REINFORCED CONCRETE DESIGN OF A PAIR OF BUNKERS FOR GRAINS

BY

SOLOMON UMORU AGIDAN1
B.Sc. Building (A.B.U.) Zaria

THE AUTHOR

The Author, Agidani Solomon Umoru was born on 15th December, 1958.

He attended L.E.A. School School Obagaji Agatu and S.I.M. primary school Gusau (1968-1972). He had his secondary school education at Government College Keffi (1973-1977), from where he proceeded to the School of Basic Studies Ahmadu Bello University, Zaria (July 1977-June 1978). At the end of his basic course, he was admitted to the Department of Building from where he graduated with B.Sc. Building (2nd Class Lower Division).

He returned to the department in 1985 to read M.Sc. Construction Technology and it is part of the fulfilment of this process that this thesis has been written.

The author is married.

DECLARATION

I hereby declare that this thesis has been composed by me and that it is the record of my own work. It has not been presented in any previous application for a higher degree.

All sources of information are specifically acknowledged by means of references at the end of the thesis.

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AGIDANI, Solomom Umoru

November, 1985.

Date

DEDICATION

This thesis is dedicated to my father

Mr. Agidani Onye and my mother

Mrs. Adobunu Agidani.

CERTIFICATION

This thesis/dissertation entitled "REINFORCED CONCRETE DESIGN OF PAIR OF BUNKERS FOR GRAINS" by Agidani, Solomon Umoru meets the regulations governing the award of the degree of Master of Science in Construction Technology of Ahmadu Bello University, and is approved for its contribution to knowledge and literary presentation.

Dr. A. A. Husseini,
(Chairman, Supervisory Committee) Date: 23/9/88

Mr. U. K. Aimiuwu,
(Member, Supervisory Committee) Date: 23/9/88

Dr. A. A. Husseini,
Head of Department of Building. Date: 5/12/88

Professor D. Saror,
Dean, Post Graduate School. Date: 26-1-89

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ABSTRACT

This thesis treats the design of a pair of bunkers, each having a capacity of 200 tons of maize, using the British code of practice CP 110 and the B.S. Cp 3 chapter five for loading (to compute wind load). As a guide to practical considerations some notes are given on such factors as filling and emptying procedures, minimum angle of hopper slope and so forth which should be of benefit to those who are meeting the subject for the first time,

Elements of a bunker are classified in the same way as elements of other structures; they have been fully analysed and designs have been carried out accordingly.

It should be emphasised that treatment of those things that make bunkers a peculiar kind of structure have been done but in no way exhaustive. Further research in future may reveal this,

Acomparative cost analysis has been included for bunkers in steel and reinforced concrete to back up the choice of the use of reinforced concrete bunkers for small and medium sized installations.

Conclusion and recommendations have been drawn and all authorities consulted have been acknowledged in the Bibliography and reference.

TABLE OF CONTENTS

LIST OF SYMBOLS

A	Cross Sectional Area of Members
Ac	Area of Concrete
Aso	Area of Congutidunal Reinforcement (for columns)
Ast	Area of Tensile Reinforcement
Asv	Cross Sectional Area of Two Legs of Stirrups
b	Width of Section
b	Breadth of Section at Level of Tension Reinforcement
d	Effective Depth of Tension Reinforcement
e	Eccentricity
eadd	Additional Eccentricity Due to Deflection of Slender Columns
F	Ultimate Load
fbs	Bond Stress
fcu	Characteristic Concrete Cube Strength
fs	Service Stress
fy	Characteristic strength of Reinforcement
fyv	Characteristic Strength of Link Reinforcement
Gk	Characteristic Dead Load
gk	Characteristic Dead Load per Unit Area or Unit Length
h	Overall Depth of Section
Hf	Depth of Bearing Stratum in Foundation
I	Second Mementia of Area
K	Stiffness of Member $\frac{EI}{L}$
La	Leaver Arm
Le	Effective Height of Column of Wall

Lex	Effective Height for Bending about the Major Axis
Ley	Effective Height for Bending about the Major Axis
Lo	Clear Height of Column between and Restraints

CHAPTER - ONE

INTRODUCTION

1.1 GENERAL INTRODUCTION

Bunkers (Shallow bins) and silos (deep bins) are vessels designed for storing, over a temporary period of time, a pre-determined quantity of dry material such as coal, coke, iron ore, gravel, grains, etc. These structures are generally elevated above the ground to facilitate gravity feed from the mouths on to trucks which transport the material from storage point to where it is to be processed. The bottom of the container is normally fixed at an elevation greater than three metres in order to allow enough head room for trucks to go beneath for loading purpose. Emphasis in this thesis is on the shallow containers (bunkers). They may be square, rectangular or circular and arranged singularly or in banks.

Square or rectangular bunkers have pyramidal bottoms with a single outlet, but a trough bottom is sometimes used with a single elongated outlet or with two or more square or circular outlets. Very large square or rectangular bunkers are often sub-divided by division walls, each compartment having its own outlet from a pyramidal bottom. Circular bunkers have conical bottoms with a single outlet.

Bunkers may be completely symmetrical or non-symmetrical about a given vertical axis. The pyramid, trough or cone may be off-set. As a matter of fact many variations are possible, dependent on plant requirements or for some mechanical reason.

Bunker installations can be seen in power stations, cement factories, gas works, iron and steel works, quarries, grains storage depots, etc. In short, at any establishment where elevated bulk storage of material is deemed for some purpose to be desirable.

In bunkers, the plane of rupture of the contained material cuts the surface of the container before meeting the opposite wall, while in silos, the plane of rupture does not cut the surface of the container. These are illustrated in figures 1.1 and 1.2.

The friction of stored materials against the bunker walls during discharge is insignificant and is, therefore, discarded in design.

Where different particulate materials are to be stored at one location, use is made of grouped bunkers with separate filling and discharge openings. This type of bunkers can be put to dual storage purpose, if the materials requiring to be stored have close dry densities. Another method of storing

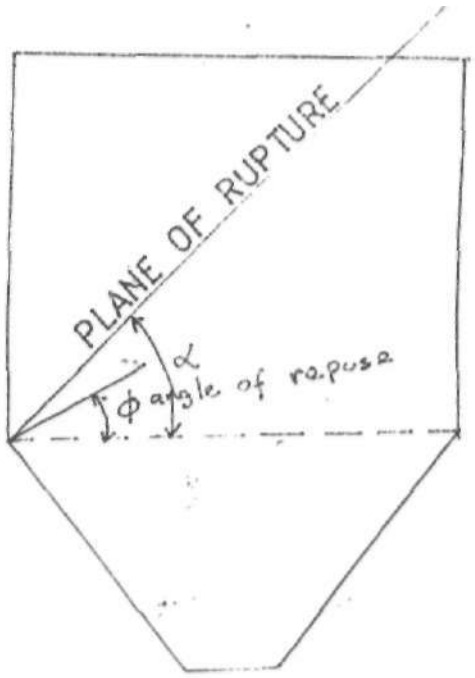


FIGURE 1.1
CROSS SECTION OF A BUNKER

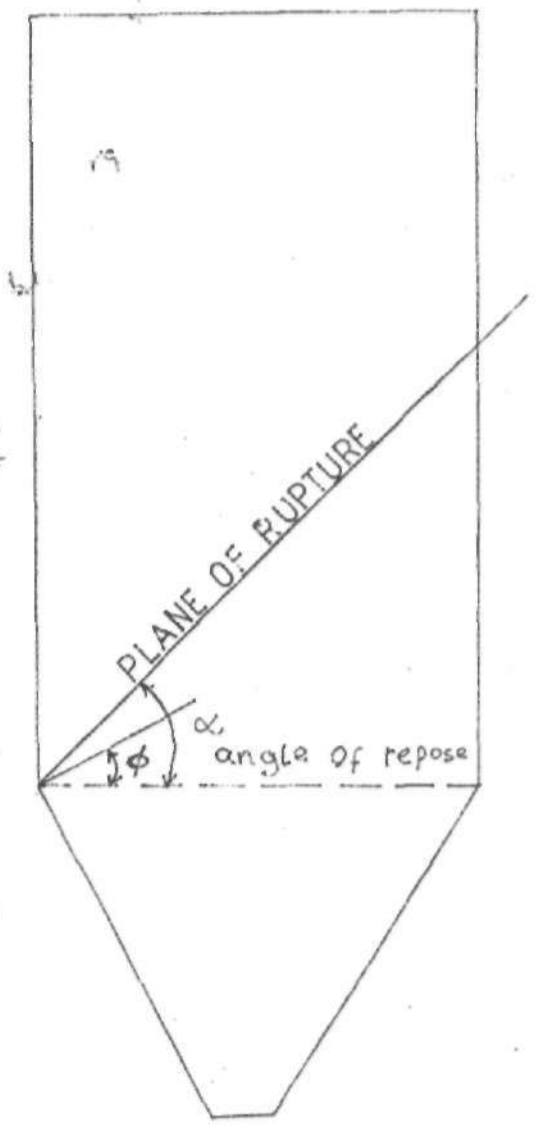


FIGURE 1.2
CROSS SECTION OF A SILO

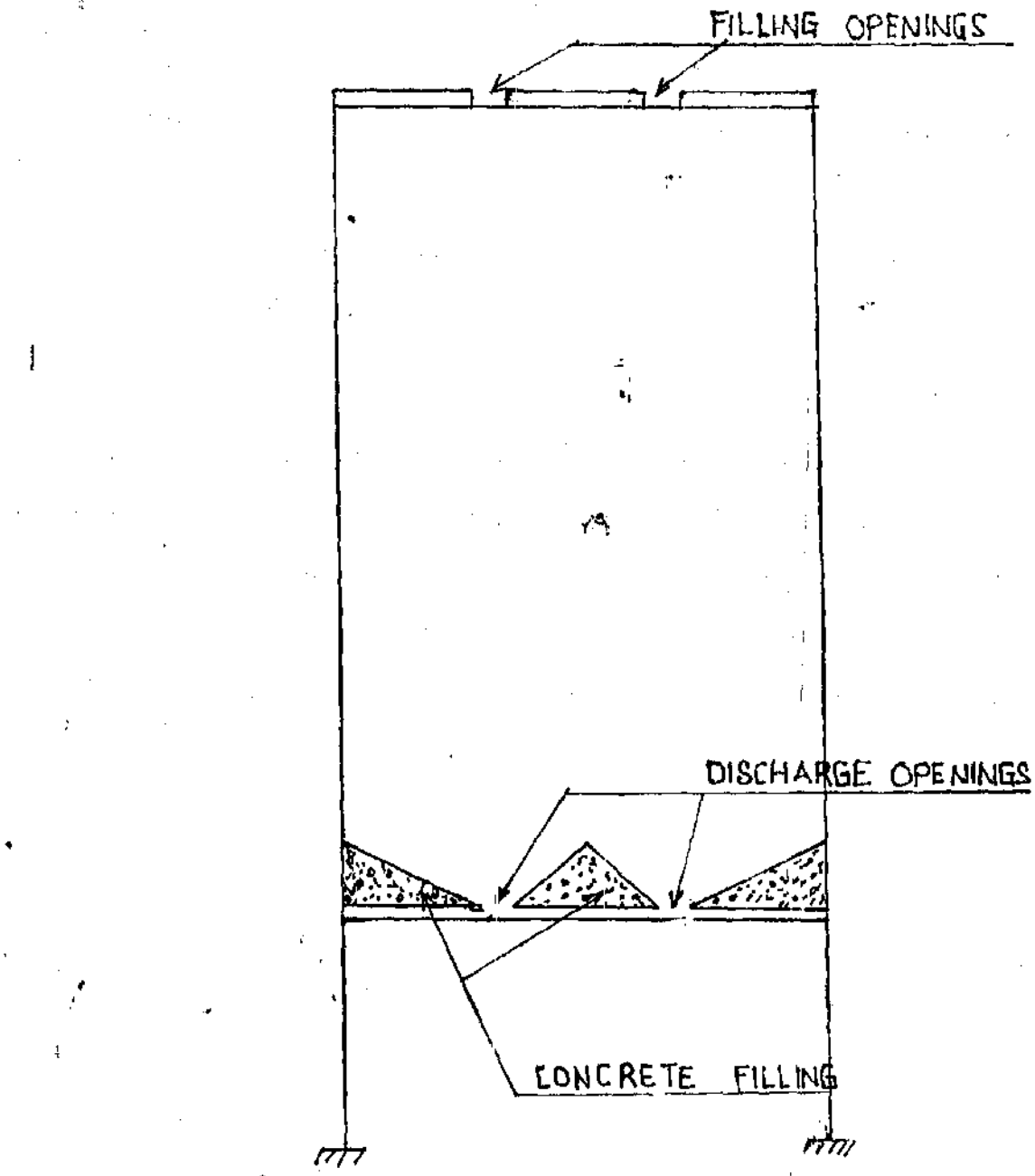


FIGURE 1.3 TROUGH TYPE BUNKER

once described maize in these words: "It is the standard with which to measure all other grains". (Schneider B.F.)[1]

Perhaps one area that deserves a great deal of attention in Nigeria's renewed effort to increase food production is the provision of storage facilities. It is only when we are able to store-away our excess grains after a bumper harvest that we can maintain and if possible step up production. It is only then that our farmers can produce as much quantity of assorted foodstuff without fear of market prices. It is only through how much of our grains we have in storage that can help us to estimate our level of production as against the level of consumption which will in turn help us to know what amount of excess we can export.

Where storage facilities are absent, the common sight is that markets are flooded in the period immediately following harvest. The farmers are forced by lack of storage facilities to sell out their produce at give away price. Soon after this period of abundant food is a period of scarcity with soaring prices of food-stuffs. High prices at this time can be explained in two ways:

1. those who did not sell early will have realised that at least 20% of the foodstuff they preserved has gone bad due to poor storage methods and so in order to cover storage losses they increase the prices and
2. the commodities are more scarce now, so going by the economic theory of demand and supply, prices soar.

From the foregoing argument therefore, it should not be surprising that of the methods of increasing agricultural productivity always discussed, reducing waste, especially those due to poor storage is usually given high priority.

The author of this thesis is a villager who has lived among farmers for a very substantial part of his life. He has therefore identified with this perenial problem. He believes that by designing a bunker for maize storage, he is contributing his 'widow's mite in enhancing knowledge in the area of design and construction of storage tanks for grains. Granted that this is successful, the author will have achieved a blue print in Nigeria's agrarian revolution.

In choosing the dimensions, the author was quite careful to come within a reasonable capacity that is affordable both in terms of cost and present storage need of most medium sized farmer organisations.

The author particularly had in mind the lower Benue River Basin Development Authority farm project at Obagaji (the author's village) which has been in dire need of storage facilities. The authorities in 1983 farming season realised about fifty tone of maize from its demonstration farm. In 1984 they stepped up production by cultivating more land and using high yield variety to one hundred and fifty (150) tons of the commodity. The projected harvest for this season is between 200 and 250 tons.

The author therefore intends to put forward a proposal to the authorities on the use of bunkers to store their grains, instead of selling the grain early at give-away prices.

1.30 DESIGN APPROACH

The design and construction of bunkers and silos are highly specialised jobs requiring both skill and experience to accomplish satisfactorily. There is no code that has come out to date with laid down procedures for the design of bunkers. The approach that has been used in this thesis therefore isto break the whole structure into elements and analysing the stresses acting on each of these elements. The elements are designed, based on the limit state theory, which is the current British standard code of practice CP110. There are cases that the author did not hold too

and as guide to those who may wish to know the cost implications of using steel or reinforced concrete to construct bunkers.

CHAPTER - TWO

DESIGN APPROACH

2.1 BUNKER TECHNOLOGY

When the design of a bunker is to be prepared, the required capacity is specified or a general sketch giving the internal dimensions is supplied. The dry density of the material must be established from available data. A bunker should never be used to store materials of greater density than that of the material for which it was designed. The slopes of the hopper should be so made that the bunker is self-clearing. In addition, in order to allow the bunker to clear itself easily and in order to prevent excessive wear on slope from rough filling, the bottom is sometimes covered with steel tiles, ceramic, etc.

To calculate the pressures on the side of the walls of a shallow bunker, one of the methods which is used for a wall retaining a filling of unlimited extent may be applied. Several useful theories and methods exist to determine the lateral pressure, some of which are Rankine and Coulomb's theories.

The friction of stored materials against the bunker walls during discharge is insignificant and is, therefore, discarded in design.

The basic principle in the construction of bunkers and silos and similar elevated structures are the

same as those of any other reinforced concrete works, but there are some special feature due particularly to the great height and limited area in plan of most of such structures. The open framework of the columns and braces which constitute the structure of bunkers, necessitates the use of scaffold which is not the same in a building. A feature of some large bunkers and most tall silos is the uniformity of horizontal cross-section throughout the height of the wall. This uniformity has led to the development and economical use of continously moving sliding formwork. Special forms are required for pyramidal and other sloping bottoms.

For the trough bottoms and low walls of long bunkers, the forms are strutted off the completed work below, so there are stages in the construction of bunker forms.

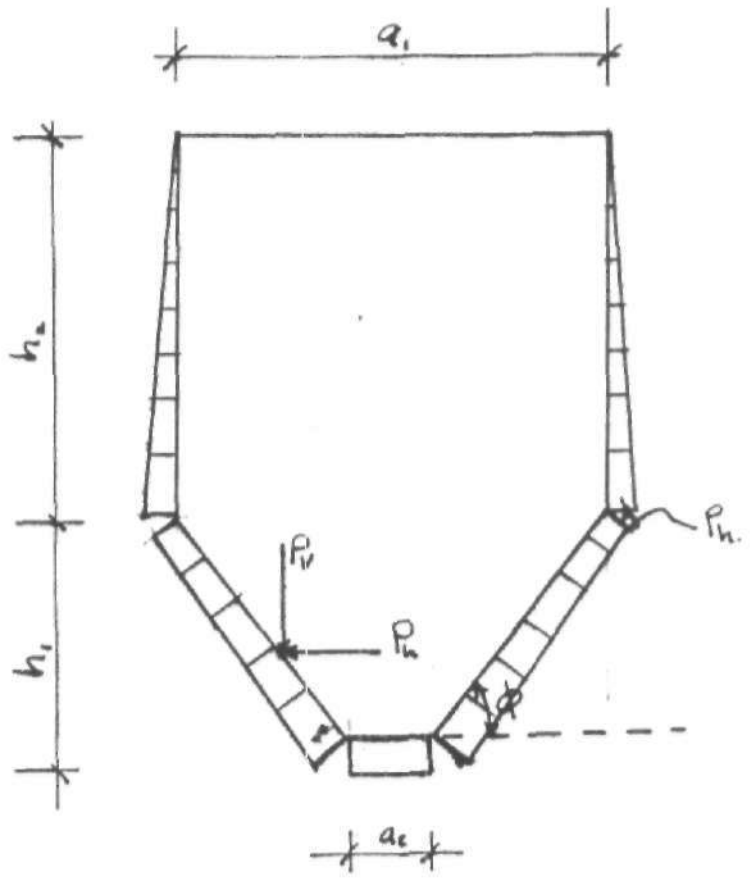


Figure 2.1: Pressures on the wall of a bunker

2.1.1 SLOPPING BOTTOMS

The slopes of the bottoms are always of such a steepness that both faces of the bottom slab require formwork. Since the hopper bottom is suspended from the walls or from beams along the top of the slopes, it is essential that the slopping bottoms should have been constructed and matured before the propping is removed from below the hopper bottom.

With the conventional design of hopper bottom (figure 2.2), the amount of formwork required is above the average for normal concrete slab construction, and, owing to the pyramidal or conical shape, the joiner's work is more complex and is made more intricate by the introduction of corner splays. When good carpenters are scarce, or their wage rates are high, it may pay off to consider the alternative of constructing a flat bottom or a trough container, making up the slopes subsequently as shown in figure 2.2b. Although both the area of framework and volume of concrete may be increased, the greater ease of placing the latter and the much simpler formwork may result in an overall saving. The wall forms, if a container with a flat bottom is being built, can be started at the outlet level instead of doing this at the top of the slopes. The corner filling can be of cheap concrete.

The fillets and surfacing can be applied as a rendering or as an anti-abrasive, tiled or other specified surface. An alternative method is to precast the hopper bottom with vertical suspension bars projecting. Bottoms can be precast in sections which are bolted together.

A factor which should not be over looked in bunker design is the slope of the walls in the trough or conical section. In order to assist in the flow of the material, the cone or trough angle should be as steep as possible. In the case of bunkers with pyramidal bottoms, the corner angle is the criterion. Whilst the sloping sides may appear to be steep enough, this corner angle may present a problem and retard flow. The corner angle for a rectangular bunker is given by

$$\phi = \tan^{-1} \sqrt{\frac{2h^2}{b^2 + a^2}}$$

and for a square bunker a = b, so

$$\phi = \tan^{-1} \frac{h \sqrt{2}}{b \sqrt{2}}$$

$$\phi = \tan^{-1} h/b$$

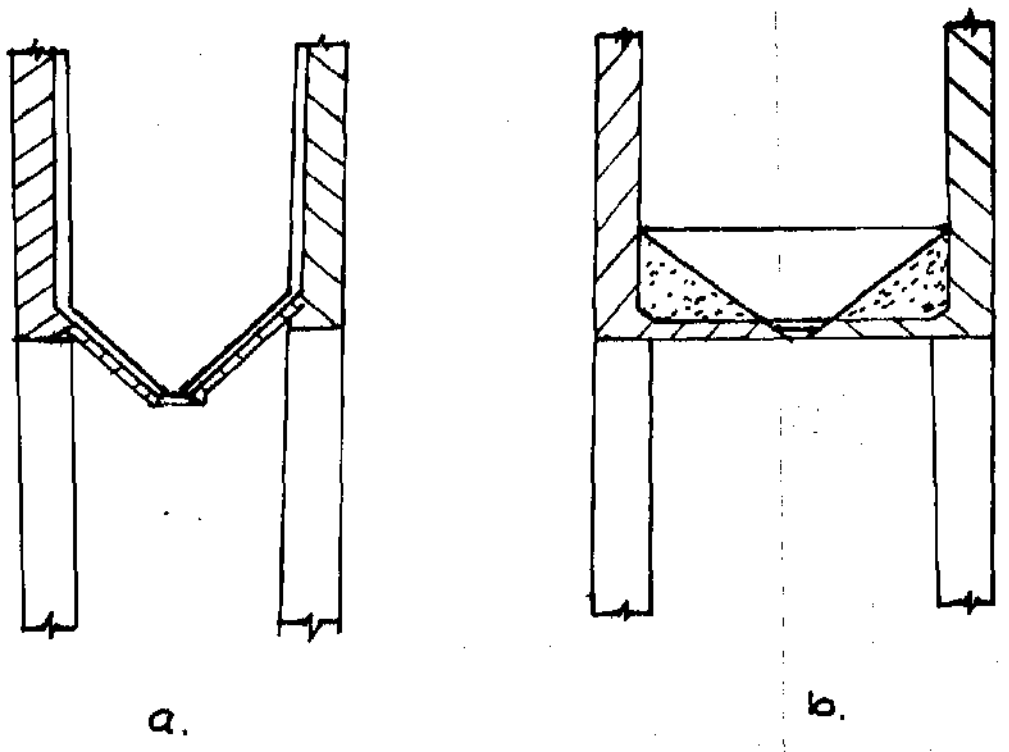


Figure 2.2 Hopper Bottoms

2.1.2 BUNKER WALLS

The vertical wall of a bunker is the main containing part of the structure. In small and medium sized bunkers, the walls are used to suspend the hoppers. In this case, the joints between the vertical walls and the sloping slabs are considered as fixed, (Figure - 2.3a). On the other hand, in large bunkers, the stresses caused by the hopper become quite significant and the presence of shrinkage stresses arising from construction joints in the vertical wall and the behaviour of modern cements make

it necessary to design a knee beam at the junction of the wall and the sloping slab to carry the hopper. In this case, this joint is considered as a free support (figure 2.3 b).

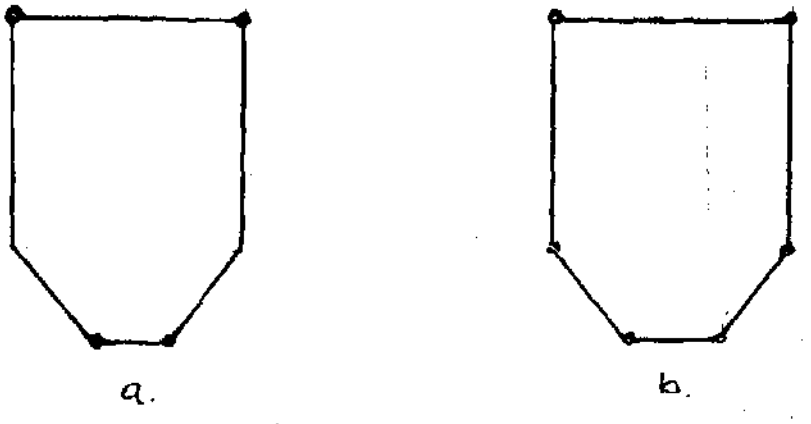


Figure 2.3: Bunker joints

2.2 FILLING PROCEDURE

There are various ways by which a bunker may be filled. All are relatively simple, and are not particularly important in so far as the strength of the bunker is concerned. Often, filling is part of a flow-line process, when the rate of filling must sometimes be related to the rate of discharging, and the production involved. Because of the different manufacturing procedures, filling operations vary from intermittent to continuous. The actual filling operation is carried out with the aid of mechanical plant-conveyors, elevators, grab cranes, etc. The one that is mostly used is the conveyor. When conveyors are used, it is usually necessary to provide support from the bunker in the form of beams spanning

from wall to wall. These beams in turn support the conveyor framework. Multi-bucket elevators may also be used.

Frequently, the top of the bunker is left completely open so that there is no restriction on the filling procedure. Alternatively it may be framed and covered with floor plates and used as a working platform, a small opening being left to receive the in-coming materials. A roof could however be placed over the opening in a bunker as a safeguard against moisture.

2.3 EMPTYING PROCEDURE

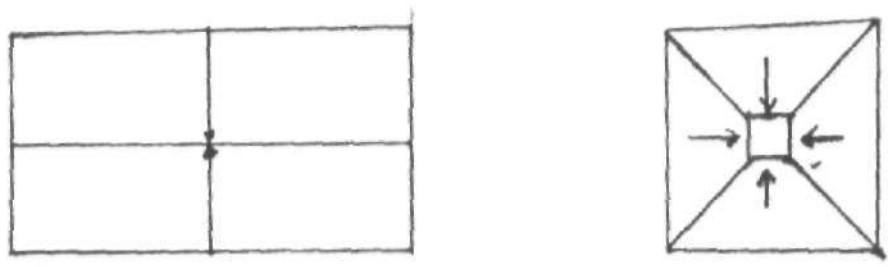
The problems of emptying the bunker are quite different from those of filling. One of the problems is that of the material arching over the outlet and where this happens, rather drastic measures are sometimes necessary. Mechanical vibrators may be used to assist movement.

In order to aid speedy discharge under the action of gravity, the hopper of bunkers is designed in the form of inverted truncated pyramid. The sloping hoppers are given an angle greater than the angle of repose by at least 10^{o}.

Two types of hopper bottoms are often encountered:

a. A rectangular hopper bottom, sloping in two directions, giving rise to two-dimesnional flow and

b. a square hopper bottom sloping in four-directions, giving rise to four dimensional flow.



(a) rectangular section (b) square section

Figure 2.4: Types of discharge.

2.4 INTRODUCTION TO LIMIT STATE DESIGN

The aim of structural design is to create a structure that is safe under worse loadhgs and that which the deformations of its members do not detract from the appearance, durability or performance of the structure during working conditions. Despite the difficulty in assessing the precise loading and variations in the strength of the concrete and steel, these requirements have to be met.

Three basic methods using factors of safety to achieve safe, workable structures have been developed; they are:

1. The permissible stress method in which ultimate strengths of the materials are divided by a factor of safety to provide design stresses which are usually within the elastic range.
2. The load factor method in which the working loads are multiplied by a factor of safety.
3. The limit state method which multiplies the working loads by factors of safety and also divides the materials ultimate strength by further partial safety factors.

The permissible stress method has proved to be a simple and useful method but it does have some serious inconsistencies. Because it is based on an elastic stress distribution it is not really applicable to a semi-plastic material such as concrete, nor is it suitable when the deformations are not proportional to the load as in slender columns. It has also been found to be unsafe when dealing with the stability of structures subject to overturning forces.

In the load factor method, the ultimate strength of the materials should be used in the calculations.

As this method does not apply factors of safety to the material stresses, it cannot directly take account of the variability of the materials and also it cannot be used to calculate the deflections or cracking at working loads.

The limit state method of design overcomes many of the short-comings of the previous two methods. This, it has achieved by applying partial factors of safety, both to the loads and to the materials strengths. The magnitude of the factors may be varied so that they may be used either with the plastic conditions in the ultimate state or with the more elastic stress range at working loads. This flexibility is particularly important if full benefits are to be obtained from the development of improved concrete and steel properties.

2.4.1 Limit States

A limit state is that dondition which once reached a structure may cease to be fit for its intended use. The design aim is therefore to avoid any such condition being reached during the expected life of the structure. The two principal types of limit state are the ultimate limit state and the serviceability limit state.

a) Ultimate Limit State:

This requires that the structure must be able to withstand, with an adequate factor of safety against collapse, the loads for which it is designed. The possibility of buckling must be taken into account, as must be the possibility of accidental damage, as caused for example, by internal explosion.

b) Serviceability Limit States:

Generally, the two most important limit states are:

1. Deflection - the appearance or efficiency of any part of the structure must not be adversely affected by deflections.

2. Cracking - Local damage due to cracking and appalling must not affect the appearance, efficiency or durability of the structure.

Other limit states that may be reached include:

3. Durability - this must be considered in terms of the proposed life of the structure and its condition of exposure.

4. Excessive vibration - which may cause discomfort or alarm as well as damage.

5. Fatigue - must be considered if cyclic loading is likely.

6. Fire resistance - this must be considered in terms of resistance to collapse, flame penetration and heat transfer.

7. Special circumstances - any special circumstances of the structure not covered by any of the more common limit states.

2.4.2 Characteristic Material Strengths

It is assumed that for any given material, the distribution of strength, taking a large number of samples will give the form shown in figure 1.4.

The characteristic strength is shown as that value below which it is unlikely that more than 5% of the results will fall.

This is given by:

$$fk = fm - 1.64s$$

where fk= characteristic strength
fm= means strength
s= standard deviation

2.4.3 Characteristic Loads

Following the above argument it should also be possible to assess loads statistically, in which case:

Characteristic loads = Mean load $\pm$ 1.64S.

In most cases it is the maximum loading on a structural member that is critical and the upper, positive value given by this expression is used, but the lower, minimum value may apply when considering stability or the behaviour of the continuous members.

These characteristic values represent the limits within which at least 90 percent of the values will lie in practice. It is to be expected that not more than 5 percent of cases will exceed the upper limit and not more than 5 percent will fall below the lower limit. They are design values which take into account the accuracy with which the loads can be predicted.

2.4.4 Partial safety factor for materials

This is given as:

$$\text{Design strength} = \frac{\text{Characteristic strength}(f_k)}{\text{Partial factor of safety}(x_m)}$$

Table 1 Partial factors of safety applied to materials (γ_f)

Limit State	Material Concrete	Steel
Ultimate	1.5	1.15
Serviceability		
Deflection	1.0	1.0
Cracking	1.3	1.0

2.4.5 Partial Safety factor for loads

Errors and in-accuracies may be due to a number of causes including among others, design assumptions and inaccuracy of calculations, possible un-usual load increases, unforseen stress redistributions and constructional inaccuracies.

These cannot be ignored, and are taken into consideration by applying a partial factor of safey (γ_k) on loadings.

So that

$$\text{design load} = \frac{\text{Characteristic load}}{\text{Partial factor of safety}(\gamma_k)} \times$$

Table 2: Partial factors of safety applied to loads (γ_k)

Local Combination	Limit State					
	Ultimate			Serviceability		
	Dead	Imp.	Wind	Dead	Imposed	Wind
Dead + Imposed	1.4(1.0)	1.6	-	1.0	1.0	-
Dead + Wind	1.4(0.9)	-	1.4	1.0	-	1.0
Dead + Imposed + Wind	1.2	1.2	1.2	1.0	0.8	0.8

2.4.6 Global Factor of Safety

The use of partial safety factors on materials and load offers considerable flexibility, which may be used to allow for special conditions such as very high standards of construction and control, or at the other extreme, where structural failure would be particularly disastrous. The global factor of safety against a particular type of failure may be obtained by multiplying the appropriate partial factors of safety. For instance, a beam failure caused by yielding of tensile reinforcement would have a factor of

$m \times f = 1.15 \times 1.4 = 1.61$ dead loads

or $1.15 \times 1.5 = 1.84$ for live loads.

In the light of all the foregoings and a lot more that may not have been mentioned therefore, the author of this thesis has considered the limit state method of design most adequate for the design of a bunker and other concrete structures. This is being accomplished in the chapters that follow.

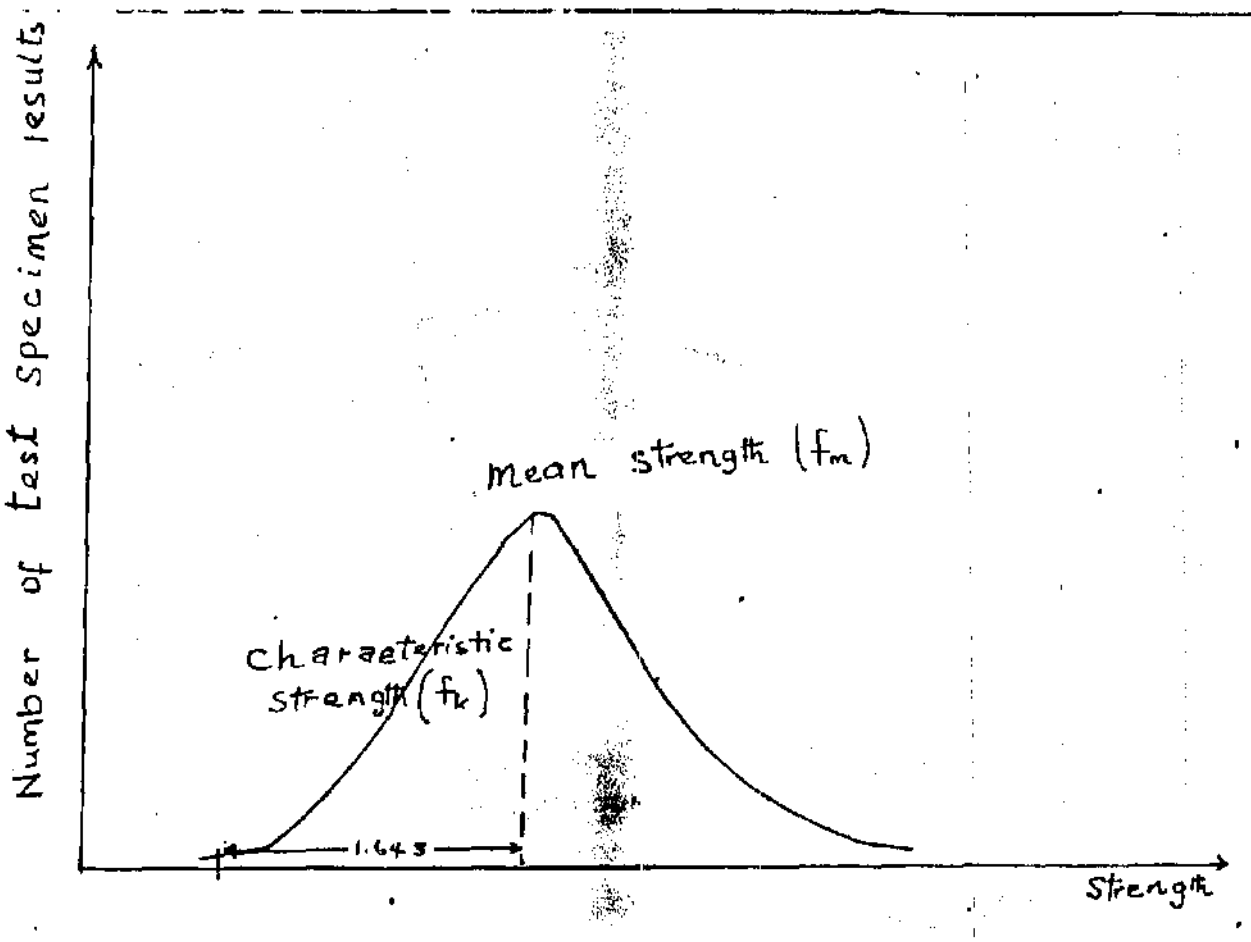


FIGURE 2.5 NORMAL FREQUENCY DISTRIBUTION OF STRENGTH

CHAPTER - THREE

3.0 ANALYSIS OF BUNKER FORCES

Having loaded the material into the bunker, the next stage is to retain it there until it is required. For all intents and purposes, all that is necessary is to provide a bunker which is strong enough to safely carry the vertical and lateral pressures, and of course its own weight.

Bunkers fall into easily definable types, and the theory applicable to one may not be suitable to another. For example, most bunkers will be surcharged as a result ofthe normal filling process. If the surchagge is taken into account the horizontal pressure, for the general run of bunkers, will be very much over-estimated. It is therefore more economical to assume the material to be flat and level at the mean height of the surcharge and design accordingly. Where the bunker is very wide in relation to its depth the effects of surcharging may need to be considered. A shallow bunker acts quite differently from a deep one. Some consideration should be given to the probable state of material as a perfectly dry granular material in one condition, a completely saturated material in another.

In the case of maize it is assumed to be completely dry.

3.1 Design Data

Density of maize = 8.4 KN/m^3

Angle of repose $\ell=35^{o}$

Capacity of two bunkers = 400 tons - (4,000 KN)

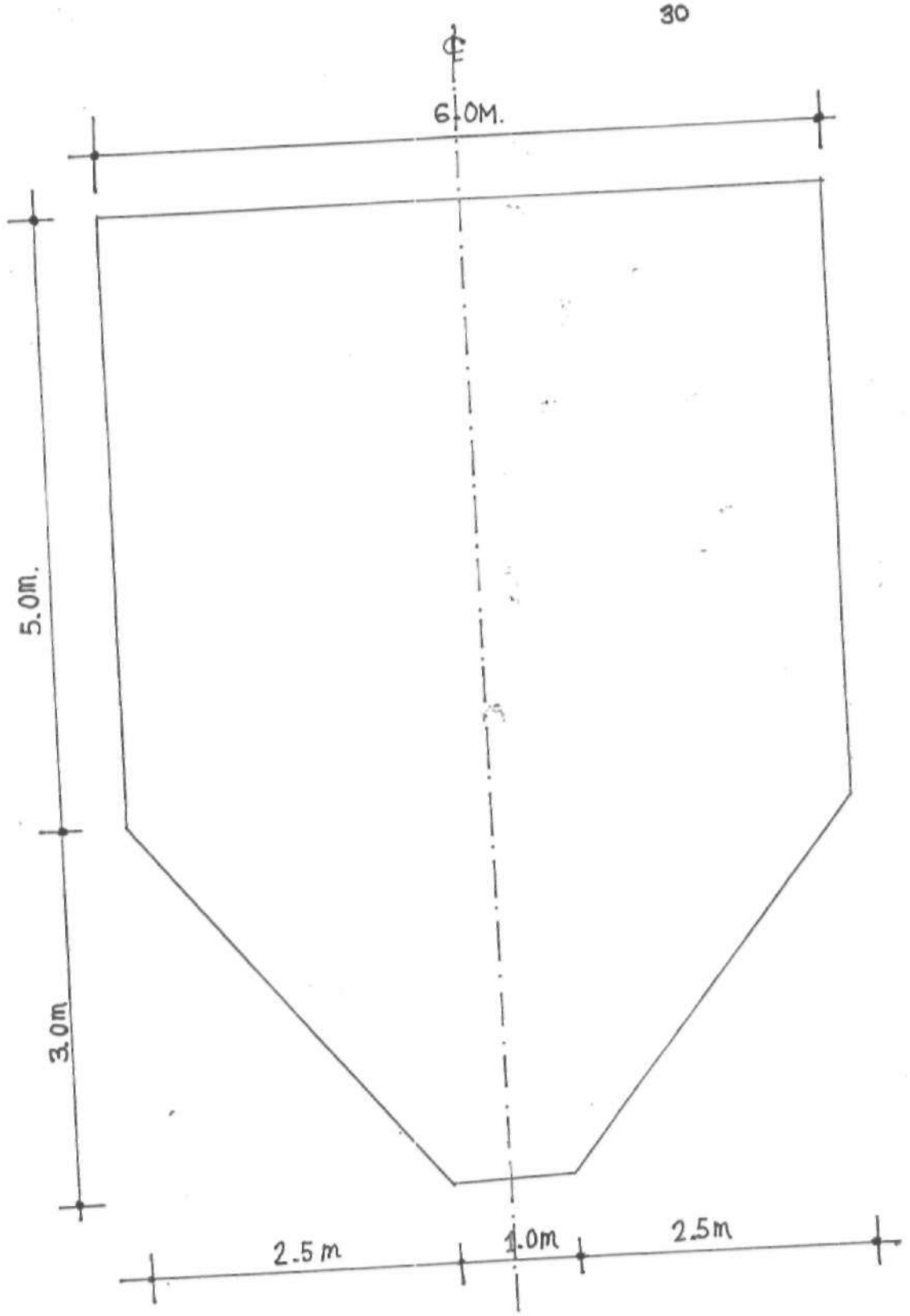


FIGURE 3.1 CROSS SECTIONAL ELEVATION OF THE BUNKER

It is required to find the dimensions of the bunkers and the angleof their discharge openings.

Trial section

Try a square section 6m x 6m and a wall height of 5m. The discharge opening should be 1m x 1m.

Total volume of one hunker $= \frac{2000\,KN}{8.4\,KN/m^3}$

$= \underline{238m^3}$

The vertical height h_2 of the sloping bottoms will be: $6(5 \times 64 \left(\frac{(6+1)}{2} h_2\right) = 238m^3$

$$180 + 21\, h_2 = 238$$

$$h_2 = \frac{58}{21}$$

$$= 2.76m$$

take $h_2 = \underline{3 \text{ metres}}$

from figure 3.1,

$$\emptyset = \tan^{-1} \frac{3.0}{2.5}$$

$$\emptyset = \underline{\underline{50.10^{\circ}}}$$

Actual Capacity of one bunker

Surface area = $5 \times 6 + \frac{6 + 1 \times 3}{2}$

= $30 + 10.5m^2$

= $40.5m^2$

Volume of bunker = 40.5×6

= $\underline{243m^3}$

Mass of material = 243×8.4

= $\underline{2,041KN}$

Total capacity of two bunkers should therefore be

$\underline{4,082KN}$

There are ten-100kg bags in one tone (or 10KN), therefore 2041KN is equivalent to 2041 bags each weighing 100kg (or 1KN). The bunker is analysed for level filling, that is a case of no-surcharge. Rankine's formula will be used to calculate the pressure intensity at the various points.

considering triangle ABC to be similar to triangle DEC, so that the length of side AB can be determined from the relation

$$\frac{AB}{DE} = \frac{BC}{EC}$$

$$\text{or} \quad AB = \frac{DE \times BD}{EC}$$

$$= \frac{2.50 \times 8}{3}$$

$$AB = \underline{6.67m}$$

Side pressure (Rankine's)

At the bottom of the wall (at D)

$$P = wh.k$$

$$K = \frac{1-\sin \phi}{1 + \sin \theta} = \frac{1-\sin 35^\circ}{1 + \sin 35^\circ}$$

$$= \frac{0.426}{1.574}$$

$$K = \underline{0.271}$$

$$P = 8.4 \times 5 \times 0.271$$

$$= \underline{11.38} \ KN/m^2$$

At the bottom of the bunker

$$P = W(h1 + h2) K$$

$$= 8.4 (5 + 3) \times 0.271$$

$$P = 18.21 \ KN/m^2$$

The total lateral pressure (Rankine's active pressure) at the bottom of the bunker.

$pa = \frac{1}{2} W (h1 + h2)^2 \times K$

$= \frac{1}{2} \times 8.4 \times 8^2 \times 0.271$

$Pa = 72.84 KN/m^2$ horizontally.

The weight of maize on triangle AEC

$\frac{1}{2} \times 6.67 \times 8 \times 8.4$

$= \underline{224KN}$ vertically downwards

The resultant of these two factors

$R = \sqrt{73^2 + 224^2}$

$= \sqrt{5329 + 50175}$

$= 235.59$

$R = \underline{236KN}$

The angle R makes with the horizontal

$= 50.19^\circ - \tan^{-1} \frac{73}{224}$

$= 50.19^\circ - 18.50^\circ$

$= \underline{32.14^\circ}$

The normal force at this point

$= R \cos 32.14^\circ$

$= 236 \times 0.8467$

$= 199.83$

$= \underline{200KN}$

The length of side AC:

$AC = 6.67^2 + 8^2$

$= \underline{10.61m}$

The length of inclined slab (DC) can be obtained by employing the idea of similar triangles as in the previous example.

$$ABC = DEC$$

$$\frac{AC}{DC} = \frac{BC}{EC}$$

$$\therefore \quad DC = \frac{AC \times EC}{BC} = \frac{10.61 \times 3}{8}$$

$$DC = \underline{3.98m}$$

The maximum pressure intensity at the mouth parallel to R therefore

$$= \frac{236 \times 2}{10.61}$$

$$= 44.49$$

$$= \underline{44.50KN}$$

the pressure intensity at this point normal to the sloping side

$$= 44.50 \cos 32.14^{\circ}$$

$$= 37.68KN$$

The pressure intensity at the top of the inclined slab (i.e at D)

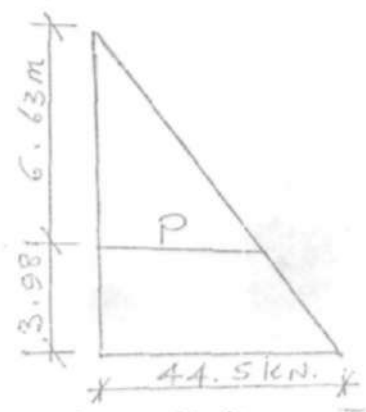


FIGURE 3.7 TRIANGULAR STRESS DISTRIBUTION.

$$P = \frac{44.50 \times 6.63}{10.61}$$

$$= \underline{27.81 KN}$$

NOTE: The resultant R represents the amount of pressure in the triangle of base AC, with an additional part AD. This must be reduced to the trapezoid hatched in figure.

Summary of all the above forces is shown in the diagram below.

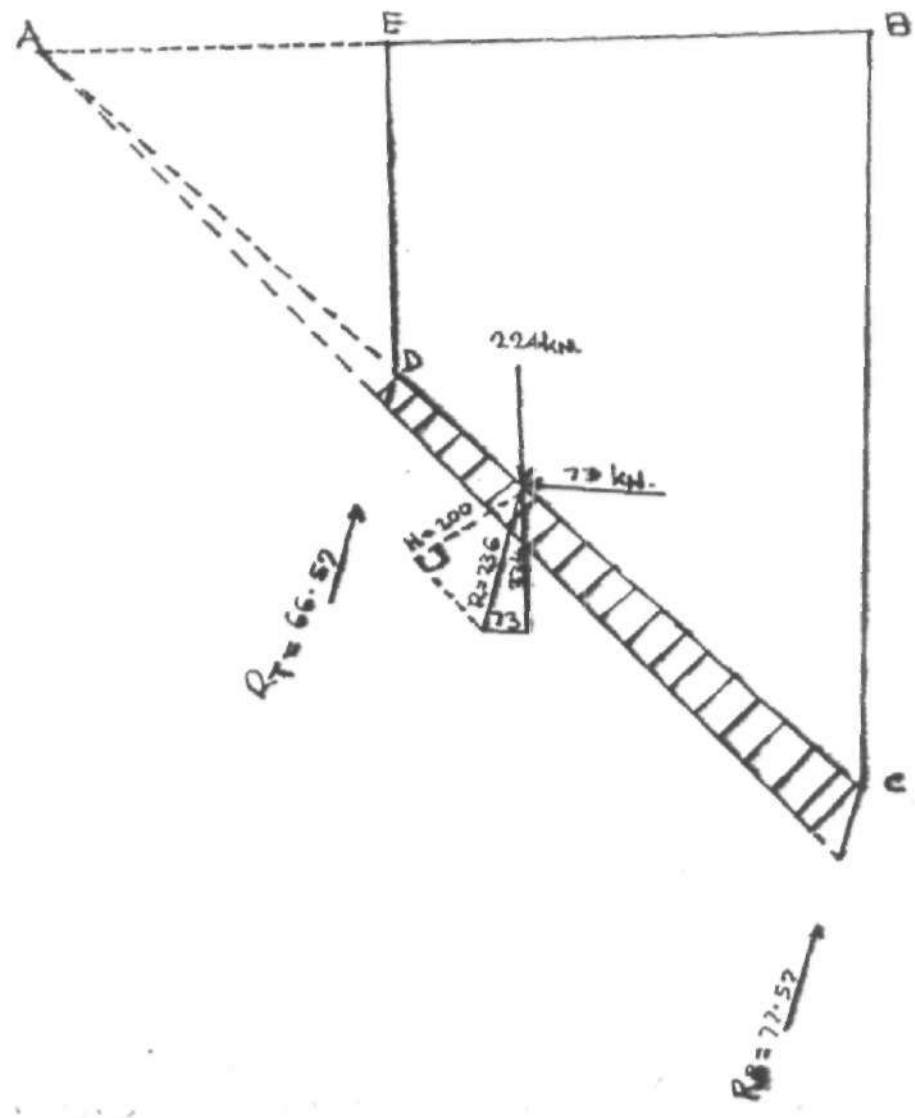


Figure 3.3 Resolution of forces

$VB = 77.43 \cos 18.05^{o}$

$= \underline{73.62KN}$

$HB = 77.43 \sin 18.05^{o}$

$= 23.99$

$= \underline{24KN}$

The maximum force acting within the vertical walls of the bunker

$L_1 = \frac{1}{2} W h1^2 k$

$= \frac{1}{2} \times 8.4 \times 5^2 \times 0.271$

$L_1 = 28.45$

say $\underline{29\ KN}$

At the top of the wall, the magnitude of this force

$= \frac{29}{3}$

$= \underline{9.66KN}$

At the bottom of the wall

$= \frac{29}{3} \times 2$

$= \underline{19.34KN}$

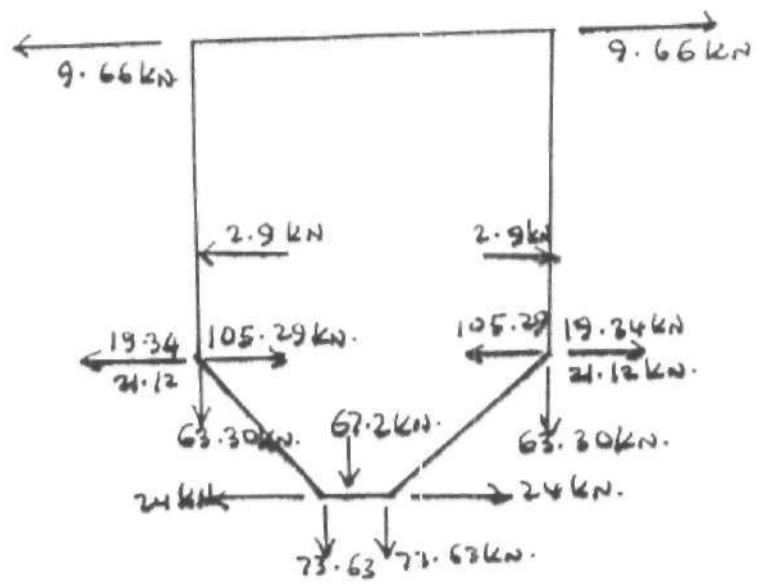


fig. 3.5 Bunker Forces

The weight of maize above the mouth of the bunker

(1m - width)

= 1.0 x 8 x 8.4 x 1

= <u>67.2KN</u>

The total vertical reaction is

171 x 2 = <u>342KN</u>

The weight of maize for one metre run of the bunker

= area x density x 1

= 40.5 x 8.4 x 1

= <u>340KN</u>

This agrees very well with 342KN obtained above.

In order to determine the maximum tension and thrusts acting on the bunker, it is necessary to include the self-weight of the bunker. The weight of the sloping slabs, assuming a thickness of 20cm is

$$0.20 \times 3.98 \times 24 \times 2$$

$$= \underline{38.21 \text{ KN}}$$

This should be added to the load of 214.46 at the mouth of the bunker to give 38.21 + 214.46

$$= \underline{252.67} \text{KN}$$

Let the tension in each sloping slab be T KN, so that

$$2T \sin 50.19^{\circ} = 252.67$$

$$T = \frac{252.67}{2 \sin 50.19^{\circ}}$$

$$T = 164.46 \text{ KN}$$

The horizontal component of this force

$$= 164.46 \cos 50.19^{\circ}$$

$$= 105.29 \text{ KN}$$

The self-weight of one sloping slab is added to the force of 171 KN to obtain the total upward force

$$= \frac{38.31}{2} + 171$$

$$= \underline{190} \text{ KN}$$

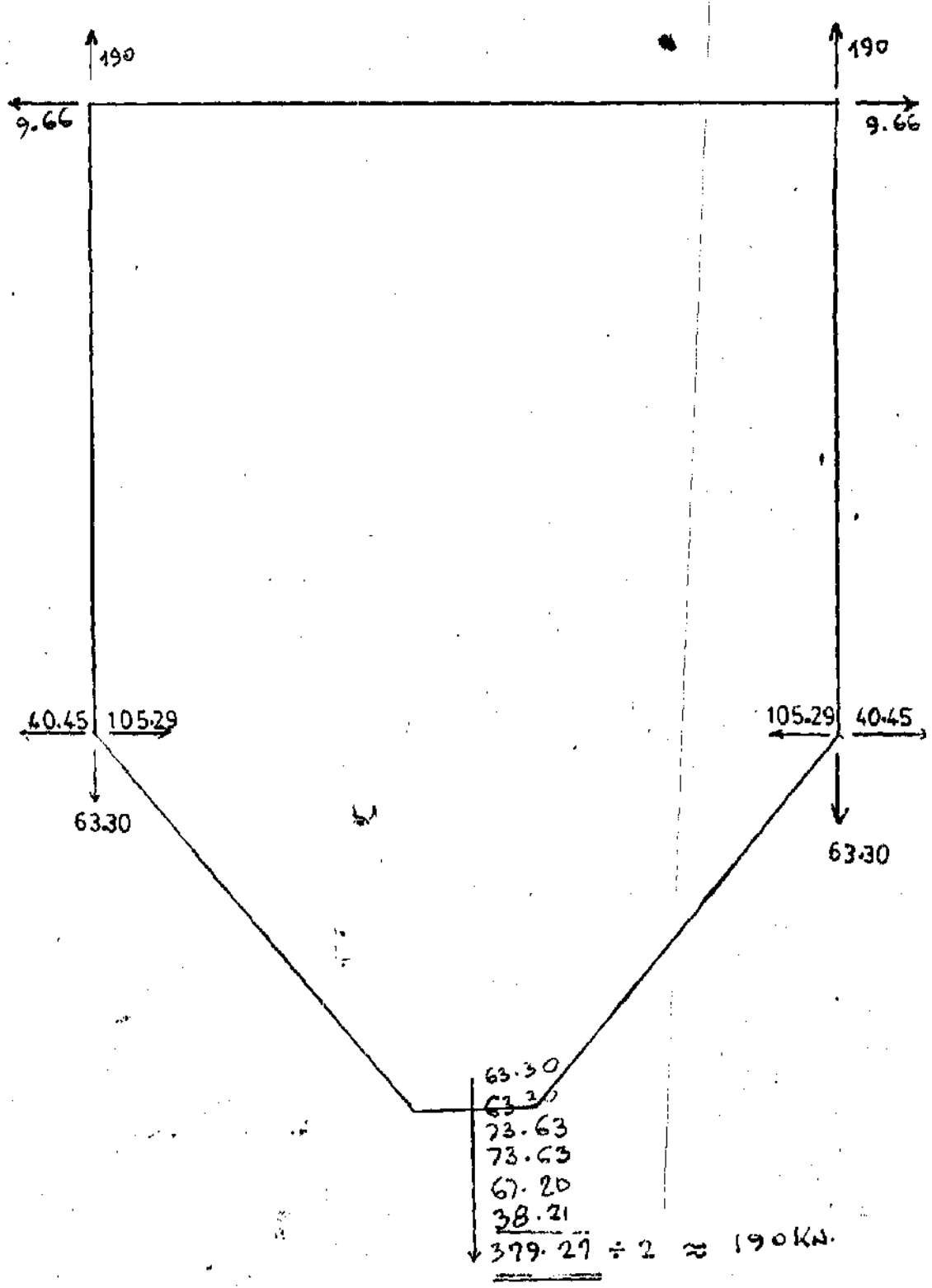


FIGURE 3.6 FINAL FORCE DIAGRAM.

CHAPTER - FOUR

DESIGN OF ELEMENTS

4.1 DESIGN OF BUNKER SLABS

A slab is a flat plate like element which carries its weight by flexure. In buildings, slabs are used for floors and roofs placed horizontally. Slabs can however be sloped to make ramps or in staircase construction. They can be cast-in-situ or precast. Slabs can be grouped into three, as solid slabs, ribbed slabs or flat slabs depending on their mode of construction.

In solid slabs, the slabs are designed between beams which rest on columns. In ribbed slabs, the span of slab is broken into small beam like structures called ribs which are supported along their transverse direction on main beams which are in turn supported on columns. In flat slabs, there are no beams. The slabs are made of very thich members, resting directly on columns.

Slab can be classified as simply supported, continuous or canti-levered. A simply supported slab is that which spans between two simple supports. A continuous slab is that which spans over at least one support, that is a slab with at least three spans. A cantilevered slab has a fixed support at one end

and unsupported at the other.

The third and last classification is; a slab can be one-way spanning or two-way. A one-way spanning slab is that which is supported on two or more of its sides with its main reinforcement running in one direction. Distribution steel is usually used in the other direction to complement the main reinforcement. A one-way slab is used where the ratio of the longer span to the shorter span is greater than two. A two-way spanning slab has its main reinforcement running both ways. A two-way slab is required where the ratio of the longer span to the shorter is less than or equal to two.

The self-weight of slabs can be reduced by forming voids in the lower surface of the slab with hollow clay blocks, light-weight permanent shuttering, removable shuttering with suitable insert and use of light-weight concrete.

In the design consideration of this project, solid slans spanning in one direction and those spanning in two directions have been used. Brief mention will however be made of flat slabs, ribbed and hollow block slabs and precast slabs.

Although the anslysis and design of slab is done at the ultimate limit state, it is the serviceability limit state of deflection that is the most important

criterion. The serviceability limit state of cracking is controlled by the rules governing bar spacing.

4.1.1 Flat Slabs

The disadvantage of the beam and slab construction is that the beams which support the slabs (solid slabs) reduce the headroom and so impose restrictions on the use of the space beneath. This disadvantage can be overcome by supporting the slabs directly on columns. This is the principle of flat slabs. As for a normal slab, a flat slab may be solid or may have recesses formed in the soffit to provide a series of ribs in two-directions. The recesses may be formed by removeable or permanent filler blocks.

4.1.2 Ribbed and Hollow Block Slab

The main advantage of this type of slab lies in the reduction in weight achieved by removing part of the concrete below the neutral axis and in the case of the hollow block slab, the blocks have holes in them and so the weight of the slab is lighter. Ribbed and hollow block slabs are economical for long spans, over say five metres. Where heavy loadings occur- such as warehouse and garages, these slabs are not considered fit.

The design of reinforced concrete slabs is not cumbersome because:

1. the breadth of the slab is already fixed and a unit breadth of one metre is used in calculations.
2. the shear stresses are usually low except where there are heavy concentrated loads.
3. compression reinforcement is seldom required.

4.1.3 Span Effective depth ratios

To avoid excessive deflection of slabs, limits are set on the span-effective depth ratios.

The minimum effective depth = $\frac{\text{the actual span}}{\text{basic ratio x M.F.}}$

M.F. is modification factor based on the area of tension steel when a slab is singly reinforced at midspan, but if a slab has both top and bottom steel at midspan the modification factors are as given in tables 10 and 11 of CP110. It can be observed that a lower service stress gives a higher modification factor and hence a smaller depth of slab.

Ordinary values are given in table 8 of CP110 with which to multiply the modification factor.

This table is reproduced below:

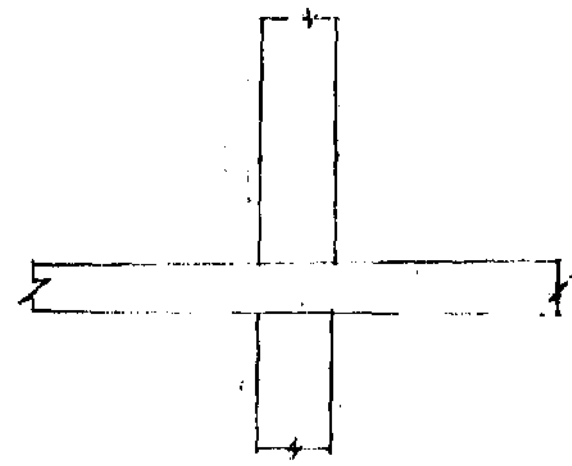
Contilever = 7

Simply supported = 20 - table 8 CP110

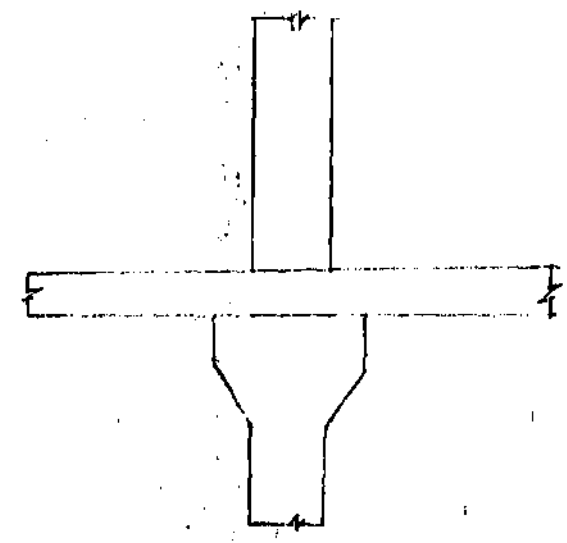
Continuous = 26

FIGURE 4.1 TYPES OF SLAB

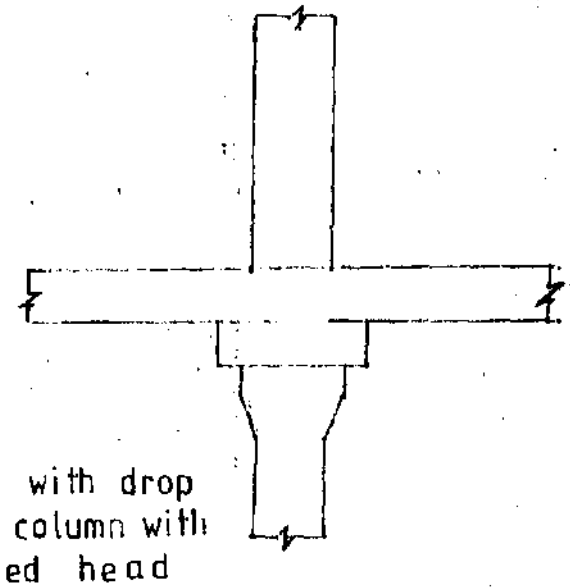
FLAT SLABS



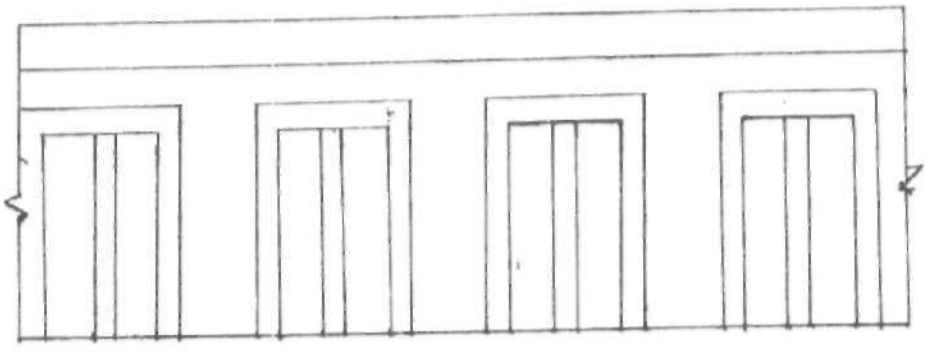
d. slab without drop and column without flared head



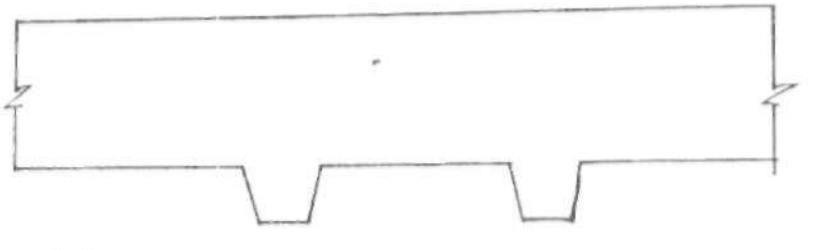
e. slab without drop and column with flared head



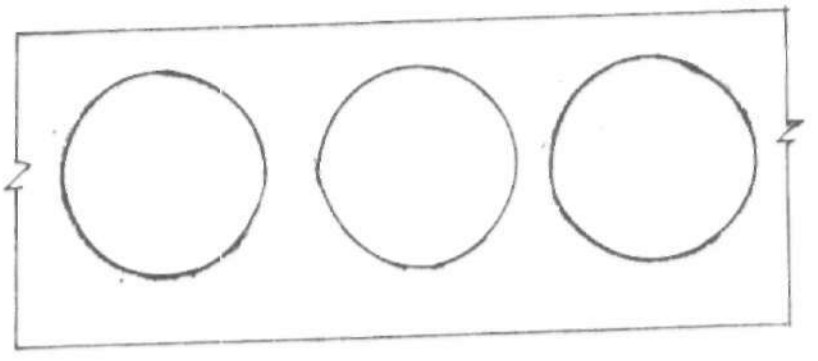
f. slab with drop and column with flared head



a. ribbed hollow-block slab



b. ribbed slab



c. voided slab

Taking some arbitrary values of the following quantities, the check is performed as follows:

effective span of slab = 4.50m (say)

effective depth = 140mm

area of tensile steel = 436mm^2/m

fly = 460N/mm^2

The slan is continuous over three supports.

Span-effective depth ratio

$$\frac{100AS}{bd} = \frac{100 \times 436}{1000 \times 140} = 0.31$$

from table 10, CP110 modification factor = 1.34

Limiting $\frac{\text{span}}{\text{effective depth}}$ -= 26 x 1.34 =34.8

Actual $\frac{\text{span}}{\text{effective depth}} = \frac{4500}{140} = 32.1$

Since limiting is greater than actual,

d = 140mm is adequate.

This procedure will be followed in the design of slabs in this text.

4.1.4 Reinforcement details

To resist cracking of concrete, codes of practice specify details such as the minimum area of reinforcement required in a section and limits to the maximum and minimum spacing of bars. Some of these rules are as follows:

(a) Minimum area of reinforcement

Min area = $\frac{0.15\ bd}{100}$ - for high yield steel

= $\frac{0.25\ bd}{100}$ - for mild steel

(b) Minimum area of secondary reinforcement.

Min area $= \frac{0.12\ bh}{100}$ - for high yield steel

$= \frac{0.15\ bh}{100}$ - for mild steel

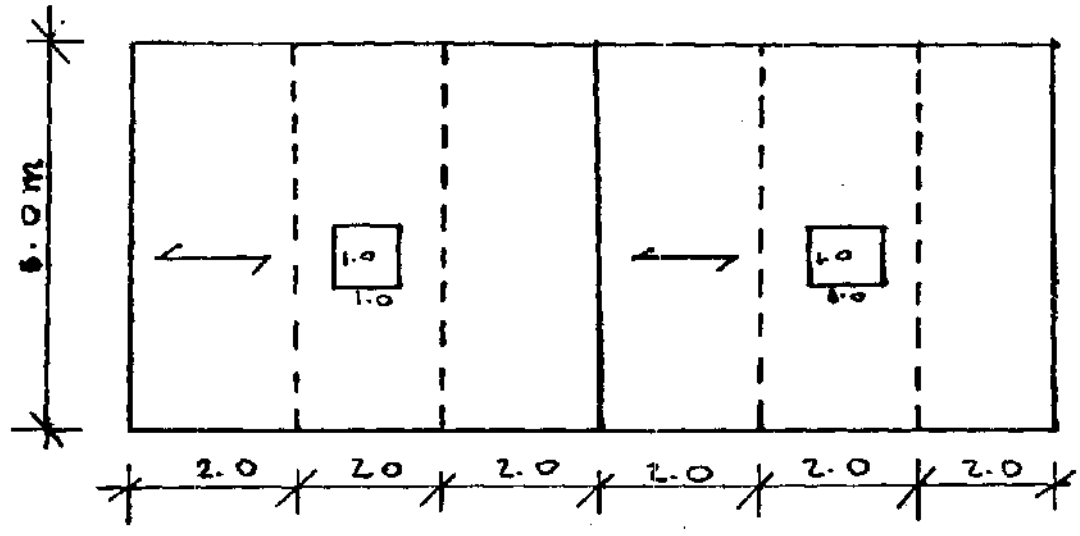
and the distance between the secondary bars should not exceed 5d.

(c) Maximum spacing of bars.

If h = 200mm or h $\leq$ 250mm and fly = 425KN/m^2

no check is required other than S $\not>$ 3d, where S is the maximum clear distance allowed between bars.

4.2 DESIGN OF COVER SLAB FOR BUNKERS



The cover slab is to be designed as a continuous solid slab.

Preliminary design of section

$\frac{\text{Basic span}}{\text{Effective depth}}$ = 26 for continuous slab

assume M.F. = 1.5

$$d = \frac{\text{span}}{\text{basic ratio} \times \text{M.F.}}$$

$$= \frac{2000}{26 \times 1.5}$$

$$= 51.3\text{mm}$$

$$h = d + \text{cover} + \emptyset/2$$

$$= 51.3 + 15 + \frac{12}{2} \quad (\emptyset \text{ 12mm bar})$$

$$h = 72.3\text{mm}$$

Try h = 100mm

4.2.1 Load Estimation

Dead Load

Self-weight ofslab = 0.10 x 24 =2.4KN/m^2

Weight of Plaster = 0.02 x 20 =0.4KN/m^2

Bitumenous felt = say $\frac{0.50 KN/m^2}{3.30\ KN/m^2}$

Liveload for flat roof with access

= 1.5 KN/m^2

4.2.2 Design Load and Moments

at ultimate limit state

n = 1.4 GK + 1.6 QK

= 1.4 x 3.30 + 1.6 x 1.5

= 4.62 + 2.40

n = 7.02 KN/m^2

total design load on each span

= 7.02 x 2

= 14.04KN/m^2

Moments

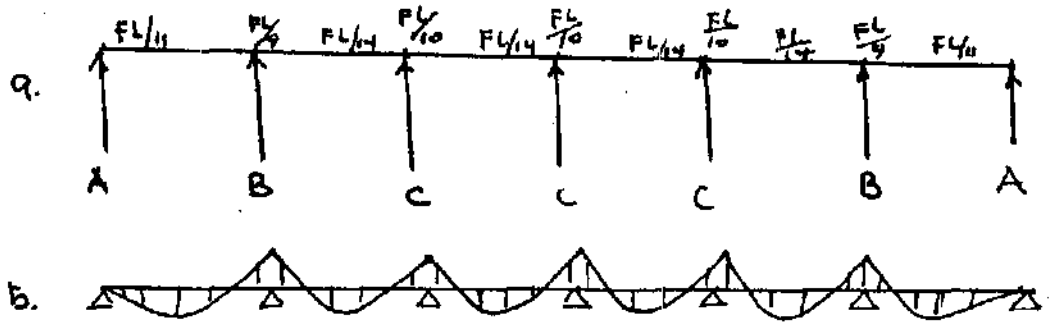


FIGURE 4.3 MOMENT DIAGRAM.

$$MUAB = \frac{FL}{11} = \frac{14.04 \times 2}{11}$$

$$= \underline{2.55} \text{ KN/m}$$

4.2.3 Reinforcement

$$\frac{mu}{fcu\ bd^2} = \frac{2.55 \times 10^6}{25 \times 1000 \times 79^2}$$

$$= 0.016$$

take la = 0.95, z = lad

$$= 0.95 \times 79$$

$$= \underline{75.05 mm}$$

$$Ast_{AB} = \frac{MUAB}{0.87\ fy}$$

$$= \frac{2.55 \times 10^6}{0.87 \times 250 \times 75.05}$$

$$= 156 mm^2$$

provide R 8 at 300mm ($168 mm^2$)

Support moment at B

$$MuB = \frac{FL}{9}$$

$$= \frac{14.04 \times 2}{9}$$

$$MuB = \underline{3.12 KN/m}$$

$$Ast = \frac{3.12 \times 10^6}{0.87 \times 250 \times 75.05}$$

$$= \underline{191 mm^2}$$

Provide R 8 at 250mm ($201 mm^2$)

Span Moment BC and CC

$$MU = \frac{FL}{14} = \frac{14.04 \times 2}{14}$$

$$= 2 \text{ KNm}$$

$$Ast = \frac{2 \times 10^6}{0.87 \times 250 \times 75.05}$$

$$= 123mm^2$$

Provide R 8 at 300mm ($168mm^2$)

Support Moment at C

$$Mu = \frac{FL}{10} = \frac{14.04 \times 2}{10}$$

$$= 2.8 \text{ KNm}$$

$$Ast = \frac{2.8 \times 10^6}{0.87 \times 250 \times 75.05}$$

$$= 172mm^2$$

Provide R 8 at 250mm ($1202mm^2$)

Check span - effective depth ratios

Span AB

$$\frac{100As}{bd} = \frac{100 \times 168}{1000 \times 75.05} = 0.22$$

from table M.F. = 2

$$\text{Limiting } \frac{\text{Span}}{\text{effective depth}} = 2 \times 26 = 52$$

$$\text{Actual } \frac{\text{Span}}{\text{effective depth}} = \frac{2000}{75.05} = 26.65$$

Since limiting is greater than the actual, the section is adequate.

Distribution steel:

$$= \frac{0.15}{100} bh$$

$$= \frac{0.15 \times 1000 \times 100}{100}$$

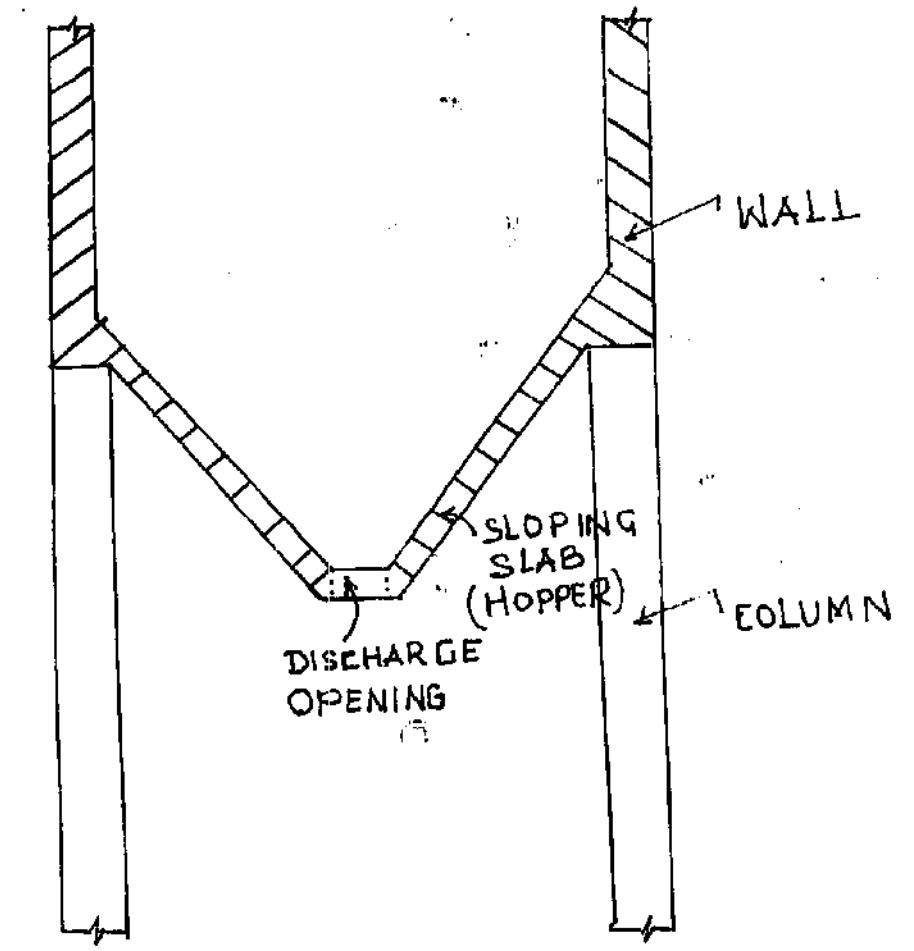
$$= 150mm^2$$

Certain problems are envisaged in the construction of an insitu cover for the bunker. Some of these problems include:

(a) Erection of forms. Since the bunker is more of less a closed container it will be difficult to erect forms for the cover after the hopper and the walls have been constructed.

(b) Removal of forms. It is imagined that if the problem in (a) is overcome, another more serious problem is that of removal of the form after the cover slab will have set. These and other less prominent problems make it imperative that covers of the sort are better pre-cast than cast-in-situ.

Structural detailing of the proposed precast slab is therefore not provided. The cover slab is also not contained in the bending schedule.



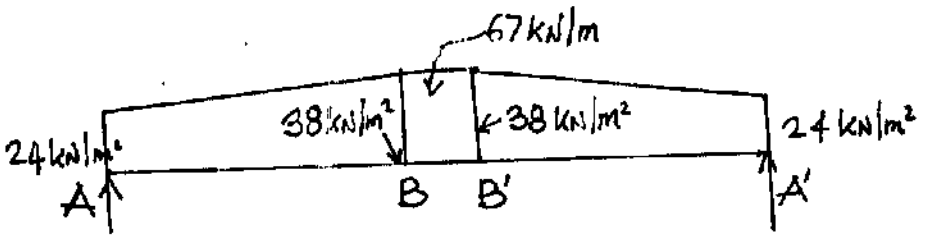
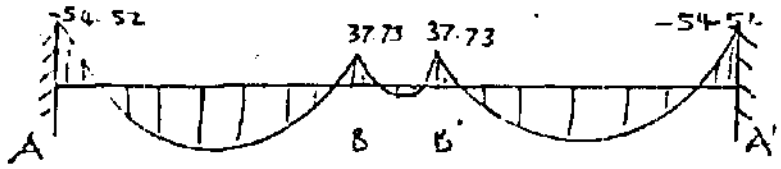


FIGURE 4.4 LOAD PATTERN OF SLOPING SLABS.

4.3.2 Moment Distribution

JOINT	A		B	B		A
NUMBER	AB	BA	BB'	B'B	B'A'	A'B'
D. F.	1.0	0.20	0.80	0.80	0.20	1.0
F.E.M.	-46.46	+53.86	-5.58	+5.58	-53.86	+46.46
		-9.66	-38.62	+38.62	+9.66	
	-4.83		+19.31	-19.31		+4.83
		-3.86	-15.45	+15.45	+3.86	
	-1.93		+ 7.73	- 7.73		+1.93
		-1.55	- 6.18	+6.18	+1.55	
	-0.78		+ 3.09	-3.09		+0.78
		-0.62	-2.47	+2.47	+0.62	
	-0.31		+1.24	-1.24		+0.31
		-0.25	-0.99	+0.99	+0.25	
	-0.13		+0.50	-0.50		+0.13
		0.10	-0.40	-0.40	+0.10	
	-0.05		+0.20	-0.20		+0.05
		-0.04	-0.16	+0.16	+0.04	
	-0.02	-0.08	-0.08	-0.08		+0.02
		-0.02	-0.06	+0.06	+0.02	
	-0.01		+0.03	-0.03		+0.01
FINAL	-54.52	+37.73	-37.73	+37.73	-37.73	+54.52

Figure 4.7 Moment diagram



4.3.3 Load Estimation

200mm thick slab is to be tried.

Self-weight of slab = 0.20 x 3.98 x 24

= 19.10 KN/m

B.M. due to self-weight of slab

$$= \frac{19.10 \times 3.98}{8}$$

= 9.5 KNm

At ultimate limit state

mu = 1.4 x 9.5 + 1.6 x 54.52

= 13.30 + 87.23

Mu = 100.53KN

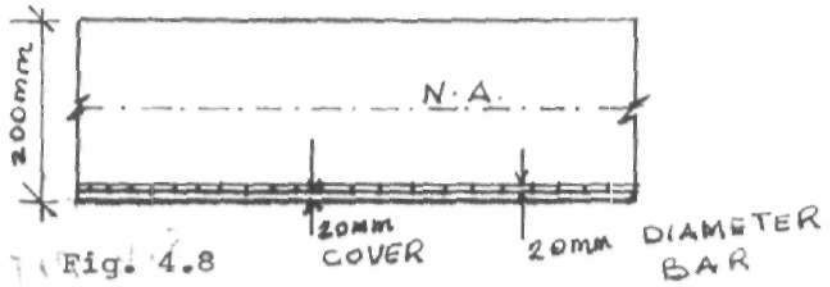


Fig. 4.8

The direct tension is transferred to the centre-line of the tensile steel. The resultant bending moment

= 100.53 - 164.46 (0.07)

= 89KNm

4.3.4 Area of Steel

$$\frac{Mu}{bd^2fcu} = \frac{89 \times 10^6}{10^3 \times 170^2 \times 25} = 0.12$$

from lever arm curve

la = 0.85

z = 0.85 x 170 = 144.5mm

Area of steel for bending

$$Ast = \frac{Mu}{0.87\ fy}$$

$$= \frac{89 \times 10^6}{0.87 \times 250 \times 144.5}$$

$$Ast = \underline{2832mm^2}$$

Provide R 25 at 150mm ($327mm^2$)

area of steel for direct tension

$$Ast = \frac{T}{fs}$$ - from table 10, CP110, for fy-250N/mm^2, fs = 145N/mm^2

$$= \frac{164.46 \times 10^3}{145}$$

$$Ast = \underline{1134mm^2}$$

Provide R 20 at 250 ($1260mm^2$)

total area of steel = $(1260 + 3270)mm^2$

$$= \underline{4530mm^2}$$

$$\frac{100As}{bd} = \frac{100 \times 4530}{1000 \times 170} = 2.66$$

From table 10, CP110 modification factor is 1.04

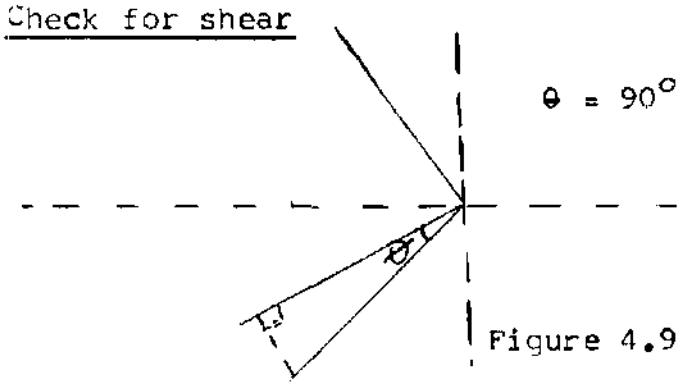
$$\therefore \text{Limiting } \frac{\text{Span}}{\text{effective depth}} = 26 \times 1.04$$

$$= 27.04$$

$$\text{Actual } \frac{\text{Span}}{\text{effective depth}} = \frac{3980}{170} = 23.41$$

Since the limiting value is greater than the actual, the section is adequate.

Check for shear



$\theta = 90^\circ - (90^\circ - 50.19^\circ) + 18.05^\circ$

$= 60.19^\circ - 18.05^\circ$

$= \underline{32.14^\circ}$

Figure 4.9

∴ Maximum shear force on the slab

$= 77.43 \cos 32.14^\circ$

$= \underline{65.56 KN}$

Shear force dur to self-weight of the slab

$= \frac{19.10}{2} = \underline{9.55 KN}$

∴ total shear force on slab

$= 9.55 + 65.56$

$V = \underline{75.11 KN}$

Shear stress $V = \frac{V}{bd}$

$= \frac{75.11 \times 10^3}{1000 \times 170}$

$= 0.44 \ N/mm^2$

$\frac{100Ast}{bd} = \frac{100 \times 4530}{1000 \times 170} = 2.66$

From table 5, CP110, Vc = $\underline{0.88}$

Es = 1.20 for h = 200mm

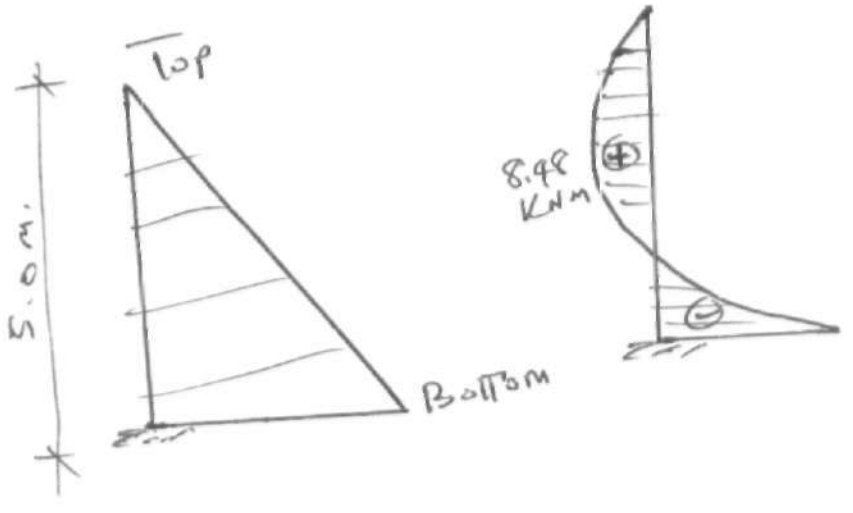
Es Vc = 1.20 x 0.88 = 1.06 > 0.44

∴ no shear reinforcement is required.

span both horizontally and vertically.

The whole load is generally transferred to the columns by the walls in the case of elevated bunkers, and when the span exceeds twice the depth of the wall, the wall can be designed as a beam. Owing to the large moments of inertia of the wall (as beam bending in a vertical plane) compared with that of columns, the beam can be assumed freely supported but the heads of the columns under the corner of the bunker should be designed to resist a bending moment equal to say, one third of the maximum positive bending moment on the beam. When the distance between the columns is less than twice the height of the wall, the reinforcement along the base of the wall should be sufficient to resist a direct tension equal to one-quarter of the total load carried by the wall.

The effect on large structures should be calculated. The vertical wall of the bunker is being considered as a propped cantilever, for the side pressure due to the contained material. The bending moment resulting from this is used to calculate the area of vertical reinforcement of the bunker wall.



a. Pressure diagram b. Stress diagram

The total Force $F = \frac{1}{2} \times 11.38 \times 5$

$= \underline{28.45KN}$ per metre width

Reactions

$$HA = \frac{F}{5}$$

$$= \frac{28.45}{5}$$

$$HA = \underline{5.69KN}$$

$$HB = \frac{4}{5}F$$

$$\frac{4 \times 28.45}{5}$$

HB = 22.76KN

Source : Steel designers Manual 4th Edition (1972)

4.4.1 Moments

M Top = 0

M Bottom = $-\frac{2}{15} \times F \times L$

$= -\frac{2}{15} \times 28.45 \times 5$

MB = - 18.97 KNm

Maximum positive moment along the span

= 0.0596 x 28.45 x 5

= 8.48KNm

The above value of support moment (MB) can be modified a bit to conform with ultimate state by multiplying by a partial safety factor for loads:

Mu = 18.98 x 1.4

Mu = 26.56 KNm

Assuming a thickness of 250mm for the wall and allowing a cover of 30mm and diameter 12mm bars

d = h - cover - Ø/2

= 250 - 30 - 6

d = 214mm

4.4.2 Aread of Steel (Reinforcement)

$$\frac{Mu}{bd^2 fcu} = \frac{26.56 \times 10^6}{1000 \times 2141 \times 25}$$

from la curve la = 0.95

but b = la d

= 0.95 x 214

= 203.3mm

Area of steel for bending

$$Ast = \frac{Mu}{0.87fy\ b}$$

$$= \frac{26.56 \times 10^6}{0.87 \times 250 \times 203.3}$$

$$Ast = 601mm^2$$

Provide R 12 at 150 centre-centre ($754mm^2$) at the inner face of the wall, and norminal reinforcement on each face throughout the section.

Horizontal Reinforcement

The area of steel that is required to take care of the horizontal reinforcement is that obtained from direct tension of the hopper on the wall.

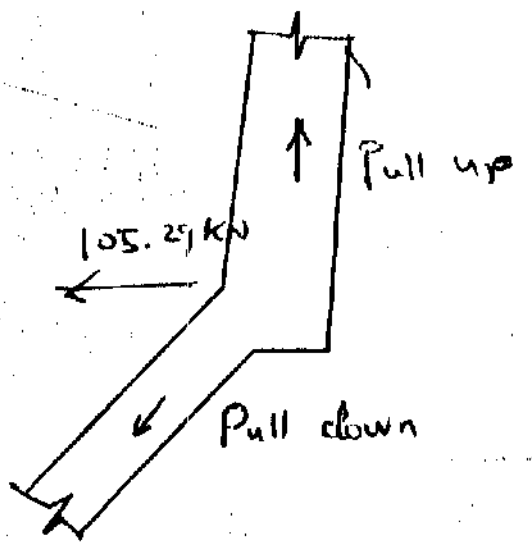


Figure 4.92: Direct tension of the hopper on the wall

Area of Steel for direct tension

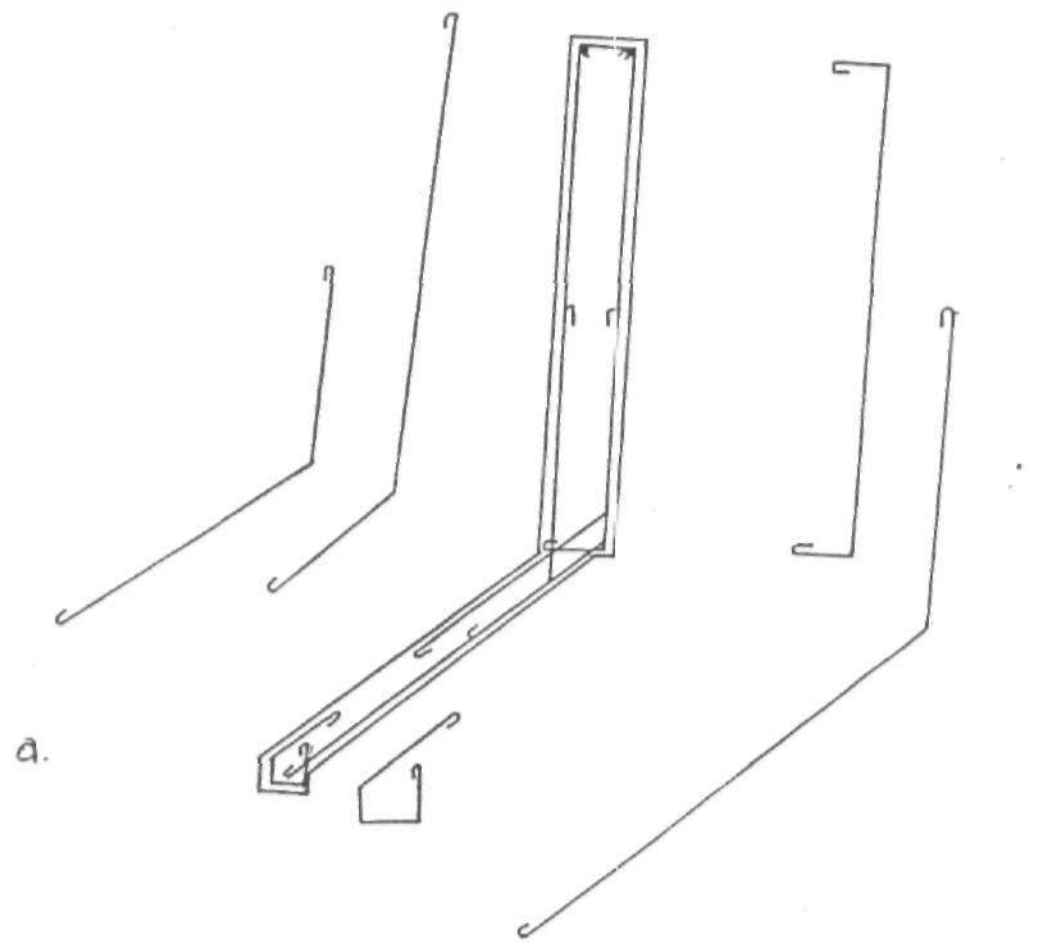
$Ast = \frac{T}{fs}$ from table 10, CP110, for fy = 250 N/mm^2 fs = 145N/mm^2

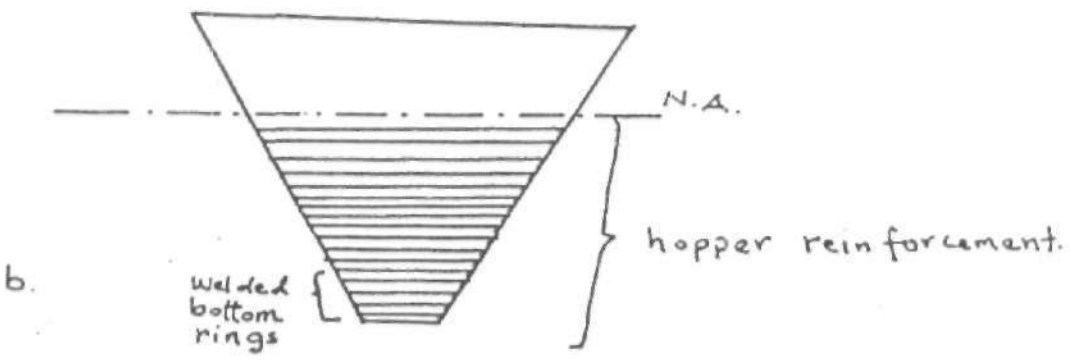
$= \frac{164.46 \times 10^3}{145}$

$= \underline{1134\ mm^3}$

Provide R 16 at 150mm (1134mm^2) horizontally from top to bottom. Provide also 4 R 16 at the bottom of the wall.

Figure 4.93 : Structural diagrams

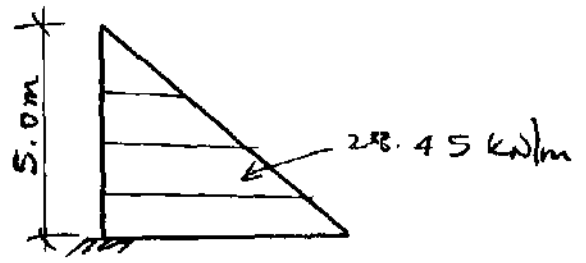




4.5 DESIGN OF INTERNAL WALL OF BUNKER

The design of the internal wall of the bunker is quite similar to that of the external one except that it experiences stresses from the two containers on either of its sides. It is expected that when this wall is designed for its critical loading arrangement, it will satisfy its stability requirements.

In this connection, it will be assumed that the critical loading arrangement occurs when one bunker is full and the other is empty.



Design as propped contilever

Moment $M_B = -\frac{2}{15} \times 28.45 \times 5$

$= -\underline{18.96 \text{ KNm}}$

$Mu = -18.96 \times 1.4$

$= -26.54 \text{ KNm}$

Using a wall thickness of 250mm and 12mm diameter bars, $d = 214mm$

$$\frac{Mu}{bd^2 fu} = \frac{26.54 \times 10^6}{103 \times 214^2 \times 25} = \underline{0.023}$$

from lever arm curve, $la = 0.95$

$\therefore \quad z = 0.95 \times 214$

$= \underline{203.3mm}$

$$Ast = \frac{M}{0.87 fy z}$$

$$= \frac{26.54 \times 10^6}{0.87 \times 250 \times 203.3}$$

$Ast = \underline{600mm^2}$ just as for the external wall.

Provide R 12 at 150mm centre to centre ($754mm^2$) vertical rods on each face from top to bottom. Provide R 16 at 150mm horizontal reinforcement as in the external wall and 4 bars, 12mm diameter at the bottom of the wall.

Span-effective depth ratio

$$\frac{100 As}{bd} = \frac{100 \times 754}{1000 \times 214} = 0.35$$

Modification factor = 1.99 table 10, CP110

Limiting $\frac{\text{Span}}{\text{effective depth}} = 1.99 \times 20 = 39.8$

Actual $\frac{\text{Span}}{\text{effective depth}} = \frac{5000}{214} = 23.36$

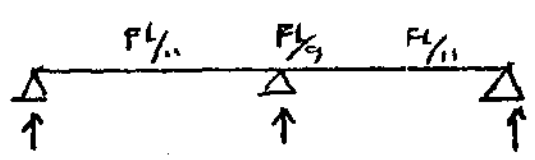
Since the limiting case is greater than the actual, the section is adequate.

4.6 DESIGN OF CONTINUOUS LONGITUDINAL BEAM AT TOP OF BUNKER

This beam runs from the end of one external wall over the internall wall to the other external wall. It serves more or less as a brace for the transverse walls. The beam is to be of section

400mm x 300mm

To stiffen the wall



Outward pressure from maize at the top

$= 9.66 \times 6$

$= \underline{57.96 KN/m}$

Figure 4.95 Longitudinal beam.

The maximum moment is at the middle support at B. This value of moment will be used for the moment at every point of the structure.

Horizontal B.M. $= \frac{57.96 \times 6}{9}$

B.M. $= \underline{38.6 KN/m}$

$$\frac{M}{bd^2 fcu} = \frac{38.64 \times 10^6}{300 \times 348^2 \times 25} = 0.04$$

From lever arm curve, la = 0.94

$$\therefore \quad z = 0.94 \times 348$$

$$= \underline{327mm}$$

$$Ast = \frac{M}{0.87fy\ z}$$

$$= \frac{38.64 \times 10^6}{0.87 \times 250 \times 327}$$

$$Ast = \underline{543mm^2}$$

Provide 4 R 16 ($804mm^2$)

This is to comprise 2 R 16 bars near each face.

4.7 DESIGN OF TRANSVERSE BEAM AT TOP

This beam spans between external walls. There is a pressure of 9.66KN/m at the top of the bunker due to the weight of retained material. Since the distance between columns is 6m, the maximum horizontal force due to this pressure

$$= 9.66 \times 6$$

on the beam = $\underline{57.96KN}$

The maximum moment at the midspan of the beam due to this force

$$= \frac{57.96 \times 6}{8}$$

$$= \underline{43.47KN/m}$$

This is accompanied by a direct tensile stress

$$= \frac{9.66 \times 6}{2}$$

$$= 28.98$$

$$= \underline{29KN}$$

Let the section of the beam be 450mm x 250 and diameter 20mm bars used with a concrete cover of 40mm.

$$\text{Effective depth } d = 450 - 40 - \frac{20}{2}$$

$$= 400mm$$

Resulting B.M. transferring the direct tension to the centre of steel in one face

$$= 43.47 - 29\ (0.4-0.225)$$

$$= 43.47 - 29\ (0.4-0.225)$$

$$M = \underline{38.40}Kn/m$$

$$\frac{M}{bd^2fcu} = \frac{38.40 \times 10^6}{250 \times 400^2 \times 25} = \underline{0.04}$$

From lever arm curve, la = 0.94

$$\therefore \quad z = 0.94 \times 400$$

$$= \underline{37}mm$$

area of steel for bending

$$Ast = \frac{M}{0.87\ fy z}$$

$$= \frac{38.40 \times 10^6}{0.87 \times 250 \times 378}$$

$$Ast = \underline{470mm^2}$$

area of steel for direct tension

$$= \frac{29 \times 10^3}{145} = 200mm^2$$

total area of steel

$$Ast = (470 + 200)\ mm^2$$

$$Ast = \underline{670mm^2}$$

Provide 4 R 16 ($804mm^2$)

This will comprise 2 R 16 near each face.

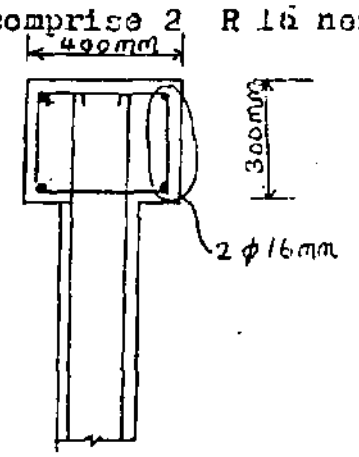


Figure 4.96: Section showing reinforcement of continuous beam at top

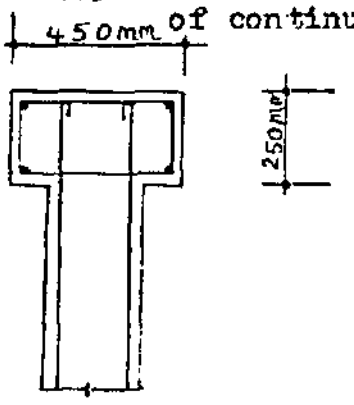


Figure 4.97: Section showing reinforcement of top transverse beam.

CHAPTER - FIVE

DESIGN OF COLUMNS AND FOOTINGS

5.1 DESIGN OF COLUMNS

The columns in a structure transmit the imposed loads on the structure to the foundation. These loads in certain cases are transferred from roofs, slabs, beams and walls. In the case of bunkers, the loads are transferred from the cover, the walls, the hopper and the beams, to the columns which in turn transfer these loads down to the foundation. Reinforced concrete columns are generally either rectangular in cross-section with separate links, circular or octagonal with helical binding. In most reinforced concrete columns, the main vertical bars are secured together by means of separate links or binders.

So many variants enter into the design of columns that it is not easy to decide readily which combinations give the most economical member, but results of tests and cost analysis carried out have shown that other factors being equal, the richer the concrete, the more economical is the column. For a square column, the minimum amount of longtitudinal reinforcement produces the cheapest member for a specified quality of concrete.

Columns are generally grouped into short and slender, each of which is considered as braced or unbraced.

5.1.1 Types of Columns

Short Column

A column is considered as short when the ratio of its effective height, le to the thickness, in both directions does not exceed 12, that is

$$\frac{lex}{h} \quad \text{and} \quad \frac{ley}{b} \leq 12$$

Slender Column

A column is considere as slender if the ratio of its effective height, le to the thickness in both directions is greater than 12.

$$\frac{lex}{h} \quad \text{and} \quad \frac{ley}{b} > 12.$$

Either of the above mentioned types of column could be braced or unbraced.

A column is considered braced in a given direction if the lateral stability of the structure as a whole is provided by walls or other suitable bracing to resist all lateral forces in that direction.

A column is said to be unbraced in a given direction when the lateral stability in that direction is provided by the column.

In a braced column, the axial forces and moments in the columns are caused by dead and

imposed loads only. On the other hand, in braced columns, the loading arrangements and the effect due to lateral loads are taken into consideration. The axial forces due to vertical loads on a column are calculated as though the beam and slab are simply supported. The critical arrangement of the ultimate loads on the beams are responsible for the bending moments in the columns.

5.1.2 Effective Height of Column

The effective height of a column is defined as the height between the points of cont - aflexture of the buckled column. The effective height is strongly dependent on the degree of fixity at both ends of the column.

Table 15 of CP110 gives some guidance on the choice of effective height in practical situations. Table 15 is reproduced below.

Table 15 CP110) Effective Column Height

	Types of Column	Effective column height
1.	Braced column properly restrained in direction at one end.	0.75 lo
2.	Braced column imperfectly restrained in direction at onr or both ends.	A value intermediate between 0.75 lo and lo depending on the efficiency of the directional restraints.
3.	Unbraced or partially braced column properly restrained in directions at one end but imperfectly restrained in direction at the other.	A value intermediate between lo and 2 lo depending upon the efficiency of the directional restraint.

5.1.3 Method of Design

Short braced column

When the wall on a short column is symmetrical or eccentricity occuring is only due to construction in-accuracies, the axial load, N is taken as:

$$N = 0.4\ f_{cu}\ A_{ct}\ 0.67\ A_{sc}\ f_y$$

If the load is truly axial load, $N = 0.45 f_{cu} A_c + 0.75 A_{sc} f_y$ with a reduction of approximately 10% allowance for accidental moment (allowance

due to construction in-accuracies) of 0.05h minimum.

Slender braced column

A slender column is liable to additional moments induced by the lateral deflection of the loaded column, the end of the column are considered fixed in position but in direction. The moments can be written as:

$$Mt = mit + Ne\ add$$

Where mi ia the maximum initial moments in the column due to the ultimate loads and Ne add is the additional moment induced by the columns deflection.

eadd is the deflection of the column due to load N.

$$eadd = \frac{Nh}{1750} \frac{(le)^2}{h} (1 - 0.0035 \frac{le}{h})$$

where h is the overall depth of the cross section in the plane of bending.

Short Unbraced Column

Short unbraced columns are analysed like braced columns but the initial moments due to vertical and lateral loads are compared with the additional moments of 0.05 Nh min and the greater value used in design.

Slender unbraced column

The additional moments due to deflection will increase due to the moments at the column ends so that the columns are in this case not fixed in po position or direction. The formular for eadd is the same as for braced columns and the maximum amount will be

$$Mi + Madd.$$

As the moments at the column ends are increased, the additional moments should taken into consideration when designing beams, bases or other members connected to the ends of such columns. If, however, the average le/h for all columns at a particular level does not exceed 20, then these additional moments can be ignored.

5.2 DESIGN OF SECTION

There are two bunkers, rigidly joined together and supported on six columns spaced at six metres centre to centre. The internal columns will be expected to carry about twice the load on the external columns due to the weight of the material.

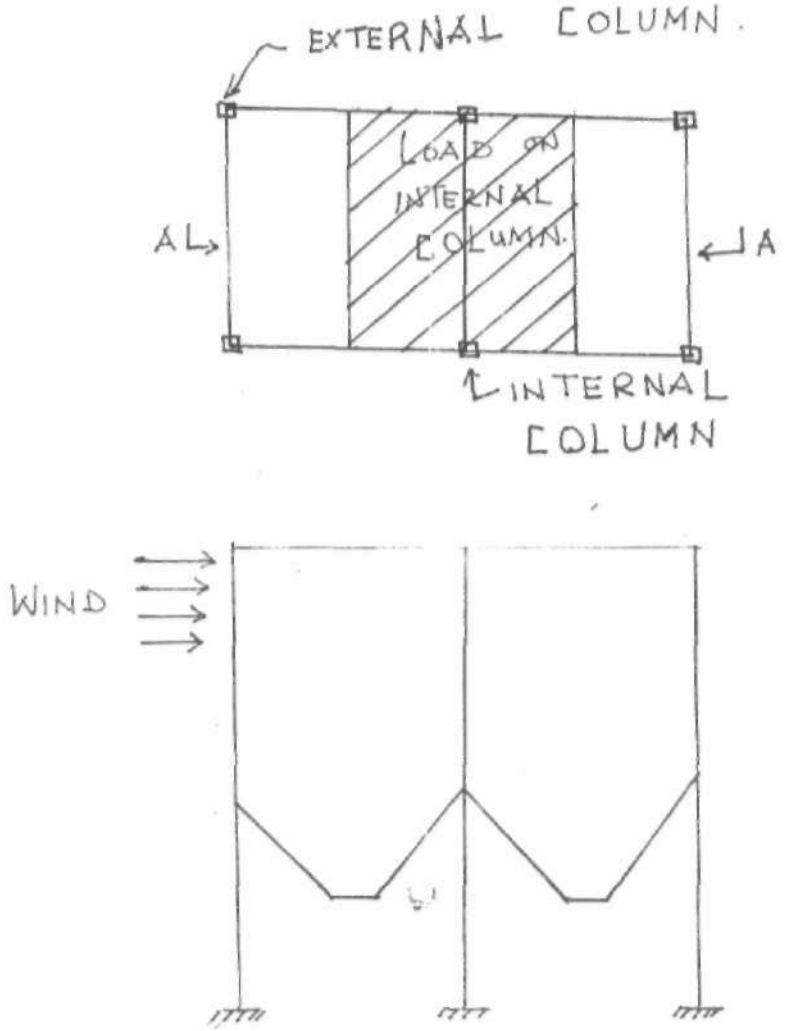


FIGURE 5.1 WIND LOADING

5.2.1 Load Estimation (dead and live loads)

Weight due to cover slab = ½ x 0.2x6x24

= 86.40KN

Load due to weight of maize = ½ x 340 x 6

= 1020.00KN

Load due to weight of wall = 0.25x5x6x24

= 180.00 KN

Load due to weight of hopper = 0.20x3.98x6x24

= 114.62 KN

" " " " " Transverse beam =

0.25x0.45x6x24 = 16.20 KN

" " " " " Longitudinal beam =

2x0.30x0.40x6x24 = 34.56 KN

= 1452KN

The liveload here = 1020KN and the dead load = 432KN

The self-weight of column, assuming a column section of 400 x 400 = 0.4 x 0.4 x 7 x 24 = 26.88

∴ the total dead load = 26.88 + 432 = 459KN

at ultimate limit state, the design column load

N = 1.4 GK + 1.6 QK

= 1.4 x 459 + 1.6 x 1020

= 642.5 + 1632

= 2275 KN

5.2.2 Wind Loading

Free standing bunkers installation will definitely be subject to the effects of wind.

The actual forces, and the proportions in which they should be applied are fully covered in B.S. CP3 chapter V. loading. It is unlikely that a fully loaded bunker, when wind is applied, will be a design criterion, but the case of the empty bunker and full wind should be investigated. A full appraisal of the effects of wind force have been fully investigated and the analysis is based on the recommendations of B.S. CP3 chapter V. loading.

Characteristic wind pressure

$$WK = 0.613Vs^2 \ N/m^2$$

$$Vs = V \times S_1 \times S_2 \times S_3$$

where Vs = Characteristic wind speed

V = basic wind speed

S_1 = Multiplying factor relationg to topology

S_2 = Multiplying factor relating to height above ground and wind breaking

S_3 = Multiplying factor relating to the life of the structure.

S_1 = May generally be taken as unity except otherwise stated.

S_2 = 0.97 (by linear interpolation)

for maximum vertical or horizontal dimesnion greater than 50 metres, topographical factor and total height of structure not exceeding 12m.

Basic wind speed

V = 42 mlsee (Zaria area)

Vs = 42 x 1 x 0.97 x 1

= 40.74m 1 sec.

all the factors S_1, S_2 and S_3 are dimensionless quantities.

$$WK = 0.613 \quad Vs^2$$

$$= 0.613 \quad (40.74)^2$$

$$WK = \underline{1017/m^2}$$

The total wind force F (or WK) on an area of A of a particular face of a structure

= WK x A x CP0 or

= WK x A x Cf

where Cf is a force coefficient 'A' is the area of the exposed portion of the structure normal to the direction of the wind.

Source: Reinforced concrete designers' handbook by Charles Reynolds and James Steedman (6th edition) page 164, table 13, Cf =1.

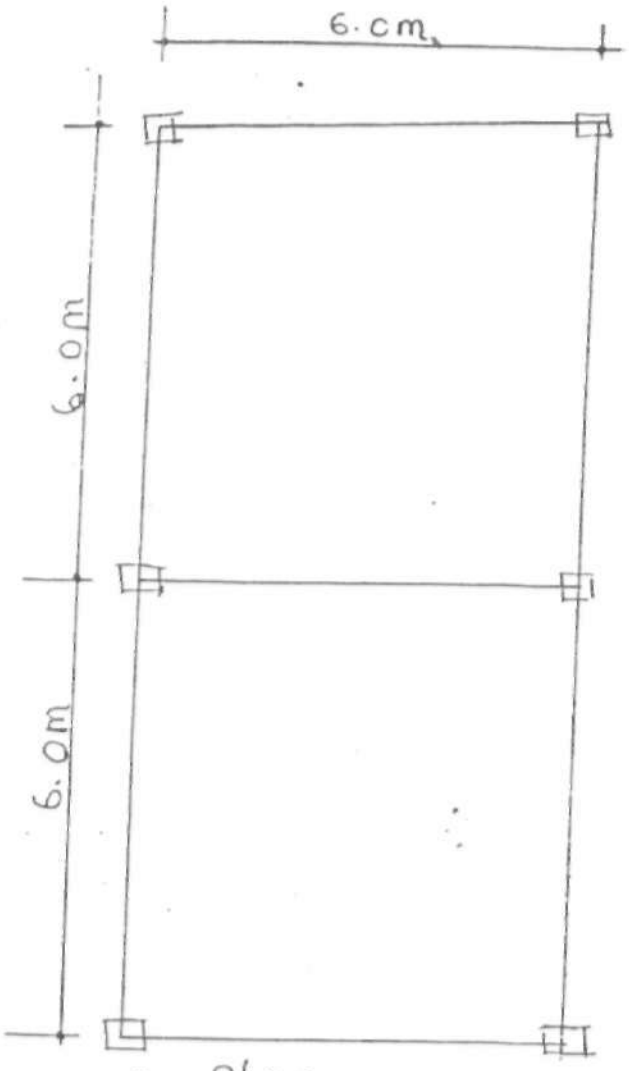
The total wind force Wk on the column

$$= 1017 \times 10^{-3} \times 8 \times 6 \times 2$$

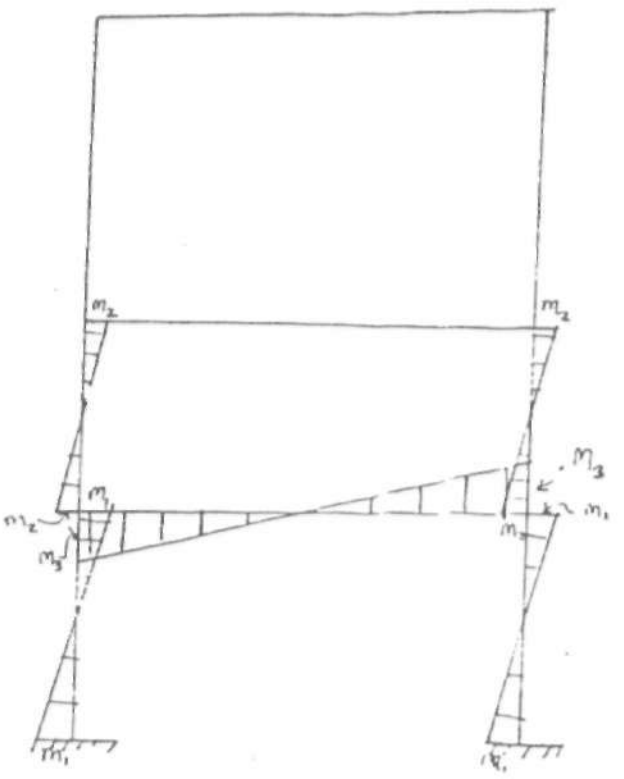
= 97.63KN

This is the condition of maximum wind effect which is supposed to occur when both bunkers are empty. Since there are six columns, the total horizontal force per column = $\frac{97.63}{6}$

= 16.27KN



a. Plan



b. Section

Figure 5.2

$$M_1 = \frac{97.63 \times 3}{2 \times n}$$

where n = 6 that is the number of columns

$$= \frac{16.27 \times 3}{2}$$

$$M_1 \quad \underline{24.41 \text{ KNm}}$$

$$M_2 = \frac{97.63 \times 4}{2 \times 6}$$

$$= \frac{16.27 \times 4}{2}$$

$$M3 = M_1 + M_2 = 24.41 + 32.54$$

$$= 56.95$$

$$M_3 = \underline{57 \text{ KNm}}$$

In order to calculate the force due to wind load on the outer column, the moment of inertia of the group is first calculated. The maximum force in any column usually occurs when the axis about which the moment of inertia is being calculated passes through two or more columns.

From figure above

$$\bar{I} = 2 \times 2 \times (0.5B)^2$$

$$= 2 \times 2 \times 0.25\ B^2$$

$$I = \underline{B^2} = \underline{12^2}$$

The force due to wind on outer rows of columns is

$$M\left(\frac{0.5B}{B^2}\right) = \frac{0.5M}{B} \text{ in each column}$$

$$= \frac{0.5 \times 32.54}{112}$$

$$= \underline{1.36KN}$$

5.2.3 Slenderness ratio

Considering the column as unbraced imperfectly restrained in direction at one or both ends, CP110 gives the value of effective column height as intermediate between Lo and 2 Lo.

Consider say le = 1.5lo

le = 1.5 x 4000

= 6000 mm

$$\text{Slenderness ratio} = \frac{le}{\text{bor } h}$$

$$= \frac{6000}{400}$$

$$= \underline{15} > 12$$

If the column section is limited to say 400 x 400 mm^2 then this column must be designed as a slender column. A larger column section will be tried, say 600 mm x 600mm.

$$\text{Slenderness ratio} = \frac{6000}{600}$$

$$= 10 < 12$$

∴ this column can be designed as short.

The total design load

$$N = 2275 + 1.35$$

$$= \underline{2277KN}$$

Since perfect condition never exist, and to allow for small eccentricity, the ultimate load should be calculated from:

$$N = 0.4\ f\ cu\ Ac + 0.67\ Asc\ fy$$

$$Asc = \frac{N - 0.4\ fcu\ Ac}{0.67\ fy}$$

$$= \frac{2277 \times 10^3 - 0.4 \times 25 \times 600^2}{0.67 \times 250}$$

$$= \frac{2277 \times 10^3 - 3600 \times 10^3}{167.5}$$

$$Asc = \underline{7898\ mm^2}$$

The negative area of steel here indicates that the concrete section alone is adequate to resist crushing. Minimum Area of steel will therefore be needed to take care of bending due to eccentricity and other imperfections.

$$Asc\ min = 1\% \times 600^2$$

$$= \underline{3600}\ mm^2$$

Provide 8 R 25 ($3930\ mm^2$)

5.2.4 Links

Diameter of links should be greater than or equal to one quarter of the largest longitudinal bar diameter = 25/4 = 6.25mm

Provide R 8 at 150mm centres (4 legs)

Spacing of Links

Maximum spacing = 12Ø

= 12 x 25

= $\underline{300 > 150mm}$

∴ It is okay.

Lap length = 20Ø + 150 mm

= 20 x 25 + 150 mm

= $\underline{650mm}$

Spacing within lap = $\frac{150}{2}$

= $\underline{75mm}$

3 R 25 mm

each face

5.3 DESIGN OF BRACE

The maximum bending moment on the brace is the sum of the column moments above and below the brace

$$= 24.41 + 32.54$$

$$= \underline{56.95\,KNm}$$

Try a section of brace 400mm x 600 mm and 25 mm bars top and bottom,

$$d = 600 - 40 - 25$$

$$= \underline{535}mm$$

$$\frac{A}{bd^2 fcu} = \frac{56.95 \times 10^6}{400 \times 535^2 \times 25}$$

$$= \underline{0.02}$$

From level arm curve, la = 0.95

$$z = 0.95 \times 535$$

$$= \underline{508.25}mm$$

$$Ast = \frac{M}{0.87\, fy\, z}$$

$$= \frac{56.95 \times 10^6}{0.87 \times 250 \times 508.25}$$

$$Ast = \underline{515}mm^2$$

Provide 2 R 20 ($628mm^2$) from column to column, top and bottom. This is necessary to take care of both the wind moments (negative) at the supports and the

Positive Moment due to the self-weight of the brace within the span. Shear reinforcement, provide ? leg links R 8 at 150mm centre to centre.

5.4 DESIGN OF FOUNDATION

A building is generally composed of a superstructure above the ground and a substructure below the ground. It is that part of a building below the ground or the substructure that is referred to as the foundation. The super-structure does not necessarily have to be a building, it could infact be a bunker as in this project. The foundations transfer and spread the loads from a structure's columns and wall into the ground. The foundation that is eventually designed will carry its imposed loads effectively if and only if the safe bearing capacity of the soil is not exceeded, if not, excessive settlement will occur.

Since the earth under the foundations is the most variable of all the materials that are considered in the design and construction of an engineering structure, it is important to have an engineering survey made of the soils under a proposed structure so that variations in the strata and soil properties can be determined.

Settlement takes place during the working life of a structure, therefore the design loading to be considered when calculating the base area should be

those that apply to the serviceability limit state and which are:

1. dead plus imposed load = 1.0GK + 1.0.QK
2. dead plus wind load = 1.0GK + 1.0.WK
3. dead plus imposed plus wind load =

1.0GK + 0.8Øk +0.8WK.

where the foundations are subject to a vertical and a horizontal load, the following rule should apply:

$$\frac{V}{P_v} + \frac{H}{PH} < 1.0$$

where V = the vertical load

H = the horizontal load

P_v = the allowable vertical load

P_h = the allowable horizontal load

The calculation to determine the structural strength of the foundations, that is the thickness of the bases and the areas of reinforcement, should be based on the loadings, and the resultant grounds pressures corresponding to the ultimate limit state.
To ensure adequate safety the stability calculations should also be for loading arrangements associated with the ultimate limit state.

It is recommended that the minimum cover to reinforcement should be not less than 75mm when the concrete is cast against the ground, or less than 40mm when the concrete is cast against a layer of building concrete.

5.4.1 Pad Footings

It is intended that the footing under the four columns supporting the weight of the bunker be designed as independent pad footings. Pad footings may be square in plan, but where there is a large moment acting about one axis it may be more economical to have a rectangular base.

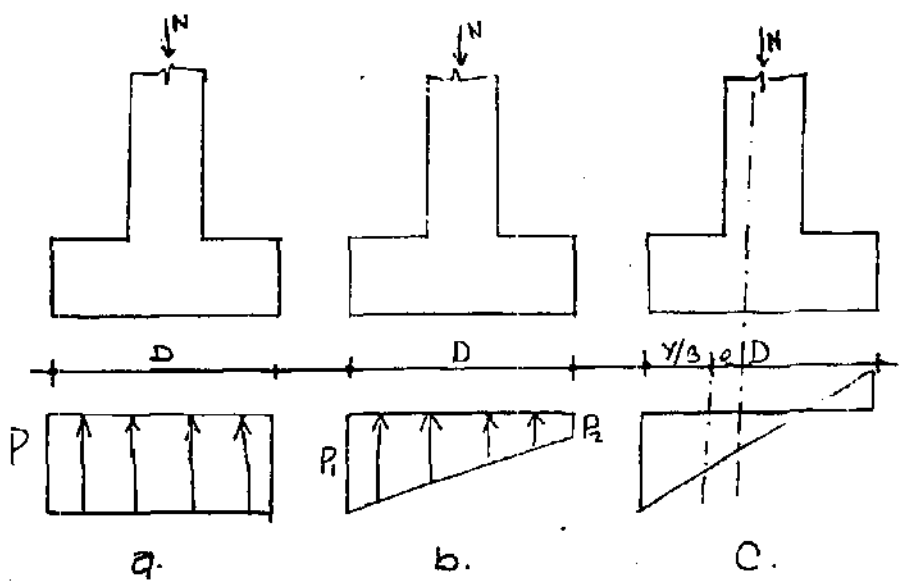


Figure 5.3

(a)	(b)	(c)
$e = 0$	$e \leq D/6$	$e > D/6$
$p = \frac{N}{BD}$	$P = \frac{N}{BD} + \frac{6m}{BD^2}$	$P = \frac{2N}{BY}$
Pressure distribution across the base of footing.		where $Y = 3(D/2 - e)$

5.4.2 Reinforcement of Pad Footings

With a square base the reinforcement to resist bonding should be distributed uniformly across the full width of the footing.

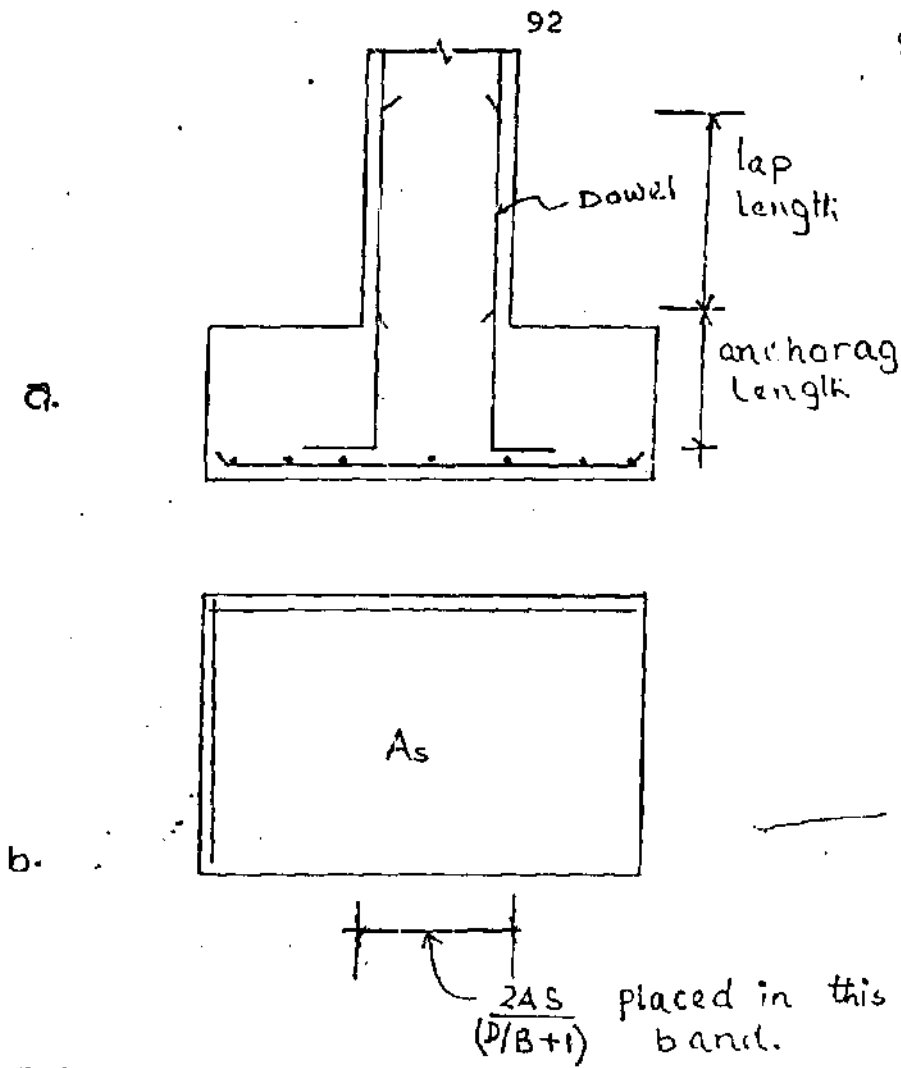


Figure 5.4:

For a rectangular base, the reinforcement in the short direction should be distributed as shown in fig. 5.4 above. If the footing should be subjected to a large overturning moment, so that there is only partial bearing, or if there is a resultant uplift force, then the reinforcement may also be required in the top face. Dowels or starter bars should extend from the footing into the column to provide continuity to the reinforcement. These dowels should be embedded in the footing an anchorage bond length and extend into the columns a full lap length. At times, a 75mm length of the column is constructed in the same concrete pour as the

In these cases the dowel lap is measured from the top of the kicker.

5.4.3 Foundation for Bunker Columns

Design data

Dead load from columns = 459.00KN

Self-weight of footing =

$3.0 \times 3.0 \times 800 \times 10^{-3} \times 24 =$

172.80KN

632KN

take total dead load to 632KN

total liveload = 1020 KN

Soil bearing capacity = 200 KN/m^2

characteristic concrete strength f_{cu} = 25 N/mm^2

characteristic concrete strength fy = 250 N/mm^2

5.4.4 Preliminary Dimensions

(a) At serviceability limit state, design

axial load = 1.0 GK + 1.00 k

= 1.0 x 650 + 1.0 x 1020

= 1670KN

Required base area = $\frac{1670}{200}$

= $8.35m^2$

Provide a base 3m x 3m = $9m^2$

(b) At ultimate limit state

design axial load = 1.4GK + 1.6QK

= 1.4 x 650 x 1.6 x 1020

= 910 + 1530

= 2440KN

earth pressure = $\frac{2440}{32}$

= 271 KN/m^2

Try an 800mm thick footing

Net upward pressure = 271-h x 24 x f

= 271 - 0.8 x 24 x 1.4

= 244 KN/m^2

(c) Ancherage length for dewels compression encharage length

= 25 x bar size

= 25 x 25

= 625 mm

Therefore a base thickness of 800mm would adequately allow for concrete cover and two layers of reinforcement below the dowels.

(d) Punching Shear - see figure.

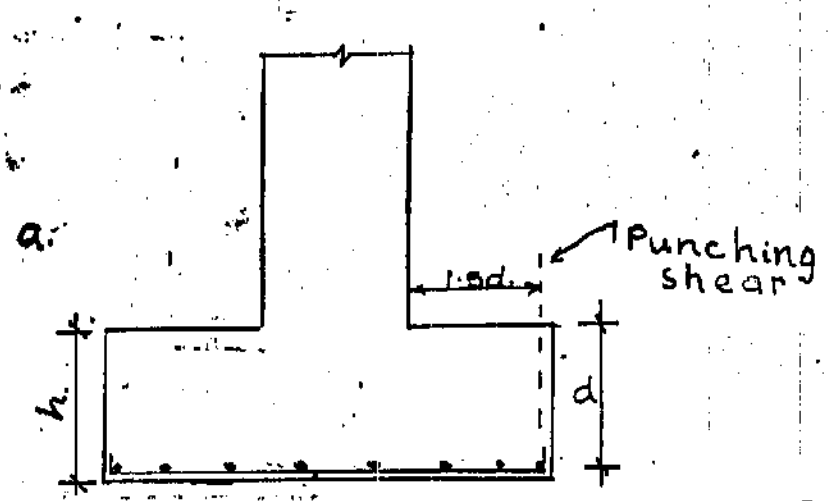


FIGURE 5.5 CRITICAL SECTION FOR DESIGN.

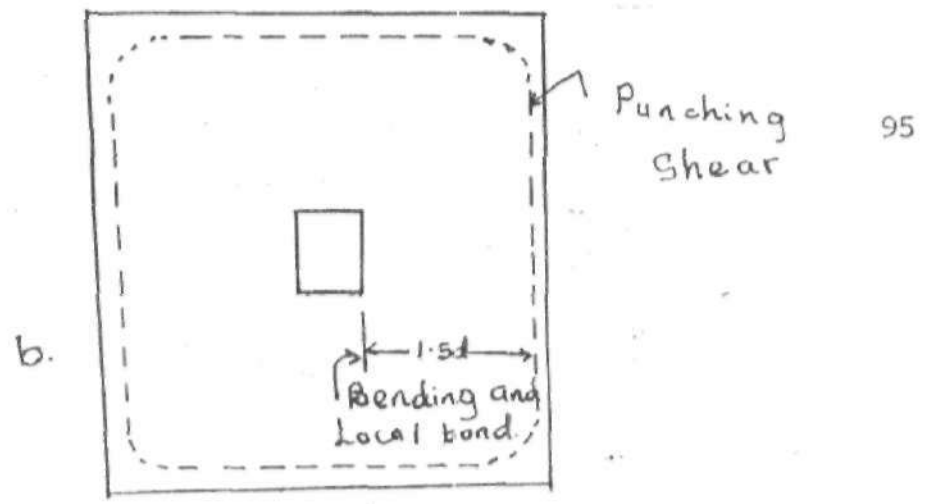


FIGURE 55 CRITICAL SECTION FOR DESIGN

The footing is constructed on a blinding layer of concrete so that the minimum cover is to be 40mm.

for h = 800mm, d = 720mm

Critical perimeter = column perimeter + 3 h

= 4 x 600 + 3 x 800

= 9936mm

area within perimeter= $(600 + 3h)^2 - (4-n)(1.5h)2$

= $(600 + 800 \times 3)^2 - (0.86)(1.5 \times 800)^2$

= $9 \times 10^6 - 1.24 \times 10^6$

= 7.94×10^6 mm^2

= $7.94 m^2$

∴ Punching shear force V = $244 (3^2 - 7.76)$

V = 303 KN

Punching shear stress $v = \frac{V}{per. \times d}$

$= \frac{303 \times 103}{9936 \times 720}$

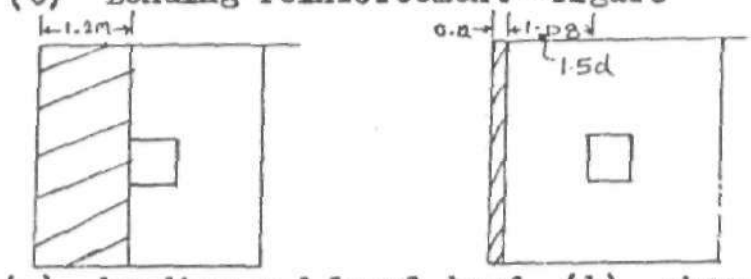
V = 0.042 N/mm^2

This value of shear stress is quite in order, so

h = 800mm is adequate.

5.4.5: Reinforcement

(c) Bending reinforcement -figure



(a) bending and local band (b) shear

figure 5.6 critical sections.

At the column face which is the critical section

$$M = (244 \times 3.0 \times 1.2) \times \frac{1.2}{2}$$

$$= \underline{527KNm}$$

for the concrete

$$u = 0.15 f_{cu} bd^2$$

$$= 0.15x25x3000x720^2x10^{-6}$$

$$= \underline{5832\ KNm}$$

$$\frac{M}{bd^2 f_{eu}} = \frac{527 \times 10^6}{3000 \times 720^2 \times 25} = 0.014$$

(g) Final check for punching shear

from table fan = 25

$$\frac{100\ As}{bd} = 0.18$$

Ultimate shear stress vc = $0.35\,KNm^2$

Punching shear was $0.04\ Nkmm^2$

therefore the section is adequate.

At the critical section for shear, 1.5d from the column face

$$V = 244 \times 3.0 \times 0.12$$

$$= \underline{87.84}\ KN$$

$$v = \frac{V}{bd}$$

$$= \frac{87.84 \times 10^3}{3000 \times 720}$$

$$v = 0.04 N/mm^2 < 0.35 N/mm^2$$

Therefore the final check for shear is satisfactory.

The section is adequate for shear.

From lever arm curve la = 0.95

$\therefore$ z = 0.95 x 720

$$= \underline{684mm}$$

$$Ast = \frac{M}{0.87\ fy z}$$

$$= \frac{527 \times 10^6}{0.87 \times 250 \times 684}$$

$$Ast = \underline{3542\ mm^2}$$

Provide R 25 at 125 mm centre to centre ($3930mm^2$)

$$\frac{100As}{bd} = \frac{100 \times 3930}{1000 \times 720}$$

$$= 0.18 > 0.15 \text{ as required}$$

Minimum spacing = 300mm

fy = 250 and 0 redistribution

(f) Local bond

At the critical section for bending shear

$V = 244 \times 3.0 \times 1.2 = 878KN$

32mm at 125.

Perimeter $Us = 2 \times 16 \times 3.14 = \frac{100.48}{24}$

approx. No. of bars $= \frac{3000}{125}$

Local bond stress $fbs = \frac{V}{Us\ d}$

$$\frac{878 \times 10^3}{24 \times 100.48 \times 720}$$

$= 0.51 < 2.0\ N/mm^2$ ∴ the section is adequate.

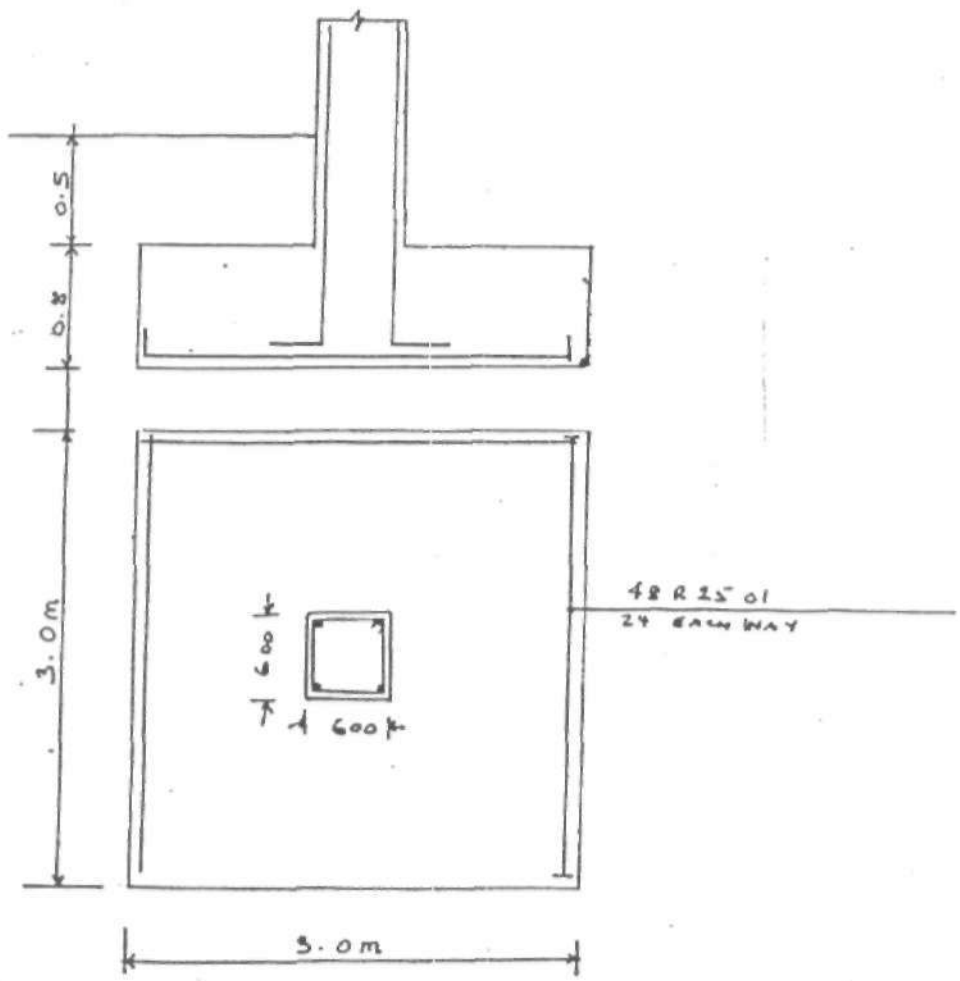


FIG 5.7 Details of reinforcement

Chapter six

6. 0 COMPARATIVE COST ANALYSIS

6.10 Shape and Dimensions

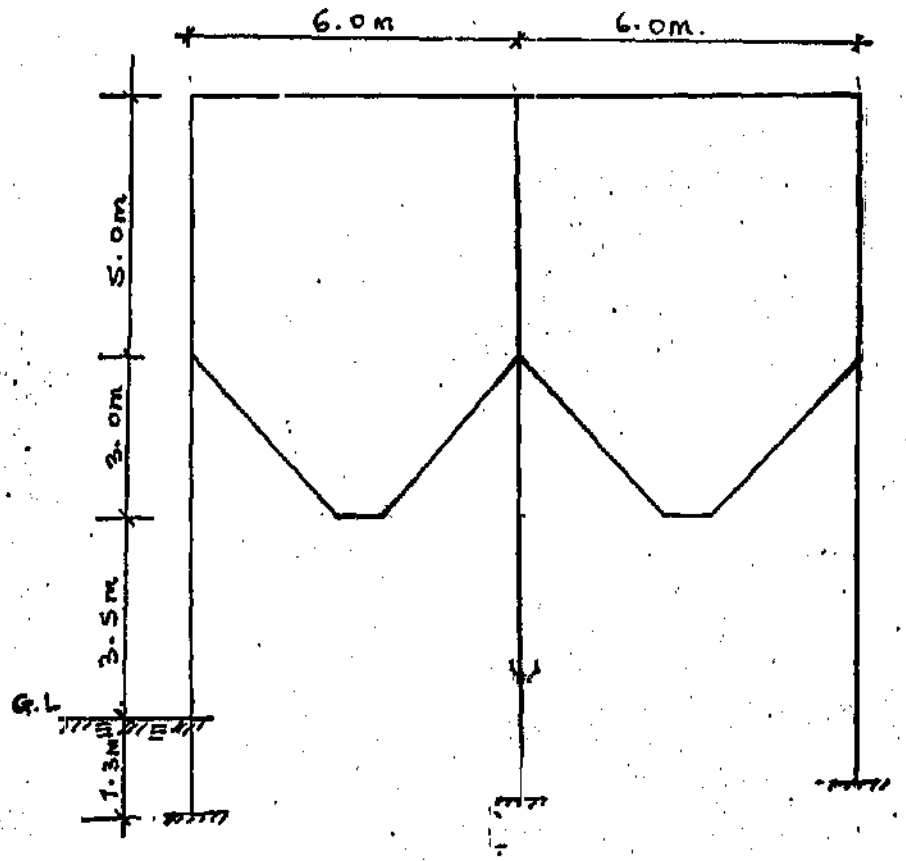


Fig. 6.1 Plan and section of Bunkers

6.20 Cost Analysis of R.C. Bunkers

Volume of Concrete

Concrete in Hopper Slab = $\frac{(1+6)}{2}3x0.2x2+\frac{(0.6x0.56)}{2}$ x 3 x 2 x 0.2

= 4.2 x 3.72

= $\underline{7.92m^3}$

Concrete in Walls = 5x6x0.2x2 + 5.6x5x0.2x2

= 12 + 11.2

= $\underline{23.2m^3}$

Cover Slab = 6 x 6 x 0.1 x 2

= $\underline{\underline{7.2m^3}}$

Columns = 0.6 x 0.6 x 12 x 6

= $\underline{\underline{25.92m^3}}$

Foundations = 3 x 3 x 0.8 x 6

= $\underline{43.2m^3}$

Brace = 0.4 x 0.6 x 6 x 6

= $\underline{8.64m^3}$

Total Volume of Concrete = $\underline{116.08m^3}$

Add 14 Per cent for Shrinkage = 0.14 x 116.08

= $\underline{16.25m^3}$

$\therefore$ Total Volume = 116.08 + 16.25

= $\underline{132.33m^3}$

Concrete Mix 1:2:4

Cement: Density = 1440kg/m^3; S.G. = 3.10

Sand: Density = 1600kg/m^3; S.G. = 2.65

Gravel: Density = 1560kg/m^3; S.G. = 2.60

Absolute Volume

Cement = $\frac{1 \times 1440}{3.10}$ = 464.50L

Sand = $\frac{2 \times 1600}{2.65}$ = 1207.56L

Gravel = $\frac{4 \times 1560}{2.60}$ = 2400L

W/C = 0.6

$\therefore$ Water = $\frac{0.6 \times 1440}{1}$ = 864L

Total Absolute Volume = 4936.05

= 4.94m^3

Quantity of Material Per m^3 of Concrete

Cement = $\frac{1 \times 1440}{4.94}$ = 291.50kg/m^3

Sand = $\frac{2 \times 1660}{4.94}$ = 647.77kg/m^3

Gravel = $\frac{4 \times 1560}{4.94}$ = 1263.16kg/m^3

Water = $\frac{0.6 \times 1440}{4.49}$ = 174.90kg/m^3

Total Quantity of Materials

Cement = 291.05 x 132.33

= 38574.20kg

Sand = 647.77 x 132.33

= 85719.40kg

Gravel = 1263 x 16 x 132.33

= 167153.96kg

Water = 174.90 x 132.33

= 23144.52kg

Cost of Materials

Cement = $\frac{35574.20}{50}$kg

= 771.48 or 772 Bags

772 Bags of Cement @ ₦15.00 Per Bag

= ₦11,580.00

Sand = 85719.40kg

Volume (in m) = $\frac{85719.40kg}{1600kg/m^3}$

= 53.57

Volume = $54m^3$

Number of Trips = $\frac{54}{4}$

= 13.5 or 14 Trips

14 Trips of Sand = @ ₦40:00 Per Trip

= ₦560.00

Gravel = 167153.96kg

Volume (in m) = $\frac{157153.96}{1560}$kg

= $107.15m^3$

Number of Trips = $\frac{107.15}{4}$

= 26.70

= 27 Trips

27 Trips @ ₦400:00 Per Trip (Chippings).

= ₦10,8000

Cost of Reinforcement = ₦4,500:00

Total Cost of Materials = ₦27,440:00

Add 20 Per cent for Labour = ₦5,488:00

Add 5 Per cent for Form Work = ₦1,372:00

Add 10 Per cent for Finishes = ₦2,744:00

Grand Toral ₦37,044:00

This amount (₦37,044:00) when compared with the cost of an equivalent size of bunkers in steel (analysis not included in the thesis) of forty five thousand Naira (₦45,000:00), leaves one with a savings of up to seven thousand, nine hundred and fifty six Naira (₦7,956:00) only.

CONCLUSION

For the present effort of the Government of Nigeria towards self-sufficiency in food production to yield the desired result, records of the present level of production of local farm produce, consumption pattern and annual production growth rate must be available. These quantities once know, can be used to plan in the long term. These will enable Government to plan our storage systems in anticipation of excess produce and to arrange for importation when production experiences a shortfall. Through this kind of an arrangement, it will be easy to decide how long it will be necessary to keep produce in storage and when such produce should be exported, used for relief operation or replaced by new farm produce.

It is believed, experience has also shown that once a grain has been properly dried, it can be stored for a fairly long period without going bad in a system of bunkers and/or silos. The efficiency of this system of storage lies in its height above the ground, which, apart from making it impracticable for rodents to attack its contents also enables trucks to go beneath it to load its contents to the points where they are needed. The use of this system of storage produces a very efficient use of space as it is purely a vertical construction. Some time and effort are

saved from the use of this system since no unloading is required; the system is self-discharging.

The author believes very strongly that there has been very little or no attention paid to the study of grain storage tanks generally in this country. He hopes that by providing a document of this sort he will be removing those things that have hitherto stood in the doorway of progress in this area of knowledge and really see this document as that which is intended to stimulate interest through its intended wide readership in this all-important area of bulk storage of grains.

No code of practice to date has come out with a clear procedure for the design of bunkers and silos, bunker and silo elements can be grouped under the usual structural elements as slabs, beams, columns and foundations. Care has been taken to fully appreciate the peculiarities of this kind of structures arising from high liveloads, high pressure intensities and structural geometry.

Bunkers and Silos fall into easily definable types depending on their size, shape and mode of discharge. They can take various shapes as circular, square, rectangular or any other form of a regular polygon in plan. The plan shape does not have any

significant effect on the performance of the system as it does have on the economy of material and ease of erection; for instance, the circular plan shape is more economical in material but more involving during construction than the rectangular shape. For this reason, the circular plan shape is thought best to use for silos, while the bunker can take any of the plan shapes depending on what criterion the design is based.

The British code of practice CP110 has been used as a guide throughout the design of the structural elements of these bunkers.

RECOMMENDATIONS

The choice between the use of a bunker, a silo, a battery of bunkers or a battery of silos depends on many factors, ranging from the quantity of material to be stored, the level and type of technology available, the location, the sub-soil condition, the length of time material might be required in storage, etc. If for a given quantity of material to be stored, it is required to choose the most economical of the two of four options, then a critical appraisal of all the points for and against each alternative must be made, together with a comparative cost analysis of these options.

A bunker or a group of bunkers made of steel or reinforced concrete will be quite ideal for our setting in Nigeria. This is because of certain problems that a silo is identified with; for instance, because of its capacity and height above ground, it cannot be erected on just any type of soil. A silo erected on a soil with a low beaming capacity stands the risk of either overturning or outright sinking.

From the comparative cost analysis carried out in Chapter six it can readily be seen that there is a great cost difference between the reinforced concrete bunker and the steel bunker. Every other thing being equal, therefore, the reinforced concrete bunker is a far cheaper option and is therefore recommended for use in small to medium sized grain depots.

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