	8T16

Table 63 Intermediate Columns Size and Reinforcement

A. GROUND FLOOR INTERMEDIATE COLUMNS

Firstly, The Column element C69 has been compared for ground floor for deteriorated and non-deteriorated structure.in the interaction diagram for intermediate column at ground floor for the compressive strength of 15.3 Mpa, It is observed that it has a maximum moment capacity of 495.2 KNm at 88.5 KN axial force. Further the maximum axial force capacity is 4120 KN for 0 moment. The design forces lie within the interaction diagram indicating the column size and the reinforcement provided is still adequate for the deteriorated structure. The interaction diagram for the deteriorated intermediate column of the ground floor is exhibited in figure E-8. Furthermore, in the interaction diagram for intermediate column at ground floor for the compressive strength of 28 Mpa, It is observed that it has a maximum moment capacity of 690 KNm at 1152 KN axial force. Further the maximum axial force capacity is 6415 KN for 0 moment. The interaction diagram of the intermediate columns for the non-deteriorated columns in the ground floor is portrayed in figure E-9.

On comparing the interactive diagrams for the intermediate columns of the ground floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 28.23% and the axial force capacity lost is 35.7%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

B. FIRST FLOOR INTERMEDIATE COLUMNS

The Column element C69 has been compared for first floor for deteriorated and non-deteriorated structure. In the interaction diagram for intermediate column at first floor for the compressive strength of 13.5 Mpa it is bserved that it has a maximum moment capacity of 112.7 KNm at 176 KN axial force. Further the maximum axial force capacity is 2200 KN for 0 moment. It is observed from the interaction diagram of intermediate columns for deteriorated first floor columns in figure 71 that although the design forces lie within the interaction diagram the capacity of the columns is utilised almost to the limit

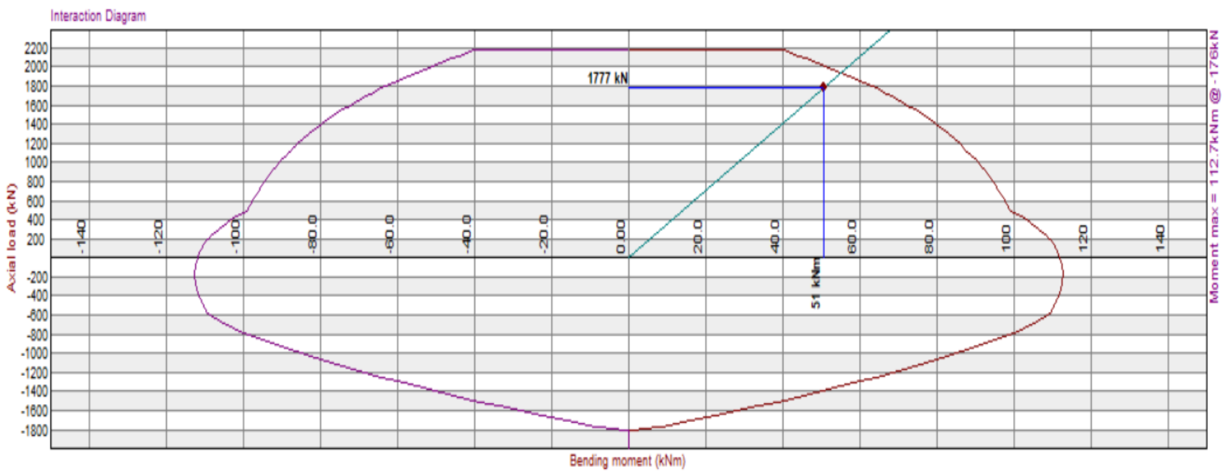


Figure 71 Interaction Diagram of Deteriorated Intermediate Column First Floor

In the interaction diagram for intermediate column at first floor for the compressive strength of 28 Mpa, It is observed that it has a maximum moment capacity of 329.3 KNm at 479 KN axial force. Further the maximum axial force capacity is 3400 KN for 0 moment. This interaction diagram of the intermediate first floor columns for non-deteriorated structure is portrayed in the figure E-10.

On comparing the interactive diagrams for the intermediate columns of the first floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 46.55% and the axial force capacity lost is 35.29%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

C. SECOND FLOOR INTERMEDIATE COLUMNS

The Column element C69 has been compared for second floor for deteriorated and non-deteriorated structure. In the interaction diagram for intermediate column at second floor for the compressive strength of 11.7 Mpa, It is observed that it has a maximum moment capacity of 303.4 KNm at -15 KN axial force. Further the maximum axial force capacity is 2000 KN for 0 moment. The interaction diagram for the intermediate deteriorated column of second floor is exhibited in the figure E-11. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated intermediate column of second floor.

In the interaction diagram for intermediate column at second floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 536.8 KNm at 523 KN axial force. Further the maximum axial force capacity is 3400 KN for 0 moment. The interaction diagram for the non-deteriorated intermediate columns of second floor is exhibited in the figure E-12.

On comparing the interactive diagrams for the intermediate columns of the second floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 43.5% and the axial force capacity lost is 45%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

D. THIRD FLOOR INTERMEDIATE COLUMNS

The Column element C69 has been compared for third floor for deteriorated and non-deteriorated structure. In the interaction diagram for intermediate column at third floor for the compressive strength of 12.5 Mpa, It is observed that it has a maximum moment capacity of 142.2 KNm at 97.5 KN axial force. Further the maximum axial force capacity is 1480 KN for 0 moment. As the design forces lie within the interaction diagram, the column and reinforcement size can be considered adequate for the deteriorated intermediate column of third floor. This interaction diagram for deteriorated intermediate columns of the third floor is exhibited in the figure E-13.

Furthermore, in the interaction diagram for intermediate column at third floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 302.9 KNm at 698 KN axial force. Further the maximum axial force capacity is 2760 KN for 0 moment. The interaction diagram for non-deteriorated intermediate columns of third floor is portrayed in the figure E-14.

On comparing the interactive diagrams for the intermediate columns of the third floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 53% and the axial force capacity lost is 46%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

E. FOURTH FLOOR INTERMEDIATE COLUMNS

The Column element C69 has been compared for Fourth floor for deteriorated and non-deteriorated structure.in the interaction diagram for intermediate column at fourth floor for the compressive strength of 18.1Mpa, it is observed that it has a maximum moment capacity of 371.9 KNm at 377 KN axial force. Further the maximum axial force capacity is 1980 KN for 0 moment. For the applied design forces the columns size and reinforcement provided is adequate. The interaction diagram for the intermediate deteriorated column of fourth floor is exhibited in the figure E-15. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated intermediate column of fourth floor.

In the interaction diagram for intermediate column at Fourth floor for the compressive strength of 28 Mpa, It is observed that it has a maximum moment capacity of 499.6 KNm at 767 KN axial force. Further the maximum axial force capacity is 2760 KN for 0 moment. The interaction diagram of the fourth floor non deteriorated intermediate columns is portrayed in the figure E-16.

On comparing the interactive diagrams for the intermediate columns of the Fourth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 25.4% and the axial force capacity lost is 28.26%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

F. FIFTH FLOOR INTERMEDIATE COLUMNS

The Column element C69 has been compared for Fifth floor for deteriorated and non-deteriorated structure. In the interaction diagram for intermediate column at fifth floor for the compressive strength of 7.20Mpa, it is observed that it has a maximum moment capacity of 191 KNm at 136 KN axial force. Further the maximum axial force capacity is 1085 KN for 0 moment. The interaction diagram for the intermediate deteriorated column of fifth floor is exhibited in the figure E-17. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated intermediate column of fifth floor.

In the interaction diagram for intermediate column at Fifth floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 375.5 KNm at 764 KN axial force. Further the maximum axial force capacity is 2300 KN for 0 moment. The interaction diagram of the fifth floor non deteriorated intermediate columns is portrayed in the figure E-18.

On comparing the interactive diagrams for the intermediate columns of the Fifth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 49.13% and the axial force capacity lost is 52.8%. Although the deteriorated column does not require retrofitting and can withstand the applied load its recommended to retrofit as the compressive strength is less than 10Mpa, along with repair to avoid propagation of deterioration.

G. SIXTH FLOOR INTERMEDIATE COLUMNS

The Column element C69 has been compared for Sixth floor for deteriorated and non-deteriorated structure. In the interaction diagram for intermediate column at sixth floor for the compressive strength of 14.70Mpa, it is observed that it has a maximum moment capacity of 289.1 KNm at 369 KN axial force. Further the maximum axial force capacity is 1410 KN for 0 moment. The interaction diagram for the intermediate deteriorated column of sixth floor is exhibited in the figure E-19. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated intermediate column of sixth floor.

In the interaction diagram for intermediate column at Sixth floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 424.1 KNm at 853 KN axial force. Further the maximum axial force capacity is 2320 KN for 0 moment. The interaction diagram of the sixth floor non deteriorated intermediate columns is portrayed in the figure E-20.

On comparing the interactive diagrams for the intermediate columns of the Sixth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 31.8% and the axial force capacity lost is 39.2%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

7.3 MIDDLE COLUMNS

In this part of the study the Middle column i.e. column that are constrained in all four faces are assessed for the loss of capacity for moment and axial force. The sizes of the columns and the reinforcement assessed for the Middle columns as per the existing site conditions are as follows.

FLOOR	COLUMNS SIZE (cm)	REINFORCEMENT
GROUND	30x100	8T25 + 12T16
FIRST	20x100	4T25 + 12T16
SECOND	20x100	4T25 + 12T16
THRID	20x80	12T16
FOURTH	20x80	12T16
FIFTH	20x80	10T16
SIXTH	20x80	10T16

Table 64: Middle Columns Size and Reinforcement

A. GROUND FLOOR MIDDLE COLUMNS

Firstly, The Column element C82 has been compared for ground floor for deteriorated and non-deteriorated structure. The figure 72 is an interaction diagram for Middle column at ground floor for the compressive strength of 15.3 Mpa. It can be observed that it has a maximum moment capacity of 881.9 KNm at 236 KN axial force. Further the maximum axial force capacity is 3480 KN for 0 moment.

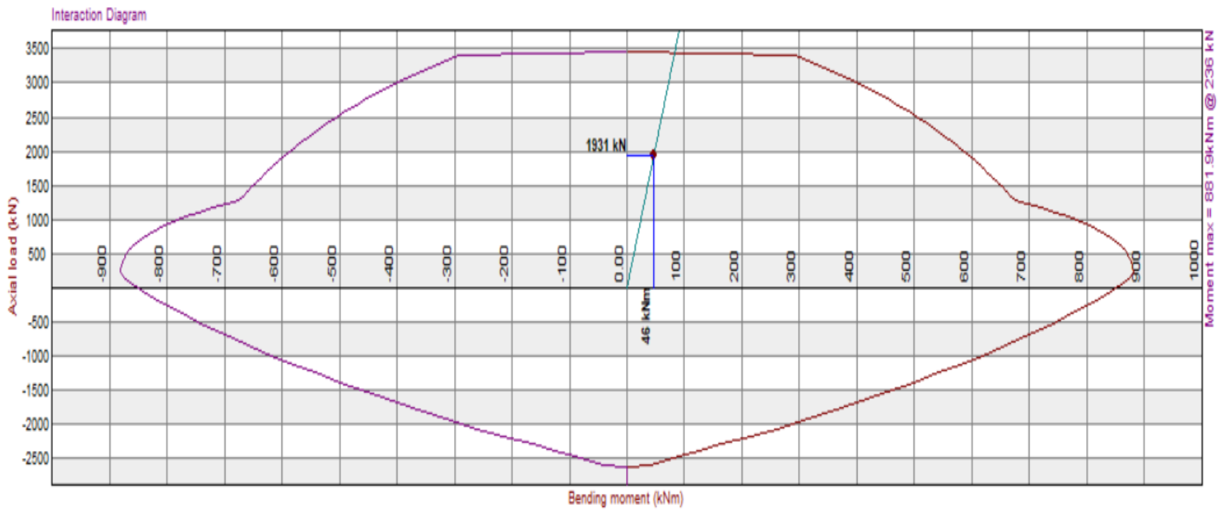


Figure 72 Interaction Diagram of Deteriorated Middle Column Ground Floor

The figure 73 is an interaction diagram for Middle column at ground floor for the compressive strength of 28 Mpa. It can be observed that it has a maximum moment capacity of 1155 KNm at 1018 KN axial force. Further the maximum axial force capacity is 5050 KN for 0 moment.

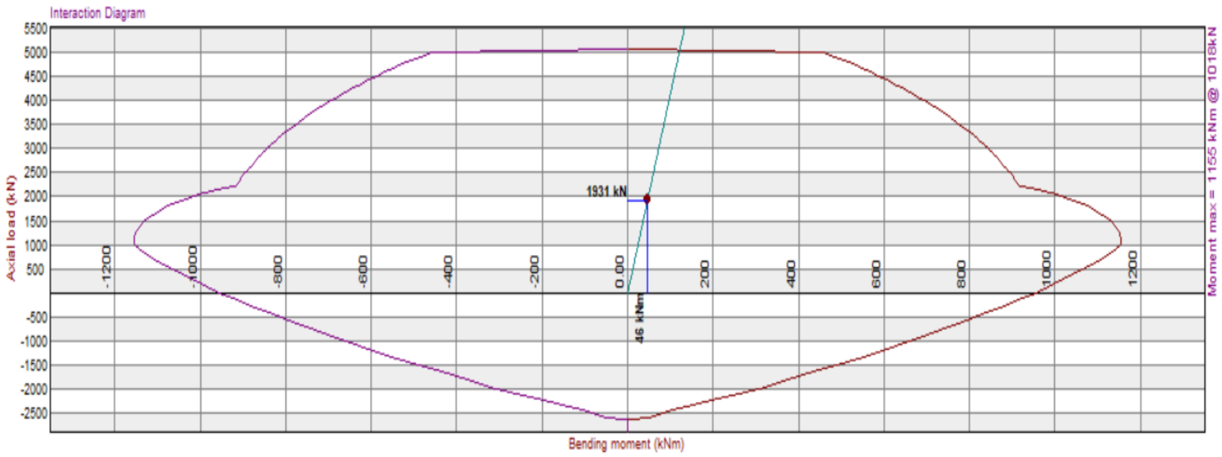


Figure 73 Interaction Diagram of Non Deteriorated Middle Column Ground Floor

On comparing the interactive diagrams for the Middle columns of the ground floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 23.6% and the axial force capacity lost is 31.08%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

B. FIRST FLOOR MIDDLE COLUMNS

The Column element C82 has been compared for first floor for deteriorated and non-deteriorated structure. In the interaction diagram for Middle column at first floor for the compressive strength of 13.5 Mpa, it is observed that it has a maximum moment capacity of 181.5 KNm at -176 KN axial force. Further the maximum axial force capacity is 2000 KN for 0 moment. The interaction diagram for the middle deteriorated column of first floor is exhibited in the figure E-21. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated middle column of first floor. Furthermore, in the interaction diagram for Middle column at first floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 292.1 KNm at 552 KN axial force. Further the maximum axial force capacity is 3400 KN for 0 moment. The interaction diagram of the first floor non deteriorated middle columns is portrayed in the figure E-22.

On comparing the interactive diagrams for the Middle columns of the first floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 37% and the axial force capacity lost is 41%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

C. SECOND FLOOR MIDDLE COLUMNS

The Column element C82 has been compared for second floor for deteriorated and non-deteriorated structure. In the interaction diagram for Middle column at second floor for the compressive strength of 11.7 Mpa, it is observed that it has a maximum moment capacity of 252.4 KNm at -210 KN axial force. Further the maximum axial force capacity is 2020 KN for

0 moment. The interaction diagram for the middle deteriorated column of second floor is exhibited in the figure E-23. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated middle column of second floor.

In the interaction diagram for Middle column at second floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 421.4 KNm at 598 KN axial force. Further the maximum axial force capacity is 3400 KN for 0 moment. The interaction diagram of the second floor non deteriorated middle columns is portrayed in the figure E-24.

On comparing the interactive diagrams for the Middle columns of the second floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 40.1% and the axial force capacity lost is 40.58%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

D. THIRD FLOOR MIDDLE COLUMNS

The Column element C82 has been compared for third floor for deteriorated and non-deteriorated structure. In the interaction diagram for Middle column at third floor for the compressive strength of 12.5 Mpa, it is observed that it has a maximum moment capacity of 88.24 KNm at 2.83 KN axial force. Further the maximum axial force capacity is 1420 KN for 0 moment. The interaction diagram for the middle deteriorated column of third floor is exhibited in the figure E-25. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated middle column of third floor.

In the interaction diagram for Middle column at third floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 161.5 KNm at 476 KN axial force. Further the maximum axial force capacity is 2510 KN for 0 moment. The interaction diagram of the third floor non deteriorated middle columns is portrayed in the figure E-26.

On comparing the interactive diagrams for the Middle columns of the third floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 45% and the axial force capacity lost is 43.4%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

E. FOURTH FLOOR MIDDLE COLUMNS

The Column element C82 has been compared for Fourth floor for deteriorated and non-deteriorated structure. In the is an interaction diagram for Middle column at fourth floor for the compressive strength of 18.1Mpa it is observed that it has a maximum moment capacity of 209.3 KNm at 222 KN axial force. Further the maximum axial force capacity is 1800 KN for 0 moment. The interaction diagram for the middle deteriorated column of fourth floor is exhibited in the figure E-27. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated middle column of fourth floor.

In the interaction diagram for Middle column at Fourth floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 287 KNm at 610 KN axial force. Further the maximum axial force capacity is 2480 KN for 0 moment. The interaction diagram of the fourth floor non deteriorated middle columns is portrayed in the figure E-28.

On comparing the interactive diagrams for the Middle columns of the Fourth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 27% and the axial force capacity lost is 27.41%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

F. FIFTH FLOOR MIDDLE COLUMNS

The Column element C82 has been compared for Fifth floor for deteriorated and non-deteriorated structure. In the interaction diagram for Middle column at fifth floor for the compressive strength of 7.20Mpa, it is observed that it has a maximum moment capacity of 90.34 KNm at 1 KN axial force. Further the maximum axial force capacity is 1000 KN for 0 moment. The interaction diagram for the middle deteriorated column of fifth floor is exhibited in the figure E-29. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated middle column of fifth floor.

Furthermore, In the interaction diagram for Middle column at Fifth floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 164.4 KNm at 436 KN axial force. Further the maximum axial force capacity is 1910 KN for 0 moment. The interaction diagram of the fifth floor non deteriorated middle columns is portrayed in the figure E-30.

On comparing the interactive diagrams for the Middle columns of the Fifth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 45% and the axial force capacity lost is 47.64%. Although the deteriorated column does not require retrofitting and can withstand the applied load its recommended to retrofit as the compressive strength is less than 10Mpa, along with repair to avoid propagation of deterioration.

G. SIXTH FLOOR MIDDLE COLUMNS

The Column element C82 has been compared for Sixth floor for deteriorated and non-deteriorated structure. In the interaction diagram for Middle column at sixth floor for the compressive strength of 14.70Mpa, it is observed that it has a maximum moment capacity of 170.4 KNm at 104 KN axial force. Further the maximum axial force capacity is 1210 KN for 0 moment. The interaction diagram for the middle deteriorated column of sixth floor is exhibited in the figure E-31. As the design forces lie within the interaction diagram the column and reinforcement size can be considered adequate for the deteriorated middle column of sixth floor.

Furthermore, In the interaction diagram for Middle column at Sixth floor for the compressive strength of 28 Mpa, it is observed that it has a maximum moment capacity of 237.9 KNm at 474 KN axial force. Further the maximum axial force capacity is 1900 KN for 0 moment. The interaction diagram of the sixth floor non deteriorated middle columns is portrayed in the figure E-31.

On comparing the interactive diagrams for the Middle columns of the Sixth floor for the compressive strength achieved from core test and a full capacity compressive strength, it is found that the moment capacity lost is up to 36% and the axial force capacity lost is 28%. Although on losing the capacity due to deterioration the column still is structurally strong to withstand the existing loads and does not require retrofitting but would require repair to avoid further propagation of deterioration.

7.4 OVERALL COMPARISION

This segment of the study looks at the three different categories of columns together to compare the loss in axial force and moment capacity of the columns. Firstly, the corner, middle and intermediate columns are compared simultaneously for the loss in axial force capacity. The figure 74 portrays the side by side comparison of the loss in axial force for different storeys. The studying the graph we can understand that the loss in capacity is more for corner column in the ground floor, first floor, third floor and fourth floor. Whereas the intermediate column has highest capacity loss in the second, fifth and sixth floor.

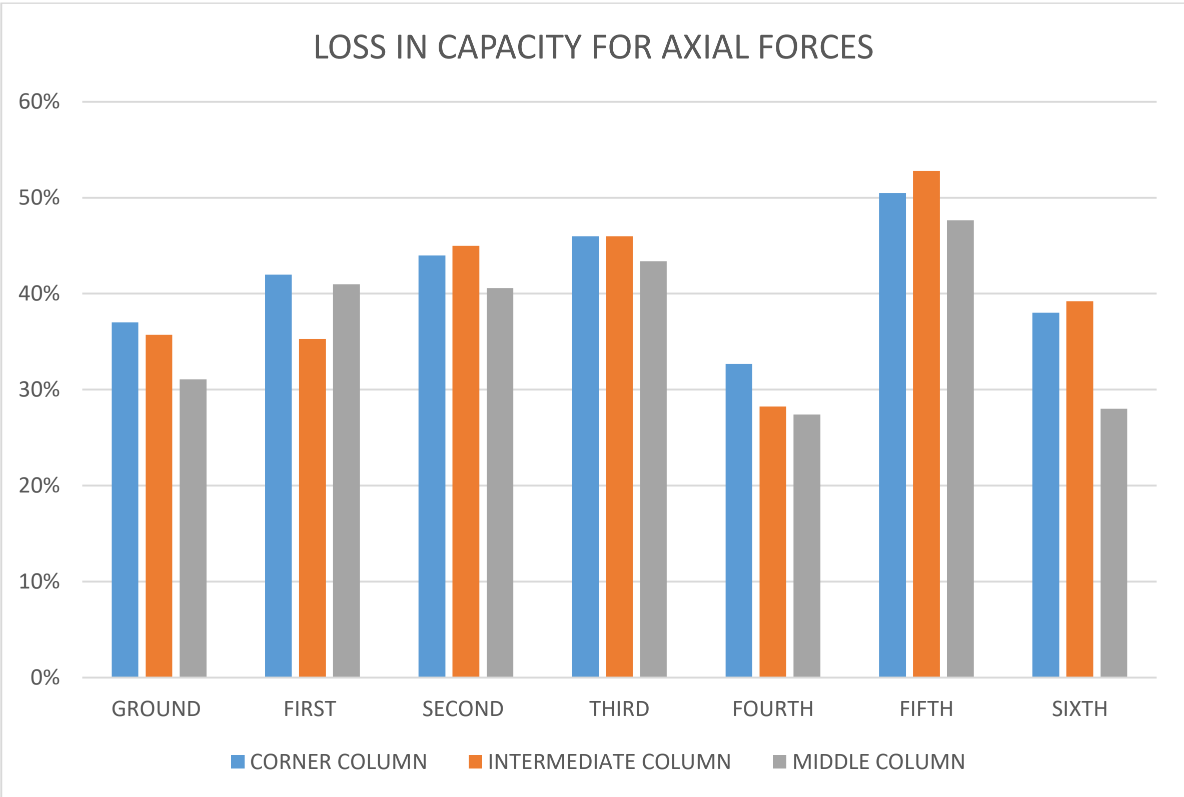


Figure 74 Axial force Capacity loss in columns.

On further observing the graphs of middle column it can be understood that it has the least amount of axial force capacity loss in all the floors except for the first floor. From the observations made in the above graphs we can understand that the columns exposed to the climatic conditions directly have more loss in axial capacity than the ones more secure from direct atmospheric contact.

Furthermore, it can also be understood that for the same loss in compressive strengths it is not necessary to have equal loss in axial force capacity of different type of columns. As observed the axial force capacity loss is more in the fifth floor and the least in the fourth floor. This observation helps us to understand that the compressive strength values are proportional to the loss of axial force capacity, although is not same for all the column and the intensity of the loss depends upon the location of the column situated in the structure.

The study also overviews the loss in capacity of moment for all the column categories simultaneously together. The figure 75 portrays the loss incapacity for moment of different categories of column at different levels of the structure. Unlike the axial forces the moment capacity loss is different. For the corner columns the maximum moment loss is obtained only at the ground floor. The intermediate columns exhibit a maximum moment loss for second, third and fifth floor. Whereas the middle columns show a maximum moment capacity loss for the fourth and the sixth floor.

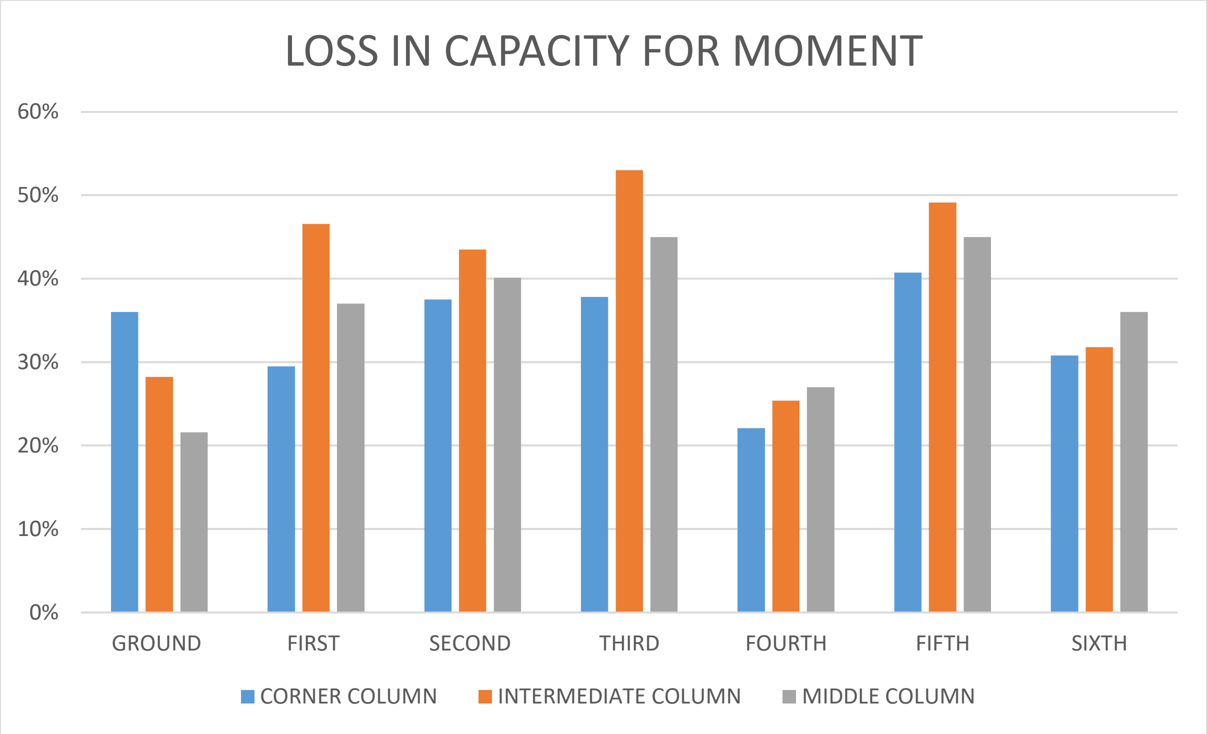


Figure 75 Momentl force Capacity loss in columns.

On further observing the graphs we can deduce that the loss in moment capacity is different for all the different categories of columns. Also it is observed that the higher loss in compressive strengths the more is the loss in moment capacity of the columns. On the basis of these observations we can presume that the loss in axial force have different impact for columns located at different locations in the structure although the loss in compressive strength is proportional to the intensity of loss overall.

The losses observed in the above study not only makes us understand that the loss in compressive strength of columns reduces the capacity but also makes us understand that the loss in capacity is not proportional to all the elements in the same floor and depends on where the structural element is located in the building.

CHAPTER 8: CONCLUSION

8.1 RESEARCH CONCLUSION

The primary intention of the study was to assess the deterioration in the semi-arid environmental conditions. It was observed from the core test data acquired from a few projects that the deterioration occurs mainly due to carbonation of the concrete and exposure to high heat. Other factors like chloride attack and sulphate attack are not prominent. On assessing the data acquired Compressive strength, Ultrasonic pulse velocity and depth of carbonation, it was observed that the mean compressive strength of the slabs is higher than that of columns and beams. Furthermore, the rate of carbonation observed is the least on the slabs and the ultrasonic pulse velocity values are the highest for slabs indicating that in deterioration the least effected are slabs amongst the structural elements. Furthermore, mean compressive strength and ultra-sonic pulse velocity values for beams were found to be slightly higher than the columns. Although the rate of carbonation is the highest in the columns, suggesting that the most effected element after deterioration is the column. On further analysis of carbonation, it was found that carbonation effects more in the structure up to second floor and then later is the most in the roof.

Later in the study a few statistical test was performed assuming hypothesis to check if the hypothesis is valid. On observing the results deduced from the T-test it was observed that the structural elements mainly beam and column exhibit similarities in effects due to deterioration whereas slabs do not portray any similarities in deterioration with the beams and columns. This indicates the deterioration of a structure effects the beams and columns and slightly similar manner rather than the slab. After achieving this more statistical test like the one way anova was conducted to infer more accurate analysis and compare all the elements simultaneously as it is a limitation in the T test. On observing the results, it indicated that the rate of carbonation, loss in compressive strength and the ultra-sonic pulse velocity of all the structural elements are not related indicating that deterioration effects all the elements of the structure differently and assumption that rate of carbonation for a certain floor and the quality of concrete after deterioration for these floors would be same cannot be made while repairing and retrofitting the structure. The elements require to be assessed individually and provided the type of repair as per its requirements and not same for all elements. On observing the normal distribution graphs for slab, beams and column it can be induced that slabs are the least effected and columns and beams are the most effected due to deterioration. In the last part of statistical analysis, a correlation was plotted to observe the effect of carbonation, ultra-sonic pulse velocity and compressive strength with each other. It was observed that the carbonation is inversely proportional to compressive strength and ultra-sonic pulse velocity.

After data analysis this study progresses ahead with structural analysis and the modelling of the structure was done with the acquired data. In this part of the study only the columns and beams were focussed on as the slab didn't exhibit much deterioration, this is inferred from the data analysed. On the structural analysis it as observed that the parametric change in the properties does not affect or produce a significant difference in the design forces obtained in the beams although deterioration has a slightly more effect on the columns in comparison to the beams. Further on observing the rigidity it is observed that the structure loses a little bit of its rigidity but not enough to have a significant impact on time period and story displacement.

In the final stage of the study, the data obtained from the structural analysis and modelling is used to further study the loss of capacity in the columns as it is the most effected structural element due to deterioration. It has been found that the loss in capacity of the columns does not only depend upon the rate of carbonation and compressive strength but also depends upon the location of the column in the structure. The loss in capacity was observed the highest in the corner column then in the intermediate column and least in the middle columns. These observations help us to understand the structural elements behaviour due to deterioration in semi-arid environmental conditions

8.2 FUTURE SCOPE OF RESEARCH.

This research initially starts with data analysis of three structural elements and as its progresses further ahead in structural analysis it narrows it two structural elements namely beams and columns. Further as it progresses in the design comparison to find the loss in capacity of the structural elements it narrows down to only columns. The study can be extended further by inculcating other structural elements in the structural analysis and design stage.

Further the obtained conclusion and data from the research can be used to generate an optimum repair and retrofitting method in terms of enhancing the structural strength with economical cost.

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APPENDIX A

CORE TEST RESULTS FOR PROJRCT P054

I. Data obtained by Non-Destructive Tests and Destructive Tests of the structural Elements as regards to the quality and strength of concrete is concerned are shown below:

a) Carbonation & Chloride content;

Structure/Core Location	Depth of Carbonation (mm)	Chloride content % (as Cl) by mass of cement
Ground Floor Columns	25~30	0.41~0.61
Second Floor Columns	25~50	
Fourth Floor Columns	20~30	
Sixth Floor Columns	26~40	
Ground Floor Top Beam	27~35	
Second Floor Top Beam	20~40	
Fourth Floor Top Beam	15~35	
Sixth Floor Top Beam	15~29	
Ground Floor Roof Slab	20~30	
Second Floor Roof Slab	10~22	
Fourth Floor Roof Slab	10~20	
Sixth Floor Roof Slab	17~25	

Moderate Carbonation & High Chloride Content in the structural elements

b) Compressive Strength of Concrete Cores;

Core Location/structure	Core No.	Compressive Strength N/mm^2
Ground Floor Columns	1	18.5
"	2	12.9
“	3	11.5
Second Floor Columns	4	17.7
“	5	13.7
"	6	8.8
Fourth Floor Columns	7	24.0
“	8	13.6
"	9	15.5
SixthFloor Columns	10	17.5

"	11	12.6
"	12	15.4
Ground Floor Beam	1	13.4
"	2	10.7
"	3	7.0
Second Floor Top Beam	4	12.5
"	5	24.8
"	6	13.8
Fourth FloorTop Beam	7	15.2
"	8	20.6
"	9	12.2
Sixth Floor Top Beam	10	13.6
"	11	17.6
"	12	30.4
Ground Floor Roof Slab	1	29.5
"	2	24.0
"	3	19.8
Second Floor Roof Slab	4	27.4
"	5	36.6
"	6	35.6
Fourth Floor Roof Slab	7	28.0
"	8	27.6
"	9	38.8
Sixth Floor Roof Slab	10	29.4
"	11	26.1
"	12	20.2

ESTIMATED IN – SITU CUBE COMPRESSIVE STRENGTH CORRECTED FOR LENGTH DIAMETER RATIO & REINFORCEMENT (N/mm²) ACCORDING TO BS EN 12504-1 : 2009

c) Delamination Survey/ Ultrasonic Pulse Velocity Measurements (core samples)
Pulse velocities of columns vary from 2.9 to 4.3 km/sec., beams vary from 2.8 to 4.7 km/sec. and slab varies 3.9 to 5.3 km/sec. On the basis of Pulse velocities measurements at locations; the classification of quality of concrete for Columns and Beams are doubtful to good and slab is good (applies to core samples only).

C -CONCRETE DURABILITY

(i) - TESTING FOR CARBONATION

Because the alkalinity of the concrete normally protects reinforcement from corrosion, testing the concrete to discover where it has lost its alkalinity through Carbonation is a very useful guide to where and when the reinforcement may rust.

CORE NO.	STRUCTURE	CARBONATION DEPTH (mm)
1	*Ground floor columns*	30
2	"	25
3	"	27
4	*Second floor Column*	25
5	"	40
6	"	50
7	*Fourth floor Column*	20
8	"	30
9	"	27
10	*Sixth floor Column*	26
11	"	40
12	"	35
1	*Ground floor top beams*	27
2	"	30
3	"	35
4	*Second Floor top beam*	40
5	"	20
6	"	32
7	*Fourth floor top beam*	27
8	"	15
9	"	35
10	*Sixth floor top beam*	27
11	"	29
12	"	15
1	*Ground floor roof slab*	20
2	"	25
3	"	30
4	*Second floor roof slab*	22
5	"	10

6	"	15
7	*Fourth floor roof slab*	15
8	"	20
9	"	10
10	*Sixth floor roof slab*	20
11	"	17
12	"	25

الإمارات العربية المتحدة - حكومة الشارقة
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CENTRAL LAB DEPARTMENT
CONSTRUCTION MATERIALS LAB. SEC

E - QUALITY AND INTEGRITY OF CONCRETE

(i) DELAMINATION SURVEY / ULTRASONIC PULSE VELOCITY MEASUREMENTS

Principle

This method, which consists of measuring the time travel of a pulse/pulses through a measured path length in the material, may be used to advantage to assess the uniformity of field concrete, to indicate changes in characteristics in concrete and in the field structures to estimate the survey of deterioration, cracking or both.

The main use of the method is in quality control of similar concrete: both lack of compaction & a change in the water/cement ratio can be detected, Laboratory data is presented below:

LABORATORY PULSE VELOCITY

Core # (Columns)	Pulse Velocity (km/sec.)
1	3.8
2	3.2
3	3.1
4	3.5
5	3.1
6	2.9
7	4.3
8	3.2
9	3.3
10	3.6
11	3.2
12	3.5
Core # (Beams)	**Pulse Velocity (km/sec.)**
1	3.3
2	3.2
3	2.8
4	3.1
5	4.3
6	3.2
7	3.5
8	4.1
9	3.2
10	3.3
11	3.6
12	4.7
Core # (Slab)	**Pulse Velocity (km/sec.)**
1	4.5
2	4.2
3	3.9
4	4.5

5	4.8
6	5.0
7	4.6
8	4.6
9	5.3
10	4.7
11	4.2
12	3.9

The velocities as determined in the laboratory (cores) columns vary from 2.9 to 4.3 km/sec., beams vary from 3.1 to 4.7. km/sec. and slab vary from 3.9 to 5.3 km/sec. According to Whitehurst, classifications of the quality of concrete on the basis of pulse velocity are:

The quality of concrete is ***excellent*** *if the velocity is greater than 4.5 km/sec.*
The quality of concrete is ***good*** *being in the range of 3.5 to 4.5 km/sec.*
The quality of concrete is ***doubtful*** *being in the range of 3.0 to 3.5 km/sec.*
The quality of concrete is ***poor*** *being in the range of 2.0 to 3.0 km/sec.*

However, the pulse velocity cannot be used as a general indicator of compressive strength. The pulse velocity in a material depends upon its elastic properties which in turn are related to the Quality and Strength of the material.

UPV testing is certainly valuable in locating faults in concrete.

APPENDIX B

CORE TEST RESULTS FOR PROJRCT P056

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ENGINEERING DEPARTMENT
ENGINEERING MATERIALS LAB. SEC.

D- CONCRETE STRENGTH EVALUATION

(i) BY CORE SAMPLES

Cores of 75 mm diameter cut from Concrete Structural Elements can be tested in compression to measure the concrete strength; It can be weighed to measure its density and analyzed chemically to determine cement content and type, chloride content, water/cement, aggregate type and grading. When cut through the reinforcement, steel condition can be examined too.

Core Location/structure	*Core No.*	*Compressive Strength N/mm^2*
Ground Floor Columns	*1*	*9.3*
"	*2*	*6.9*
"	*3*	*15.3*
Third Floor Columns	*4*	*12.7*
"	*5*	*23.4*
"	*6*	*9.2*
Fifth Floor Columns	*7*	*18.8*
"	*8*	*14.3*
"	*9*	*9.4*
Ground Floor Top Beam	*1*	*16.0*
"	*2*	*16.5*
"	*3*	*27.1*
Third Floor Top Beam	*4*	*17.7*
"	*5*	*26.4*
"	*6*	*24.2*
Fifth FloorTop Beam	*10*	*23.0*
	11	*22.9*
	12	*19.5*
Ground Floor Roof Slab	*1*	*19.3*
"	*2*	*23.2*
"	*3*	*22.3*
Third Floor Roof Slab	*4*	*16.4*
"	*5*	*21.1*
"	*6*	*26.2*
Fifth Floor Roof Slab	*7*	*24.2*
"	*8*	*22.7*
"	*9*	*21.8*

FOR LENGTH DIAMETER RATIO & REINFORCEMENT (N/mm^2) ACCORDING TO BS EN 12504 – 1 :2009

SECTION 2 - NON-DESTRUCTIVE AND DESTRUCTIVE TESTING OF CONCRETE

C -CONCRETE DURABILITY

(i) - TESTING FOR CARBONATION

Because the alkalinity of the concrete normally protects reinforcement from corrosion, testing the concrete to discover where it has lost its alkalinity through Carbonation is a very useful guide to where and when the reinforcement may rust.

CORE NO.	STRUCTURE	CARBONATION DEPTH (mm)
1	*Ground floor columns*	40
2	"	52
3	"	40
4	*Third floor columns*	35
5	"	40
6	"	28
7	*Fifth Floor Column*	18
8	"	20
9	"	35
1	*Ground floor top beams*	25
2	"	20
3	"	25
4	*Third floor top beam*	25
5	"	30
6	"	28
7	*Fifth Floor top beam*	26
8	"	30
9	"	28
1	*Ground floor roof slab*	19
2	"	25
3	"	28
4	*Third floor roof slab*	22
5	"	26
6	"	29
7	*Fifth Floor Roof Slab*	28
8	"	25
9	"	20

E - QUALITY AND INTEGRITY OF CONCRETE

(i) DELAMINATION SURVEY / ULTRASONIC PULSE VELOCITY MEASUREMENTS

Principle

This method, which consists of measuring the time travel of a pulse/pulses through a measured path length in the material, may be used to advantage to assess the uniformity of field concrete, to indicate changes in characteristics in concrete and in the field structures to estimate the survey of deterioration, cracking or both.

The main use of the method is in quality control of similar concrete: both lack of compaction & a change in the water/cement ratio can be detected, Laboratory data is presented below:

LABORATORY PULSE VELOCITY

Core # (Columns)	Pulse Velocity (km/sec.)
1	3.0
2	2.6
3	3.4
4	3.0
5	4.3
6	2.8
7	3.9
8	3.4
9	2.9
Core # (Beams)	**Pulse Velocity (km/sec.)**
1	3.4
2	3.5
3	4.5
4	3.6
5	4.5
6	4.5
7	4.3
8	4.2
9	4.1
Core # (Slab)	**Pulse Velocity (km/sec.)**
1	4.0
2	4.3
3	4.1
4	3.8
5	4.2
6	4.5
7	4.5
8	4.3
9	4.2

APPENDIX C

CORE TEST RESULTS FOR PROJRCT P068

الإمارات العربية المتحدة – حكومة الشارقة
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United Arab Emirates – Sharjah Government
SHARJAH CITY MUNICIPALITY

~~CENTRAL LAB DEPARTMENT~~
CONSTRUCTION MATERIALS LAB. SEC.

b) Compressive Strength of Concrete Cores;

Core Location/structure	Core No.	Compressive Strength N/mm^2
Ground Floor Columns	1	10.4
"	2	18.9
"	3	32.3
First Floor Columns	4	16.5
"	5	13.1
"	6	23.9
Second Floor Columns	7	21.5
"	8	19.9
"	9	21.6
Third Floor Columns	10	13.2
"	11	14.7
"	12	12.3
Fourth Floor Columns	13	15.7
"	14	25.4
"	15	15.9
Fifth Floor Columns	16	21.3
"	17	17.4
"	18	28.7
Sixth Floor Columns	19	14.4
"	20	22.8
"	21	13.9
Ground Floor Beam	1	12.7
"	2	17.7
"	3	17.6
First Floor Top Beam	4	18.8
"	5	14.2
"	6	14.6
Second Floor Top Beam	7	22.6
"	8	12.8
"	9	16.2
Third Floor Top Beam	10	9.6
"	11	10.9
"	12	10.3
Fourth Floor Top Beam	13	13.2
"	14	11.0
"	15	9.1
Fifth Floor Top Beam	16	18.6
"	17	13.4

	18	*18.4*
Sixth Floor Top Beam	*19*	*16.6*
"	*20*	*15.2*
"	*21*	*12.3*
Ground Floor Roof Slab	*1*	*36.1*
"	*2*	*29.7*
"	*3*	*38.0*
First Floor Roof Slab	*4*	*22.4*
"	*5*	*25.4*
"	*6*	*24.9*
Second Floor Roof Slab	*7*	*23.7*
"	*8*	*30.6*
"	*9*	*25.5*
Third Floor Roof Slab	*10*	*16.4*
"	*11*	*22.1*
"	*12*	*21.9*
Fourth Floor Roof Slab	*13*	*31.0*
"	*14*	*33.2*
"	*15*	*28.8*
Fifth Floor Roof Slab	*16*	*24.0*
"	*17*	*24.5*
"	*18*	*31.9*
Sixth Floor Roof Slab	*19*	*24.4*
"	*20*	*29.8*
"	*21*	*35.5*

ESTIMATED IN – SITU CUBE COMPRESSIVE STRENGTH CORRECTED FOR LENGTH DIAMETER RATIO & REINFORCEMENT (N/mm^2) ACCORDING TO BS EN 12504-1 : 2009

c) Delamination Survey/ Ultrasonic Pulse Velocity Measurements (core samples)
Pulse velocities of columns vary from 3.2to 4.8 km/sec., beams vary from 3.0 to 3.9 km/sec. and slab varies from 3.6 to 5.1 km/sec. On the basis of Pulse velocities measurements at locations; the classification of quality of concrete for Columns & Beam are doubtful to good and Slab are good (applies to core samples only).

2. <u>External RCC Elements</u>
In the aggressive external environment general conditions of exposure, in absence of proper maintenance, High carbonation , High chloride content resulted in cracking and spalling of columns which were repaired in past and the walls were painted from both the side externally as well as internally long back hence the exact extent of deterioration is not known .

C -CONCRETE DURABILITY

(i) - TESTING FOR CARBONATION

Because the alkalinity of the concrete normally protects reinforcement from corrosion, testing the concrete to discover where it has lost its alkalinity through Carbonation is a very useful guide to where and when the reinforcement may rust.

CORE NO.	STRUCTURE	CARBONATION DEPTH (mm)
1	*Ground floor columns*	25
2	"	30
3	"	17
4	*First floor columns*	27
5	"	32
6	"	22
7	*Second floor columns*	28
8	"	42
9	"	37
10	*Third floor columns*	25
11	"	27
12	'	24
13	*Fourth floor columns*	20
14	"	15
15	"	19
16	*Fifth floor columns*	26
17	"	30
18	"	22
19	*Sixth floor columns*	25
20	"	21
21	"	28
1	*Ground floor top beams*	32
2	"	30
3	"	25
4	*First Floor top beam*	18
5	"	21
6	"	24
7	*Second floor top beam*	29
8	"	31
9	"	34

10	*Third floor top beam*	48
11	"	47
12	"	36
13	*Fourth floor top beam*	28
14	"	36
15	"	40
16	*Fifth floor top beam*	29
17	"	36
18	"	26
19	*Sixth floor top beam*	27
20	"	29
21	"	34
1	*Ground floor roof slab*	36
2	"	32
3	"	28
4	*First floor Roof Slab*	33
5	"	58
6	"	40
7	*Second floor roof slab*	30
8	"	36
9	"	29
10	*Third floor roof slab*	38
11	"	32
12	"	34
13	*Fourth floor roof slab*	28
14	"	36
15	"	38
16	*Fifth floor roof slab*	24
17	"	25
18	"	22
19	*Sixth floor roof slab*	30
20	"	33
21	"	35

Please note that concrete that is contaminated with chlorides may fail to protect the reinforcement even if no alkalinity has been lost through carbonation. The test for Carbonation at all core locations indicates that the depth of carbonation of columns vary from 15 to 42 mm, Beams vary from 18 to 48 mm and slab vary from 22 to 58 mm .

E - QUALITY AND INTEGRITY OF CONCRETE-

(i) DELAMINATION SURVEY / ULTRASONIC PULSE VELOCITY MEASUREMENTS

Principle

This method, which consists of measuring the time travel of a pulse/pulses through a measured path length in the material, may be used to advantage to assess the uniformity of field concrete, to indicate changes in characteristics in concrete and in the field structures to estimate the survey of deterioration, cracking or both.

The main use of the method is in quality control of similar concrete: both lack of compaction & a change in the water/cement ratio can be detected, Laboratory data is presented below:

LABORATORY PULSE VELOCITY

Core # (Columns)	Pulse Velocity (km/sec.)
1	3.2
2	3.8
3	4.8
4	3.6
5	3.3
6	4.5
7	4.3
8	3.9
9	4.3
10	3.3
11	3.4
12	3.2
13	3.6
14	4.6
15	3.6
16	4.4
17	3.7
18	4.7
19	3.4
20	4.4
21	3.2
Core # (Beams)	**Pulse Velocity (km/sec.)**
1	3.2
2	3.7
3	3.7
4	3.8
5	3.3
6	3.3

7	3.9
8	3.3
9	3.5
10	3.0
11	3.2
12	3.2
13	3.2
14	3.1
15	3.0
16	3.8
17	3.4
18	3.8
19	3.6
20	3.5
21	3.2
Core # (Slab)	**Pulse Velocity (km/sec.)**
1	4.9
2	4.7
3	5.1
4	4.4
5	4.6
6	4.5
7	4.4
8	4.8
9	4.6
10	3.6
11	4.5
12	4.4
13	4.8
14	4.9
15	4.7
16	4.5
17	4.6
18	4.8
19	4.6
20	4.8
21	4.9

The velocities as determined in the laboratory (cores) columns vary from 3.2 to 4.8 km/sec., beams vary from 3.0 to 3.9 km/sec. and slab vary from 3.6 to 5.1 km/sec. According to Whitehurst, classifications of the quality of concrete on the basis of pulse velocity are:

The quality of concrete is ***excellent*** *if the velocity is greater than 4.5 km/sec.*
The quality of concrete is ***good*** *being in the range of 3.5 to 4.5 km/sec.*
The quality of concrete is ***doubtful*** *being in the range of 3.0 to 3.5 km/sec.*

APPENDIX D

CORE TEST RESULTS FOR PROJRCT P069

الإمارات العربية المتحدة - حكومة الشارقة
بلدية مدينة الشارقة
United Arab Emirates – Sharjah Government
SHARJAH CITY MUNICIPALITY

CENTRAL LAB DEPARTMENT
CONSTRUCTION MATERIALS LAB. SEC.

<u>SECTION 2 - NON-DESTRUCTIVE AND DESTRUCTIVE TESTING OF CONCRETE</u>

<u>C -CONCRETE DURABILITY</u>

<u>(i) - TESTING FOR CARBONATION</u>

Because the alkalinity of the concrete normally protects reinforcement from corrosion, testing the concrete to discover where it has lost its alkalinity through Carbonation is a very useful guide to where and when the reinforcement may rust.

CORE NO.	STRUCTURE	CARBONATION DEPTH (mm)
1	*Ground floor columns*	50
2	"	40
3	“	35
4	*First floor columns*	48
5	“	55
6	“	55
7	*Second floor columns*	30
8	"	31
9	“	27
10	*Third floor columns*	25
11	"	20
12	“	19
13	*Fourth floor columns*	50
14		35
15	"	45
16	*Fifth floor columns*	40
17	“	30
18	“	35
1	*First floor top beams*	30
2	“	15
3	“	25
4	*Second Floor top beam*	40
5	"	35
6	“	30
7	*Third floor top beam*	18
8	"	20
9	“	25
10	*Fourth floor top beam*	23
11	"	17
12	“	20

13	*Fifth Floor top beam*	20
14	"	25
15	"	35
1	*Ground floor roof slab*	7
2	"	10
3	"	14
4	"	10
5	"	15
6	"	10
7	*First Floor Roof Slab*	12
8	"	15
9	"	10
10	*Second floor roof slab*	13
11	"	15
12	"	12
13	*Third floor roof slab*	16
14	"	19
15	"	20
16	*Fourth floor roof slab*	7
17	"	9
18	"	10
19	*Fifth floor roof slab*	10
20	"	11
21	"	15

Please note that concrete that is contaminated with chlorides may fail to protect the reinforcement even if no alkalinity has been lost through carbonation. The test for Carbonation at all core locations indicates that the depth of carbonation of columns vary from 19 to 55 mm, Beams vary from 15 to 40 mm and slab vary from 07 to 20 mm .

D- CONCRETE STRENGTH EVALUATION

(i) BY CORE SAMPLES

Cores of 75 mm diameter cut from Concrete Structural Elements can be tested in compression to measure the concrete strength; It can be weighed to measure its density and analyzed chemically to determine cement content and type, chloride content, water/cement, aggregate type and grading. When cut through the reinforcement, steel condition can be examined too.

Core Location/structure	*Core No.*	*Compressive Strength N/mm^2*
Ground Floor Columns	*1*	7.3
"	*2*	23.4
"	*3*	22.3
First Floor Columns	*4*	14.5
"	*5*	15.6
"	*6*	17.2
Second Floor Columns	*7*	11.2
"	*8*	23.9
"	*9*	18.2
Third Floor Columns	*10*	25.1
"	*11*	20.9
"	*12*	13.9
Fourth Floor Columns	*13*	11.3
"	*14*	16.3
"	*15*	Damaged Core
Fifth FloorColumns	*16*	11.0
"	*17*	16.2
"	*18*	*14.6*
First floor top beams	*1*	15.3
"	*2*	22.4
"	*3*	16.2
Second Floor top beam	*4*	15.5
"	*5*	19.9
"	*6*	19.6
Third floor top beam	*7*	13.4
"	*8*	13.8
"	*9*	14.3
Fourth floor top beam	*10*	18.7
"	*11*	12.6
"	*12*	22.9
Fifth Floor top beam	*13*	17.4

"	*14*	*10.5*
"	*15*	*8.5*
Ground Floor Roof Slab	*1*	**31.2**
"	*2*	**19.5**
"	*3*	**Damaged Core**
"	*4*	**41.0**
"	*5*	**17.9**
"	*6*	**34.4**
First Floor Roof Slab	*7*	**29.1**
"	*8*	**37.5**
"	*9*	**29.9**
Second Floor Roof Slab	*10*	**24.1**
"	*11*	**19.7**
"	*12*	**20.3**
Third Floor Roof Slab	*13*	**15.4**
"	*14*	**15.8**
"	*15*	**12.8**
Fourth Floor Roof Slab	*16*	**29.3**
"	*17*	**14.9**
"	*18*	**32.6**
Fifth Floor Roof Slab	*19*	**20.2**
"	*20*	**18.2**
"	*21*	*15.2*

FOR LENGTH DIAMETER RATIO & REINFORCEMENT (N/mm²) ACCORDING TO BS EN 12504 – 1 :2009

Compressive Strength of Columns vary from 7.3~ 25.1 N/mm^2 Beams vary 8.5 ~ 22.9 N/mm^2 and Slab vary from 12.8 ~ 41.0 N/mm^2 .
- More non-destructive techniques are employed to determine the quality of concrete in general

(For details see the attached Core Compressive Strength test results) after section-3

E - QUALITY AND INTEGRITY OF CONCRETE-

(i) DELAMINATION SURVEY / ULTRASONIC PULSE VELOCITY MEASUREMENTS

Principle

This method, which consists of measuring the time travel of a pulse/pulses through a measured path length in the material, may be used to advantage to assess the uniformity of field concrete, to indicate changes in characteristics in concrete and in the field structures to estimate the survey of deterioration, cracking or both.

The main use of the method is in quality control of similar concrete: both lack of compaction & a change in the water/cement ratio can be detected, Laboratory data is presented below:

LABORATORY PULSE VELOCITY

Core # (Columns)	**Pulse Velocity (km/sec.)**
1	2.7
2	4.5
3	4.4
4	3.5
5	3.6
6	3.7
7	3.1
8	4.2
9	3.8
10	4.4
11	4.0
12	3.5
13	3.1
14	3.6
15	2.7
16	3.2
17	3.6
18	3.5
Core # (Beams)	**Pulse Velocity (km/sec.)**
1	3.5
2	4.3
3	3.6
4	3.5
5	4.0
6	3.9
7	3.3
8	3.4
9	3.5
10	3.8
11	3.2

12	4.3
13	3.7
14	3.1
15	2.9
Core # (Slab)	**Pulse Velocity (km/sec.)**
1	4.8
2	3.9
3	3.5
4	5.3
5	3.8
6	5.1
7	4.7
8	5.0
9	4.7
10	4.4
11	3.9
12	4.0
13	3.4
14	3.5
15	3.2
16	4.7
17	3.6
18	4.8
19	4.2
20	3.9
21	3.5

The velocities as determined in the laboratory (cores) columns vary from 2.7 to 4.5 km/sec., beams vary from 2.9 to 4.3 km/sec. and slab vary from 3.2 to 5.3 km/sec. According to Whitehurst, classifications of the quality of concrete on the basis of pulse velocity are:

The quality of concrete is ***excellent*** *if the velocity is greater than 4.5 km/sec.*

The quality of concrete is ***good*** *being in the range of 3.5 to 4.5 km/sec.*

The quality of concrete is ***doubtful*** *being in the range of 3.0 to 3.5 km/sec.*

The quality of concrete is ***poor*** *being in the range of 2.0 to 3.0 km/sec.*

However, the pulse velocity cannot be used as a general indicator of compressive strength. The pulse velocity in a material depends upon its elastic properties which in turn are related to the Quality and Strength of the material.

UPV testing is certainly valuable in locating faults in concrete.

APPENDIX E

INTERACTION DIAGRAM FOR COLUMN DESIGN

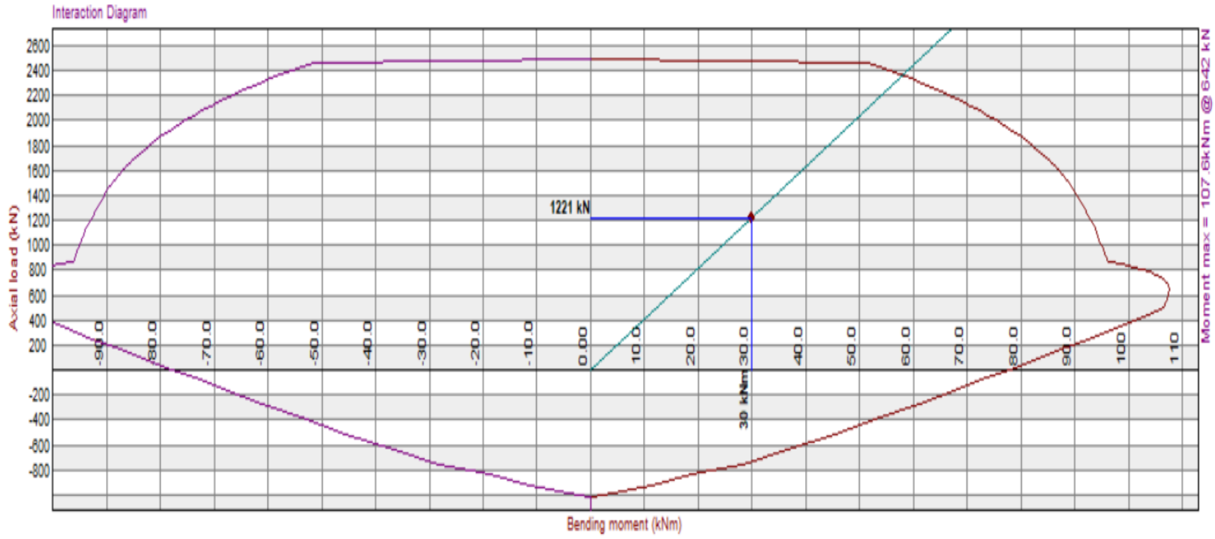

Figure E- 1 Interaction Diagram of Non Deteriorated Corner Column Second Floor

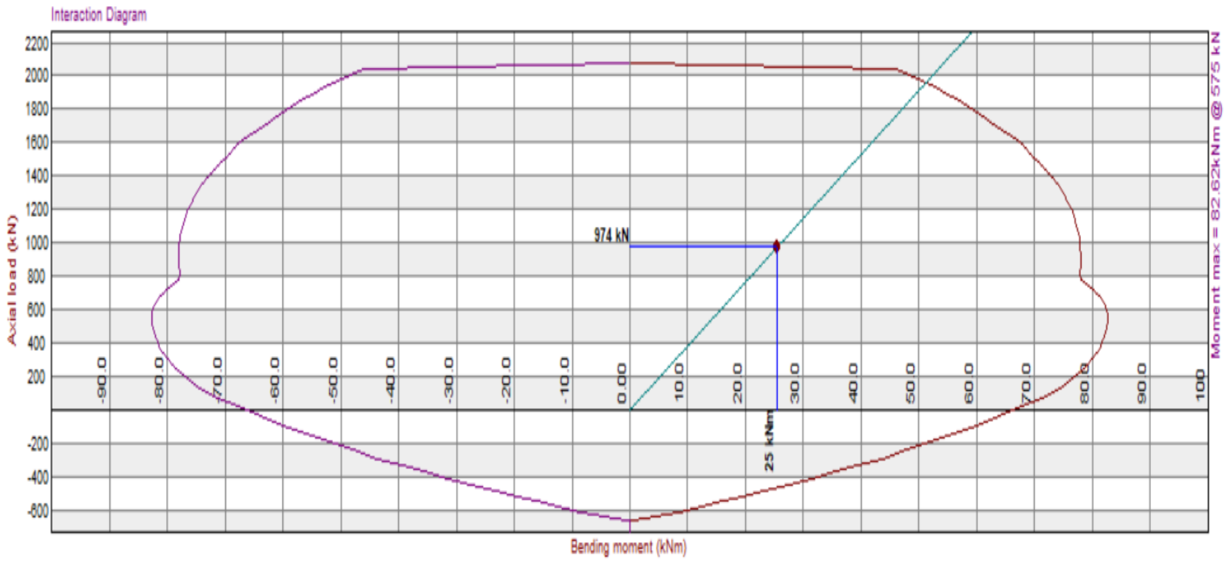

Figure E- 2 Interaction Diagram of Non Deteriorated Corner Column Third Floor

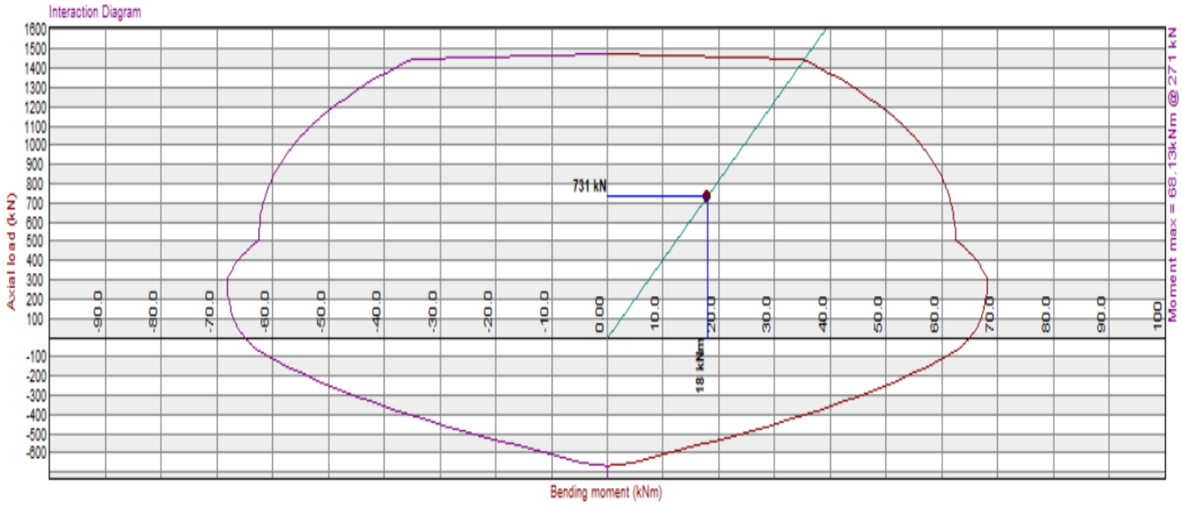

Figure E- 3 Interaction Diagram of Deteriorated Corner Column Fourth Floor

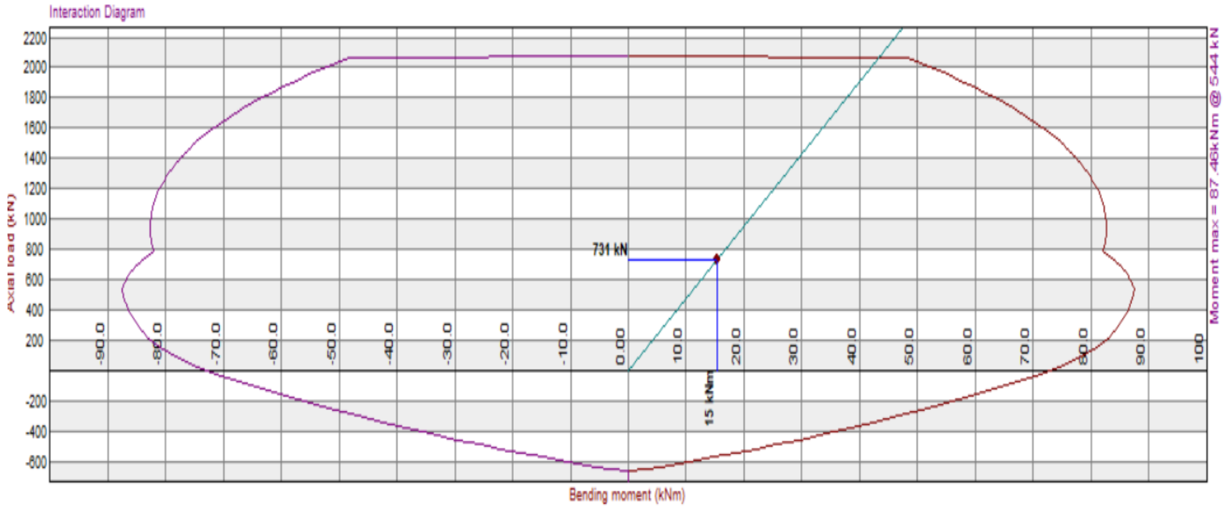

Figure E- 4 Interaction Diagram of Deteriorated Corner Column Fourth Floor

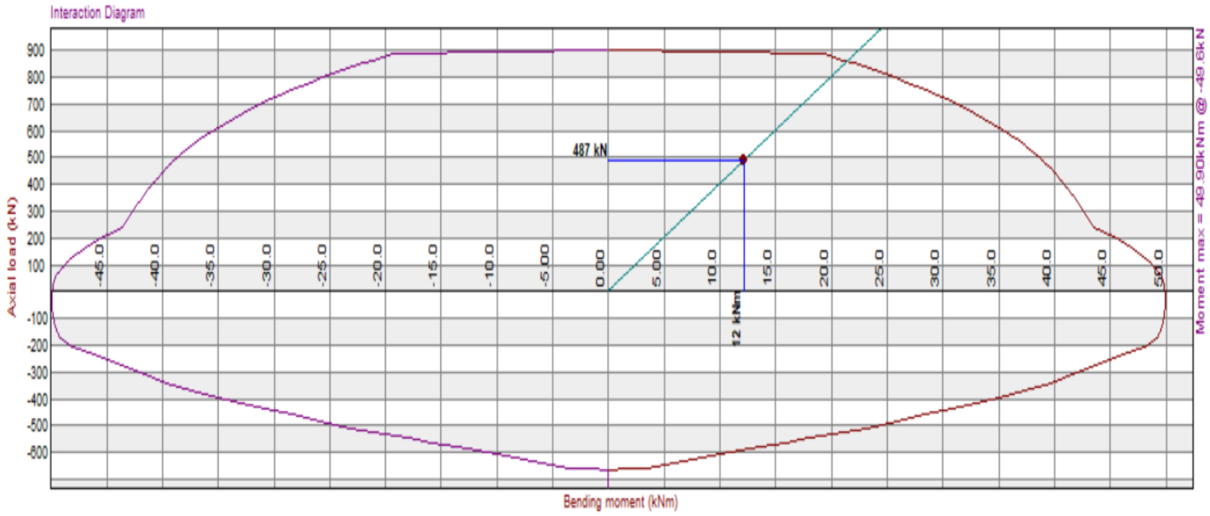

Figure E- 5 Interaction Diagram of Deteriorated Corner Column Fifth Floor

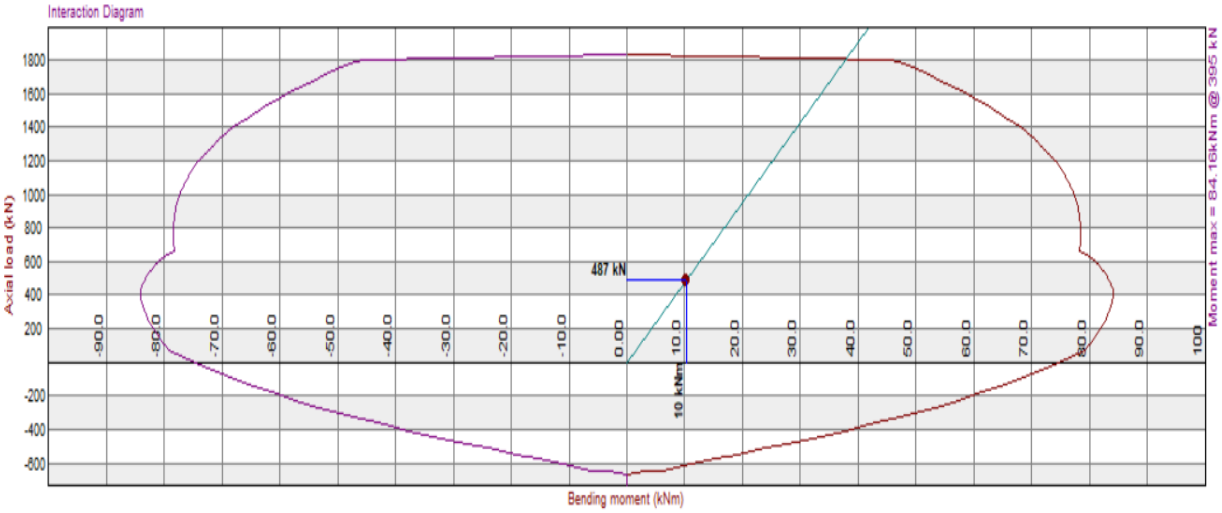

Figure E- 6 Interaction Diagram of Non Deteriorated Corner Column Fifth Floor

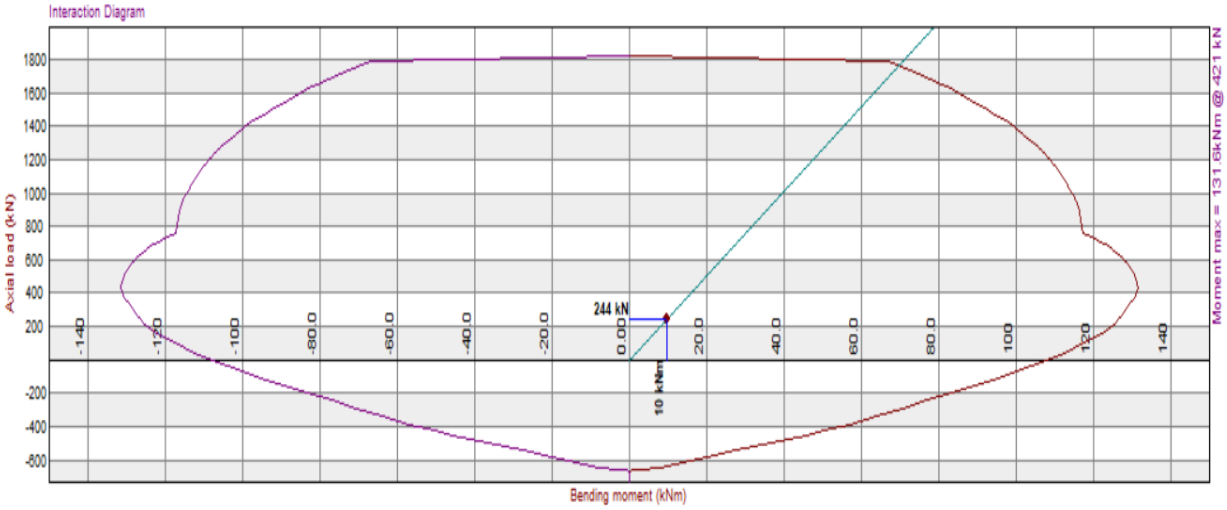

Figure E- 7 Interaction Diagram of Non Deteriorated Corner Column Sixth Floor

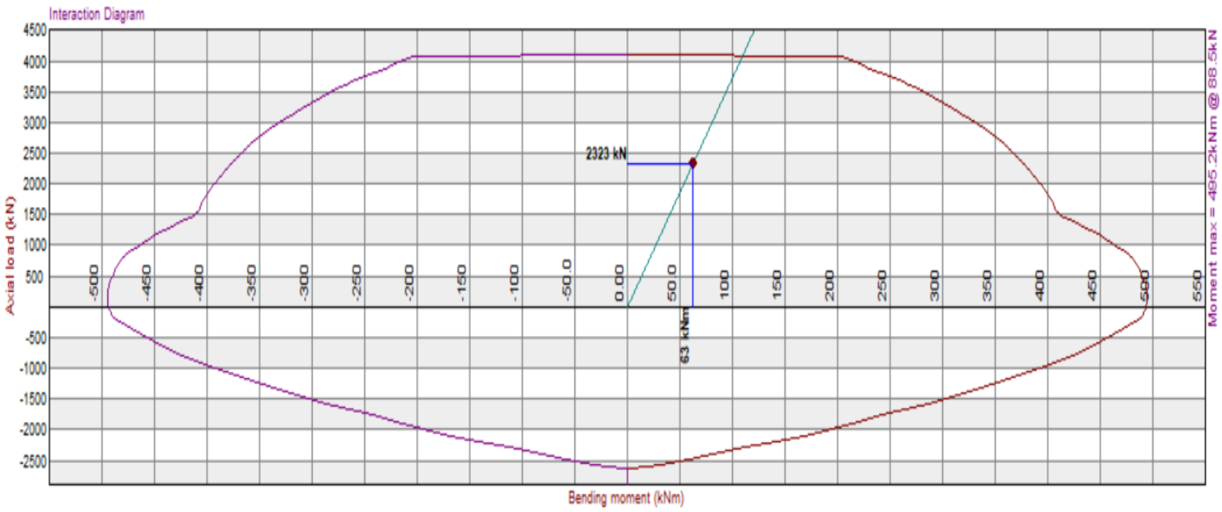

Figure E- 8 Interaction Diagram of Deteriorated Intermediate Column Ground Floor

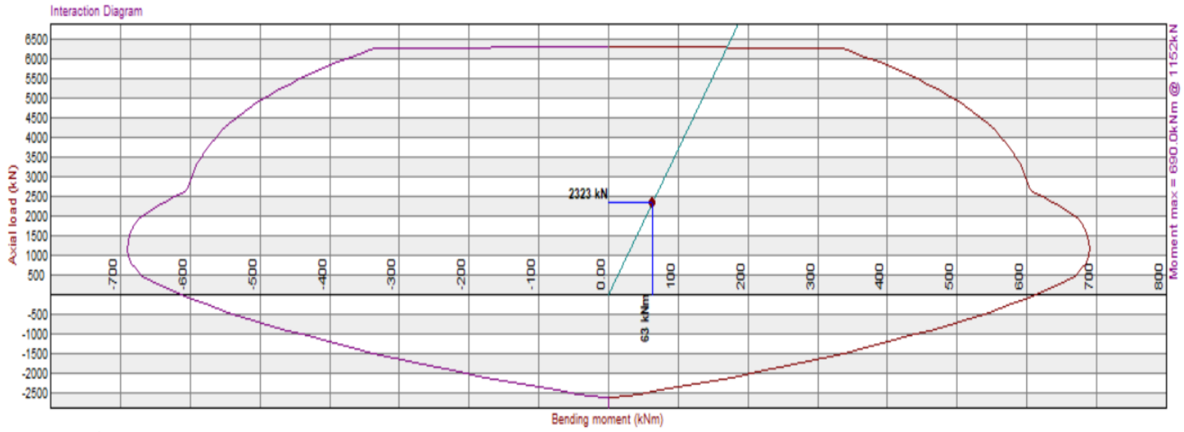

Figure E- 9 Interaction Diagram of Non Deteriorated Intermediate Column Ground Floor

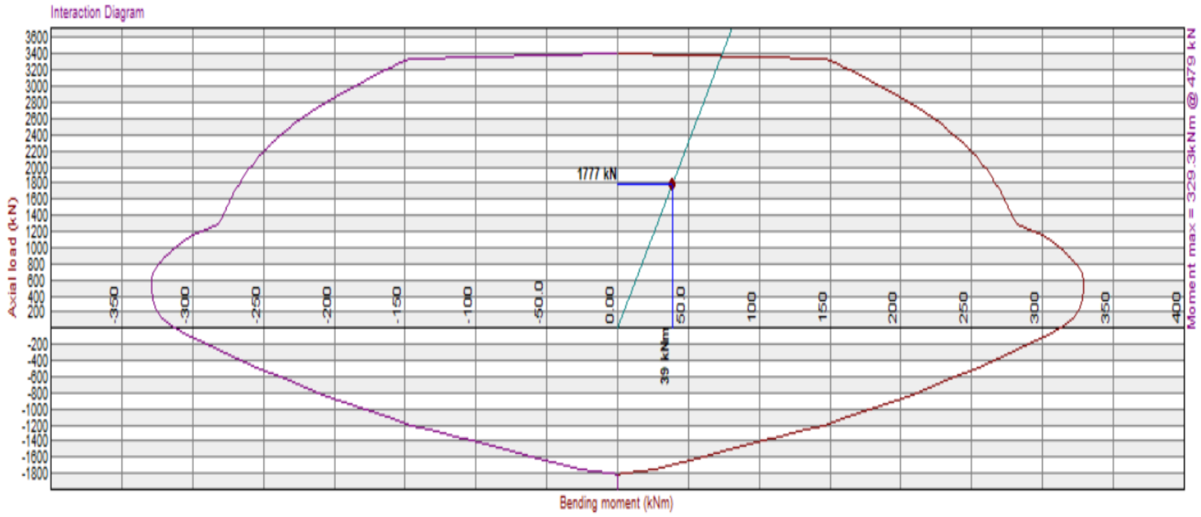

Figure E- 10 Interaction Diagram of Non Deteriorated Intermediate Column First Floor

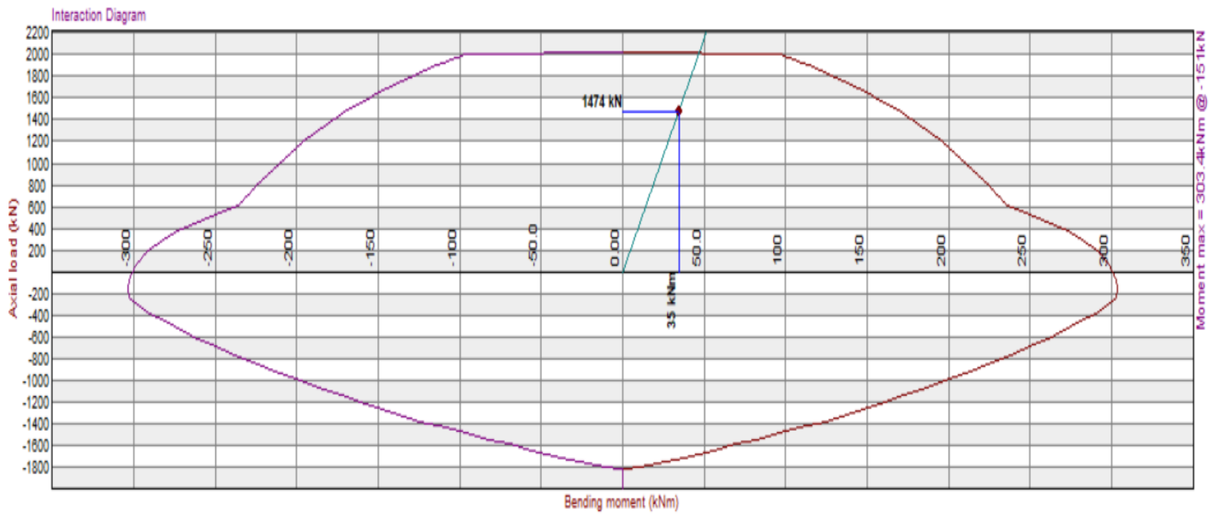

Figure E- 11 Interaction Diagram of Deteriorated Intermediate Column Second Floor

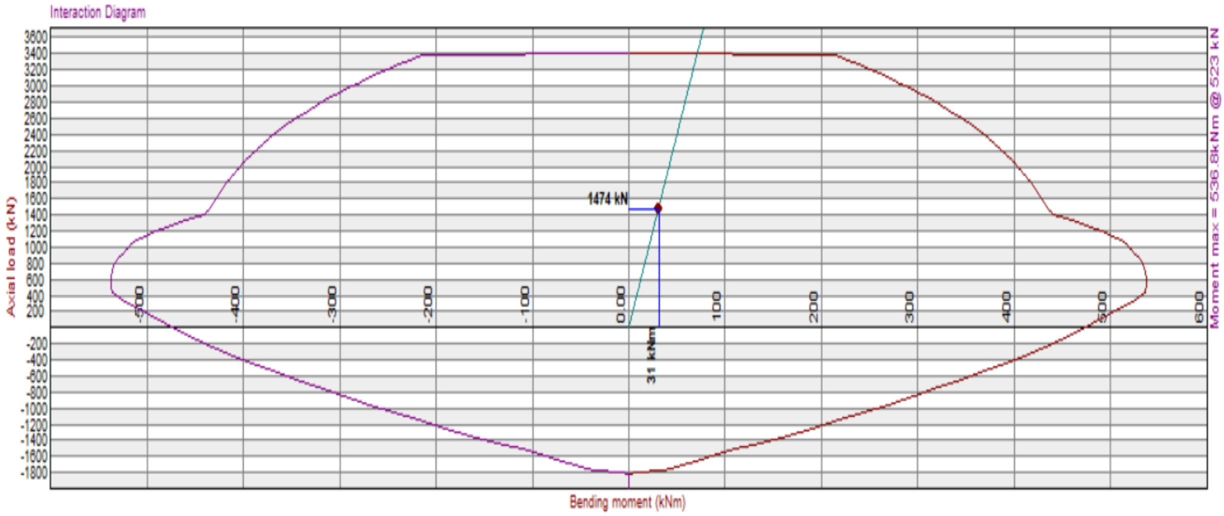

Figure E- 12 Interaction Diagram of Non Deteriorated Intermediate Column Second Floor

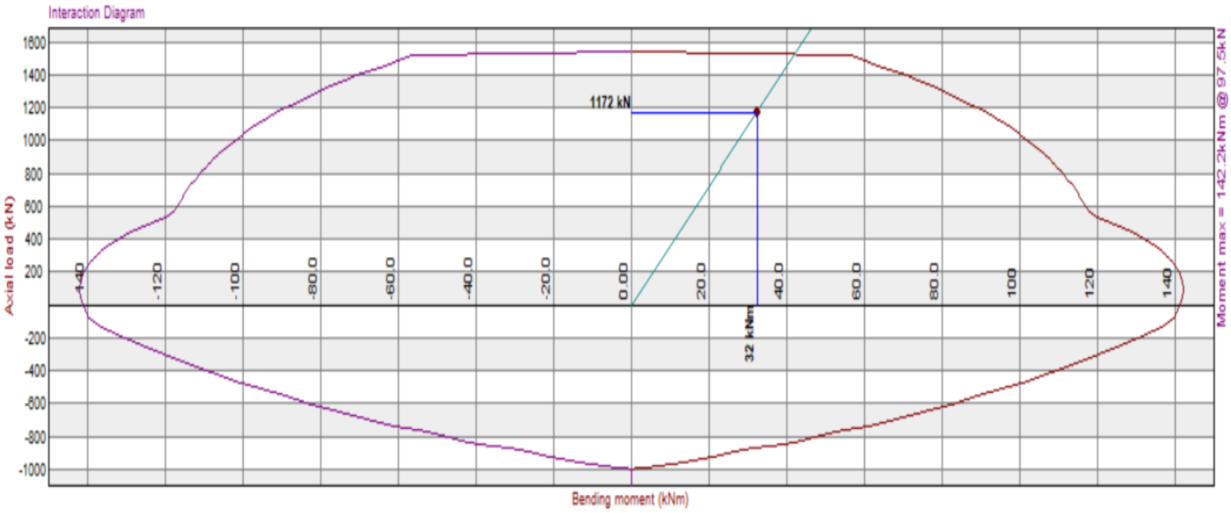

Figure E- 13 Interaction Diagram of Deteriorated Intermediate Column Third Floor

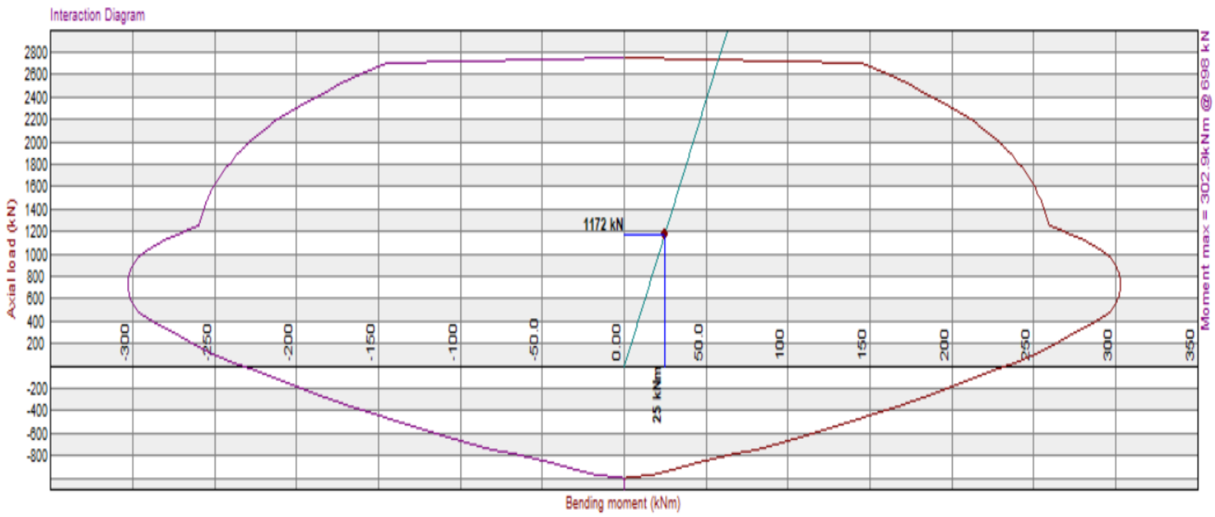

Figure E- 14 Interaction Diagram of Non Deteriorated Intermediate Column Third Floor

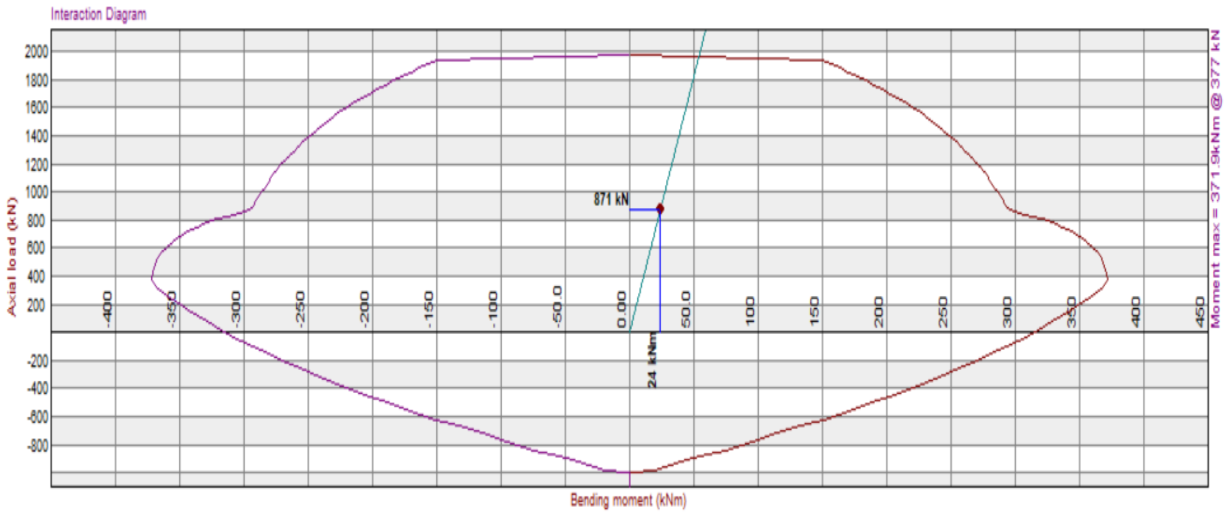

Figure E- 15 Interaction Diagram of Deteriorated Intermediate Column Fourth Floor

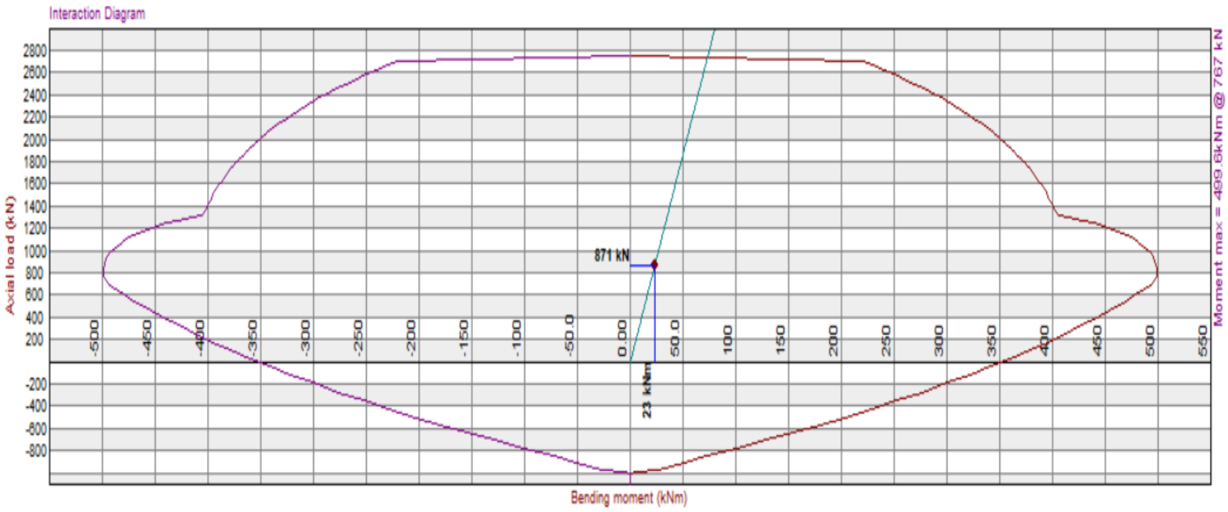

Figure E- 16 Interaction Diagram of Non Deteriorated Intermediate Column Fourth Floor

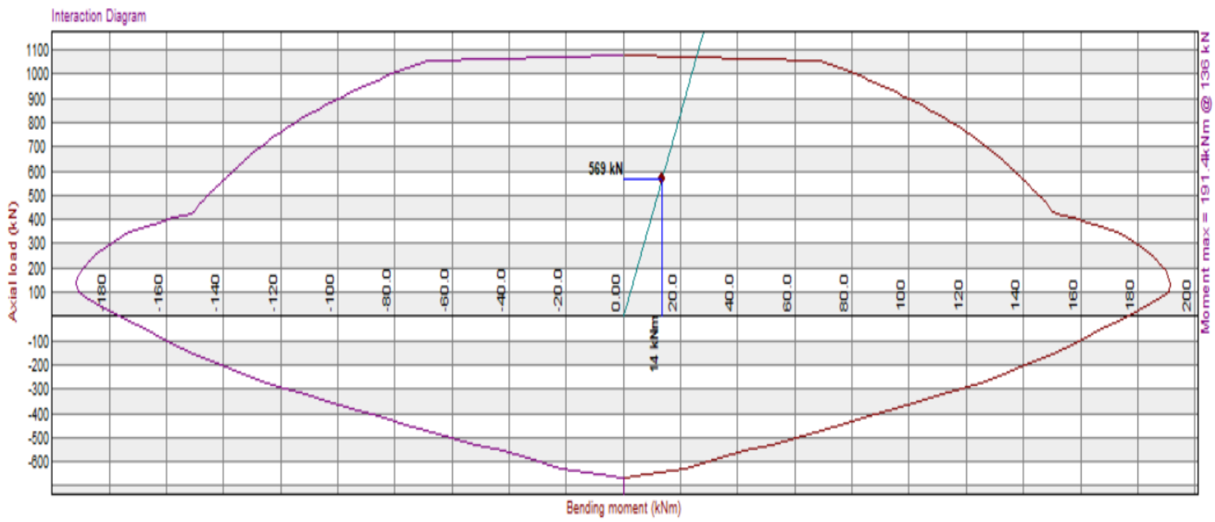

Figure E- 17 Interaction Diagram of Deteriorated Intermediate Column Fifth Floor

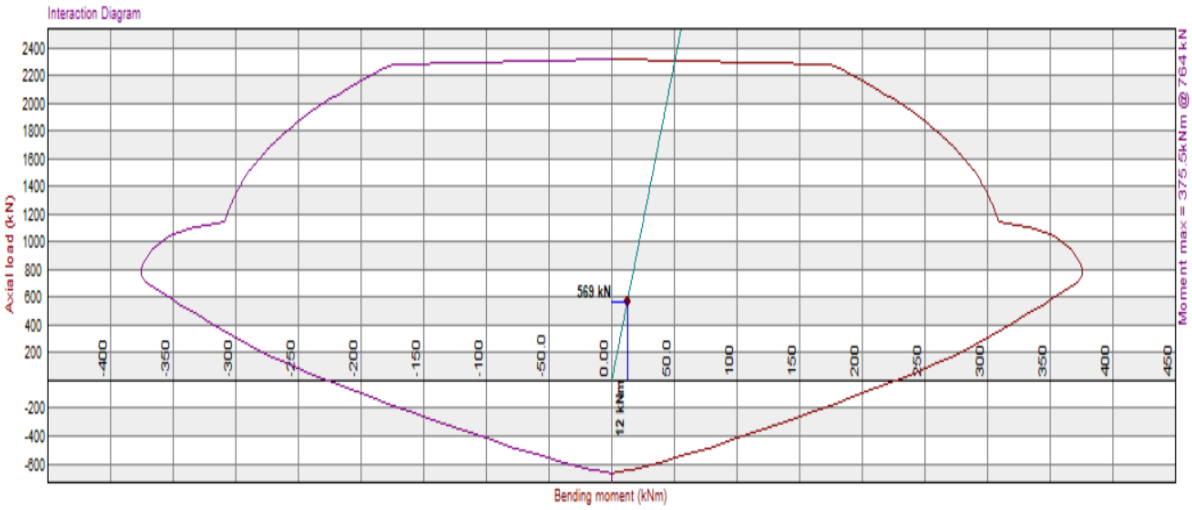

Figure E- 18 Interaction Diagram of Non Deteriorated Intermediate Column Fifth Floor

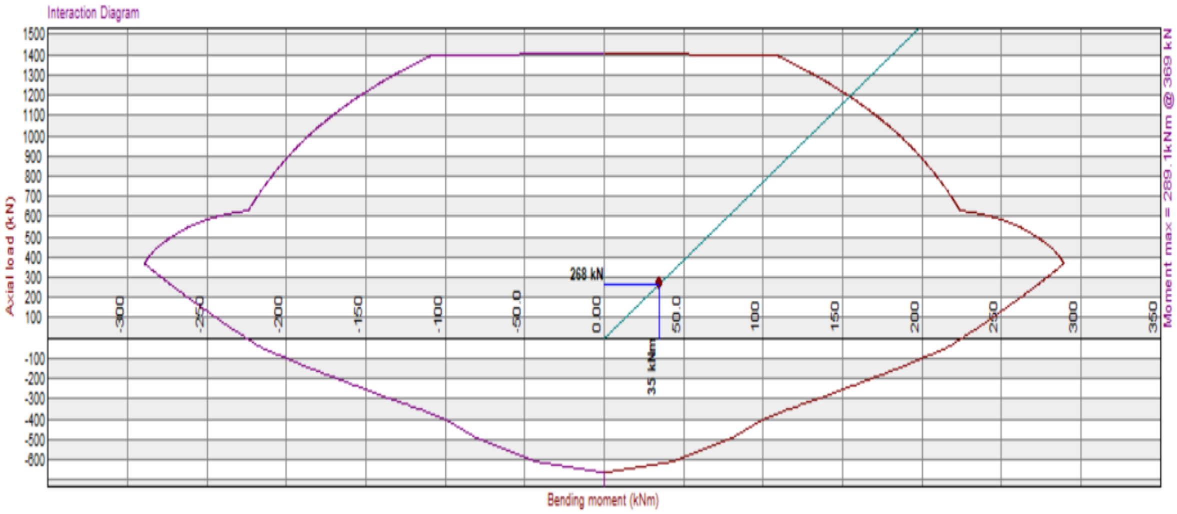

Figure E- 19 Interaction Diagram of Deteriorated Intermediate Column Sixth Floor

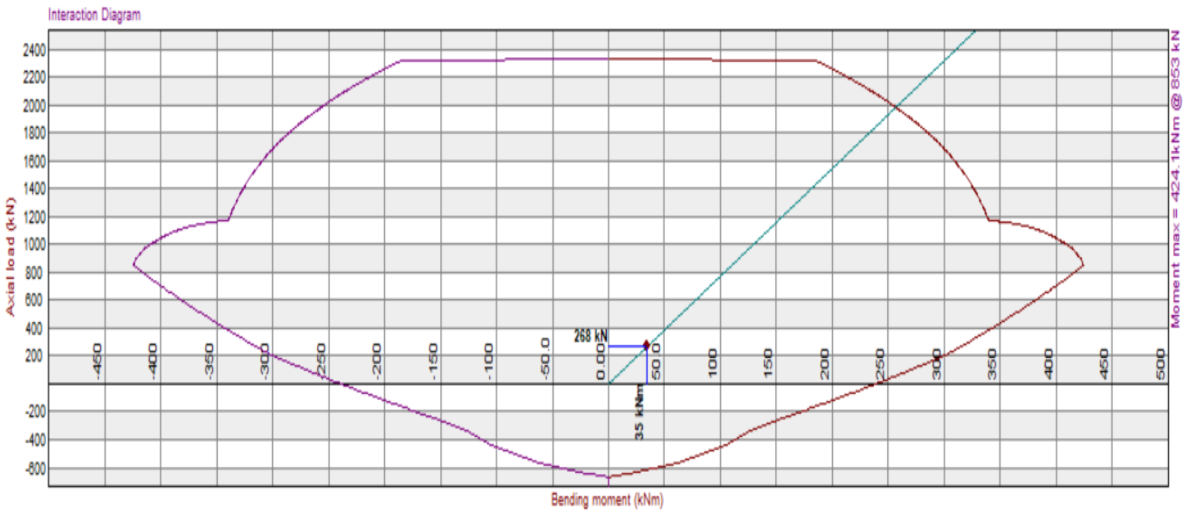

Figure E- 20 Interaction Diagram of Non Deteriorated Intermediate Column Sixth Floor

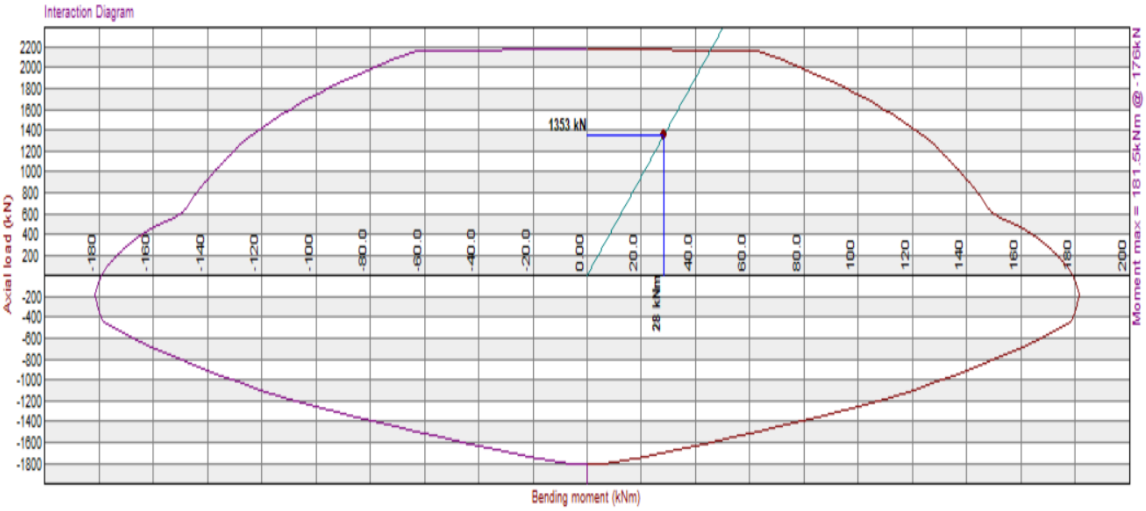

Figure E- 21 Interaction Diagram of Deteriorated Middle Column First Floor

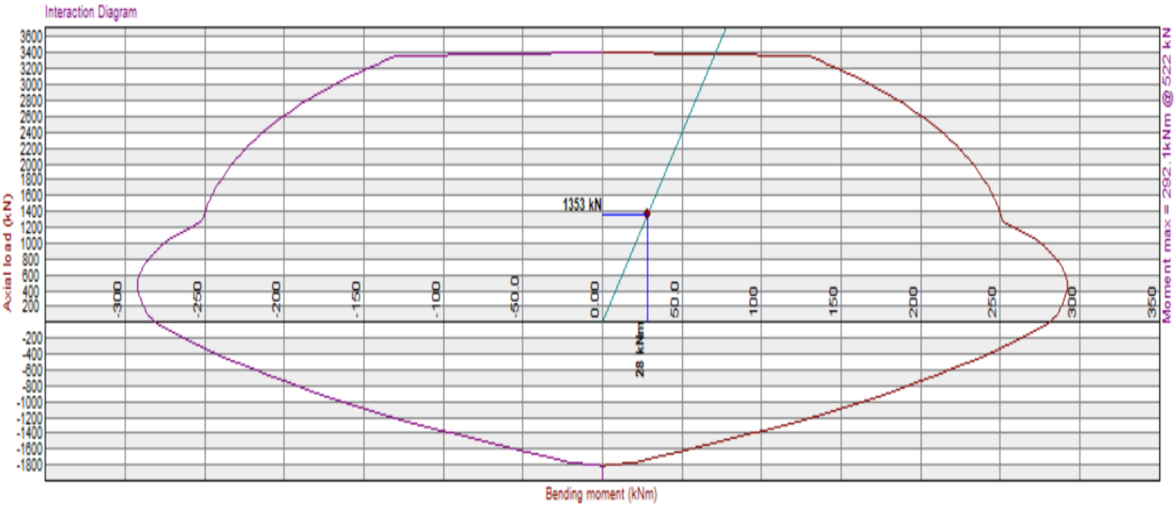

Figure E- 22 Interaction Diagram of Non Deteriorated Middle Column First Floor

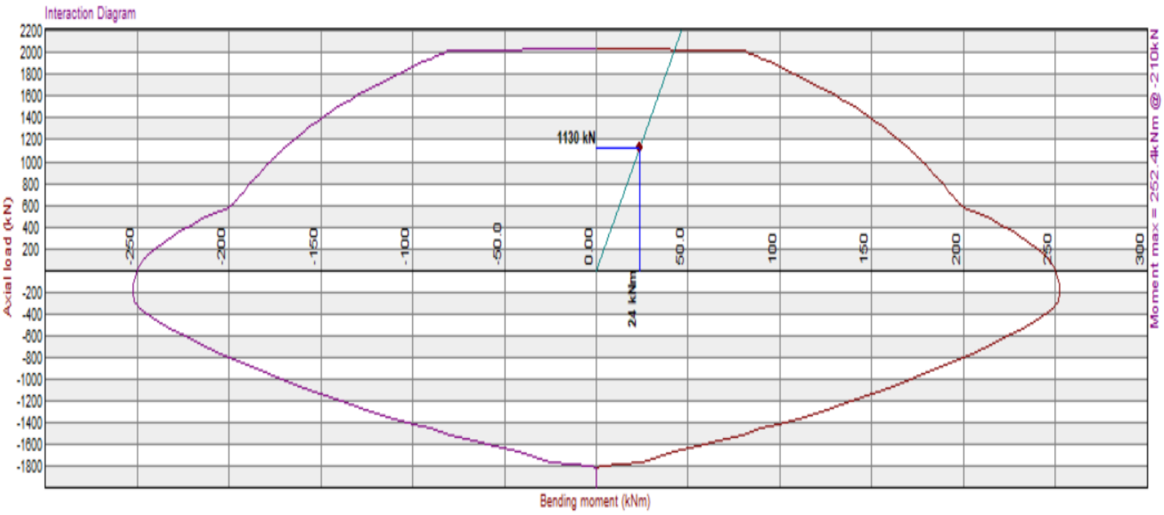

Figure E- 23 Interaction Diagram of Deteriorated Middle Column Second Floor

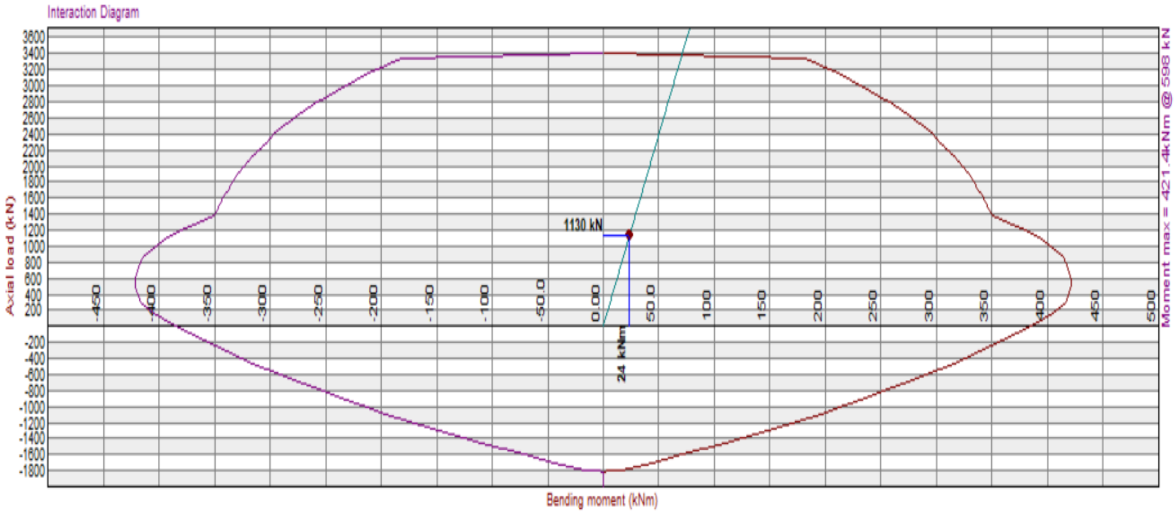

Figure E- 24 Interaction Diagram of Non Deteriorated Middle Column Second Floor

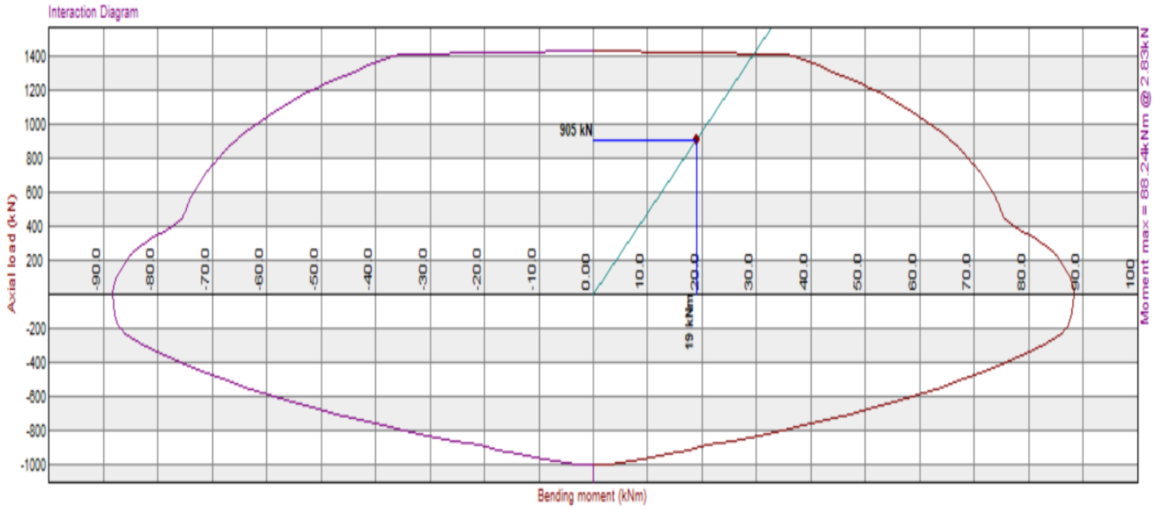

Figure E- 25Interaction Diagram of Deteriorated Middle Column Third Floor

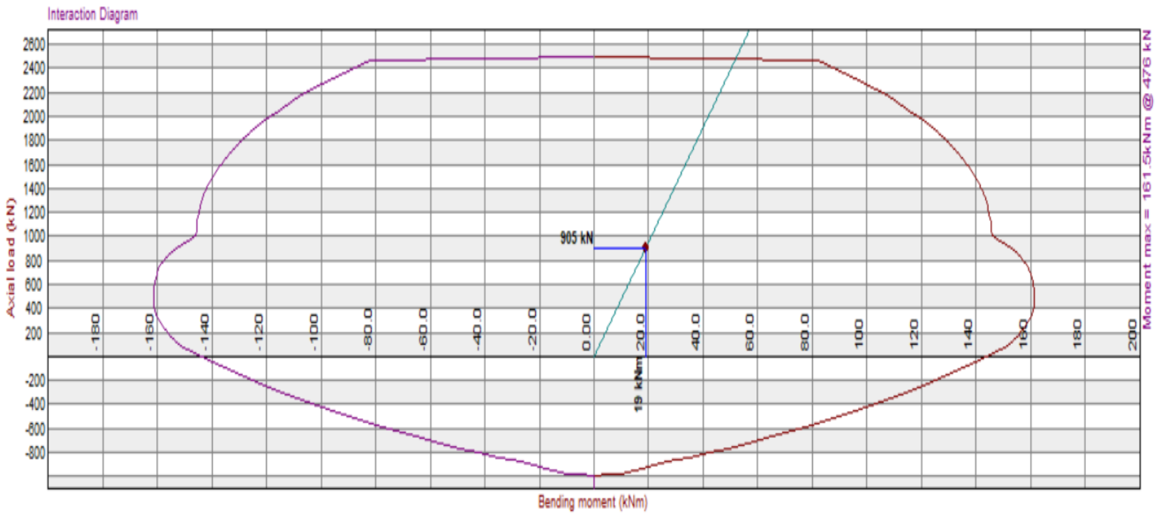

Figure E- 26 Interaction Diagram of Non Deteriorated Middle Column Third Floor

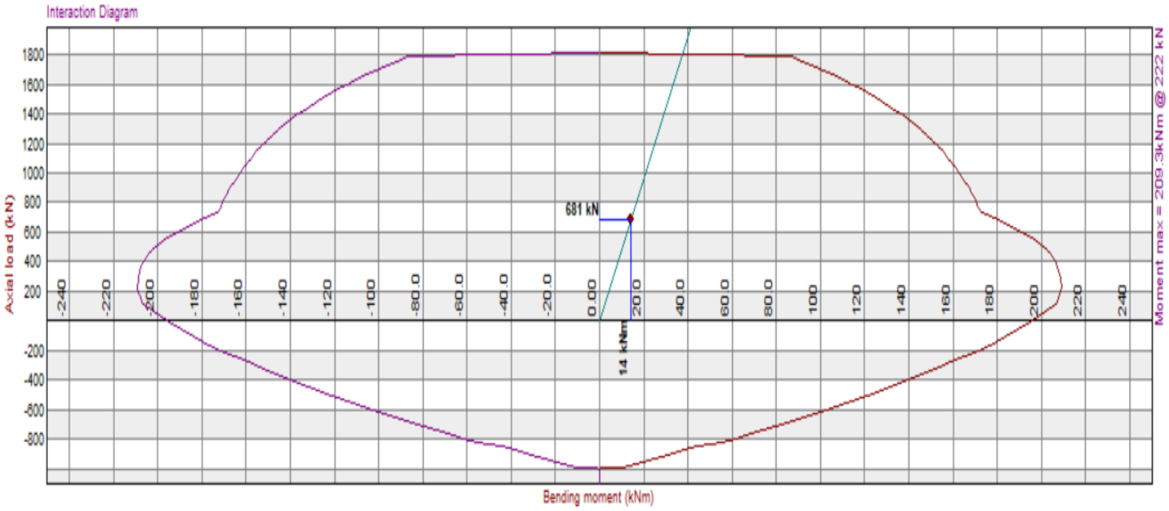

Figure E- 27 Interaction Diagram of Deteriorated Middle Column Fourth Floor

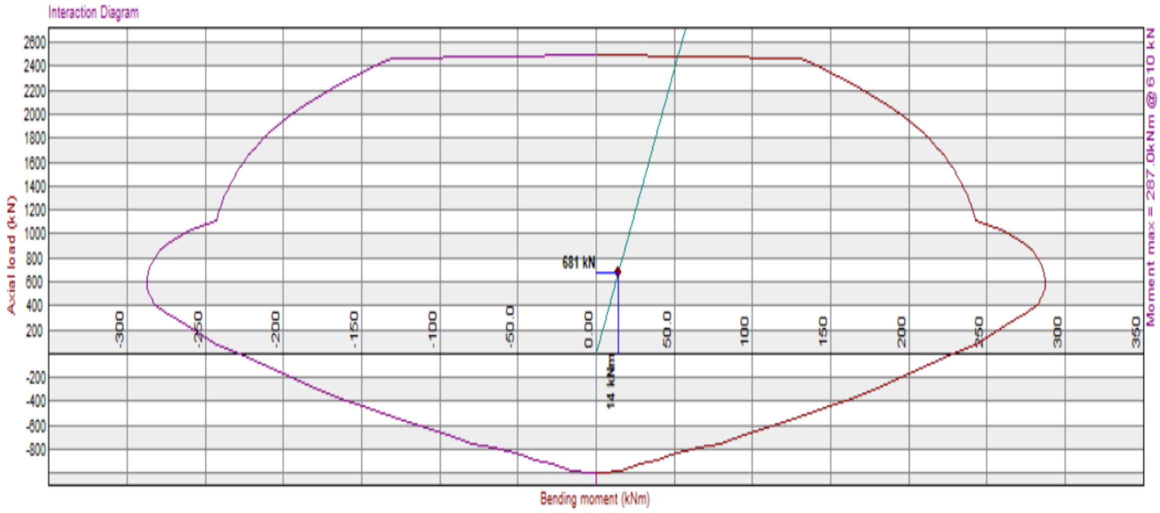

Figure E- 28 Interaction Diagram of Non Deteriorated Middle Column Fourth Floor

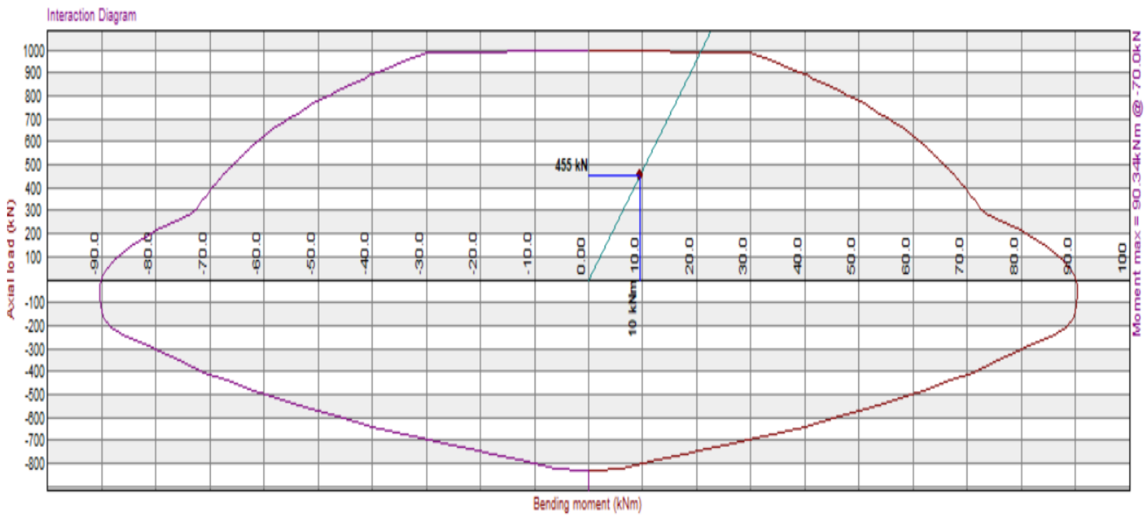

Figure E- 29 Interaction Diagram of Deteriorated Middle Column Fifth Floor

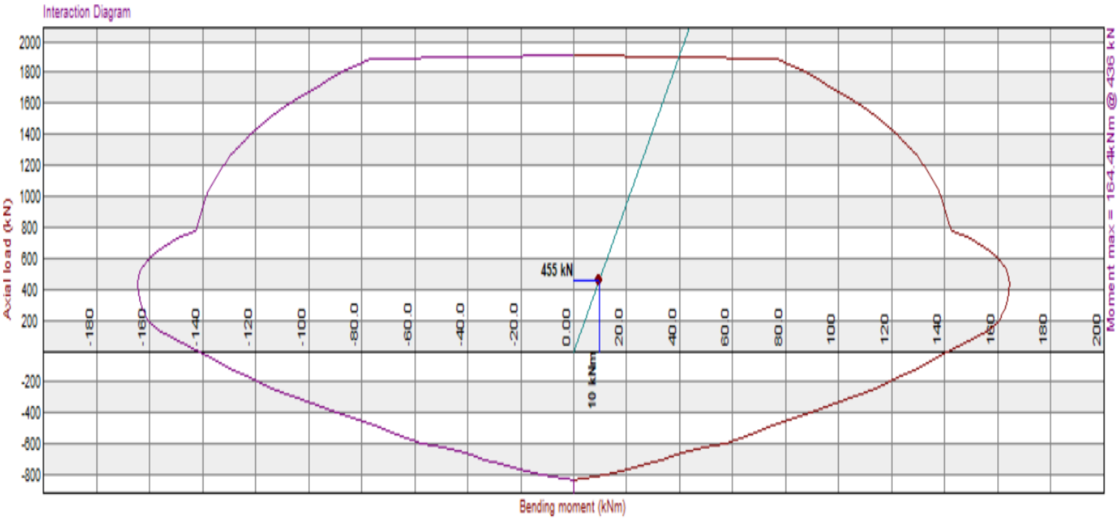

Figure E- 30 Interaction Diagram of Non Deteriorated Middle Column Fifth Floor

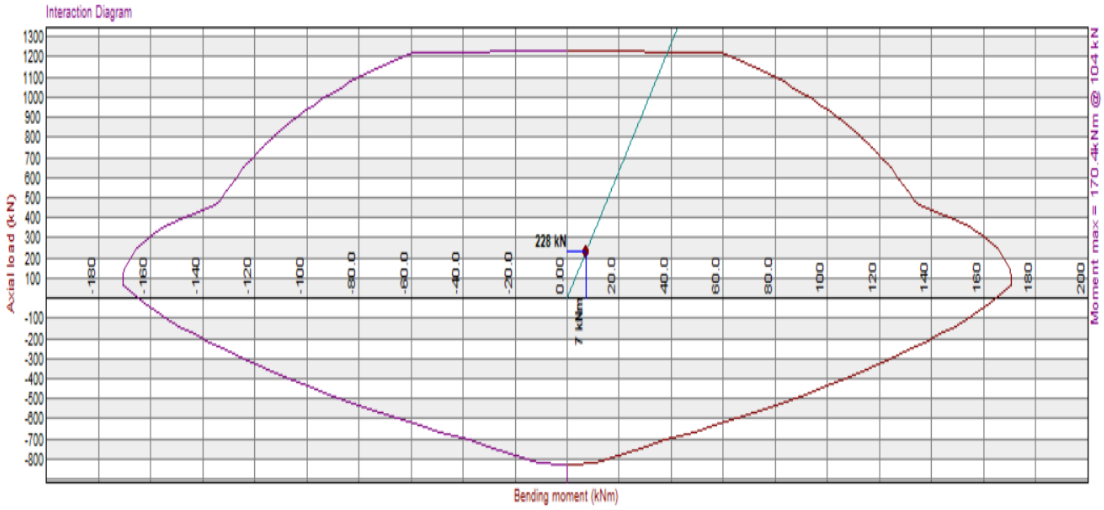

Figure E- 31 Interaction Diagram of Deteriorated Middle Column Sixth Floor

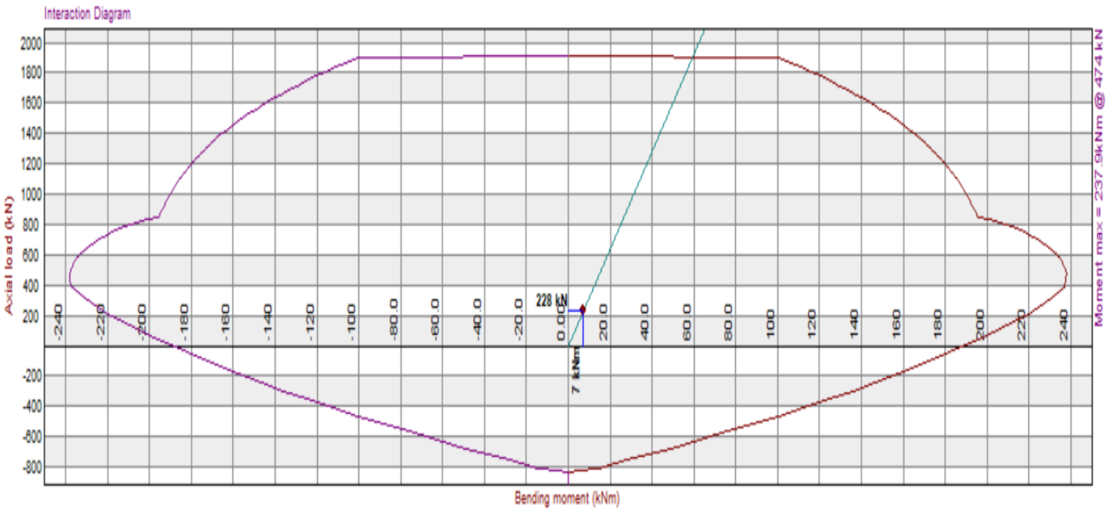

Figure E- 32 Interaction Diagram of Non Deteriorated Middle Column Sixth Floor