EXPERIMENTAL INVESTIGATION INTO THE STRUCTURAL PROPERTIES OF PRECAST REINFORCED CONCRETE PLANKS

BY

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DECLARATION

I declare that the work in the thesis entitled “Experimental Investigation into the Structural Properties of Precast Reinforced Concrete Planks has been performed by me in the Department of Building under the supervision of Drs. M.M. Garba and O.G. Okoli.

The information derived from the literature has been duely acknowledged in the text and a list of references provided. No part of this thesis was previously presented for another degree or diploma at any university.

Anejo, Joseph Aje September 28, 2007

Name of Student	Signature	Date

CERTIFICATION

This thesis entitled "EXPERIMENTAL INVESTIGATION INTO THE PROPERTIES OF PRECAST REINFORCED CONCRETE PLANKS" by Anejo, Joseph Aje meets the regulations governing the award of the degree of Master of Science of Ahmadu Bello University, Zaria, and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This work is dedicated to:

Ada Anejo my wife and my children **Eve Ene Anejo**, **Naomi Ehi Anejo** and **Hannah Ladi Oine Anejo**

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I am immensely grateful to the chairman of my supervisory committee Dr. M.M. Garba for his valuable advice and guidance as well as the interest he has put into this work to ensure that the work was completed. The effort and contribution of Dr. G.O. Okoli my co-supervisor is also acknowledged and appreciated.

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ABSTRACT

The aim of this work was to carry out experimental investigation into the properties of precast reinforced concrete planks. Three different thicknesses of planks were used for the tests; 60, 50 and 40mm. Two values of concrete cover to reinforcement were used; 20mm and 15mm. Two grades of concrete were also used; grades 25 and 30. One type of reinforcement was used with average diameter and yield strength of 3.3mm and 318N/mm^2 respectively. A total of 36 specimens were tested. The tests were carried out on the planks under static point loads to determine the mode of failure, maximum (failure) loads and the corresponding deflection as the loads were varied. The planks failed, in all cases, by the cracking of the concrete in the tension zone. The maximum load at which failure occurred varied depending on the plank thickness, the concrete cover to the reinforcement as well as the grade of concrete. The deflection of the planks also varied with the variation in each of these parameters. One thing was common in all cases; the load-deflection curve for each case was similar in shape to the stress-strain curve for steel at the initial stage of loading. The maximum practical load was 2.7kN for the 15mm cover using the grade 25 concrete and 2.9kN for the grade 30 concrete. The maximum design (theoretical) load was 1.9kN. To compare the maximum load with the design load, a ratio was defined (the differential load ratio). In all the tests, it was only in four cases that the value exceeded 0.20. All of them occurred in the 60mm thick planks. A minimum plank thickness of 60mm; minimum concrete cover of 15mm, a maximum aggregate size of 10mm, a minimum load differential ratio of 0.21 and a minimum yield strength of 318N/mm^2 were recommended for the design and production of reinforced concrete planks.

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ABBREVIATIONS AND SYMBOLS

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
A_c	area of concrete
A_s	area of tension reinforcement
A'_s	area of compression reinforcement
a_{cr}	distance of crack width considered to the surface of the nearest reinforcement
BS	British Standard
c_{min}	minimum cover to tension reinforcement
d	effective depth
E	elastic modulus
E_c	elastic modulus in concrete
E_s	elastic modulus in steel
F_{des}	design load
F_{max}	maximum load a section can carry
f_{cu}	characteristic cube strength of concrete
f_y	characteristic strength of steel
I	second moment of area of a section
G_k	characteristic dead load
i, k	number of cycles of stress
k_1	ratio of the average compressive stress to the characteristic concrete strength
k_2	ratio of the depth of the centroids of the stress block to the neutral axis depth

ℓ	span of beam

M	bending moment
M_D	total damage contribution accumulated from a history of loadings in fatigue
N	number of loading cycles to failure
Q_k	characteristic live load
R	radius of curvature
S	ratio of the maximum stress to the short-term static strength
w	load intensity
x	depth to neutral axis
x_w	depth of Whitney's stress block
β	differential load ratio
γ_m	partial safety factor
ϵ	strain
ϵ_{cu}	maximum concrete strain
ϵ_m	average strain at the level where crack is being considered
ϵ_s	steel strain in tension
ϵ_{sc}	steel strain in compression
ϵ_0	concrete strain when peak stress is reached
ϕ	slope between two points
Δ	deflection of one point measured from the tangent at another point
σ	standard deviation

CHAPTER 1

INTRODUCTION

1.1 Background

The structural properties of a structural member are the behaviours of the member under the effect of loading. The loading could cause the member to be subjected to any of tensile, compressive, shear, torsional stresses or any combination of these stresses. The member fails if the stress or any of their combinations acting on the member exceeds the internal resistance offered by it to the stress or any such combination. A structural member therefore is any member in any structure whose function includes having to resist external load.

Reinforced concrete planks are thin structural members; the overall depth being very small compared to the span. They are most times subjected to flexure. They have applications in the following areas:

(i) as treads in staircases. This reduces the amount of dead load on the whole structure and allows for ease of construction.

(ii) as planks in steel-framed pedestrian foot bridges

(iii) as footway slabs, which span over service ducts on road bridges and sometimes across drains and similar constructions.

Loading of the planks can be static or dynamic. The dynamic type is in the form of impact load generated as a result of users having to run over the planks and sometimes

accidentally dropping the planks during handling. Also, because people have to walk over and over again, it is also subjected to cyclic or repeated loading. The areas of application as listed above indicate that usage is common and failures have also occurred, sometimes in a dramatic manner. The modes of failures have resulted from a lot of factors. These factors, according to Blades et al (1991), include incorrect analysis, design and/or detailing.

According to Martin et al (1994), the load bearing capacity of a flexural member is indicated by the moment of resistance of the cross-sectional dimension of the member under consideration. For this reason, Breen (1991) pointed out that most times, the analysis focuses on the local behaviour of the section rather than the overall action of the member under consideration. However, (designers) according to Troitsky (1976) have shown that the distribution of material in structural members is the most efficient way to resist stress, and may not only affect economies in material, but can also result in much stronger structure.

Varghese (1996) listed experimental investigation as one of the two methods of design of reinforced concrete members. This is more so if the member is small enough to be subjected to a loading condition in the region of that to which it will be subjected during its service period.

This work was intended to carry out a theoretical investigation into the load carrying capacity of the reinforced concrete planks. The planks were then subjected to experimental investigation to establish the mode of failure as well as failure loads. The deflections of the planks were also measured under the loads. The behaviour of the

planks under experimental investigation was then compared with the theoretical analysis. To this end, the effects of the thickness of the planks, the amount of concrete cover for the reinforcement as well as the strength of concrete were determined. This was aimed at providing a basis for recommending the thinnest plank thickness for structural use.

1.2 Need for the Study

The reinforced concrete planks find usage in many areas. Their behaviour under loading needs investigation because they sometimes form very crucial components of structures as in treads of fire escape in buildings. Because of their usage, they are usually subjected to random loading. For this reason, their mode of failure needs to be investigated when they are subjected to actual loading or in service. This is to give a clear picture of the mode of failure. Also, because the actual failure load was compared with the ultimate design load, a comparison could be made between the load at which a structural member fails and the ultimate load bearing capacity determined from structural calculations. Because the thicknesses of the planks were varied, it provided a basis for recommending the thinnest section for planks during design.

1.3 Aim and Objectives

Aim

The aim of this study is to investigate experimentally the structural properties of reinforced concrete planks with a view to finding the thinnest possible section for the planks.

Objectives

The above aim was pursued through the following objectives:

(i) To design and produce concrete mixes to attain two different values of specified strength.

(ii) To cast reinforced concrete planks of various thicknesses using the mixes so designed.

(iii) To carry out tests on the planks to determine failure loads under static loading with corresponding deflection at each stage of the loading.

(iv) To recommend the thinnest section of planks for structural use.

1.4 Methodology

A total of 36 specimens were produced and tested. Two different concrete mixes were designed to achieve specified strengths of 25 and 30 N/mm^2 respectively and the mixes produced. Three different plank thicknesses of 60, 50 and 40mm were used. They all had uniform lengths and widths of 1200mm and 250mm respectively. The concrete cover to the reinforcement was also varied, 20mm and 15mm. For each concrete cover, three specimens were produced and tested and the average of the results taken. For each of these concrete covers, the ultimate load was first of all calculated theoretically. The failure load was then determined experimentally. These two values were compared. Also, the deflections of the planks under load were measured. The relationships between the loadings and deflections have been presented graphically.

1.5 Scope and Limitation

The loads that come on a structural element could be static or dynamic. On application these loads cause deformation (deflection, compression, twisting, extension, etc.). The

element cracks, buckles, snaps, is crushed, or fail in any other manner when its load carrying capacity is exceeded. The response of an element to the application of load is its the structural properties. Only the application of static load with its effects has been considered in this work. The only form of deformation that was measured was deflection. The maximum load as well as the mode of failure was determined.

The reading of the deflection was not done using a digital instrument but a ruler due to lack of instrumentation. The error due to parallax might have effect on the results obtained. The reading of the loads and observation of the deflection were done by two different people at the same time, though at same time; the slight time lag between the reading of the loads and communication of loads to the observer of the deflection may also have effect on the results obtained.

CHAPTER 2 LITERATURE REVIEW

2.1 The Use of Precast Concrete Products

Precast concrete products are the many types and varieties of concrete units which are cast in moulds either in a factory or on site, are not built into the structure until they have fully hardened (Murdock and Brook 1990). Factory production has certain advantages over site manufacture and these include ease of supervision and control, more efficient compaction and possibility of moist curing. The higher level of supervision and control exercised entails higher quality, higher labour productivity and better workmanship in the use of precast concrete. The manufacturing cost is therefore cut down. Also, because it is easy to use high strength concrete, member sizes can be reduced thereby reducing the overall dead load of the precast concrete structure (Airapetov 1986). Specifically, precast concrete products are superior in terms of homogeneity of concrete, dimensional precision and quality of the surfaces.

Prefabrication of structural members, according to Vambersky (1994), stands for high quality with regards to strength, stiffness and durability. It offers a wide variety of spans, shapes and bearing capacities. The manufacture and erection of precast concrete structures is fast and requires only a relatively small crew of qualified workers who work in more favourable working circumstances. There are however, some specific requirements of precast concrete technology, which need to be fully respected for the advantages of precast concrete to be fully achieved. For example, functional (expansion, contraction, shrinkage etc) joints exist in precast reinforced concrete structures. These joints have to be effectively sealed to ensure air-tightness and water-proofing. In open joints, provision should be made to adequately drain them so as not to allow water get

trapped in the joints with the attendant consequences. Also, as far as it is practicable, where there are reinforcement stick-outs that will require to be joined to other reinforcement in other components, they should be butt-jointed to allow for axial transfer of load. This is to avoid the additional bending moments that are typical of lap-spliced connections. The butt-jointing, according to Stoll and Evstratov (1987), is best achieved by welding.

According to Bljuger (1988), precast construction consists, in essence, of transferring most of the work from the construction site to the prefabrication plant. For this reason the structural system has to be broken down into elements which are capable of off-site prefabrication, and of subsequent assembly with appropriate joints.

2.1.1 Areas of application of precast concrete in construction

Because of its time saving character and high quality, precast concrete technology finds application today in very many areas of construction. Murdock and Brook (1990) group precast concrete products into five categories. These are

- architectural cast stone and other types of ornamental concrete
- concrete steps, paving flags, etc., which have to be wear resisting
- load bearing structural units
- roofing tiles
- concrete blocks.

Precast concrete also finds usage in the construction of drains.

With specific regards to building, Vambersky (1994) lists the areas of application of application of precast concrete as follows:

- foundation piles
- foundation beams
- superstructure: columns, walls, stability cores, beams, floors
- stairs and parapets
- partitions
- facades
- decorations
- building accessories such as planters etc.

Precast concrete is also used in conjunction with other structural materials like steel, insitu concrete, masonry, timber, etc., a process described by Vambersky (2004) as hybrid construction.

It is evident from the above that precast concrete is used for both load bearing and non-load bearing components of buildings; in effect, precast concrete finds application in construction in all areas both for structural and non structural purposes.

2.2 Concrete Planks

McMullan (1988) gives two definitions of a plank, ascribing one each to UK and USA. It is firstly defined as a flat length of sawn softwood of 50-100mm thick and over 300mm wide (UK version). Secondly, it is defined as timber more than 25mm thick that is laid horizontally like floorboard (USA version). Both definitions refer to timber. But one thing that is common to the two definitions is that the thickness is thin compared to the other dimensions, the width is at least three times the thickness of the plank. One other fact also is that it is laid horizontally.

Using the above analogy, a concrete plank can therefore be described as a concrete element that has a rectangular cross section with a thickness of not more than one third of the width of the section and the span more than three times the width and usually not cast monolithically with its supports.

From the forgoing, it is seen that reinforced concrete planks have thin sections compared to their width and span. It has application in many areas. It is used as treads in staircases, as cover to drainages and service ducts in bridge. It is also used in some cases to provide covering to roofs in the form of nailable concrete. According to Chudley and Greeno (1998), it is used as a component in what is described as plank and pot composite floor system.

In all the areas of application mentioned above, the planks are placed horizontally and therefore behave as slabs. The major kind of stress to which it is subjected therefore is flexure. That is, it tends to bend under its self weight as well as the external loads it might have to carry.

Concrete planks do not differ from other concrete elements in terms of composition. Like other concrete products, its basic constituents are cement, fine and coarse aggregates which could be normal or light weight and of course water that has to be used for mixing. Because of the fact that the sections are usually 'thin', lower sizes of coarse aggregates are used with a maximum size of 10mm as indicated by Blades et al (1991). This is to allow for adequate concrete cover to the reinforcement and also to allow for proper compaction of the concrete around the reinforcement.

Because of the thinness of the section, large diameter reinforcing rods are not suitable. For this reason, the planks are usually reinforced with wire mesh (Federal Concrete Products, 2004). Reinforced concrete planks where used as structural members are subjected to the same design procedures. Blades et al (1991) used the Australian Standard AS 3600 to design similar elements on the basis of the ultimate strength theory.

From the discussion so far, it is clear that reinforced concrete planks are like other reinforced concrete elements and could be subjected to similar analysis/design and production procedures. It implies therefore that the basic theory of reinforced concrete design *i.e.* load estimation as well as section analysis to determine load carrying capacity are applicable to reinforced concrete planks. Also, curing is important for the planks for the same reasons it is carried out in other concrete elements. Federal Concrete Products (2004) specifies that the planks should be naturally cured under cover. All these point to the fact that adopting the ultimate limit state design procedure as specified in BS 8110 – 1997 for the design of the reinforced concrete planks will also lead to safe designs of reinforced concrete planks.

However, some of the serviceability limit states according to BS 8110-1997 might be difficult to achieve. Because of the thinness of the sections it is difficult to introduce double (*i.e.* top and bottom) layers of reinforcement, which will therefore limit the level of thinness of the section from the consideration of strength.

2.3 Reinforced Concrete Planks and Ferrocement

In structural based application, there are many thin concrete or cement-based products that are used. The reinforcing material is usually used to differentiate between them. The most prominent among these other thin-section products is ferrocement. It is therefore important to differentiate between reinforced concrete planks and ferrocement.

Ferrocement is both a product and a process. As a product it is defined by Naaman (2000) as a thin composite made with a cement based mortar matrix reinforced with closely spaced layers of relatively small diameter wire mesh. The process involved in the production of ferrocement is in itself referred to as ferrocement. The mesh used in ferrocement may be made of metallic or fibre reinforced polymeric materials. For this reason, ferrocement also belongs to the family of thin laminated composites. It has application in many areas of construction for both marine and terrestrial structures such as boats, fishing vessels, barges, water tanks, silos, shells, and monolithic and prefabricated housing structures.

The mortar mixture generally has a sand-cement ration of 1.5 to 2.5 and a water cement ratio of 0.35 to 0.5. Ferrocement has a very high percentage of reinforcement compared to the normal reinforced concrete. Reinforcement makes up about 5% to 6% of the ferrocement volume according to Cement Association of Canada (2006).

Ferrocement has many advantages for structural purposes. According to Hartog (1988), it can be shaped in any form; formed into sections that are less than 25mm in thickness

and assembled over a light framework. The material is very dense, but structures made from it are light in weight. It is watertight.

In contrast, reinforced concrete is cast in sections and needs extensive and very solid formwork to support the weight of the concrete. Because ferrocement is reinforced in two directions, it has homogenous-isotopic properties in two directions (Wang et al 2004). Because of the usually high reinforcement ratio, ferrocement generally has a high tensile strength and a high modulus of rupture. The specific surface of reinforcement in ferrocement is very high compared to that in concrete. For this reason larger bond forces develop with the matrix resulting in lower average crack spacing and width than in reinforced concrete.

Ferrocement forms one of the many thin cement based materials. Others are reinforced with fibre e.g. glass fibre, and because the fibres are in layers and joined together in the matrix to form sections, the products are generally referred to as laminates and their strength, according to Mobasher (2005), depends on the number of layers as well as the orientation of the layers in relation to one another.

It has been shown that ferrocement differ from reinforced concrete planks in terms of the amount of reinforcement to matrix ratio as well as the low level of thinness of the section that can be achieved. The focus in this work is not ferrocement and so the method of its section analysis and design is therefore not discussed.

2.4. Effects of Loads on Structural Members

A structural member may be subjected to axial load (tension and compression), flexure, shear or torsion, or any combination of these (Kong and Evans 1996). These, according to Mosley et al (1999), cause distortion of the member with resulting stresses and strains. To carry out the analysis and design of a structural member, it is necessary to have knowledge of the relationship between these stresses and strains. This is more so when dealing with reinforced concrete, which is a composite material. The reason is that in this case the analysis of the stresses on a cross-section of a member must consider the equilibrium of the forces in the concrete and steel, and also the compatibility of the strains across the cross-section.

2.5. Stress-Strain Characteristics of Steel and Concrete

The loads on a structure, as stated earlier, cause distortion of the members with consequent stresses and strains in the concrete and the reinforcement in reinforced concrete structure. To be able to carry out the analysis and design of a member, it is important to have some knowledge of the relationship between these stresses and strains. Reinforced concrete is a composite material; therefore the analysis of the stresses on a cross-section of a member must consider the equilibrium of the forces in the concrete and steel as well as compatibility of the strains across the cross-section. The behaviour of reinforced concrete member under load is therefore closely related to the stress-strain characteristics of the steel reinforcement and the concrete.

2.5.1 Steel

The behaviours of steel under both tensile and compressive loads are similar, being linear in the elastic range up to the yield stress according to Mosley et al (1999). The stress-strain curves are as shown in figure 2.1.

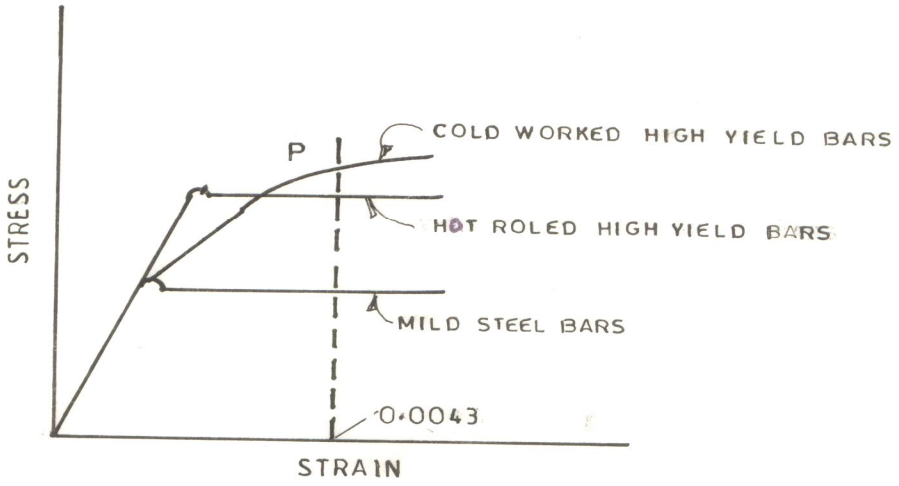


Figure 2.1 Actual Stress-strain curves for steel (Kong and Evans, 1996).

There are many types of reinforcement and the common ones used for reinforced concrete according to Hughes (1987) are rolled mild steel bars, hot rolled deformed bars, and cold worked deformed bars. Both mild steel bars and hot-rolled high-yield bars have definite yield points. The cold worked deformed bars have higher yield point because the cold working can also raise the yield point it has no definite yield point, therefore a value is used, called the proof stress (point p in figure 2.1.) at a strain of 0.0043 is chosen for design purposes.

For the purpose of design, BS 8110 idealises the stress-strain curves for the reinforcement to the one shown in figure 2.2 and it is applicable to both tension and compression.

Within the elastic range, the relationship between stress and strain is given by

Stress = elastic modulus x strain

$= E_s \epsilon_s$ - (2.1)

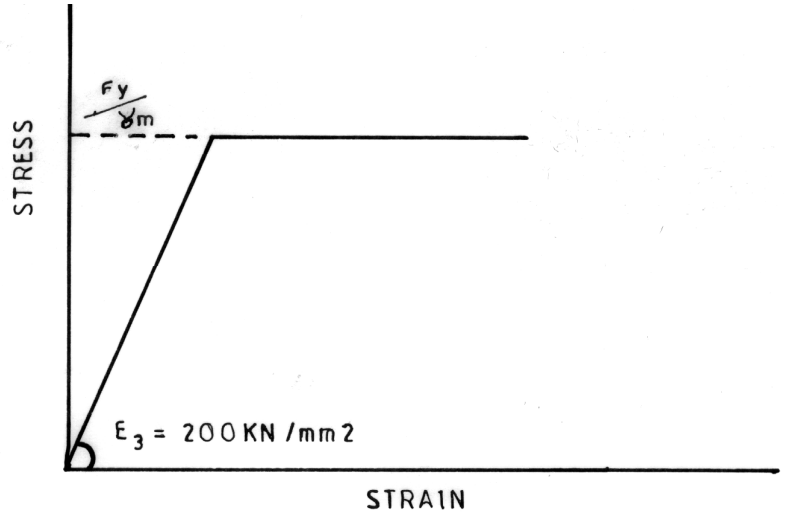


Figure 2.2 Design Stress – Strain Curve (BS 8110, 1997)

2.5.2 Concrete

Concrete is a variable material, having a wide range of strengths and stress-stain curves. The exact shape of stress-strain curves depends very much on the concrete strength (Kong and Evans 1996). Generally, the behaviour of structural concrete is represented by a parabolic stress-strain relationship up to a strain ϵ_0 from which point strain increases at constant stress. The parabolic shape of the curves tends to flatten with the reduction in concrete strength when the applied stress increases beyond approximately 70%.

Figure 2.3 shows typical curves for short term loading. Up to about one third of the maximum stress, the curve is more or less straight; from that stress up to the maximum stress, it is curved ascending, and beyond that it is curved descending. However, with

respect to the range of concrete strength used in practical design, it has been stated generally that

i) up to about half of the maximum stress, the stress-stain curve may be approximated by a straight line

ii) the peak stress is reached at a strain of about 0.002

iii) visible cracking and disintegration of the concrete does not occur until the strain reaches about 0.0035

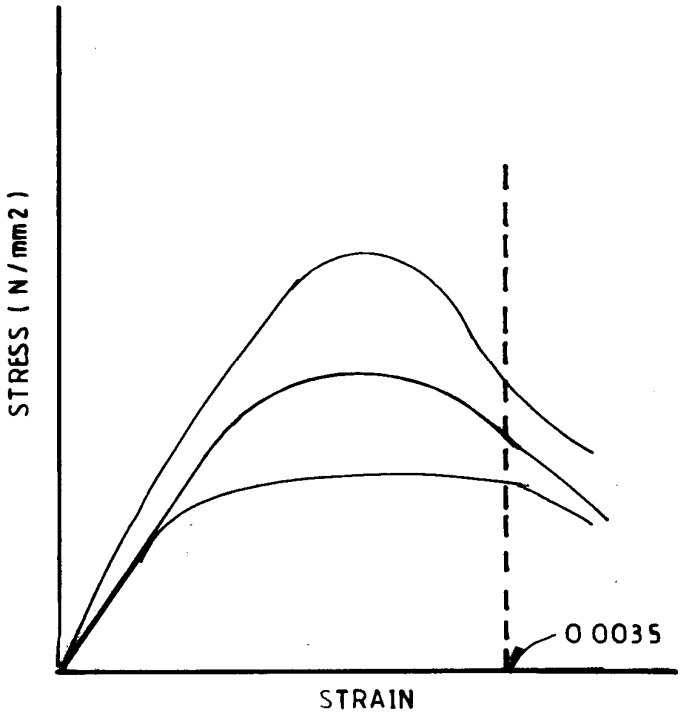


Figure 2.3 Actual Stress-Strain Curves for Concrete (Kong and Evans, 1996)

For design purposes, BS 8110 uses the idealized curve shown in figure 2.4.

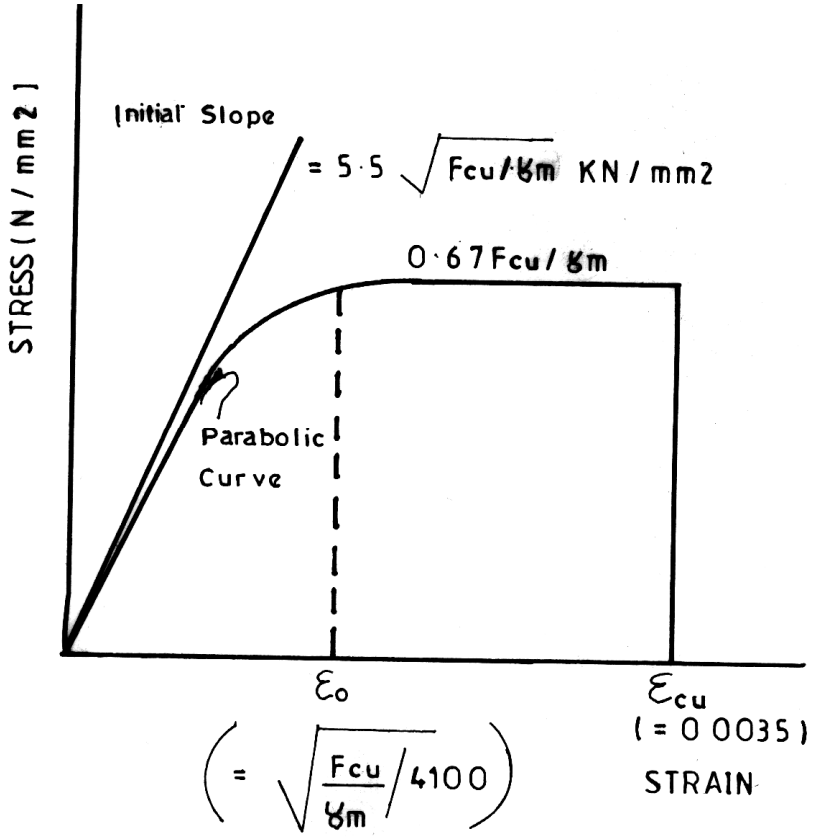


Figure 2.4 Design Stress-Strain Curve (BS 8110, 1997).

2.6 Strains and Stresses Distribution across a Reinforced Concrete Section

In reinforced concrete the theory of bending assumes that the concrete will crack in the tensile regions and that, after cracking all tension is carried by the reinforcement. Also, plane sections of a structural member (at least for practical purposes) are assumed to remain plain after straining implying that across that section the distribution of strain must be linear.

Extensive research works have been carried out in the past decades to give good insight into the behaviour of reinforced concrete in bending under ultimate loads. These include the work by Evans (1943) and that of ACI-ASCE Committee 327 (1956). They provide the basis of current design methods. The following assumptions are made:

(a) the strains in the concrete and the reinforcing steel are directly proportional to the distances from the neutral axis, at which the strain is zero
(b) the ultimate limit of collapse is reached when the concrete strain at the extreme compression fibre reaches a specified value ϵ_{cu}
(c) at failure, the distribution of concrete compressive stresses is defined by an idealized stress/strain curve.
(d) the tensile strength of the concrete is ignored
(e) the stresses in the reinforcement are derived from the appropriate stress/strain curve.

Figure 2.5 shows the cross section of a member under bending and the resulting strain diagram with three different types of stress distribution in the concrete.

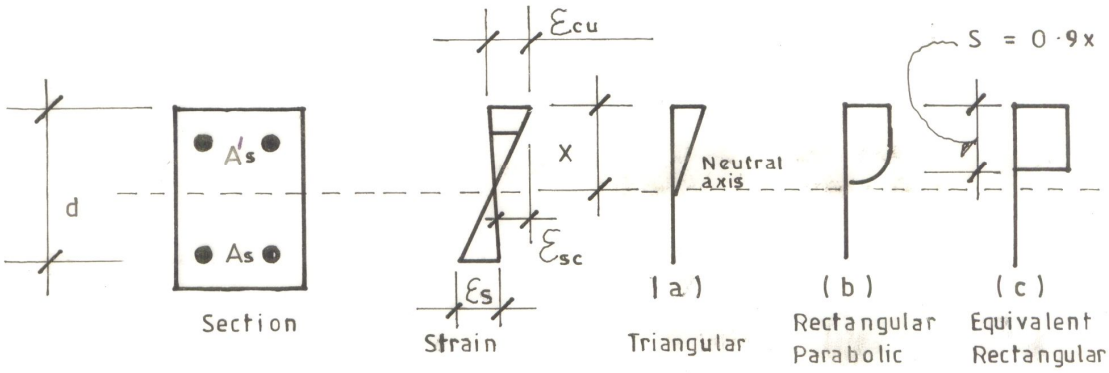


Figure 2.5 Section with strain diagrams and stress blocks (Mosley et al, 1999)

The stress distribution diagram is generally referred to as the stress block and it is actually the stress/strain curve drawn with a horizontal axis for stress and a vertical axis for strain.

From figure 2.5 there are three stress distribution types: the triangular, the rectangular – parabolic and then the equivalent rectangular stress blocks.

i) The triangular stress distribution applies when the stresses are very nearly proportional to strains. This condition occurs generally at loading levels encountered under working conditions and is therefore used at serviceability limit state.

ii) The rectangular – parabolic stress block represents the distribution at failure when the compressive strains are within the plastic range and it is associated with the design for the ultimate limit state.

iii) The equivalent rectangular stress block is a simplified alternative to the rectangular-parabolic distribution.

In the reinforced concrete section, there is compatibility of strains between the reinforcement and the adjacent concrete. Therefore the steel strains ϵ_s in tension and ϵ_{sc} in compression can be determined from the strain diagram. The depth of the neutral axis, x and the maximum concrete strain, ϵ_{cu} and the steel strains is given by

$$\epsilon_s = \epsilon_c\left(\frac{d-x}{x}\right) \quad - \quad (2.2)$$

and also

$$\epsilon_{sc} = \epsilon_{cu}\left(\frac{x-d}{x}\right) \quad - \quad (2.3)$$

where d is the effective depth of the beam and d is the depth of the compression reinforcement.

The strains having been determined from the above equations the stresses in the reinforcement can then be evaluated from figure 2.2 in conjunction with equation (2.1).

2.7 Properties of some Proposed Stress Blocks

A large number of ultimate strength theories have been developed since the beginning of the last century, they however differ, basically in the shape assumed for the stress block as stated by Evans (1943). If therefore the properties of the stress block are expressed in general terms, the ultimate strength equations can then be derived from the principle of mechanics. The two relevant properties of the stress block are the ratios

k_1 and k_2. k_1 is the ratio of the average compressive stress to the characteristic concrete strength f_{cu} and k_2 is the ratio of the depth of the centroid of the stress block to the neutral axis depth.

2.7.1 Hognestad *et al*

Over the years much research has been carried out by, among others, Hognestad (1952), Hognestad *et al* (1955), Rusch (1960) and Mattock *et al* (1961) to study the properties of the stress block. The tests carried out by Hognestad (1952) and Hognestad *et al* (1955) have got considerable influence on modern design approach. The results are summarized in figure 2.6.

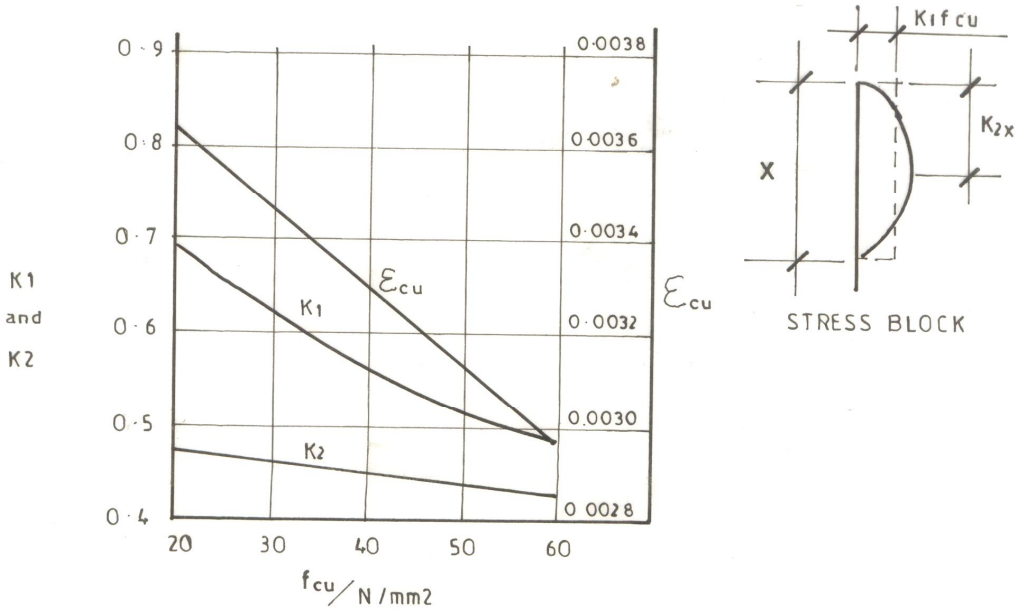


Figure 2.6 Properties of Hognestad et al's Stress block

The properties of the concrete stress block are expressed in terms of k_1 and k_2 as earlier defined.

From the figure the ultimate strain varies with concrete strength. However, current design codes assume, for simplicity, that ϵ_{cu} has a definite value regardless of the concrete strength.

2.7.2 Whitney's equivalent rectangular block

The concept of an equivalent rectangular block was first proposed by Whitney (1942) and it is adopted by both BS 8110 (1997) and the ACI Building Code (1983). Whitney discovered that if the actual stress block was replaced by an imaginary rectangular block of intensity equal to 0.85 multiplied by the cylinder strength f_c and of such a depth x_w that the area $0.85f_c\ x_w$ was equal to that of the actual block, then the centroids of the two blocks were very nearly at the same level as shown in figure 2.7.

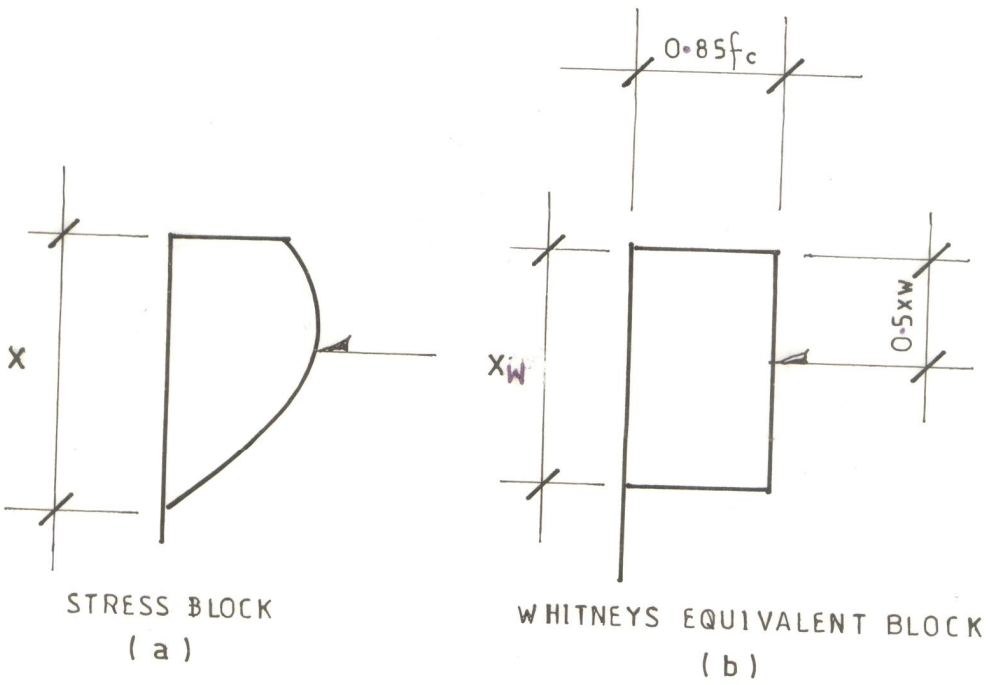


Figure 2.7 Whitney's equivalent rectangular stress block

The depth x_w of Whitney's stress block is not directly related to the neutral axis depth x. While x is to be determined from the strain distribution, x_w is determined from the condition of equilibrium.

This method as proposed by Whitney is rarely applied in modern design directly. It is however, of historical significance and deserves mentioning.

2.7.3 The rectangular parabolic block

A rectangular parabolic stress block is used to provide a more rigorous analysis of the reinforced concrete section. It is the type that is adopted by BS 8110 for design calculations. It is similar to the stress-stain curve for concrete, with a maximum stress of $0.45f_{cu}$ at the ultimate strain of 0.0035. The parabolic part of the stress block is assumed to end at a stain $\epsilon_0 = \sqrt{fcu}/5000$. The stress block is as shown in figure 2.8.

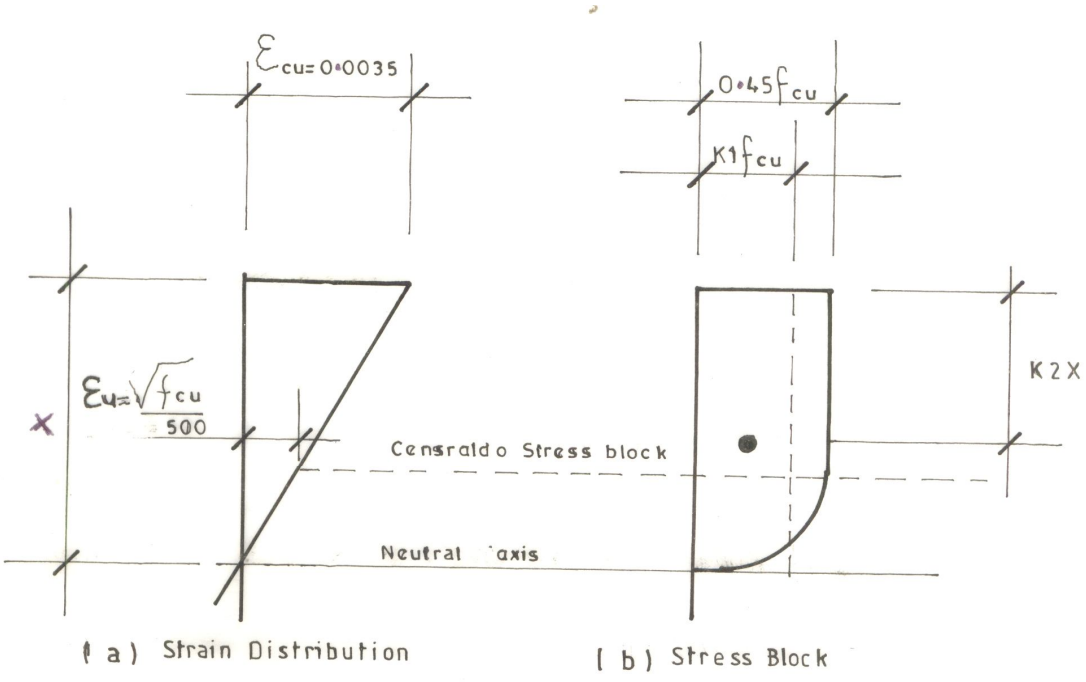


Figure 2.8 Rectangular – Parabolic stress block (BS 8110, 1997)

Using the properties of parabola as given in Coates et al (1988) the following expressions are obtained for k1 and k_2

$k_1 = 0.45\ (1-\sqrt{fcu}/52.5)$ - (2.4)

$k_2 = \dfrac{\left(2-\sqrt{fcu}\right)^2 + 2}{4\left(3-\sqrt{fcu}\right)}$ - (2.5)

2.7.4. The equivalent rectangular stress block

Most times in the design of reinforced concrete structures the design is done for the ultimate limit state and then checks are carried out to ensure that the conditions for serviceability limit state are met. For this reason BS 8110 states that the moment of resistance may be determined using a simplified rectangular stress block having an intensity of $0.45f_{cu}$ extending from the compression face to a depth of 0.9x as shown in figure 2.9.

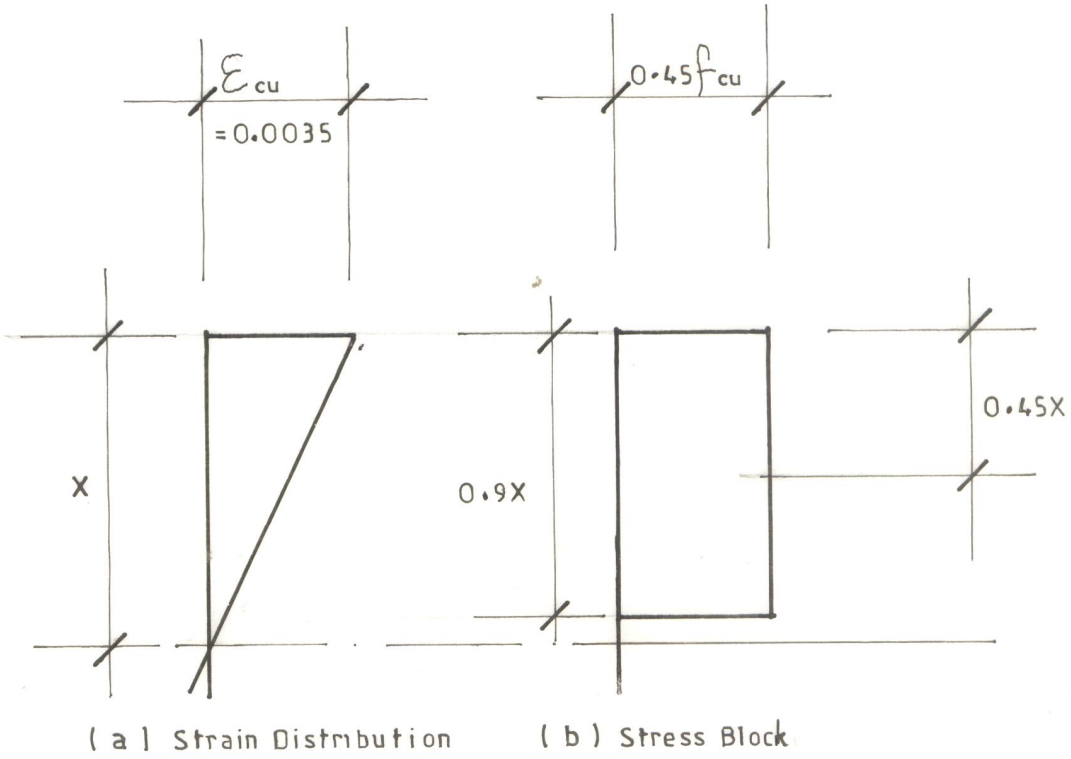


Figure 2.9 Simplified design stress block for ultimate limit state (BS 8110, 1997).

Evaluating equations 2.4 and 2.5 based on the more precise rectangular parabolic stress block, k_1 and k_2 have typical values of 0.40 and 0.45 respectively for the range of concrete grades used in most practical reinforced concrete design. This makes the areas of both types of stress blocks to be approximately equal. Therefore the moment of resistance of the section under consideration will be similar using calculations based on any one of the two stress blocks, a fact that justifies the adoption of the simplified stress block in BS 8110.

2.7.5 The triangular stress block

The triangular stress block, according to Mosley et al (1999) applies to elastic conditions during the serviceability limit state. Generally in practice, it is not used in design calculations except for liquid retaining structures, or for the calculation of crack widths and deflections. Using the triangular stress block, the cross-section can be considered as

(a) cracked in tension zone

(b) un-cracked with the concrete resisting a small amount of tension

(c) partially cracked section

According to Kong and Evans (1996) case (a) is the classical elastic theory which is only relevant now in the calculation of crack widths. Case (b) is important in pre-stress concrete and is not given consideration in this work. Case (c) is currently used for calculation of deflections.

2.8 Serviceability Limit State

The deflection of a structure or any of its parts must not affect the appearance or efficiency of the structure; similarly, any cracking of the concrete must not adversely affect its appearance or durability. More efficient design procedures than before are now available to designers. This has enabled designers therefore to satisfy the ultimate limit state requirements with lighter but more stressed structural members. The serviceability limit state has therefore become a much more important design consideration than in the past.

Of the serviceability limit states, the ones that are normally considered in current designs are those of excessive flexural deflection and excessive flexural cracking as indicated by ACI (1985), ACI (1984), Branson (1977) and, Swamy and Spanos (1985).

In practical designs, the serviceability limit state requirements are met by the following straight forward procedures as given in BS 8110 (1997) part 1:

(a) deflections are controlled by simply limiting the span/effective depth ratio
(b) crack widths are controlled by simply limiting the maximum spacing of tension reinforcement.

However, there are times when it is needful to estimate how a particular structure will behave; like when it is necessary to predict the deflection of a structure.

2.8.5 **Deflection**

Deflection in beams, according to Williams and Todd (2000), can be caused by both bending moment and the shear force. However, if the beam is relatively long compared to the depth, which is usually the case (except for deep beams); shear deflection is small and therefore negligible.

Excessive deflections have, among others, the following consequences in a structure as stated in ACI (1984) and ACI (1985);

(i) sagging floors

(ii) roofs that do not drain properly

(iii) damaged partitions and finishes

It is generally required that neither the efficiency nor appearance of a structure is to be harmed by deflections that will occur during its lifespan. BS 8110 (1997) limits the final deflection to the following maximum values:

(a) span/250
(b) span/500 or 20mm, whichever is less, after the construction of partitions or application of finishes.

There are many facts which may have significant effects on deflections and are difficult to allow for. The more important ones according to Mosley et al (1999) are;

(i) support restraints must be estimated on the basis of simplified assumptions which will have varying degree of accuracy
(ii) the precise loading and duration cannot be predicted and errors in dead loading may have significant effect
(iii) a cracked member will behave differently to one that is un-cracked. This may be a problem in lightly reinforced members where working load may be close to the cracking limits.
(iv) the effect of screeds finishes and partitions are very difficult to assess.

Norris et al (1994) have also shown that deflection varies depending on whether the element is simply supported at the two ends or in the "plate mode" i.e. simply supported all round.

As already stated, deflections in structural members are expected to be small (i.e. in the region of span/250 and span/500 in case of beams). This therefore means that acceptable deflections occur while the behaviour of the beam is within the elastic limit. The implication is that the behaviour of the beam should be guided by the elastic theory of bending.

The elastic bending theory is based on the expression.

$$M = EI\frac{d^2y}{dx^2} \qquad - \qquad (2.6)$$

For small deflections, the term d^2y/dx^2 is approximately equal to the reciprocal of the curvature; therefore

$$M = EI\,\frac{1}{R} \qquad - \qquad (2.7)$$

Equation (2.7) expresses the moment area theorem. 1/R is the curvature at the point the moment M is acting.

As shown in equation (2.6), in the elastic beam theory, the slopes and deflections of beams are expressed in terms of the properties of the M/EI diagram. Within the elastic limit for structural members, the quantity M/EI is equal to the curvature 1/R as can be derived from equation (2.7). For this reason the moment area theorems may be rephrased as curvature-area theorems which according to Kong et al (1983) are as follows;

(a) The change in slope Ø between two points A and B on a member is equal to the area of the curvature diagram between the two points

$$Ø = \int (1/R)dx$$

R is the radius of curvature of the typical element dx.

(b) The deflection Δ of point B, measured from the tangent at point A, is equal to the moment of the curvature diagram taken about point B whose deflection is desired

$$\Delta = \int x\,(1/R)\,dx$$

x is measured from B

For calculating the deflections of concrete structures, the curvature-area theorems have the following advantages over the moment area theorems according to Kong and Evans (1996).

(i) Unlike the moment area theorems, the curvature-area theorems express the purely geometrical relations between the slopes, Ø, the deflections, Δ and the curvature 1/R. Since the relations are purely geometric, their validity is independent of the mechanical properties of the materials. This means that the curvature area theorems are equally applicable regardless of whether the structure is elastic, plastic or elasto-plastic.

(ii) Unlike the moment-area theorems, the curvature-area theorems can be used even where the deformations are caused by other effects than bending moments, e.g. by shrinkage and creep. Once the curvatures are known, the slopes and deflections are completely defined by the curvature area theorems; whether the curvatures have been caused by bending moments or by shrinkage and creep does not affect the results.

However, the use of the moment-area theorems still gives good results as shown by the findings of Climaco and Regan (2001).

2.8.6 Cracking

According to Illston et al (1979), concrete is a two phased material, either as coarse aggregates in a matrix of mortar or as aggregates in a matrix of hardened cement paste. The hardened cement paste provides the strength that allows for concrete to be used for structural purposes and is therefore the fundamental material. The aggregate is important because it provides the rigidity and dimensional stability to concrete. One important consequence of the development of cracks in concrete therefore is the reduction in the effective area to resist the applied load.

In reinforced concrete, cracks form when the concrete tensile strength is exceeded, according to Gunther and Mehlhorn (1991). That is, cracking emanates from, in addition to other sources, overloading in relation to the actual strength of the structural member, but this is the consequence of inadequate design, or construction not conforming to specification. In reinforced concrete in service, tension is induced in the reinforcing steel and the surrounding concrete. Surface crack is therefore inevitable but with proper structural design and detailing, the cracks are very narrow and barely noticeable. Besides the visible cracks on the surface of the structural member, inner cracks form at the reinforcing bar ribs (or at the steel/concrete interface). In a beam

under load, as the load on the beam increases, the number of cracks, and consequently the crack spacing rapidly reach a nearly constant value which does not change significantly with further increase in load. Under such a situation the crack width directly over a reinforcement bar increases with concrete cover and with the average strain at the level at which the crack is considered (Beeby, 1971). Also, according to ACI 207. 2R – 90, stress induced cracks have maximum width at the surface of the concrete and tapers towards the steel, but the difference in width may decrease with time. This implies that the crack width increases with the height to which the crack penetrates. BS 8110: Part 2 gives the crack width formula as follows:-

$$\text{design surface crack width} = 3a_{cr}\epsilon_m / (1 + 2[a_{cr} - c_{min}]) \quad - \quad (2.8)$$

For a concrete member under load, when the tensile strength decreases, the compressive strength increases. For any stress less than 60% of the tensile strength microcracks are usually ignored (Bangash 1989). Evans (1962) and Welch (1972) both agree that at a value of 75% of the tensile strength unstable cracks begin to develop in concrete.

From the consideration of energy dissipation, it is easier to extend an existing crack than to form a new one. This is the reason why, under an applied load, each subsequent crack occurs under a higher load than the proceeding one. The total number of cracks developed is determined by the size of the concrete member, and the distance between cracks depends on the maximum size of aggregate present (Hansen 1968).

The importance of cracking and the minimum width at which a crack is considered significant, depend on the function of the structural members and on the conditions of exposure of the concrete. Reis et al (1965) suggested the following values for permissible crack widths, and they still offer good guidance.

Interior members	0.35mm
Exterior members under normal exposure condition	0.25mm
Exterior members exposed to particularly aggressive environment	0.15mm

Observers have not been quite agreed on this, but the minimum crack width that can be seen by the naked eye according to Neville (1997) is 0.13mm. Very fine cracks are common but not harmful from structural strength consideration; therefore intensive search for cracks is not beneficial. However, the causes, evaluation and repair of non-structural cracks are fully considered in ACI 224. R-90, and by the Concrete Society Technical Report No.22.

2.9 Fatigue Strength

Two types of failure in fatigue can take place in concrete (Neville and Brooks 1994). In the first type, failure occurs under a sustained load (or a slowly increasing load), near but below the strength under an increasing load. This is known as static fatigue or creep rupture. The second type occurs under cyclic or repeated loading and is simply known as fatigue. In both cases a time-dependent failure occurs only at stresses, which

are greater than a certain threshold value but smaller than the short-term static strength.

2.9.5 Static fatigue

Static fatigue occurs only when concrete is loaded in the range where unstable crack growth begins. Under such a sustained load, there will be slow crack growth both at the interface and in the matrix. Eventually, these cracks will grow until they reach some critical size, and fracture will then occur. Under this type of loading, the strain at failure will be higher than in short-term tests. This, according to Minders and Young (1993), is because more cracking has time to take place.

Under low rates of loading, static fatigue occurs when the stress exceeds about 70 to 80% of the short-term strength according to Neville and Brooks (1994). This threshold value represents the onset of rapid development of microcracks, which eventually link up, and result in failure. A similar behaviour also occurs under a sustained load where a certain load is applied fairly quickly and then held constant. Above a similar threshold of about 70 to 80% of the short-term strength, the sustained load will result in failure eventually. At loadings below the threshold, failure will not occur and the member will only continue to creep.

2.9.6 Fatigue

Fatigue, according to Minders and Young (1993), may be defined as the phenomenon by which a material is caused to fail by repeated application of loads that are not large enough to cause failure in a single application. This implies that under repeated stresses, some initial progressive permanent structural change occurs in a structural

member. This change which is sometimes referred to as fatigue damage, appears to involve primarily the growth of cracks as earlier stated, although some viscous flow (or creep) may also take place. Fatigue occurs for compressive, tensile and flexural loadings. Generally the factors that affect static strength in concrete will have similar effect on fatigue. Also the moisture condition of the concrete has only marginal effect on the fatigue strength which is negligible.

For a constant range of alternating stress, the fatigue strength decreases as the number of cycles increases. This statement is illustrated by the curves shown in figure 2.10 which relates S to N where S is the ratio of the maximum stress to the short-term static strength and N is the number of cycles to failure.

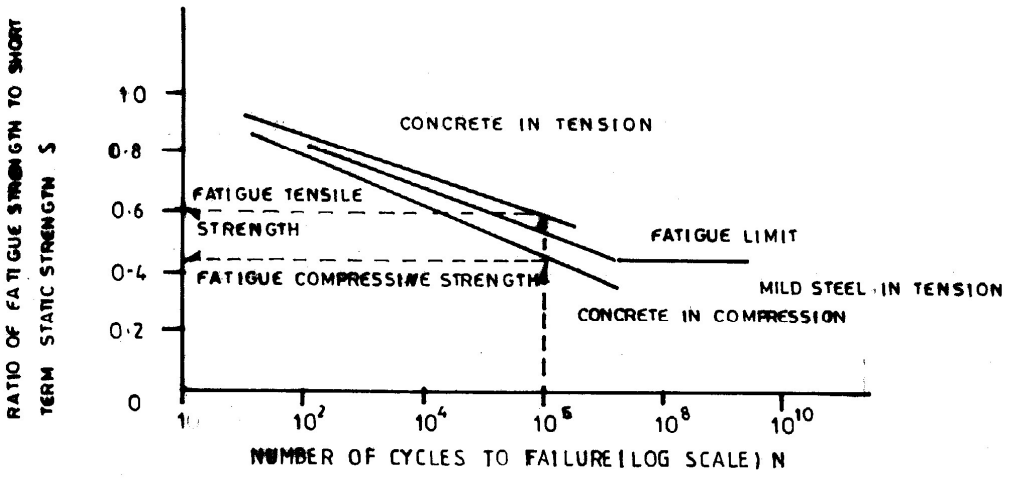


Figure 2.10 Typical relations between the fatigue strength and the number of cycles for concrete and mild steel with a minimum stress of zero. (Neville and Brooks, 1994)

The maximum value of S below which no failure occurs is known as endurance limit. From the S – N curve shown, whereas mild steel has an endurance limit of about 0.5 which means that when S<0.5, N is infinity, concrete does not appear to have a corresponding limit. Because of the large variation in the properties of concrete, for a

given cycle of stress, it is not easy to precisely determine the number of cycles to failure.

For the forgoing reason therefore, to test a structural member made of reinforced concrete for fatigue, it is hereby suggested that a minimum number of cycles be set for a given period of time. If the element does not fail under this particular number of cycles it should be taken to have met the fatigue strength requirements for its purpose.

In a practical situation, the range of stress to which a structural member is subjected to determine its fatigue strength varies. It is therefore the whole history of the cyclic loading that is relevant. The Miner's rule assumes that failure will occur if the total damage contribution M accumulated from a history of loadings is equal to unity. The individual damage contribution of one particular loading is equal to the ratio of the number of cycles, n_i, at a given cycle of stress I to the number of cycles to failure, N_i, at that stress. Therefore:

$$M = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \text{------} + \frac{n_k}{N_k} = k \sum_{i=1} \frac{n_i}{N_i} = 1$$

For any cyclic stress I, the value of N_i is obtained from the appropriate S – N curve. S – N curves however have very large scatter associated with them, Miner's rule is not very accurate (Neville and Brooks 1994).

2.10 Impact

Impact strength is important in driving piles, in foundations for machines exerting impulsive loading, and also when accidental impact is possible as might occur when

handling precast concrete members. Impact loading is also considered in highway bridges design whereby the impact effect is taken into consideration by increasing the live load by a certain percentage. However, the live loads specified in design for the normal use of the structure are assumed to include adequate allowance for ordinary impact conditions. In the reinforced concrete planks under consideration, investigation was carried out on its behaviour in service. This means therefore that impact as indicated in the examples mentioned earlier will not be given any special consideration in the current investigation. However, Blades et al (1991) report that the static failure load is up to eight times higher than the impact load required to cause failure.

The experimental investigations are carried out as indicated in the following chapter.

CHAPTER 3 MATERIALS AND METHOD

3.1. Experimental Programme

A total of thirty six specimens of reinforced concrete planks (eighteen each of grades 25 and 30 concrete) were fabricated and tested. The planks were designed on the basis of the specifications of BS 8110: Structural use of concrete; 1997 revision. The loads were applied in 1kN increments and then the necessary observations made within five minutes for each specimen. The reinforcement was also tested to determine its yield stress. This yield stress value was used in the design of the planks. For each of the parameters investigated, three specimens were used and then average taken to give the result.

3.2. Test Specimens

Each test specimen was a plank made of reinforced concrete and simply supported. The planks were of varying thicknesses of 60, 50 and 40 millimetres and of uniform length and width of 1200mm and 250mm respectively. The plan dimensions chosen were to make them to be in the region of those used in practice. Live load of 3.0kN/m^2 was used to conform to BS 6399: Part 1:1984 Code of Practice for Dead and Imposed Loads. The value of the load was based on the proposed use of the planks, stair treads for a public building.

3.3. Materials

The planks used in the tests were of reinforced concrete, the two basic materials being concrete and reinforcement.

3.3.1. Concrete

The concrete used in the test specimens was of normal weight using crushed granite aggregates with maximum size of 10mm. This maximum size was chosen because of the proposed thicknesses of the planks. The sand as well as the gravels used was supplied in one batch each. The concrete was made using one brand of cement. Effort was made to achieve two pre-determined strengths of 25 N/mm^2 and $30N/mm^2$. The actual cube strength of the concrete was obtained by averaging the compressive strength of three 150 x 150 x 150 cubes at the age of 28 days.

(i) ***Concrete mix design***

The concrete mix was designed using the British Department of Environment method for normal weight concrete developed in 1975 and revised in 1988. (The detail mix design procedure is as shown in appendix B).

The materials properties used are as follows:

Table 3.1: Concrete Materials Properties

Material	**Density (kg/m^3)**	**Specific Gravity**
Cement	1440	3.15
Sand	1600	2.65
Crushed stone	1540	2.65
Water	1000	1.00

To ease compaction of the concrete in the moulds, medium workability of 75mm was desired and therefore targeted. The cubes were tested to determine the achieved strength. Plate 3.1 shows one of the cubes during testing while plate 3.2 shows the cube after crushing. The results showing the achieved strengths for the two grades of concrete are shown in table 3.2.



Plate 3.1: Crushing of concrete to determine achieved strength. (Civil Engineering Department Laboratory, Kaduna Polytechnic, 2006)



Plate 3.2: Concrete cube after crushing (Civil Engineering Department Laboratory, Kaduna Polytechnic, 2006)

Table 3.2 Strengths of concrete

(a) 25 N/mm^2 target strength

Specimen No.	Failure Load (kN)	Area of Test Cube (mm^2)	Concrete Cube Strength (N/mm^2)
SP1	630	22500	28
SP2	600	22500	27
SP3	650	22500	29
Average	627	22500	28

(b) 30 N/mm^2 target strength

Specimen No.	Failure Load (kN)	Area of Test Cube (mm^2)	Concrete Cube Strength (N/mm^2)
SP1	815	22 500	36
SP2	790	22 500	35
SP3	820	22 500	36
Average	803	22 500	36

(Source: Laboratory Work Report, 2006)

3.3.2 Reinforcement

The reinforcement was of one size of wire fabrics obtained from one source. The wire fabrics have average diameter of 3.3mm and average yield strength of 318N/mm^2. The load carrying capacity of the planks section was varied based on the spacing of the reinforcement. The spacing therefore depended on the concrete cover which determined the effective depth and hence the moment of resistance of the section.

The reinforcement was tested in order to obtain the yield strength. To this end three samples were tested. These were cut directly from the reinforcement fabrics to be used in reinforcing the planks. The results showing the yield loads and consequently the yield strength are presented as shown in Table 3.2.

Table 3.3 Yield Strength of Steel Reinforcement (Wire Fabrics)

Specimen No.	Yield Load (kN)	Cross Sectional Area of Reinf't (mm^2)	Yield Strength (N/mm^2)
1	2.75	8.6	320
2	2.95	8.6	343
3	2.50	8.6	291
Average	2.75	8.6	318

(Source: Laboratory Work Report, 2006)

3.4 Fabrication of Specimens

The wires that constitute the reinforcement fabrics were cut into lengths of 1160mm allowing for concrete cover of 20mm at each end of the wires. Some other short lengths of wires were provided at right angles to the main bars. These were to act as distribution reinforcement.

The desired concrete cover was achieved by using spacer blocks of sand/cement mortar prepared specifically for that purposed.

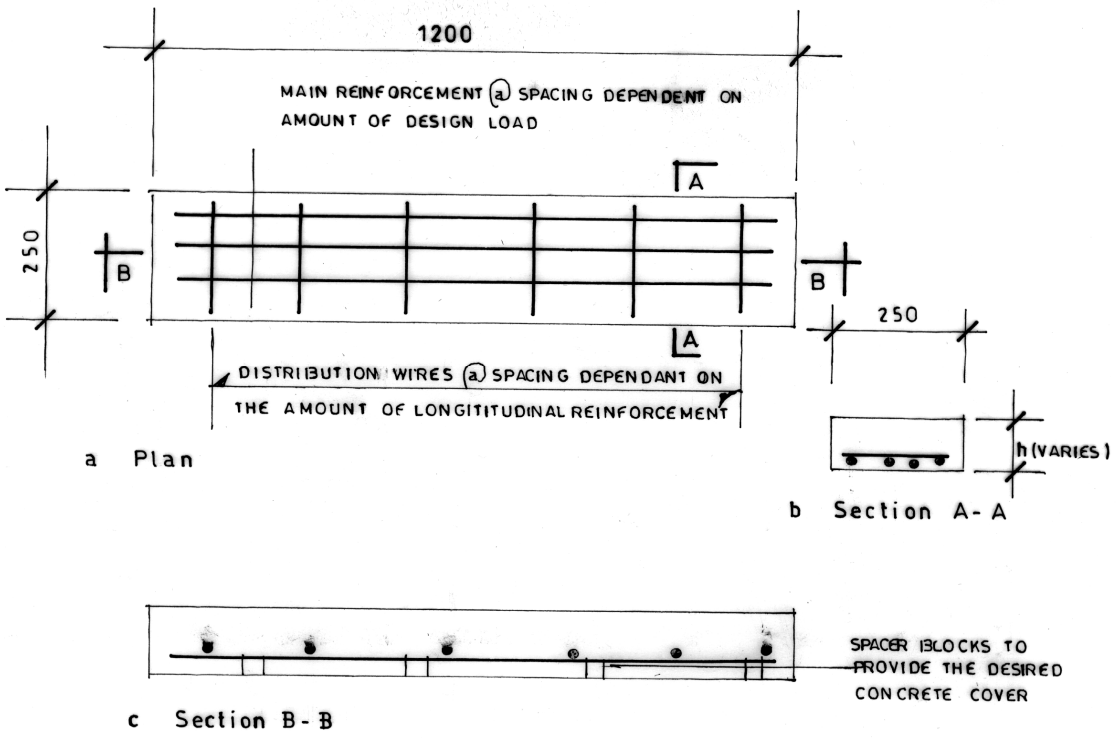


Figure 3.1 Typical arrangement of reinforcement in the specimens.

As shown in figure 3.1 the main reinforcement was placed under, parallel to the span of the planks while the distribution reinforcement was provided at its top at right angles. The prepared reinforcement fabric was then placed in moulds.

Two different concrete covers of 20 and 15 millimetres were used for each selected plank thickness. The values of the design loads varied depending on the thickness of the respective planks. These therefore were the factors that determined the amount of reinforcement.

3.5. Identification of Specimens

A total of thirty six specimens were cast and tested during the investigation. The specimens were numbered to allow for easy identification of each one of them.

An alpha-numerical code was used to identify the specimens. The alphabets indicate the thickness of the planks while the numbers indicate the concrete cover. The classification is as shown in table 3.3.

Table 3.4 Classification of Samples

Plank Thickness (mm)	1(20)	2(15)	No. of Specimens
A (60)	A1	A2	6
B (50)	B1	B2	6
C (40)	C1	C2	6

For each parameter, three specimens were tested. The identification numbers were as given in table 3.4

Table 3.5: Specimens Identification Numbers

Plank	Concrete Cover (mm)	
Thickness (mm)	1(20)	2(15)
A (60)	A1 A11, A12, A13	A2 A21, A22, A23
B (50)	B1 B11, B12, B13	B2 B21, B22, B23
C (40)	C1 C11, C12, C13	C2 C21, C22, C23

Note:

(i) The alphabet represents the plank thickness

(ii) The first digit represents the concrete cover while the second digit represents the specimen number in that category e.g. B21 means specimen has a thickness of 50mm, concrete cover of 15mm and is number 1 in that set.

(iii) The average reading for the three specimens for each plank thickness and concrete cover will be taken to represent that of the particular plank.

Example for B1 = $\frac{B11 + B12 + B13}{3}$

3.6 Design of the Concrete Planks

The design of the planks was based on the provisions of BS8110 - 1997 Revision. The planks were intended to act as treads to the staircase in an office building. The total design load varied depending on the thickness of the respective planks. The amount of reinforcement for each thickness of the planks depended on the concrete cover in the respective cases. The 30mm thick slab was analysed but did not meet the requirements for use as a structural member based on the provisions of BS8110 -1997. It was therefore discarded. The reinforcement details are shown in table 3.5. The design procedures are shown in appendix B.

Table 3.6 Details of Planks Reinforcement Details

(a) For grade 25 concrete

Thickness (mm)	**Concrete Cover (mm)**	**Area of Tensile Reinf't, A_s (mm^2)**
60	20	91.9
	15	80.7
50	20	114.7
	15	96.1
40	20	165.6
	15	126.1

b) For grade 30 concrete

Thickness (mm)	Concrete Cover (mm)	Area of Tensile Reinf't, A_s (mm^2)
60	20	91.9
	15	80.7
50	20	110.4
	15	96.1
40	20	165.6
	15	120.4

3.7 Experimental Procedure

Some quantities of sand and gravel were procured and thoroughly air-dried in the laboratory. After drying, the desired amount of the coarse aggregates was sieved for the entire experimental programme and kept in a dried state. The gravels consisted of sizes ranging from 4.5mm to 10mm. No special attempt was made to sieve the sand apart from the fact that any noticeable impurities and unsuitable particle sizes were removed.

Batching of the cement, sand and gravel was by weight. The mixing of the concrete was done manually. The sand was first of all mixed thoroughly with the cement. The coarse aggregates were then spread evenly on the top. Water was then added and the mix thoroughly turned. The moulds were prepared and oiled before the casting in each instance. The reinforcement wire fabric as prepared was then placed inside the mould. The spacer blocks were attached to the bottom of the reinforcement mesh to give the desired concrete cover. Because of the small thickness of the planks, the moulds were filled in one single layer and thoroughly compacted. The top surfaces of the specimens

were finished smooth about an hour after casting. This was to provide a level surface for the testing.

Curing was done at room temperature by constant watering and being covered back with the polythene sheets as shown in plate 3.3.



Plate 3.3: Planks under polythene sheets during curing (Civil Engineering Department Laboratory, Kaduna Polytechnic, 2006)

The planks were then removed from under the polythene sheets after 27 days and demoulded. Plate 3.4 shows the planks shortly before being demoulded. They were left to dry under fan overnight as shown in plate 3.5, before being tested.



Plate 3.4: Planks shortly before demoulding (Civil Engineering Department Laboratory, Kaduna Polytechnic, 2006)



Plate 3.5: Some of the Planks Left to Dry Before Testing (Civil Engineering Department Laboratory, Kaduna Polytechnic, 2006)

All the specimens were tested in a 3000kN capacity SEIDNER electric-hydraulic pump powered universal testing machine. They were tested under the same rate of loading, failures occurring, in each case, within three minutes.

A simple device (a pointer) was attached to the specimen to measure the deflection on a calibrated scale (plastic rule) that was attached to the testing machine. Measurement of deflection was read to the nearest millimeter.

CHAPTER 4 RESULTS

4.1 Observations

On application of the load, the deflection was gradual. All the specimens failed by the cracking of the concrete in the tension zone. Plate 4.1 shows one of the planks after failure.



Plate 4.1: One of the Planks Showing Crack after Failure (Civil Engineering Department Laboratory, Kaduna Polytechnic, 2006)

In each of the cases after the maximum load was reached, the deflection became rapid until each specimen collapsed and the value of the load at collapse was less than the maximum load recorded. In all cases, cracks were initiated at loads approximately equal to 1kN. Cracks developed faster in cases where the point of load coincided with the

position of the cross wires. Once cracks were initiated, they developed very quickly compared to the time it took for them to be initiated. At collapse, most of the specimens cut completely into two, and in each of such cases, the cutting was approximately at the mid span, the point of load application. In most of the cases only one major crack was observed which was at failure. The other minor cracks, some barely visible, occurred on the two sides of the major one. These cracks were noted to be closely spaced with distances of not more than 10mm between successive cracks. There was no difference in the rate of propagation of cracks with respect to the two grades of concrete under similar rates of loading once the cracks were initiated. However, it took relatively longer time for the cracks to be initiated for the grade 30 concrete than for the grade 25 concrete specimens. The crack width at maximum load and at failure was wider for the specimens with the 20 mm concrete cover.

Failure loads varied depending on the grade of concrete. It was observed that specimens produced using grade 30 concrete had higher failure loads than the grade 25 concrete specimens. The failure load values also varied depending on the amount of concrete cover. For the respective grades of concrete, the failure load for a particular plank thickness was higher for the 15mm concrete cover than for the 20mm concrete cover. The deflection also was lower for each value of load in the corresponding cases for the 15mm concrete cover than for the 20mm concrete cover. Also, the deflection was less than for the grade 30 concrete for similar amount of load.

4.2 Detailed Results

The tables showing the detailed results obtained from the experimental investigations are presented in appendix C. Figures 4.1, and 4.2 are the graphical representations of the results obtained from the experimental investigations. The graphs show the loadings in relation to the corresponding deflections. The graphs were combined for the two values of the concrete cover for each plank thickness. This was to allow for ease of comparison of the behaviour of each plank thickness for each of the two values of concrete cover.

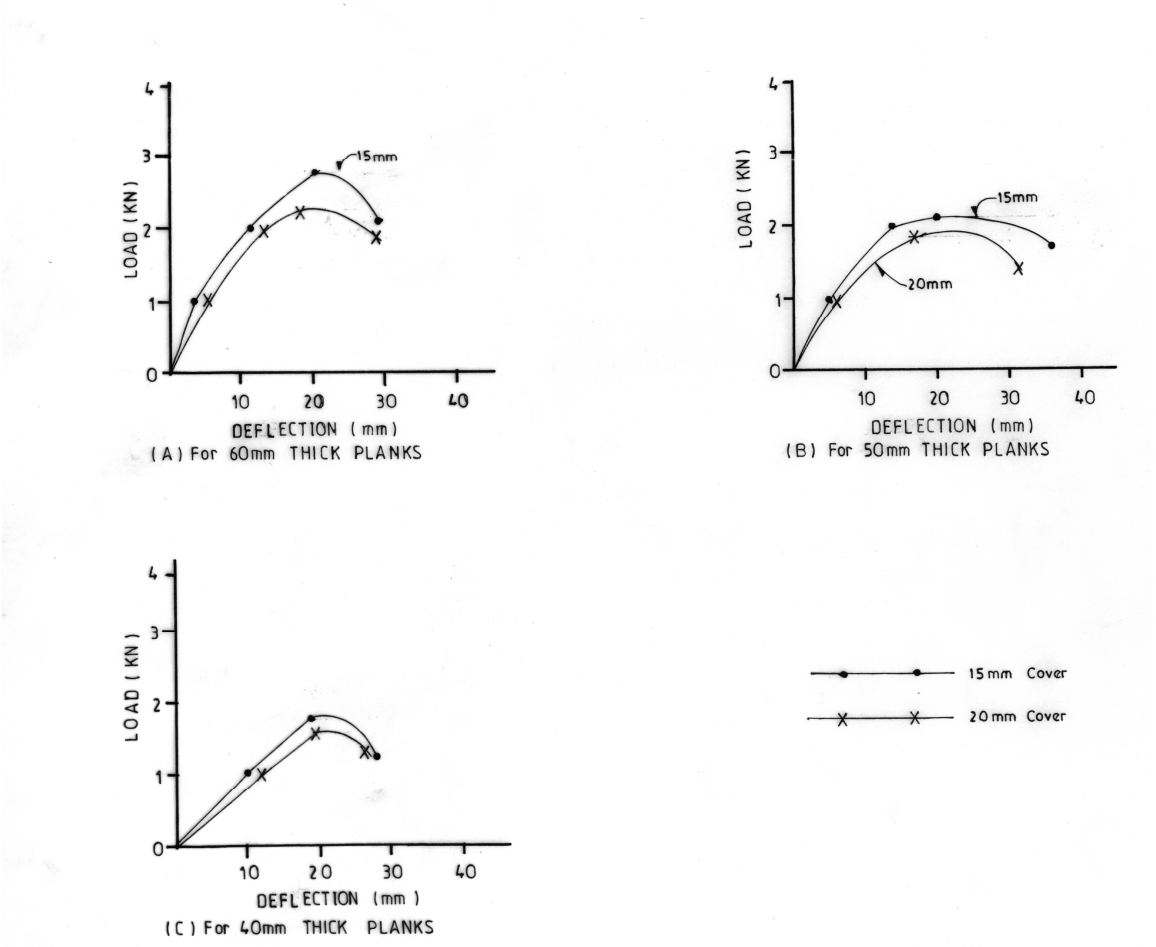


Figure 4.1: Load – Deflection curves for Grade 25 concrete planks (Laboratory Work Report, 2006)

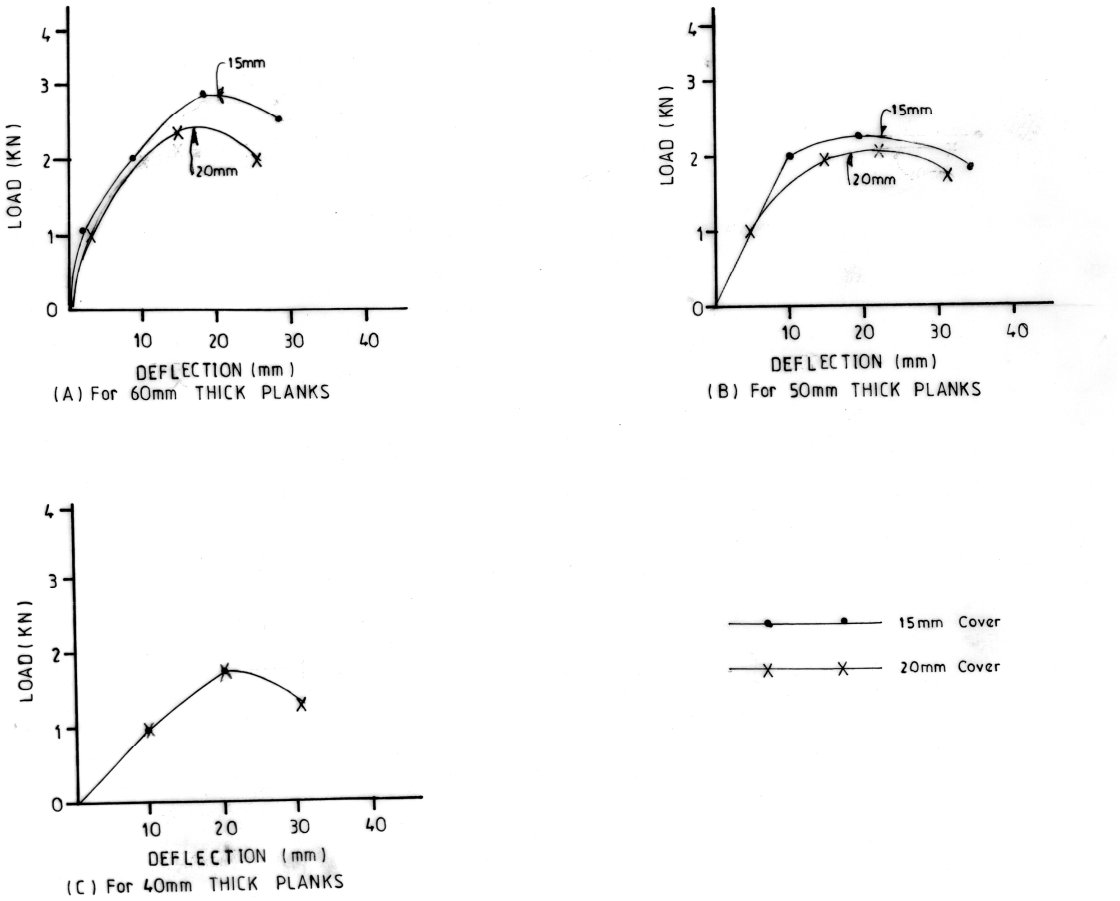


Figure 4.2: Load – Deflection curves for Grade 30 Concrete planks (Laboratory Work Report, 2006)

4.2 Comparison of Design Loads with Failure Loads

To compare the two values of load, the following ratio was defined.

$$\beta = \frac{F_{max} - F_{des}}{F_{des}} \quad - \quad (4.1)$$

where β = differential load ratio

F_{max} = maximum load

F_{des} = design load

This was aimed at establishing a benchmark to be achieved for a particular plank to be adjudged suitable to function as a structural member.

For each case of comparison, results obtained using equation (4.1) have been presented in the tables that follow. Table 4.1 is for grade 25 concrete planks and table 4.2 is for grade 30 concrete planks.

Table 4.1: Differential Load Ratios for Grade 25 Concrete Planks.

(a) For 60mm thick planks

Concrete cover (mm)	**F_{des} (kN)**	**F_{max} (kN)**	$\frac{F_{max} - F_{des}}{F_{des}}$
20	1.9	2.3	0.21
15	1.9	2.7	0.42

(b) For 50mm thick planks

Concrete cover (mm)	**F_{des} (kN)**	**F_{max} (kN)**	$\frac{F_{max} - F_{des}}{F_{des}}$
20	1.8	1.9	0.06
15	.8	2.1	0.17

(c) For 40mm thick planks

Concrete cover (mm)	**F_{des} (kN)**	**F_{max} (kN)**	$\frac{F_{max} - F_{des}}{F_{des}}$
20	1.7	1.6	-0.06
15	1.7	1.7	0

(Source: Laboratory Work Report, 2006)

Table 4.2 Differential Load Ratios for Grade 30 Concrete Planks

(a) For 60mm thick planks

Concrete cover (mm)	**F_{des} (kN)**	**F_{max} (kN)**	$\frac{F_{max} - F_{des}}{F_{des}}$
20	1.9	2.4	0.26
15	1.9	2.9	0.53

(b) For 50mm thick planks

Concrete cover (mm)	**F_{des} (kN)**	**F_{max} (kN)**	$\frac{F_{max} - F_{des}}{F_{des}}$
20	1.8	2.1	0.17
15	1.8	2.2	0.22

(c) For 40mm thick planks

Concrete cover (mm)	**F_{des} (kN)**	**F_{max} (kN)**	$\frac{F_{max} - F_{des}}{F_{des}}$
20	1.7	1.6	-0.06
15	1.7	1.7	0

(Source: Laboratory Work Report, 2006)

CHAPTER 5 DISCUSSION

5.1 Effect of Plank Thickness

The thickness of the plank has a direct influence on the load carrying capacity of the planks. It was discovered that the maximum load the planks can carry decreased with the reduction in the plank thickness. The 40mm thick planks had the lowest value for the respective grade of concrete and concrete cover while the 60mm thick planks had the highest load carrying capacity in the respective cases. This is due to the fact that the value of the effective depth and hence the moment of resistance was lowest for the 40mm thick plank and highest for the 60mm thick planks, the value being directly proportional to the overall depth of the section. Also, the load carrying capacity of an element depends on the quantity of materials provided and distributed within the element to resist the applied stresses as stated by Troisky (1976). Concrete acts together with the reinforcement to resist stress in reinforced concrete. The quantity of concrete in the 40mm plank being comparatively low and there is a limit to the amount of steel that can be provided; the load carrying capacity was low.

From the result, the deflection was higher at similar loads for the thinner sections. Theoretically, the deflection is inversely proportional to the quantity EI, the flexural rigidity, which depends on the section properties of the element and the material from which it is made. The value of E is constant for a particular grade of concrete; I varies with the sectional dimensions. Therefore for thinner sections EI will be lower and the deflection value will consequently be higher. Because the deflection is more for the thinner sections and the concrete having low tensile strength, the cracks also developed

at lower loads for these thinner sections of the planks. This is because the rate of deflection being faster with respect to the loads for the thinner planks, the cracks will therefore develop faster.

5.2 Effect of Concrete Cover to Reinforcement

The load at which the planks fail was higher for the 15mm cover than for the 20mmcover. Theoretically, the effective depth d, is higher for 15mm cover. This therefore increased the value of the lever arm and hence the moment of resistance of the section. Practically, where the cover was thicker, the reinforcement was located nearer the centroid of the section. The section acted more like mass concrete compared to where the cover was thinner, and the reinforcement located closer to the extreme fibers in the tension zone (Blades et al, 1991). Since concrete is weak in tension, the reinforcement being further away for the 20mm cover from the tension zone than for the 15mm cover, the failure occurs more quickly.

Also, the widths of the cracks were wider at failure for the 20mm cover. This was because the development of the crack has to extend deeper into the concrete planks before the reinforcement had to be reached. The reinforcement that provided resistance to cracking is located further away from the tension face than for the 15mm cover.

The deflections of the planks were higher in all cases for the 20mm cover than for the 15mm concrete cover for corresponding cases of loading and plank thickness. This was because for the 15mm concrete cover, the steel is more stressed at the tension zone and its behaviour also has more influence on the behaviour of the planks. Steel has higher value of modulus of elasticity E which has an inverse relationship with deflection.

If the value of E is higher, the deflection will be lower. The value of difference in the deflection at a particular load is not much for the two values of concrete cover, the reason being the small difference of 5mm between the values of the concrete cover. As seen, it still showed some difference. However, it was observed that there was no marked difference in the case of 40mm thick planks.

Cracks were developed faster in planks with 20mm concrete cover compared to those with 15mm concrete cover. This was because the reinforcement in the later case is located closer to the fibre having maximum stress in the tension zone. Steel is better in resisting tension than concrete. Therefore the cracks will take a longer time to develop and propagate in this case than in the case of the 20mm concrete cover because of the higher tensile resistance of steel.

5.3 Effect of Concrete Strength

The loads at which the planks failed in almost all cases varied with the variation in the strength of concrete. Two grades of concrete were used, grade 25 and grade 30. For corresponding cases (*i.e.* similar thickness and similar cover) the planks made from grade 30 concrete failed at higher loads than those made from grade 25 concrete. The grade 25 concrete had higher water/cement ratio than the grade 30 concrete. The porosity as the water dried out and concrete hardened was therefore higher for the grade 25 concrete. This results in reduction of the concrete strength (Roy and Gonda, 1973). For this reason the grade 25 concrete will resist lesser amount of load as indicated in the results obtained.

Cracks developed faster in specimens made using grade 25 concrete. This is because the flexural strength of concrete increases with increase in compressive strength of concrete according to Neville and Brooks (1994), though the relation is not a direct proportionality. For this reason the resistance to flexural stress is higher for the grade 30 concrete. The higher resistance implies that the initiation and propagation of cracks will be faster in the specimens made with grade 25 concrete.

With respect to deflection, for the thicker sections the deflection was more for grade 25 concrete; with the 40mm thick plank, there was no pronounced difference for the two grades of concrete.

For the above reason also, the deflections were also higher for grade 25 concrete planks than for the grade 30 concrete.

5.4 Comparison of Design Loads with Maximum Loads

As seen generally from tables 4.1 to 4.2, in almost all the cases, the differential load ratios were higher for grade 30 concrete than for grade 25 concrete. Also the values were higher for the 15mm concrete cover than for the 20mm concrete cover for both grades of concrete. The behaviour of some of the planks in this respect was highly unsatisfactory with the 40mm plank recording a value of -0.06 in a particular instance. The results generally show that the behaviours of the 50mm and 40mm planks under load are unstable. The 60mm thick planks gave encouraging values ranging between 0.21 at the lower end to a maximum of 0.53.

CHAPTER 6 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary of Findings

1 The planks failed in all cases by the cracking of the concrete in the tension zone as shown in plate 4.1.

2. The maximum load at which failure occurred varied depending on the plank thickness, the concrete cover to the reinforcement as well as the grade of concrete.

3. The deformation of the planks varied with the variation in each of these parameters mentioned in item 2 above.

4. The load -deflection curve for each case was similar in shape to the stress-strain curve for steel at the initial stage of the loading.

5. The maximum load obtained was 2.7KN for the 15mm cover using the grade 25 concrete and 2.9KN for the grade 30 concrete.

6. In all the tests, it was only in three cases that the value of load differential ratio as defined in (4.1) exceeded 0.25. All these cases occurred in the 60mm thick planks.

7. The behaviour of the 40mm plank was highly unreliable, and with the higher grade of concrete, the load-deflection curves were not distinguishable for the two values of concrete cover as indicated in figure 4.2.

6.2 Conclusion

From the investigation carried out and the results obtained, the following conclusions were drawn.

1. The theoretical (design) load a structural element can carry is usually less than the maximum experimental (or actual) load that the element can carry.

2. The thickness of a structural member subjected to bending has a direct influence on the structural behaviour of that particular member. The flexural strength is higher the thicker a section is. The deflection is lower for any given load for the thicker sections than for the thinner ones.

3. The concrete cover to the reinforcement influences the structural behaviour. The relationship is an inverse one i.e. the thinner the concrete cover for the reinforcement, the higher the amount of load the element can carry. The deflection on the other hand reduces comparatively for the lower value of concrete cover to the reinforcement.

4. The ductility of a reinforced concrete element in flexure is enhanced when the concrete cover to the reinforcement in the tension zone is less. The steel being closer to the extreme fibre in the tension zone controls the behaviour of the reinforced concrete element in bending. This was responsible for the curves taking a shape close to that of the stress-strain curve for the steel.

5. The rigidity of a reinforced concrete member under flexure increases with the reduction in the concrete cover to the reinforcement. This can be seen from the curve as those curves for the lesser amount of cover have higher slope. Higher slope of the load-deformation curve is an indication of higher modulus of rigidity.

6. Cracks are more quickly initiated in elements made using lower grades of concrete in comparison to those made using higher grades of concrete.

7. Equation (4.1) reproduced below has been proposed to give an indication of the amount of load a member can carry above the value of the design load.

$$\beta = \frac{F_{max} - F_{des}}{F_{max}}$$

8. Even though it is desired that structural elements should be made as thin as possible to increase their efficiency (*i.e.* being more stressed), reducing the size below certain limits will serve no structural purpose at all. This is the reason why the behaviour of the 40mm thick planks was so unpredictable. It therefore cannot be relied upon as a structural member.

9. The structural behaviour of reinforced concrete planks in theory agrees largely with the behaviour of such elements under test loads.

6.3 Recommendations

1. The differential load ratio as earlier defined should not be less than 0.20. This is to take into consideration the fact that, even though partial safety factors were used in

the design, the effect of impact load was not specifically designed for. This extra amount of load resistance will be adequate to take care of any normal impact load that may be expected in the use for which the planks have been designed.

2. The least practicable amount of concrete cover should be used for the reinforcement as this will increase the load carrying capacity of the planks. For this reason a minimum concrete cover of 15mm is recommended to take into consideration the durability requirement.

3. A minimum yield strength of 318 N/mm^2 is recommended for reinforcement fabrics to be used in the production of the concrete planks.

4. A maximum aggregate size of 10mm is recommended for use in making the concrete for the planks.

5. Only the 60mm thick planks showed consistency in behaviour and it is therefore recommended as the minimum thickness for the planks in agreement with the recommendation of Blades *et al* (1991) in this regard.

6. The effect of the use of higher grades of concrete and reinforcement than have been used could be investigated to check if the minimum plank thickness could further be reduced.

7. The economy of using lower grades of these materials over the use of higher grades of these materials could also be investigated as another area of further investigation.

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APPENDIX A: CONCRETE MIX DESIGNS

Grade 25 concrete

Target mean strength = characteristic strength + 1.64σ

There was close supervision, batching was by weight. Standard deviation value of 4 was therefore used.

Therefore, target mean strength = 25 + 1.64 x 4 = 32 N/mm^2.

From table A2 (the tables used are also attached) the compressive strength of a mix made with Ordinary Portland Cement and water/cement ratio of 0.50 is 49 N/mm^2. Therefore the free water/cement ratio for the required mean compressive strength is approximately equal to 0.6. However, by reference to table 3.3 of BS 8110, for nominal cover of 20mm and mild condition of exposure, maximum free water/cement ratio permitted is 0.55. The lower value was adopted.

Adopted value of water/cement ratio = 0.55.

From table A1 the approximate workability requires a slump of 75mm so that the approximate free water content required for crushed coarse aggregates with a maximum size of 10mm is 250 kg/m^3 of concrete from table A3.

The cement content = $\frac{250}{0.55}$ = 455kg/m^3

According to table 3.3 of BS 8110, the minimum cement content = 325kg/m^3< 455kg/m^3. Therefore estimated cement content is satisfactory.

For aggregate with bulk specific gravity of 2.65, the wet density of concrete is 2400kg/m^3.

∴ Total aggregate content = 2400 – (250 + 455) = 1695kg/m^3 of concrete.

From figure A1, for free water cement ration of 0.55, a slump of 75mm and maximum size of aggregates of 10mm, fine aggregate content is 42% of total aggregate content.

The fine aggregate content = 0.42 x 1695 = 712 kg/m^3 of concrete.

The coarse aggregate content = 1695 – 712 = 983 kg/m^3 of concrete

Estimated quantities in kg/m^3 of concrete are as follows

Cement: 455 kg/m^3

Fine Aggregates: 712 kg/m^3

Coarse Aggregates: 983 kg/m^3

Added water: 250 kg/m^3

Mix ratio: 1:1.6:2.2/0.55.

Number of planks per day = 6

For 60mm thick planks

Volume of concrete required per day = 6 x 1.2 x 0.25 x 0.06 = 0.108m^3

Add 10% for waste = $\underline{0.011m^3}$

$0.119m^3$

Materials needed for the day

Cement	=	455 x 0.119	=	54.2kg
Sand	=	712 x 0.119	=	84.7kg
Gravel	=	983 x 0.119	=	117kg
Water	=	250 x 0.119	=	29.8kg

For 50mm thick planks

Volume of concrete required per day = 6 x 1.2 x 0.25 x 0.05 = 0.09m^3

Add 10% for waste = $\underline{0.009m^3}$
$0.099m^3$ per day

Materials needed for the day

Cement	=	455 x 0.099	=	45.1kg
Sand	=	712 x 0.099	=	70.5kg
Gravel	=	983 x 0.099	=	97.3kg
Water	=	250 x 0.099	=	24.8kg

For 40mm thick planks

Volume of concrete required per day = 6 x 1.2 x 0.25 x 0.04 = $0.072m^3$

Add 10% for waste = $\underline{0.007m^3}$
$0.079m^3$ per day

Materials needed for the day

Cement	=	455 x 0.79	=	36kg
Sand	=	712 x 0.079	=	56.3kg
Gravel	=	983 x 0.079	=	77.7kg
Water	=	250 x 0.079	=	19.8kg

Grade 30 concrete

Target mean strength = 30 + 1.64 x 4 = 37 N/mm^2.

From table A2 the compressive strength of a mix made with ordinary Portland cement and water/cement ratio of 0.50 is 49 N/mm^2. Therefore the free water/cement ratio for the required mean compressive strength of 37 N/mm^2 is 0.52. However, from Table 3.3 of BS 8110, for nominal cover of 20mm and mild condition of exposure, maximum free

water/cement ratio permitted is 0.50. The lower value is used. Therefore the water/cement value adopted is 0.50.

From table A1, the approximate workability requires a slump of 75mm, so the approximate free water content required for crushed coarse aggregates with maximum size of 10mm is 250kg/m^3 of concrete.

The cement content required = $\frac{250}{0.50}$ = 500kg/m^3

According to table 3.3 of BS 8110 the minimum cement content = 350kg/m^3.

Therefore the estimated cement content is satisfactory.

For aggregate with bulk specific gravity of 2.65, the wet density of concrete is 2400kg/m^3.

∴ Total aggregate content = 2400 – (250 + 500) = 1650 kg/m^3 of concrete.

From figure A1, for free water/cement ratio of 0.50, a slump of 75mm and maximum aggregate size of 10mm, fine aggregate content is 40%

∴ The fine aggregate content = 0.40 x 1650 = 660kg/m^3

The coarse aggregate content = 1650 – 660 = 990kg/m^3

Estimated quantities in kg/m^3 of concrete are as follows.

Cement: 500 kg/m^3

Fine aggregates: 660 kg/m^3

Coarse aggregates: 990 kg/m^3

Added water: 250 kg/m^3

Mix ratio: 1:1.3:2/0.5

Number of planks per day = 6.

For 60mm thick planks

Total volume of concrete required per day = 0.119m^3

Materials needed for the day

Cement = 500 x 0.119 = 59.5kg

Sand = 660 x 0.119 = 78.5kg

Gravel = 990 x 0.119 = 117.8kg

Water = 250 x 0.119 = 29.8kg

For 50mm thick planks

Number of planks per day = 6

Total volume of concrete required per day = 0.099m^3 per day

Materials needed for the day

Cement = 500 x 0.099 = 49.5kg

Sand = 660 x 0.099 = 65.3kg

Gravel = 990 x 0.099 = 98kg

Water = 250 x 0.099 = 24.8kg

For 40mm thick planks

Number of planks per day = 6

Total volume of concrete required = 0.079m^3 per day

Materials needed for the day

Cement = 500 x 0.079 = 39.5kg

Sand = 660 x 0.079 = 52.1kg

Gravel = 990 x 0.079 = 78.2kg

Water = 250 x 0.079 = 19.8kg

Table A1: Workability, slump, and compacting factor of concretes with 19 or 38mm maximum size of aggregate.

Degree of Workability	Slump mm	Compacting factor	Use for which concrete is suitable
Very low	0-25	0.78	Roads vibrated by power-operated machines. At the more workable end of this group, concrete may be compacted in certain cases with hand-operated machines.
Low	25-50	0.85	Roads vibrated by hand-operated machines. At the more workable end of this group, concrete may be manually compacted in roads using aggregate of rounded or irregular shape. Mass concrete foundations without vibration or lightly reinforced sections with vibration.
Medium	25-100	0.92	At the less workable end of this group, manually compacted flat slabs using crushed aggregates. Normal reinforced concrete manually compacted and heavily reinforced sections with vibrations.
High	100-175	0.95	For sections with congested reinforcement. Not normally suitable for vibration.

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Table A2: Approximate compressive strengths (N/mm^2) of concrete mixes made with a free-water/cement ratio of 0.5

Type of Cement	Type of Coarse Aggregate	Compressive strengths (N/mm2) Age (days) 3	7	28	91
Ordinary Portland (OPC) or Sulphate-Resisting Portland (SRPC)	Uncrushed	22	30	42	49
	Crushed	27	36	49	56
Rapid-Hardening Portland (RHPC)	Uncrushed	29	37	48	54
	Crushed	34	43	55	61

Table A3: Approximate free-water contents (kg/m^3) required to give various levels of workability

Slump (mm)		0-10	10-30	30-60	60-180
Vebe time(s)		>12	6-12	3-6	0-3
Maximum Size Aggregate (mm)	Type of aggregate				
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

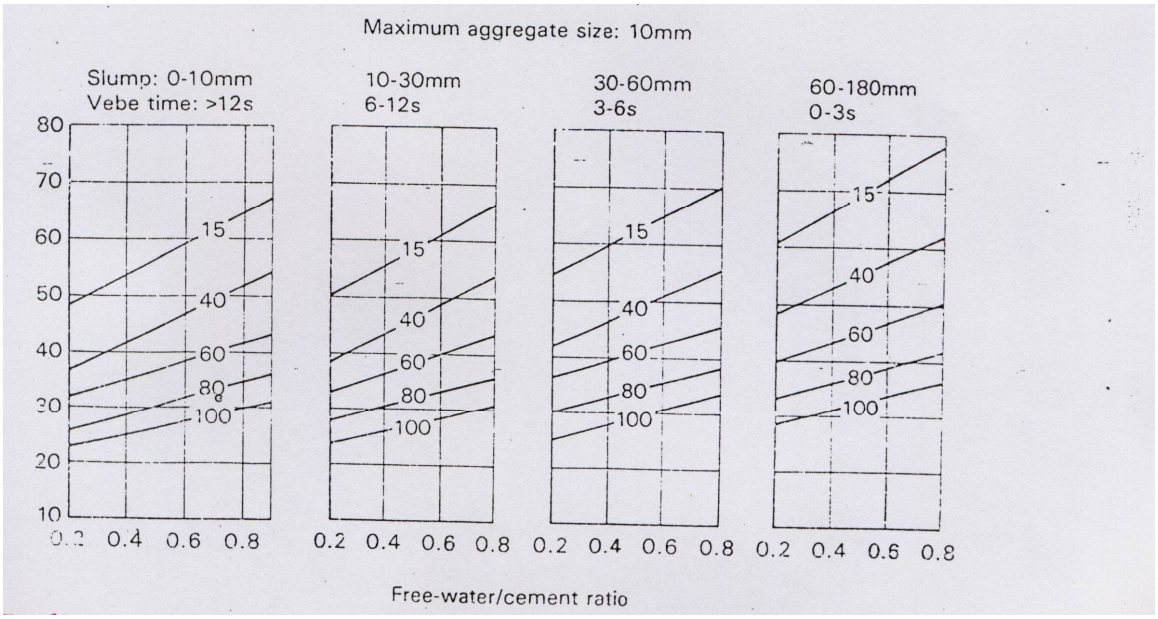


Figure A1: recommended proportions of fine aggregate according to percentage passing a 600µm sieve

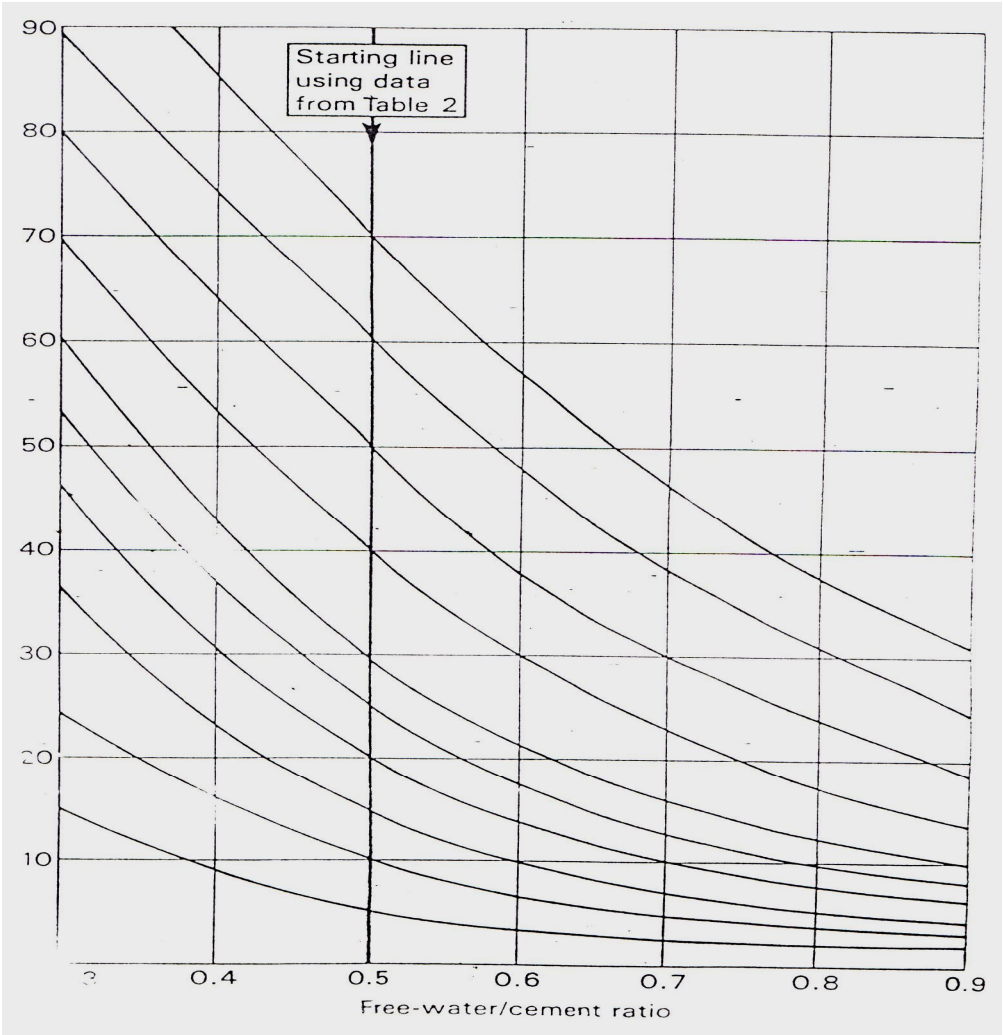


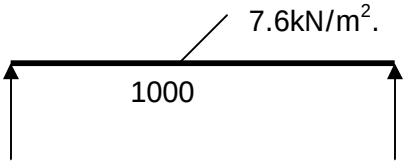
Figure A2: Relationship between compressive strength and free water/cement ratio

APPENDIX B: DESIGN OF REINFORCED CONCRETE PLANKS

For 60mm thick planks.

Loading	Per Square Metre
Self weight 0.06 x 24	= 1.44kN/m^2
Finishes	= 0.6kN/m^2
Characteristic Dead Load	= 2. 0kN/m^2 G_k=2.0kN/m^2
Characteristic Live Load	= 3. 0kN/m^2 Q_k=3.0kN/m^2
Design Load = $1.4G_K + 1.6Q_K$	
= 1.4 x 2 + 1.6 x 3	= 7.6kN/m^2 n = 7.6kN/m^2

Ultimate Moment



Max span moment = $\frac{n\ell^2}{8}$

= $7.6x1^2/8$ = 1kNm/m

Reinforcement

For Grade 25 Concrete

(i) 20mm concrete cover, d = 60 – 20 – 3/2 = 38.5 ≈ 38mm

$k = M/f_{cu}bd^2 = \frac{1x10^6}{25x1000x38^2}$ = 0.03, z = 0.95d = 36

$A_s = M/0.95f_yz = \frac{1x10^6}{0.95x318x36}$ = 91.9mm^2

(ii) 15mm Cover d = 43

$k = 1x106/25x1000x43^2$ = 0.02, Z = 0.95d = 41

$A_s = 1x10^6/0.95x318x41$ = 80.7mm^2

For Grade 30 Concrete

(i) 20mm concrete cover

A_s = 91.9mm^2

(ii) 15mm concrete cover

A_s =80.7mm^2

For 50mm thick planks

Loading		Per Square Metre
Self weight 0.05 x 24	=	1.2kN/m^2
Finishes	=	0.6kN/m^2
Characteristic dead load	=	1.8kN/m^2. G_k=1.8kN/m^2
Characteristic live load	=	3.0kN/m^2. Q_k= 3.0kN/m^2
Design Load = 1.4 x 1.8 + 1.6 x 3	=	7.3kN/m^2 n = 7.3kN/m^2

Ultimate BM

$M = \frac{7.3 \times 1^2}{8}$ = 0.9kNm/m

Reinforcement

For Grade 25 Concrete

(i) 20mm cover d = 50 – 20 - 3/2 = 28

$k = \frac{0.9 \times 10^6}{25 \times 1000 \times 28^2}$ = 0.05, Z = 0.94d = 26

$A_s = \frac{0.9 \times 10^6}{0.95 \times 318 \times 26}$ = 114.6mm^2/m

(ii) 15mm cover d = 33, z = 0.95d = 31

$A_s = \frac{0.9 \times 10^6}{0.95 \times 318 \times 31}$ = 96.1mm^2/m

For Grade 30 Concrete

(i) 20mm cover d = 28

$k = 0.9x10^6/30x1000x28^2$ = 0.04, z = 0.95d = 27

$A_s = \frac{0.9 \times 10^6}{0.95x318x27}$ = $110.4mm^2/m$

(ii) 15mm cover d = 33

$k = 0.9x10^6/30x1000x33^2$ = 0.03, z = 0.95d = 31

$A_s = \frac{0.9 \times 10^6}{0.95x318x31}$ = $96.1mm^2/m$

For 40mm thick planks

Loading		Per square metre
Self weight 0.04x24	=	$1.0kN/m^2$
Finishes	=	$0.6kN/m^2$
Characteristic dead load	=	$1.6kN/m^2$. $G_k=1.6KN/m^2$
Characteristic live load	=	$3.0kN/m^2$. $Q_k = 3.0KN/m^2$
Design Load = 1.4 x 1.6 + 1.6 x 3	=	$6.7kN/m^2$. $n = 6.6KN/m^2$

Ultimate BM

$M = \frac{6.7 \times 1^2}{8} = 0.8\text{KNm/m}$

Reinforcement

For Grade 25 Concrete

(i) 20mm cover d = 40 – 20 3/2 = 18mm

$k = \frac{0.8\text{x}10^6}{25\text{x}1000\text{x}18^2} = 0.1$

$z = d\,[0.5 + \sqrt{(0.25 - \frac{0.1}{0.9})}] = 0.87d = 16$

$As = \frac{0.8\text{x}10^6}{0.95\text{x}318\text{x}16} = 165.6\text{mm}^2/\text{m}$

(ii) 15mm cover d = 23mm

$k = \frac{0.8\text{x}10^6}{25\text{x}1000\text{x}23^2} = 0.06$

$z = d\,[0.5 + \sqrt{(0.25 - \frac{0.06}{0.9})}] = 0.93d = 21$

$A_s = \frac{0.8\text{x}10^6}{0.95\text{x}318\text{x}21} = 126.1\text{mm}^2/\text{m}$

For Grade 30 Concrete

(i) 20mm cover, d = 18mm

$k = \frac{0.8\text{x}10^6}{30\text{x}1000\text{x}18^2} = 0.08,\quad z = 0.90d = 16$

$A_s = \frac{0.8\text{x}10^6}{0.95\text{x}318\text{x}16} = 165.6\text{mm}^2/\text{m}$

(ii) 15mm cover d = 23mm

$k = \frac{0.8 \times 10^6}{25 \times 1000 \times 23^2} = 0.05, \quad z = 0.94d = 22$

$A_s = \frac{0.8 \times 10^6}{0.95 \times 318 \times 22} = 120.4mm^2$

APPENDIX C: EXPERIMENTAL RESULTS

Table C1 Loading of the 60mm planks for grade 25 concrete

(a) For 20mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	5	
2	13	
2.3	18	Maximum load
2	28	Collapse load

(b) For 15mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	4	
2	11	
2.7	20	Maximum load
2.1	28	Collapse load

Table C2 Loading for the 50mm planks for grade 25 concrete

(a) For 20mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	6	
1.9	17	Maximum load
1.5	30	Collapse load

(b) For 15mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	5	
2	14	
2.1	20	Maximum load
1.8	36	Collapse load

Table C3 Loading of the 40mm planks for grade 25 concrete

(a) For the 20mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	12	
1.6	20	Maximum load
1.3	26	Collapse load

(b) For the 15mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	10	
1.7	19	Maximum load
1.3	27	Collapse load

Table C4: Loading of the 60mm planks for grade 30 concrete

(a) For the 20mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	3	
2	10	
2.4	15	Maximum load
2	25	Collapse load

(b) For the 15mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	2	
2	9	
2.9	18	Maximum load
2.5	28	Collapse load

Table C5: Loading of the 50mm planks for grade 30 concrete

(a) For the 20mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	5	
2	15	
2.1	22	Maximum load
1.7	32	Collapse load

(b) For the 15mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	5	
2	10	
2.2	19	Maximum load
1.9	35	Collapse load

Table C6 Loading of the 40mm planks for grade 30 concrete

(a) For the 20mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	10	
1.7	20	Maximum load
1.3	30	Collapse load

(b) For the 15mm concrete cover

Load (kN)	Deflection (mm)	Remark
1	10	
1.7	20	Maximum load
1.4	30	Collapse load