MINIMUM ECCENTRICITY IN THE ULTIMATE STRENGTH ANALYSIS

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Thesis submitted in partial fulfilment of the requirements for the award of the degree of MASTER OF ENGINEERING (in Civil Engineering) of Ahmadu Bello University, Zaria, Nigeria.

May, 1980

THESIS APPROVAL

We hereby recommend that the thesis prepared under our supervision by Mr. Olusanya Sunday Solomon entitled:

"Minimum eccentricity in the ultimate strength analysis"

be accepted in partial fulfilment of the requirements for the award of the degree of Master of Engineering (in Civil Engineering) of Ahmadu Bello University, Zaria.

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DEDICATION

Your sustained efforts to care for me since my childhood days has brought me thus far. In sincere appreciation of all that effort, this thesis is dedicated to you, dear uncle,

Mr. Jacob Adeyanju Oluọdẹ.

O. S. Solomon

May, 1980.

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Ahmadu Bello University,
Zaria

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NOTATION

Notation is generally in accordance with CP 110, and the principal symbols are listed below. Other symbols are defined in the text where necessary.

A_s'	Area of compression reinforcement
A_s	Area of tension reinforcement
b	Width of section
d	Effective depth of tension reinforcement
d'	Depth to compression reinforcement
d_2	Depth to reinforcement
E_s	Modulus of elasticity of steel (= 200kN/mm^2)
e	Eccentricity
f_{cu}	Characteristic concrete cube strength
f_y	Characteristic strength of reinforcement
h	Overall depth of section in plane of bending
M	Bending moment due to ultimate loads
N	Ultimate axial load
x	Neutral axis depth
x_c	Depth of concrete in compression
α	Strength ratio
β	Moment ratio
ε	Strain
ρ	Reinforcement ratio
φ	Stress ratio

ABSTRACT

The nominal allowance for eccentricity due to construction tolerances as specified in three different codes of practice (ACI 318-77, CP 110, and CEB-FIP Model Code) was studied and compared with actual values of construction errors measured on building sites. It was found that the three specifications do not compare favourably with one another, neither do they agree very well with site results. Some inherent flaws in the way these codes specify the nominal eccentricity was also discussed.

An alternative specification was then proposed which not only corrects these flaws but also compares favourably with site results. Unlike in the three codes referred to above the proposed specification may be built-in into the design formulae and it was shown that this simplifies the ultimate strength design formulae.

Finally, this alternative proposal was utilized to develop a set of formulae which allow for a uniform approach to the design of concrete sections under different combinations of bending moments and axial loads.

CHAPTER ONE

INTRODUCTION AND LITERATURE REVIEW

1.1 It is an accepted fact that man is yet to achieve perfection. On building construction sites therefore, various tolerances should, and usually are, allowed so that a column designed as 300 x 300 mm for example should not be expected to be constructed to precisely this size; if a tolerance of say 5mm is allowed, then a column of sides anywhere between 295mm and 305mm will be passed as being satisfactory. A similar thing applies when eccentricities are considered. This thesis will study eccentricity errors on rectangular reinforced concrete elements, and only single axis bending will be considered.

To allow for construction inaccuracies and to provide sufficient safety for reinforced concrete elements, an additional eccentricity e_1 say, is introduced into design calculations at the point through which the resultant of the external forces acting on the elements passes, and in the most unfavourable direction /1.1/. This will have the effect of increasing the ultimate load effect. Construction inaccuracies include the following:

i. horizontal alignment error,

ii. transfer alignment error, and

iii. vertical alignment error.

These errors are illustrated in fig. 1.1 and discussed in detail in chapter 2.

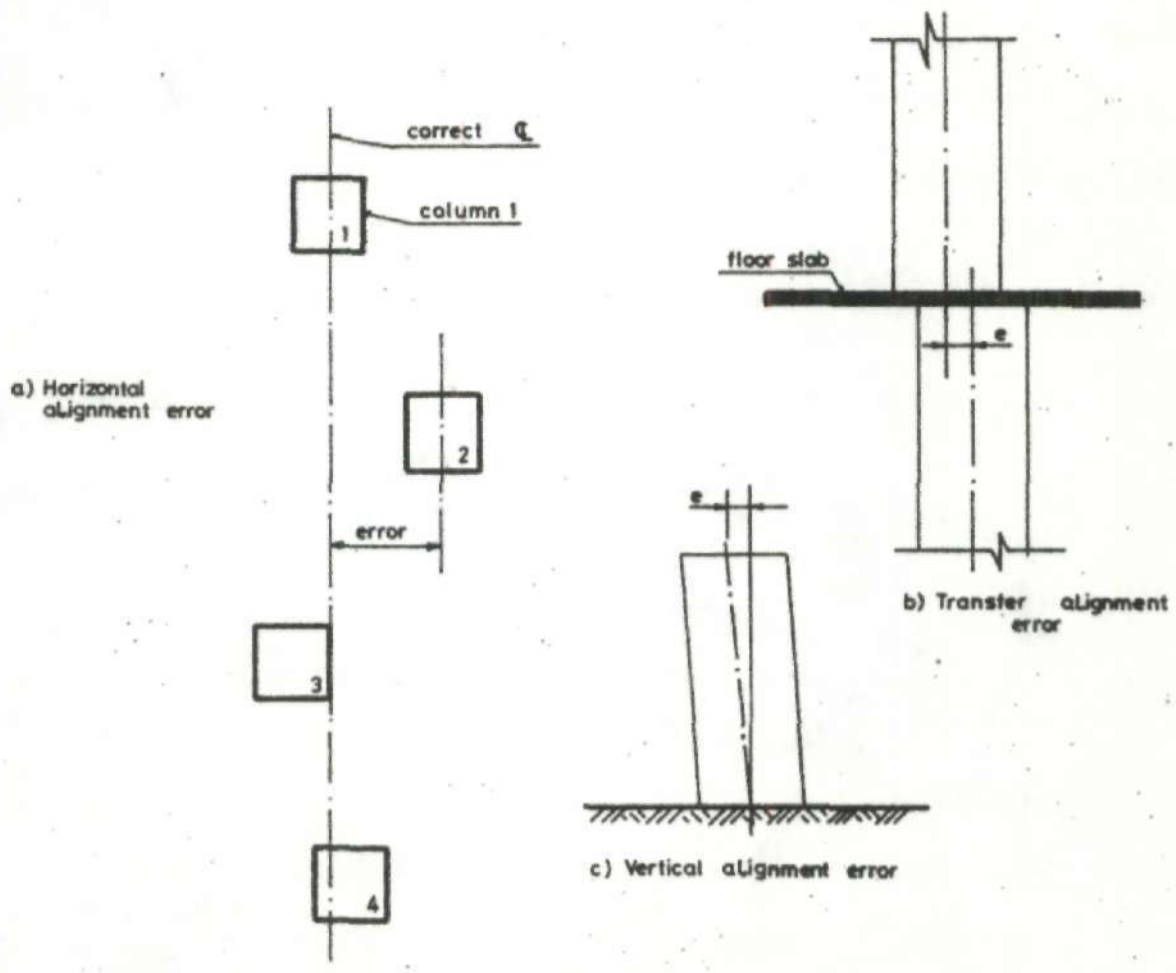


FIG. 1.1 CONSTRUCTION ERRORS

The safety aspect is intended to take account of the uncertainty resulting from the following:-

i. Reinforcement inclination; possible deviations of the reinforcing bars from the principal directions of the axial stresses.

ii. Workmanship; possibility that the construction is not executed by skilled craftsmen under efficient supervision, and that it is not erected in reasonably clement weather.

Construction errors may have various undesirable consequences. Consider for example, the horizontal alignment error illustrated in fig. 1.2. Due to the alignment error along Row A, the span of beam B, (l_2), is greater than designed, (l_1). This will increase the bending moment on the beam and may result in bending failure. Column A is also subjected to an additional moment due to the additional eccentricity e_1 introduced during construction. This may cause material failure in the column.

Problems that may arise from transfer and vertical alignment errors are not unsimilar to those discussed above, and the minimum eccentricity specified in codes of practice is intended as a safeguard against these problems.

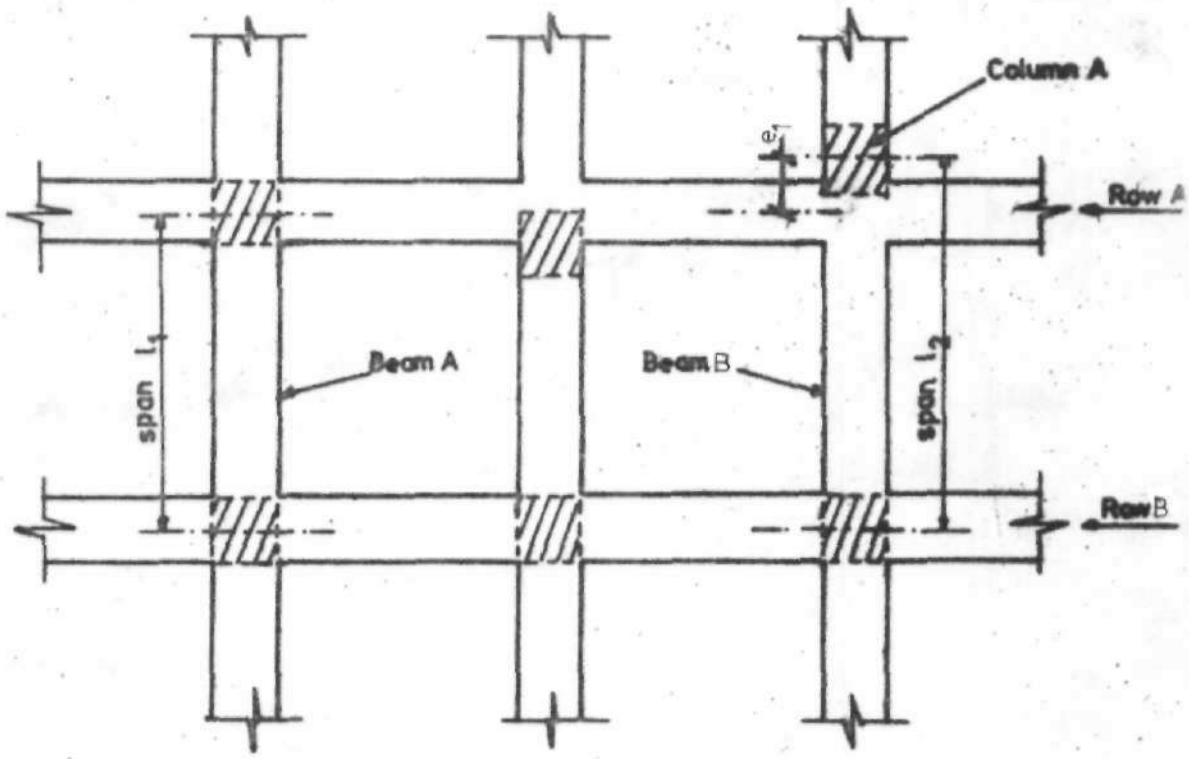


FIG 1.2 HORIZONTAL ALIGNMENT ERROR (PLAN VIEW)

The CEB-FIP "Model code for concrete structures" /1.1/, hereinafter referred to as the CEB code specifies that the minimum eccentricity due to construction tolerances shall not be less than h/30 (ie 0.03h), where h is the minimum depth of section in the plane of bending. For elements subjected to a theoretically axial compressive force, the CEB code makes the following additional specifications:

i. the above minimum eccentricity shall not be less than 20mm,

ii. as an alternative to introducing the minimum eccentricity, we may increase the load and material partial safety factors by 20% each, thus having the effect of decreasing the ultimate axial resistance force.

The British Standard CP110 "Code of Practice for the Structural use of Concrete" /1.2, 1.3/, herein-after referred to as the CP code, specifies h/20 (ie 0.05h) as the minimum eccentricity, h being as defined above. To allow for this nominal eccentricity of 0.05h in axially loaded sections, the CP code increases the partial safety factors by about 10% and builds this into a design formula which it then specifies; that is, the ultimate axial load is reduced from

$$N = 0.45 f_{cu} A_c + 0.72 f_y A_{sc} \qquad (1.1)$$

$$\text{to} \quad N = 0.4 f_{cu} A_c + 0.67 f_y A_{sc} \qquad (1.2)$$

The American Code ACI 318-77 "Building Code Requirements for Reinforced Concrete" /1.4, 1.5, 1.6/, hereinafter referred to as the ACI code, specifies the minimum eccentricity as (0.6 + 0.03h) inches, that is, (15.2 + 0.03h)mm. The 1971 version of the ACI code /1.7/ specified 0.10h as the minimum eccentricity, subject to an absolute minimum of 25mm. No special specification is made for axially loaded sections by the ACI code.

The requirements for minimum eccentricity by the three codes above are illustrated diagramatically in fig. 1.3.

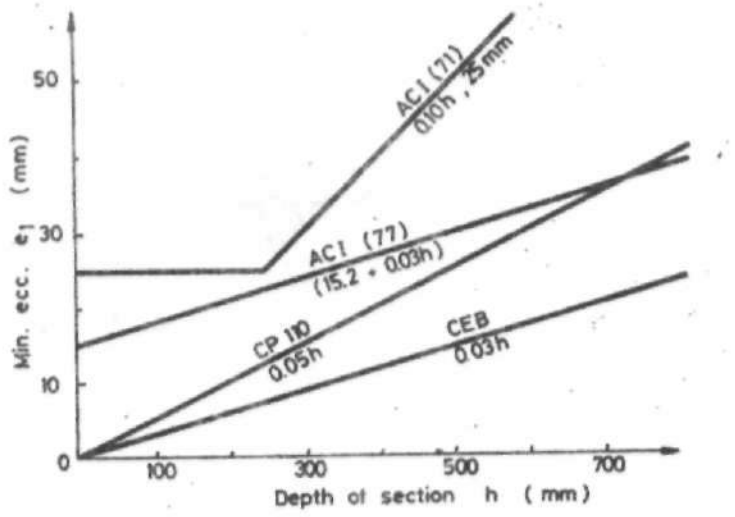


FIG. 1.3 REQUIREMENTS FOR MINIMUM ECCENTRICITY.

1.2 Observations on the Codes' specifications

1. The eccentricity error is made during the construction stage of the element, and once this error has been made, it remains there permanently, affecting all the bending moments that may be applied. Therefore any allowance introduced to correct this error should also be of a permanent nature. The specifications of the three codes above however are not permanent; instead they are applied only when the computed eccentricities are less than the minimum eccentricity, and completely ignored or "cut off" on elements with computed eccentricities greater than or equal to the minimum eccentricity. As an illustration, consider a section of depth 200mm. The CP code allows a

minimum eccentricity of 10mm for this section. If the computed eccentricity on this section is less than 10mm (say 7mm), then the minimum eccentricity of 10mm will be used in design; if however the computed eccentricity is greater than 10mm, say 15mm, then the latter value is used and the minimum eccentricity is ignored, not unmindful of the fact that an error has already been introduced in the section during construction. There is therefore a discontinuity between sections which have computed eccentricities less than the minimum eccentricity and those with computed eccentricities greater than or equal to the minimum eccentricity.

Viewed mathematically, for a computed eccentricity e_2 and for a minimum eccentricity e_1, the three codes recommend that although the total eccentricity may be as large as $(e_1 + e_2)$ due to construction errors, it may be specified as:

i. e_1, if $e_2 < e_1$ (1.3)

ii. e_2, if $e_2 \geq e_1$ (1.4)

The error, E, introduced by this specification if expressed as a ratio of the total eccentricity $(e_1 + e_2)$ is therefore

i. $$E = \frac{(e_1 + e_2) - e_1}{(e_1 + e_2)}$$ from eqn 1.3

$$= \frac{e_2}{e_1 + e_2}$$ when $e_2 \geq e_1$ (1.5)

or ii. $$E = \frac{(e_1 + e_2) - e_2}{(e_1 + e_2)}$$ from eqn 1.4

$$= \frac{e_1}{e_1 + e_2} \quad \text{when } e_2 \neq e_1 \qquad (1.6)$$

This error E increases from zero per cent when $e_2 = 0$ (eqn 1.5) to a maximum of 50% when $e_2 = e_1$ (eqn 1.6) then diminishes as e_2 increases further (eqn.1.6). At $e_2 = 9e_1$ for example, eqn 1.6 shows that the error is 10%, and when $e_2 = 19e_1$ the error is 5%. The variation of this error is shown graphically in fig 1.4. Ideally the total eccentricity $(e_1 + e_2)$ should be specified and used in design, as the error in this case will be zero.

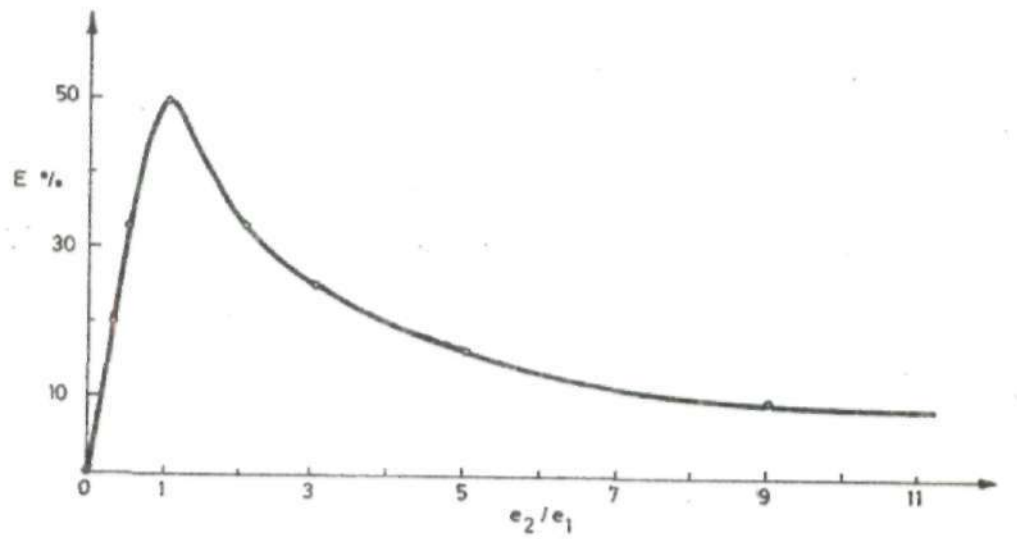


FIG. 1.4 SPECIFICATION ERROR FOR MINIMUM ECCENTRICITY.

ii. All the three codes specify the minimum eccentricity as a function of the section depth h. In the CEB and CP codes for example, the minimum eccentricities vary in arithmetic progression with the section depth, so that the minimum

eccentricity allowed in a section of depth 400mm say, is twice that allowed in a section of depth 200mm; (it will be slightly less than twice the value in the ACI code). From mere logic, it does not sound too convincing to argue that if an error e_1, say, is made while constructing a 200mm size column then an error four times this magnitude (ie 4 e_1) will be made when constructing an 800mm size col. Actual site results however need to be obtained and studied to see if the above statement is true or not.

iii. Although the basic aim of introducing minimum eccentricity is to correct for construction errors, fig 1.3 shows that the specifications for the minimum eccentricity by the various codes differ appreciably. This disparity in specified eccentricities is further evident from Table 1.1 where the minimum eccentricity on four arbitrarily chosen section sizes are tabulated.

Table 1.1 Comparison of minimum eccentricities on some section sizes.

Section size b x h,mm	Min Ecc. specified, mm		
	CEB (h/30)	ACI (15.2+0.03h)	CP110 (0.05h)
250x250	8.3	22.7	12.5
250x500	16.7	30.2	25.0
300x700	23.3	36.2	35.0
700x300	10.0	24.2	15.0

1.3 In the light of the observations made in section 1.2 we may be prompted to ask: do these codes' specifications compare favourably with actual errors on the construction sites? If no, can we propose an alternative specification? Will this proposed specification correct the disparities? To answer these and other such questions, we need to take measurements on actual construction sites; this is discussed in the next chapter.

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CHAPTER TWO

FIELD MEASUREMENTS

2.1 A field survey was carried out on three building sites under construction in order to obtain actual field data on construction errors. The first site surveyed (site 1) is the New Crop Protection building, while site 2 is the New Agronomy building. These two buildings are 150m apart and are located 200m behind the Department of Agriculture, Ahmadu Bello University main campus, Zaria. Site 3 is the New Administration Block Building for the Nigeria Leather Research Institute, opposite the Bassawa Army Barracks Zaria, along the old Zaria - Kano road. The firm of Messrs A.G. Ferrero and Sons was constructing the building at site1 while Nasara Enterprises Zaria was handling site 2, and Onimisi Enterprises Kaduna was in charge of site 3.

Eighteen columns were surveyed at site 1, thirty-three at site 2 and thirty-seven at site 3, making a total of eighty-eight columns surveyed. The general layout of the columns at the three sites are shown in figs 2.1, 2.2 and 2.3.

There modes of construction errors were investigated:

i. Horizontal alignment: The error in constructing the ground floor columns so that they are properly aligned

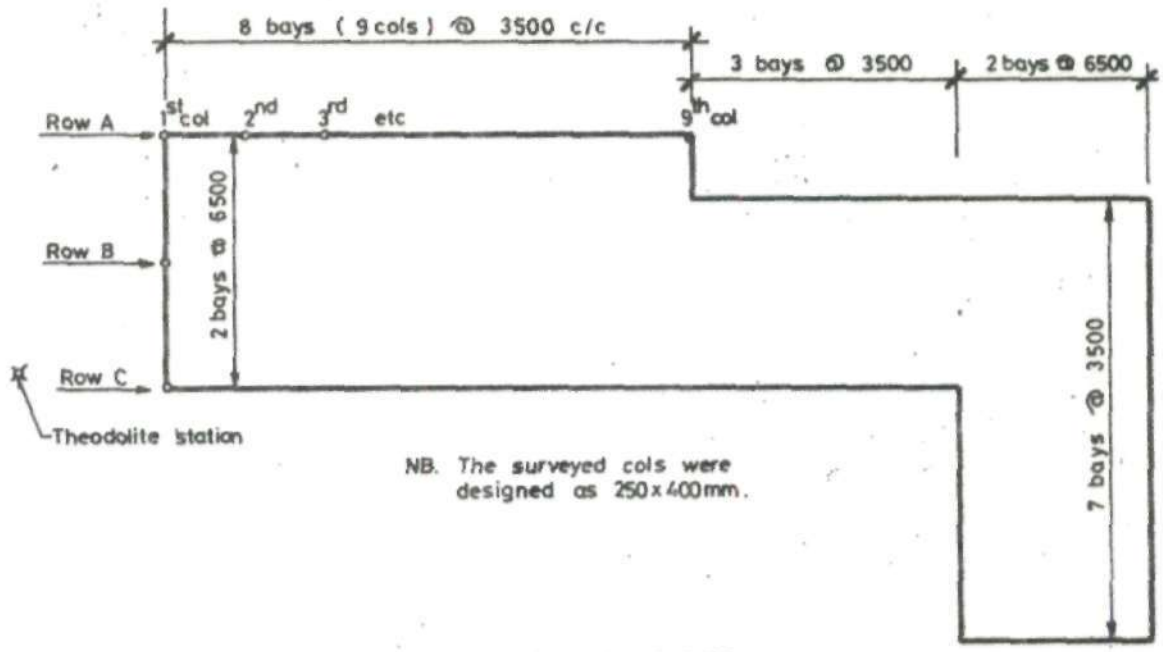


FIG. 2.1 LAYOUT OF THE COLUMNS AT SITE 1.

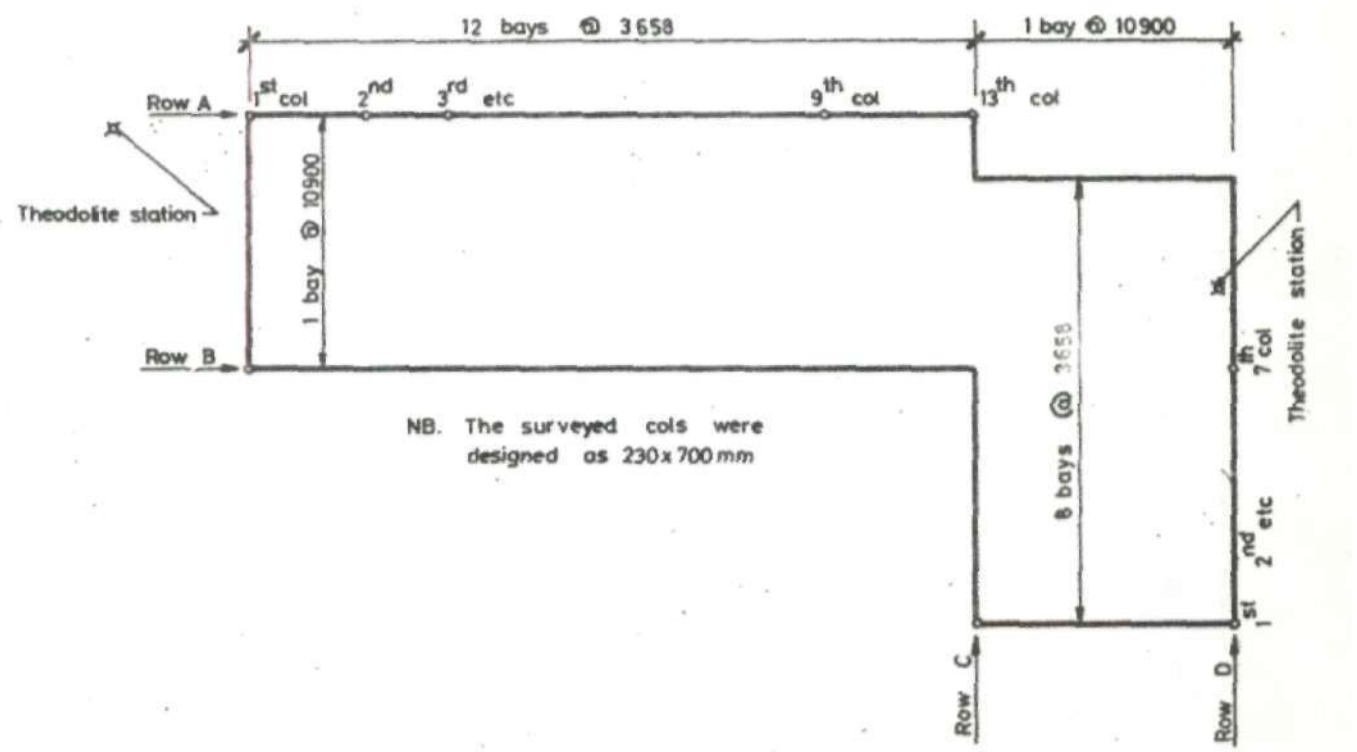


FIG. 2.2 LAYOUT OF THE COLUMNS AT SITE 2.

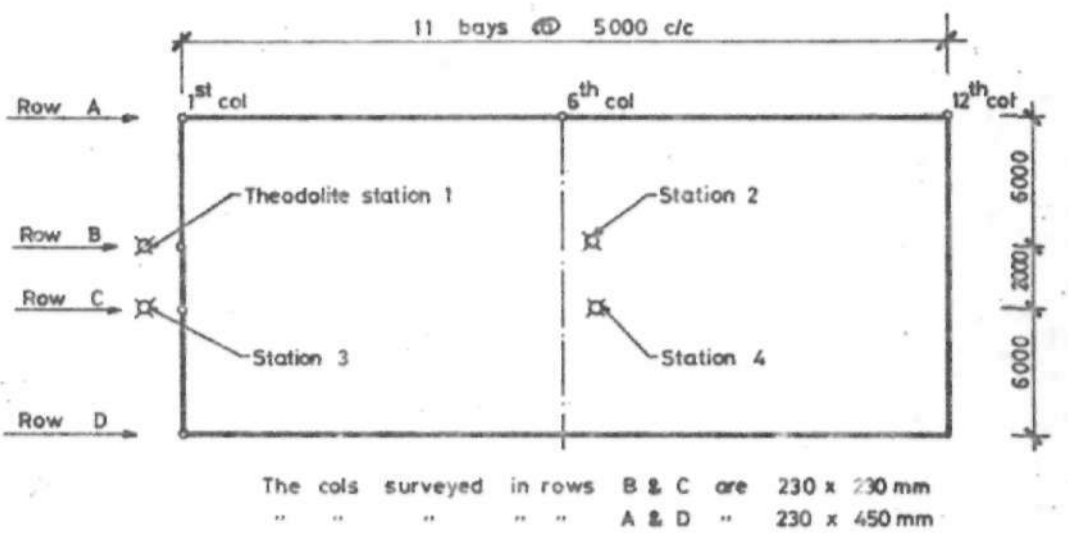


FIG. 2.3 LAYOUT OF COLUMNS AT SITE 3.

along a straight line, (see fig 1.1a).

ii. Floor to floor (transfer) alignment: the lack of coincidence between the centre-lines of the columns on two successive floors, the columns being on top of each other but separated by a floor slab, (see fig 1.1b).

iii. Vertical alignment: the error in constructing the columns vertically from the ground floor to the top floor; that is, the inclination. (see fig 1.1c).

The error data so collected were statistically analysed to obtain the characteristic mean eccentricity e, which is the eccentricity limit above which not more than 5% of the site results may be expected to fall; the probability of 5% defectives being the generally adopted value in limit state design, /1.1, 1.2, 1.3/.

$$\text{ie} \quad e = \bar{e} + z\sigma \qquad (2.1)$$

where e = characteristic mean eccentricity,

$\bar{e}$ = sample mean eccentricity,

z = probability factor (= 1.64 for 5% defectives),

σ = standard deviation.

The standard deviation gives a measure of variability of a given statistical observation, and is defined as the root

mean square of the deviation of the observations from their mean. For a sample x_1, x_2, x_3, ..., x_n of size n therefore,

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{n}} \qquad (2.2)$$

where $\bar{x} = \sum_{i=1}^{n} x_i/n$ (2.3)

= sample mean.

When estimating the standard deviation of the entire observations (population) from a sample of it, an error is introduced by assuming the sample mean to be the population mean, the latter being unknown in this case. It can be proved /2.1/ that the error so introduced may be corrected by replacing n in the denominator of eqn 2.2 by (n-1). For practical purposes, it seems debatable to introduce this refinement into data such as those resulting from tests on concrete or similar variable materials, because the inherent variation is quite high and the inference underlying such a correction presupposes a degree of accuracy and reliability not generally in keeping with such test results, /2.2/; moreover, for values of n exceeding about 30, it is usually of little practical significance whether n or (n-1) is used, /2.1/.

If $\bar{x}$ is a decimal number that has been rounded off, we accumulate a large error by using the standard deviation formula as given in eqn 2.2, /2.3/; to avoid this, eqn 2.2 may be expressed in the following form, (and this is the form that will be used in this thesis):

from eqn 2.2, $\sigma^2 = \Sigma(x - \bar{x})^2/n$

$$= \Sigma(x^2 - 2x\bar{x} + \bar{x}^2)/n$$

$$= (\Sigma x^2 - 2\bar{x}\Sigma x + n\bar{x}^2)/n$$

replacing $\bar{x}$ by $\Sigma x/n$ from eqn 2.3,

$$\sigma^2 = \frac{\left(\Sigma x^2 - \frac{(\Sigma x)^2}{n}\right)}{n}$$

$$\text{ie. } \sigma = \sqrt{\frac{\Sigma x^2 - \frac{(\Sigma x)^2}{n}}{n}} \qquad (2.4)$$

The probability factor Z is defined as the deviation of the variable x from the mean, expressed in multiples of the standard deviation.

$$\text{ie } Z = (x - \bar{x})/\sigma \qquad (2.5)$$

2.2 Ground floor horizontal alignment

A Wild Transit Theodolite was stationed at about 2.5m from the end of each row of columns to be surveyed (see fig 2.1)

and at such a position that offsets could be read on a scale-rule placed perpendicularly on the edge of any of the columns in the row. The theodolite was then levelled /2.4/ and offsets read at four levels on each column, between the ground floor level (0.00m) and 2.10m above floor level, the levels being 0.70m apart. At each of these levels, one reading each was taken on the two edges of the column face considered; these readings were later averaged to give the offset at the centre of the column face.

The construction error in the horizontal alignment was computed relative, arbitrarily, to the line joining the nearest and the farthest columns to the theodolite station (ie line C9 - C1 in fig 2.4).

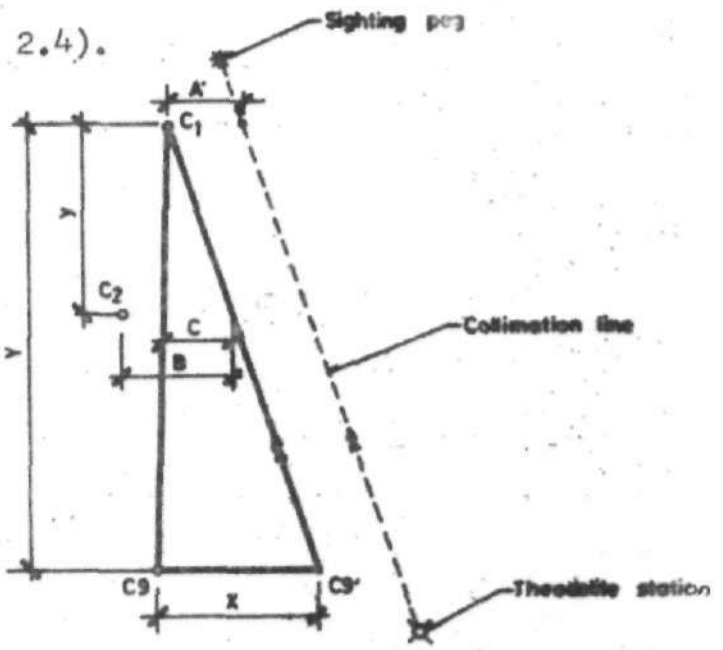


FIG. 2.4

To do this, the reduced offsets B at each column level relative to the floor level offset of the column farthest from the theodolite station (ie column C1 in fig 2.4) was first computed, this being equivalent to shifting the collimation (sight) line to C1C9′. If there were no construction errors all the columns would lie on line C1C9 and their correct offsets C, to line C1C9′ can be computed by proportions using the principles of similar triangles, with triangle C1C9C9' as a base. The difference between C and B gives the construction error in the horizontal alignment.

The first floor slabs were yet to be constructed when the horizontal alignment readings were taken.

Table 2.1 Offsets along row C (site 1)

Col No.	Ruler Position	Offsets at column edge (mm) Ruler ht above floor level (m) 0.00	0.70	1.40	2.10	Column Spacing (m)	Cummulative col. spacing $y=\Sigma y_i$ (m)
	Edge 1	83.0	80.0	81.0	76.0	0.000	
C9	Edge 2	80.0	76.0	74.0	71.0	0.000	
	Mean	81.5	78.0	77.5	73.5	0.000	0.000
	Edge 1	88.0	87.0	88.0	84.0	3.275	
C8	Edge 2	103.0	103.0	100.0	97.00	3.290	
	Mean	95.5	95.0	94.0	90.5	3.283	3.283
	Edge 1	112.0	110.0	116.0	115.0	3.317	
C7	Edge 2	115.0	116.0	115.0	111.0	3.293	
	Mean	113.5	113.0	115.5	113.0	3.305	6.588
	Edge 1	135.0	128.0	126.0	125.0	3.270	
C6	Edge 2	140.0	136.0	130.0	127.0	3.280	
	Mean	137.5	132.0	128.0	126.0	3.275	9.863
	Edge 1	154.0	156.0	162.0	160.0	3.302	
C5	Edge 2	151.0	154.0	156.0	156.0	3.305	
	Mean	152.5	155.0	159.0	158.0	3.304	13.167
	Edge 1	150.0	158.0	161.0	161.0	3.280	
C4	Edge 2	158.0	162.0	166.0	166.0	3.290	
	Mean	154.0	160.0	163.5	163.5	3.285	16.452

	Edge 1	152.0	147.0	149.0	151.0	3.315	
C3	Edge 2	158.0	145.0	145.0	151.0	3.310	
	Mean	155.0	146.0	147.0	151.0	3.313	19.765
	Edge 1	139.0	136.0	137.0	138.0	3.295	
C2	Edge 2	149.0	149.0	144.0	145.0	3.295	
	Mean	144.0	142.5	140.5	141.5	3.295	23.060
	Edge 1	161.0	155.0	156.0	153.0	3.285	(total col
c1	Edge 2	161.0	154.0	153.0	154.0	3.290	spacing Y=)
	Mean	161.0	154.5	154.5	153.5	3.288	26.348

Table 2.2 Construction errors along row C (site 1)

Column No.	Ht above floor level (m)	Offsets (mm)			Error e_1 (=C-B), mm
		Measured A	Reduced B (=A-A')	Reqd C (=$\frac{Y}{Y}$.X)	
C9	0.00	81.5(=A')	0.0	0.0	0.0
	0.70	78.0	-3.5	0.0	3.5
	1.40	77.5	-4.0	0.0	4.0
	2.10	73.5	-8.0	0.0	8.0
C8	0.00	95.5	14.0	9.9	-4.1
	0.70	95.0	13.5	9.9	-3.6
	1.40	94.0	12.5	9.9	-2.6
	2.10	90.5	9.0	9.9	0.9
C7	0.00	113.5	32.0	19.9	-12.1
	0.70	113.0	31.5	19.9	-11.6
	1.40	115.5	34.0	19.9	-14.1
	2.10	133.0	31.5	19.9	-11.6
C6	0.00	137.5	56.0	29.8	-26.2
	0.70	132.0	50.5	29.8	-20.7
	1.40	128.0	46.5	29.8	-16.7
	2.10	126.0	44.5	29.8	-14.7
C5	0.00	152.5	71.0	39.7	-31.3
	0.70	155.0	73.5	39.7	-33.8
	1.40	159.0	77.5	39.7	-37.8
	2.10	158.0	76.5	39.7	-36.8
C4	0.00	154.0	72.5	49.6	-22.9
	0.70	160.0	78.5	49.6	-28.9
	1.40	163.5	82.0	49.6	-32.4
	2.10	163.5	82.0	49.6	-32.4

C3	0.00	155.0	73.5	59.6	-13.9
	0.70	146.0	64.5	59.6	-4.9
	1.40	147.0	65.5	59.6	-5.9
	2.10	151.0	69.5	59.6	-9.9
C2	0.00	144.0	62.5	69.6	7.1
	0.70	142.5	61.0	69.6	8.6
	1.40	140.5	59.0	69.6	10.6
	2.10	141.5	60.0	69.6	9.6
C1	0.00	161.0	79.5(=X)	79.5	0.0
	0.70	154.5	73.0	79.5	6.5
	1.40	154.5	73.0	79.5	6.5
	2.10	153.5	72.0	79.5	7.5

Table 2.3 Eccentricity errors along row C, site 1 (absolute values)

e_1(mm)	e_1^2 (mm^2)	e_1(mm)	e_1^2 (mm^2)
0.0	0.00	36.8	1354.24
3.5	12.25	22.9	524.41
4.0	16.00	28.9	835.21
8.0	64.00	32.4	1049.76
4.1	16.81	32.4	1049.76
3.6	12.96	13.9	193.21
2.6	6.76	4.9	24.01
0.9	0.81	5.9	34.81
12.1	146.41	9.9	98.01
11.6	134.56	7.1	50.41
14.1	198.81	8.6	73.96
11.6	134.56	10.6	112.36
26.2	686.44	9.6	92.16
20.7	428.49	0.0	0.00
16.7	278.89	6.5	42.25
14.7	216.09	6.5	42.25
31.3	979.69	7.5	56.25
33.8	1142.44	Σ=501.7	Σ=11537.87
37.8	1428.84		

Sample mean eccentricity $\bar{e} = \frac{\Sigma e_1}{n}$ from eqn 2.3

$$= \frac{501.7}{36}$$

$$= 13.936\text{mm}$$

Standard deviation $\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$ from eqn 2.4

$$= \sqrt{\frac{11537.87 - \frac{(501.7)^2}{36}}{36}}$$

$$= 11.237\text{mm}$$

Characteristic mean eccentricity $e = \bar{e} + (1.64\sigma)$ from eqn 2.1

$$= 13.936 + (1.64 \times 11.237)$$

$$= 32.364\text{mm}$$

Table 2.4 Offsets along row B (site 1)

Col No	Ruler Position	Offsets at column edge (mm) Ruler ht above floor level (m) 0.00	0.70	1.40	2.10	Column spacing (m)	Cummulative col spacing $y=\Sigma y_i$ (m)
	Edge 1	108.0	110.0	110.0	110.0	0.000	
B9	Edge 2	112.0	116.0	118.0	120.0	0.000	
	Mean	110.0	113.0	114.0	115.0	0.000	0.000
	Edge 1	97.0	89.0	90.0	97.0	3.300	
B8	Edge 2	103.0	101.0	103.0	110.0	3.300	
	Mean	100.0	95.0	96.5	103.5	3.300	3.300
	Edge 1	87.0	85.0	83.0	83.0	3.280	
B7	Edge 2	95.0	93.0	90.0	84.0	3.290	
	Mean	91.0	89.0	86.5	83.5	3.285	6.585
	Edge 1	113.0	116.0	125.0	132.0	3.273	
B6	Edge 2	112.0	121.0	125.0	131.0	3.295	
	Mean	112.5	118.5	125.0	131.5	3.284	9.869
	Edge 1	106.0	105.0	107.0	105.0	3.320	
B5	Edge 2	104.0	101.0	98.0	93.0	3.290	
	Mean	105.0	103.0	102.0	99.0	3.305	13.174
	Edge 1	86.0	83.0	85.0	81.0	3.290	
B4	Edge 2	95.0	94.0	95.0	90.0	3.295	
	Mean	90.5	88.5	90.0	85.5	3.293	16.467

	Edge 1	82.0	78.0	74.0	76.0	3.290	
B3	Edge 2	95.0	93.0	90.0	89.0	3.277	
	Mean	88.5	85.5	82.0	82.5	3.284	19.751
	Edge 1	111.0	112.0	118.0	118.0	3.310	
B2	Edge 2	111.0	112.0	114.0	110.0	3.318	
	Mean	111.0	112.0	116.0	114.0	3.314	23.065
B1	Edge 1	97.0	96.0	92.0	92.0	3.280	(total col
	Edge 2	94.0	89.0	88.0	89.0	3.270	spacing Y=)
	Mean	95.5	92.5	90.0	90.5	3.275	26.340

Table 2.5 Construction errors along row B (site 1)

Column No.	Ht above floor level (m)	Offsets (mm)			Construction error e_1 (=C-B)mm
		Measured A	Reduced B (=A-A')	Reqd C ($=\frac{y}{Y}.X$)	
B9	0.00	110.0(=A')	0.0	0.0	0.0
	0.70	113.0	3.0	0.0	-3.0
	1.40	114.0	4.0	0.0	-4.0
	2.10	115.0	5.0	0.0	-5.0
B8	0.00	100.0	-10.0	-1.8	8.2
	0.70	95.0	-15.0	-1.8	13.2
	1.40	96.5	-13.5	-1.8	11.7
	2.10	103.5	-6.5	-1.8	4.7
B7	0.00	91.0	-19.0	-3.6	15.4
	0.70	89.0	-21.0	-3.6	17.4
	1.40	86.5	-23.5	-3.6	19.9
	2.10	83.5	-26.5	-3.6	22.9
B6	0.00	112.5	2.5	-5.4	-7.9
	0.70	118.5	8.5	-5.4	-13.9
	1.40	125.0	15.0	-5.4	-20.4
	2.10	131.5	21.5	-5.4	-26.9
B5	0.00	105.0	-5.0	-7.3	-2.3
	0.70	103.0	-7.0	-7.3	-0.3
	1.40	102.5	-7.5	-7.3	0.2
	2.10	99.0	-11.0	-7.3	3.7
B4	0.00	90.5	-19.5	-9.1	10.4
	0.70	88.5	-21.5	-9.1	12.4
	1.40	90.0	-20.0	-9.1	10.9
	2.10	85.5	-24.5	-9.1	15.4

B3	0.00	88.5	-21.5	-10.9	10.6
	0.70	85.5	-24.5	-10.9	13.6
	1.40	82.0	-28.0	-10.9	17.1
	2.10	82.5	-27.5	-10.9	16.6
B2	0.00	111.0	1.0	-12.7	-13.7
	0.70	112.0	2.0	-12.7	-14.7
	1.40	116.0	6.0	-12.7	-18.7
	2.10	114.0	4.0	-12.7	-16.7
B1	0.00	95.5	-14.5(=X)	-14.5	0.0
	0.70	92.5	-17.5	-14.5	3.0
	1.40	90.0	-20.0	-14.5	5.5
	2.10	90.5	-19.5	-14.5	5.0

Table 2.6 Eccentricity errors along row B
Site 1 (absolute values)

e_1(mm)	e_1^2(mm^2)	e_1 (mm)	e_1^2 (mm^2)
0.0	0.00	3.7	13.69
3.0	9.00	10.4	108.16
4.0	16.00	12.4	153.76
5.0	25.00	10.9	118.81
8.2	67.24	15.4	237.16
13.2	174.24	10.6	112.36
11.7	136.89	13.6	184.96
4.7	22.09	17.1	292.41
15.4	237.16	16.6	275.56
17.4	302.76	13.7	187.69
19.9	396.01	14.7	216.09
22.9	524.41	18.7	349.69
7.9	62.41	16.7	278.89
13.9	193.21	0.0	0.00
20.4	416.16	3.0	9.00
26.9	723.61	5.5	30.25
2.3	5.29	5.0	25.00
0.3	0.09	Σ=385.3	Σ=5905.09
0.2	0.04		

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{385.3}{36}$$

$$= 10.703\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{5905.09 - \frac{(385.3)^2}{36}}{36}}$$

$$= 7.034\text{mm}$$

$$e = \bar{e} + (1.64\sigma)$$

$$= 10.703 + (1.64 \times 7.034)$$

$$= 22.239\text{mm}$$

Table 2.7 Eccentricity errors at site 1

Site/row	Σe_1 (mm)	Σe_1^2 (mm^2)	No. of readings n
1C	501.7	11537.87	36
1B	385.3	5905 .09	36
1	887.0	17442.96	72

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{887.0}{72}$$

$$= 12.319\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{17442.96 - \frac{(887)^2}{72}}{72}}$$

$$= 9.513\text{mm}$$

$$e = \bar{e} + (1.64\sigma)$$

$$= 12.319 + (1.64 \times 9.513)$$

$$= 27.920\text{mm}.$$

Table 2.8 Offsets along row A (site 2)

Col No.	Ruler Position	Offsets at column edge (mm) Ruler ht above floor level(mm) 0.00	0.70	1.40	2.10	Column spacing (m)	Cummulative Col. Spacing $y=\Sigma y_i$ (m)
	Edge 1	115.0	112.0	109.0	105.0	0.000	
A9	Edge 2	125.0	124.0	125.0	119.0	0.000	
	Mean	120.0	118.0	117.0	112.0	0.000	0.000
	Edge 1	129.0	128.0	126.0	118.0	3.484	
A8	Edge 2	121.0	125.0	128.0	119.0	3.456	
	Mean	125.0	126.5	127.0	118.5	3.470	3.470
	Edge 1	88.0	86.0	82.0	77.0	3.433	
A7	Edge 2	83.0	88.0	86.0	77.0	3.432	
	Mean	85.5	87.0	84.0	77.0	3.433	6.903
	Edge 1	95.0	90.0	85.0	72.0	3.455	
A6	Edge 2	90.0	87.0	82.5	74.0	3.462	
	Mean	92.5	88.5	83.8	73.0	3.459	10.362
	Edge 1	60.0	55.0	47.0	46.0	3.400	
A5	Edge 2	58.0	48.0	48.0	45.0	3.445	
	Mean	59.0	51.5	47.5	45.5	3.423	13.785
	Edge 1	50.0	48.0	49.0	54.0	3.455	
A4	Edge 2	49.0	47.0	46.0	50.0	3.456	
	Mean	49.5	47.5	47.5	52.0	3.456	17.241

	Edge 1	35.5	37.5	40.0	40.0	3.440	
A3	Edge 2	30.0	31.0	30.0	30.0	3.436	
	Mean	32.8	34.3	35.0	35.0	3.438	20.679
	Edge 1	22.5	17.0	17.0	12.0	3.465	
A2	Edge 2	28.5	24.5	23.0	21.5	3.482	
	Mean	25.5	20.8	20.0	16.8	3.474	24.153
	Edge 1	13.0	18.0	20.0	18.5	3.445	(Total col
A1	Edge 2	9.0	13.0	12.0	10.5	3.450	spacing Y =)
	Mean	11.0	15.5	16.0	14.5	3.448	27.601

Table 2.9 Construction errors along row A (site 2)

Column No.	Ht above Floor level (m)	Offsets (mm)			Constructn Error e_1 (=C-B), mm
		Measured A	ReducedB (=A-A')	Reqd C (=$\frac{Y}{X}$.X)	
A9	0.00	120.0(=A')	0.0	0.0	0.0
	0.70	118.0	-2.0	0.0	2.0
	1.40	117.0	-3.0	0.0	3.0
	2.10	112.0	-8.0	0.0	8.0
A8	0.00	125.0	5.0	-13.7	18.7
	0.70	126.5	6.5	-13.7	-20.2
	1.40	127.0	7.0	-13.7	-20.7
	2.10	118.5	-1.5	-13.7	-12.2
A7	0.00	85.5	-34.5	-27.3	7.2
	0.70	87.0	-33.0	-27.3	5.7
	1.40	84.0	-36.0	-27.3	8.7
	2.10	77.0	-43.0	-27.3	15.7
A6	0.00	92.5	-27.5	-40.9	-13.4
	0.70	88.5	-31.5	-40.9	-9.4
	1.40	83.8	-36.2	-40.9	-4.7
	2.10	73.0	-47.0	-40.9	6.1
A5	0.00	59.0	-61.0	-54.4	6.6
	0.70	51.5	-68.5	-54.4	14.1
	1.40	47.5	-72.5	-54.4	18.1
	2.10	45.5	-74.5	-54.4	20.1
A4	0.00	49.5	-70.5	-68.1	2.4
	0.70	47.5	-72.5	-68.1	4.4
	1.40	47.5	-72.5	-68.1	4.4
	2.10	52.0	-68.0	-68.1	-0.1

A3	0.00	32.8	-87.2	-81.7	5.5
	0.70	34.3	-85.7	-81.7	4.0
	1.40	35.0	-85.0	-81.7	3.3
	2.10	35.0	-85.0	-81.7	3.3
A2	0.00	25.5	-94.5	-95.4	-0.9
	0.70	20.8	-99.2	-95.4	3.8
	1.40	20.0	-100.0	-95.4	4.6
	2.10	16.8	-103.2	-95.4	7.8
A1	0.00	11.0	-109.0(=X)	-109.0	0.0
	0.70	15.5	-104.5	-109.0	-4.5
	1.40	16.0	-104.0	-109.0	-5.0
	2.10	14.5	-105.5	-109.0	-3.5

Table 2.10 Eccentricity errors along row A, site 2 (absolute values)

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)
0.0	0.00	20.1	404.01
2.0	4.00	2.4	5.76
3.0	9.00	4.4	19.36
8.0	64.00	4.4	19.36
18.7	349.69	0.1	0.01
20.2	408.04	5.5	30.25
20.7	428.49	4.0	16.00
12.2	148.84	3.3	10.89
7 .2	51.84	3.3	10.89
5.7	32.49	0.9	0.81
8.7	75.69	3.8	14.44
15.7	246.49	4.6	21.16
13.4	179.56	7.8	60.84
9.4	88.36	0.0	0.00
4.7	22.09	4.5	20.25
6.1	37.21	5.0	25.00
6.6	43.56	3.5	12.25
14.1	198.81	Σ =272.1	Σ=3387.05
18.1	327.61		

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{272.1}{36}$$

$$= 7.558\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{3387.05 - \frac{(272.1)^2}{36}}{36}}$$

$$= 6.079\text{mm}$$

$$e = \bar{e} + (1.64\sigma)$$

$$= 7.558 + (1.64 \times 6.079)$$

$$= 17.528\text{mm}$$

Table 2.11 Offsets along row B (site 2)

Col No.	Ruler Position	Offsets at column edge (mm) Ruler ht above floor level(m) 0.00	0.70	1.40	2.10	Column spacing (m)	Cummulative col. spacing $y=\sum y_i$ (m)
	Edge 1	164.0	167.0	170.0	179.0	0.000	
B9	Edge 2	165.0	168.0	167.0	164.0	0.000	
	Mean	164.5	167.5	168.5	171.5	0.000	0.000
	Edge 1	163.0	168.0	173.0	185.0	3.465	
B8	Edge 2	166.0	170.0	173.0	183.0	3.460	
	Mean	164.5	169.0	173.0	184.0	3.463	3.463
	Edge 1	172.0	177.0	181.0	179.0	3.440	
B7	Edge 2	163.0	168.0	172.0	168.0	3.435	
	Mean	167.5	172.5	176.5	173.5	3.348	6.811
	Edge 1	162.0	163.0	169.0	172.0	3.440	
B6	Edge 2	157.0	158.0	164.0	163.0	3.455	
	Mean	159.5	160.5	166.5	167.5	3.448	10.259
	Edge 1	169.0	170.0	170.0	172.0	3.440	
B5	Edge 2	162.0	170.0	167.0	164.0	3.446	
	Mean	165.5	170.0	168.5	168.0	3.443	13.702
	Edge 1	170.0	175.0	177.0	182.0	3.465	
B4	Edge 2	174.0	177.0	178.0	184.0	3.485	
	Mean	172.0	176.0	177.5	183.0	3.475	17.177

	Edge 1	167.0	170.0	171.0	170.0	3.430	
B3	Edge 2	162.0	165.0	162.0	158.0	3.428	
	Mean	164.5	167.5	166.5	164.0	3.429	20.606
	Edge 1	154.0	157.0	155.0	152.0	3.450	
B2	Edge 2	150.0	154.0	156.0	156.0	3.460	
	Mean	152.0	155.5	155.5	154.0	3.455	24.061
	Edge 1	159.0	163.0	164.0	168.0	3.446	(total col
B1	Edge 2	157.0	159.0	162.0	161.0	3.459	spacing Y=)
	Mean	158.0	161.0	163.0	164.0	3.453	27.514

Table 2.12 Construction errors along row B (site 2)

Column No.	Ht above floor level (m)	Offsets (mm)			Constructn error e_1 (=C-B), mm
		Measured A	ReducedB (=A-A')	Reqd C (=$\frac{y}{Y}$.X)	
B9	0.00	164.5(=A')	0.0	0.0	0.0
	0.70	167.5	3.0	0.0	-3.0
	1.40	168.5	4.0	0.0	-4.0
	2.10	171.5	7.0	0.0	-7.0
B8	0.00	164.5	0.0	-0.8	-0.8
	0.70	169.0	4.5	-0.8	-5.3
	1.40	173.0	8.5	-0.8	-9.3
	2.10	184.0	19.5	-0.8	-20.3
B7	0.00	167.5	3.0	-1.6	-4.6
	0.70	172.5	8.0	-1.6	-9.6
	1.40	176.5	12.0	-1.6	-13.6
	2.10	173.5	9.0	-1.6	-10.6
B6	0.00	159.5	-5.0	-2.4	2.6
	0.70	160.5	-4.0	-2.4	1.6
	1.40	166.5	2.0	-2.4	-4.4
	2.10	167.5	3.0	-2.4	-5.4
B5	0.00	165.5	1.0	-3.2	-4.2
	0.70	170.0	5.5	-3.2	-8.7
	1.40	168.5	4.0	-3.2	-7.2
	2.10	168.0	3.5	-3.2	-6.7
B4	0.00	172.0	7.5	-4.1	-11.6
	0.70	176.0	11.5	-4.1	-15.6
	1.40	177.5	13.0	-4.1	-17.1
	2.10	183.0	18.5	-4.1	-22.6

B3	0.00	164.5	0.0	-4.9	-4.9
	0.70	167.5	3.0	-4.9	-7.9
	1.40	166.5	2.0	-4.9	-6.9
	2.10	164.0	-0.5	-4.9	-4.4
B2	0.00	152.0	-12.5	-5.7	6.8
	0.70	155.5	-9.0	-5.7	3.3
	1.40	155.5	-9.0	-5.7	3.3
	2.10	154.0	-10.5	-5.7	4.8
B1	0.00	158.0	-6.5(=X)	-6.5	0.0
	0.70	161.0	-3.5	-6.5	-3.0
	1.40	163.0	-1.5	-6.5	-5.0
	2.10	164.5	0.0	-6.5	-6.5

Table 2.13 Eccentricity errors along row B site 2 (absolute values)

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)
0.0	0.00	6.7	44.89
3.0	9.00	11.6	134.56
4.0	16.00	15.6	243.36
7.0	49.00	17.1	292.41
0.8	0.64	22.6	510.76
5.3	28.09	4.9	24.01
9.3	86.49	7.9	62.41
20.3	412.09	6.9	47.61
4.6	21.16	4.4	19.36
9.6	92.16	6.8	46.24
13.6	184.96	3.3	10.89
10.6	112.36	3.3	10.89
2.6	6.76	4.8	23.04
1.6	2.56	0.0	0.00
4.4	19.36	3.0	9.00
5.4	29.16	5.0	25.00
4.2	17.64	6.5	42.25
8.7	75.69	Σ=252.6	Σ=2761.64
7.2	51.84		

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{252.6}{36}$$

$$= 7.017\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{2761.64 - \frac{(252.6)^2}{36}}{36}}$$

$$= 5.242\text{mm}$$

$$e = \bar{e} + (1.64\sigma)$$

$$= 7.017 + (1.64 \times 5.242)$$

$$= 15.614\text{mm}$$

Table 2.14 Offsets along row C (site 2)

Col No	Ruler Position	Offsets at column edge (mm) Ruler ht above floor level (m) 0.00	0.70	1.40	2.10	Column spacing (m)	Cummulatice col.spacing $y=\Sigma y_i$ (m)
	Edge 1	127.0	129.0	131.0	130.0	0.000	
C1	Edge 2	118.0	120.0	123.0	119.0	0.000	
	Mean	122.5	124.5	127.0	124.5	0.000	0.000
	Edge 1	123.0	130.0	136.0	132.0	3.430	
C2	Edge 2	126.0	130.0	125.0	120.0	3.405	
	Mean	124.5	130.0	130.5	126.0	3.418	3.418
	Edge 1	123.0	125.0	127.0	132.0	3.470	
C3	Edge 2	114.0	114.0	111.0	106.0	3.470	
	Mean	118.5	119.5	119.0	119.0	3.470	6.888
	Edge 1	103.0	102.0	109.0	109.0	3.450	
C4	Edge 2	96.0	97.0	102.0	103.0	3.440	
	Mean	99.5	99.5	105.5	106.0	3.445	10.333
	Edge 1	124.0	132.0	134.0	137.0	3.431	
C5	Edge 2	118.0	123.0	128.0	134.0	3.430	
	Mean	121.0	127.5	131.0	135.5	3.431	13.764
	Edge 1	104.0	103.5	98.0	97.0	3.420	
C6	Edge 2	102.0	109.0	110.0	106.0	3.410	
	Mean	103.0	106.3	104.0	101.5	3.415	17.179
	Edge 1	153.0	151.0	144.0	137.0	3.483	(total col
C7	Edge 2	144.0	141.0	132.0	126.0	3.480	spacingY=)
	Mean	148.5	146.0	138.0	131.5	3.482	20.661

NB. Column C7 is the nearest column to the theodolite station (see fig 2.2)

Table 2.15 Construction errors along row C (site 2)

Column No.	Ht above floor level(m)	Offsets (mm)			Constrn error e_1 (=C-B), mm
		Measured A	Reduced B (=A-A')	Reqd C (=$\frac{y}{Y}$.X)	
C1	0.00	122.5(=A')	0.0	0.0	0.0
	0.07	124.5	2.0	0.0	-2.0
	1.40	127.0	4.5	0.0	-4.5
	2.10	124.5	2.0	0.0	-2.0
C2	0.00	124.5	2.0	4.3	2.3
	0.07	130.0	7.5	4.3	-3.2
	1.40	130.5	8.0	4.3	-3.7
	2.10	126.0	3.5	4.3	0.8
C3	0.00	118.5	-4.0	8.7	12.7
	0.07	119.5	-3.0	8.7	11.7
	1.40	119.0	-3.5	8.7	12.2
	2.10	119.0	-3.5	8.7	12.2
C4	0.00	99.5	-23.0	13.0	36.0
	0.70	99.5	-23.0	13.0	36.0
	1.40	105.5	-17.0	13.0	30.0
	2.10	106.0	-16.5	13.0	29.5
C5	0.00	121.0	-1.5	17.3	18.8
	0.07	127.5	5.0	17.3	12.5
	1.40	131.0	8.5	17.3	8.8
	2.10	135.5	13.0	17.3	4.3
C6	0.00	103.0	-19.5	21.6	41.1
	0.07	106.3	-16.2	21.6	37.8
	1.40	104.0	-18.5	21.6	40.1
	2.10	101.5	-21.0	21.6	42.6

C7	0.00	148.5	26.0(=X)	26.0	0.0
	0.07	146.0	23.5	26.0	2.5
	1.40	138.0	15.5	26.0	10.5
	2.10	131.5	9.0	26.0	17.0

Table 2.16 Eccentricity errors along row C site 2 (absolute values)

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)
0.0	0.00	29.5	870.25
2.0	4.00	18.8	353.44
4.5	20.25	12.5	156.25
2.0	4.00	8.8	77.44
2.3	5.29	4.3	18.49
3.2	10.24	41.1	1689.21
3.7	13.69	37.8	1428.84
0.8	0.64	40.1	1608.01
12.7	161.29	42.6	1814.76
11.7	136.89	0.0	0.00
12.2	148.84	2.5	6.25
12.2	148.84	10.5	110.25
36.0	1296.00	17.0	289.00
36.0	1296.00	Σ=434.8	Σ=12568.16
30.0	900.00		

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{434.8}{28}$$

$$= 15.529\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{12568.16 - \frac{(434.8)^2}{28}}{28}}$$

$$= 14.413\text{mm}$$

$$e = \bar{e} + (1.64\sigma)$$

$$= 15.529 + (1.64 \times 14.413)$$

$$= 39.166\text{mm}$$

Table 2.17 Offsets along row D (site 2)

Col No	Ruler Position	Offsets at column edge(mm) Ruler ht above floor level (m) 0.00	0.70	1.40	2.10	Column spacing (m)	Cummulative col.spacing $y=\Sigma y_i$ (m)
	Edge 1	109.0	108.0	92.0	93.0	0.000	
D1	Edge 2	104.0	101.0	95.0	101.0	0.000	
	Mean	106.5	104.5	93.5	97.0	0.000	0.000
	Edge 1	112.0	110.0	100.0	93.0	3.435	
D2	Edge 2	116.0	117.0	114.0	108.0	3.430	
	Mean	114.0	113.5	107.0	100.5	3.433	3.433
	Edge 1	125.0	123.0	112.0	116.0	3.420	
D3	Edge 2	120.0	117.0	113.0	120.0	3.415	
	Mean	122.5	120.0	111.5	118.0	3.418	6.851
	Edge 1	140.0	139.0	135.0	132.0	3.458	
D4	Edge 2	150.0	159.0	152.0	162.0	3.440	
	Mean	145.0	149.0	143.5	147.0	3.449	10.300
	Edge 1	148.0	147.0	144.0	141.0	3.460	
D5	Edge 2	155.0	162.0	159.0	152.0	3.445	
	Mean	151.5	154.5	151.5	146.5	3.453	13.753
	Edge 1	157.0	156.5	157.0	153.0	3.420	
D6	Edge 2	154.0	156.5	154.0	160.0	3.400	
	Mean	155.5	156.5	155.5	156.5	3.410	17.163
	Edge 1	161.0	160.0	162.0	149.0	3.495	(total col
D7	Edge 2	170.0	164.0	166.0	166.5	3.485	spacingY=)
	Mean	165.5	162.0	164.0	157.8	3.490	20.653

NB. Column D7 is the nearest column to the theodolite station (see fig 2.2)

Table 2.18 Construction errors along row D (site 2)

Col. No.	Ht above floor level(m)	Offsets (mm)			Constrn error e_1 (=C-B), mm
		Measured	ReducedB (=A-A')	Reqd C ($=\frac{Y}{Y}.X$)	
D1	0.00	106.5(=A')	0.0	0.0	0.0
	0.70	104.5	-2.0	0.0	2.0
	1.40	93.5	-13.0	0.0	13.0
	2.10	97.0	-9.5	0.0	9.5
D2	0.00	114.0	7.5	9.8	2.3
	0.70	113.5	7.0	9.8	2.8
	1.40	107.0	0.5	9.8	9.3
	2.10	100.5	-6.0	9.8	15.8
D3	0.00	122.5	16.0	19.6	3.6
	0.07	120.0	13.5	19.6	6.1
	1.40	111.5	5.0	19.6	14.6
	2.10	118.0	11.5	19.6	8.1
D4	0.00	145.0	38.5	29.4	-9.1
	0.70	149.0	42.5	29.4	-13.1
	1.40	143.5	37.0	29.4	-7.6
	2.10	147.0	40.5	29.4	-11.1
D5	0.00	151.5	45.0	39.3	-5.7
	0.07	154.5	48.0	39.3	-8.7
	1.40	151.5	45.0	39.3	-5.7
	2.10	146.5	40.0	39.3	-0.7
D6	0.00	155.5	49.0	49.0	0.0
	0.70	156.5	50.0	49.0	-1.0
	1.40	155.5	49.0	49.0	0.0
	2.10	156.5	50.0	49.0	-1.0

D7	0.00	165.5	59.0(=X)	59.0	0.0
	0.07	162.0	55.5	59.0	3.5
	1.40	164.0	57.5	59.0	1.5
	2.10	157.8	51.3	59.0	7.7

Table 2.19 Eccentricity errors along row D, site 2 (absolute values)

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)
0.0	0.00	11.1	123.21
2.0	4.00	5.7	32.49
13.0	169.00	8.7	75.69
9.5	90.25	5.7	32.49
2.3	5.29	0.7	0.49
2.8	7.84	0.0	0.00
9.3	86.49	1.0	1.00
15.8	249.64	0.0	0.00
3.6	12.96	1.0	1.00
6.1	37.21	0.0	0.00
14.6	213.16	3.5	12.25
8.1	65.61	1.5	2.25
9.1	82.81	7.7	59.29
13.1	171.61	Σ=163.5	Σ=1593.79
7.6	57.76		

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{163.5}{28}$$

$$= 5.839\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{1593.79 - \frac{(163.5)^2}{28}}{28}}$$

$$= 4.777\text{mm}$$

$$e = \bar{e} + (1.64\sigma)$$

$$= 5.839 + (1.64 \times 4.777)$$

$$= 13.673\text{mm}$$

Table 2.20 Eccentricity errors at site 2

Site/row	Σe_1 (mm)	Σe_1^2 (mm²)	No. of readings n
2A	272.1	3387.05	36
2B	252.6	2761.64	36
2C	434.8	12568.16	28
2D	163.5	1593.79	28
2	1123.0	20310.64	128

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{1123}{128}$$

$$= 8.773\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{20310.64 - \frac{(1123)^2}{128}}{128}}$$

$$= 9.039\text{mm}$$

$$e = \bar{e} + (1.64\sigma)$$

$$= 8.773 + (1.64 \times 9.039)$$

$$= 23.597\text{mm}$$

Table 2.21 Offsets along row B, from station 1 site 3 (see fig 2.3)

Col No.	Ruler Position	Offsets at column edge (mm) Ruler ht above floor level (m) 0.00	0.70	1.40	2.10	Column spacing (m)	Cummulative col.spacing $y=\Sigma y_i$ (m)
	Edge 1	105	93	80	75		
B6	Edge 2	100	87	80	80		
	Mean	102.5	90	80	77.5	0.000	0.000
	Edge 1	86	87	91	101		
B5	Edge 2	80	88	96	104		
	Mean	83	87.5	93.5	102.5	4.790	4.790
	Edge 1	88	92	95	96		
B4	Edge 2	85	86	90	96		
	Mean	86.5	89	92.5	96	4.800	9.590
	Edge 1	91	75	76	70		
B3	Edge 2	87	80	76	76		
	Mean	89	77.5	76	73	4.795	14.385
	Edge 1	94	88	82	77		
B2	Edge 2	97	87	82	76		
	Mean	95.5	87.5	82	76.5	4.798	19.183
	Edge 1	97	85	80	75		(total col
B1	Edge 2	95	87	82	79		spacing $Y_1=$)
	Mean	96	86	81	77	4.786	23.969

Table 2.22 Offsets along row B, from station 2 site 3

Col No.	Ruler Position	Offsets at column edge (mm) Ruler ht above floor level (m) 0.00	0.70	1.40	2.10	Column spacing (m)	Cummulative col.spacing $y=\Sigma y_i$ (m)
	Edge 1	83	68	57	50		
B12	Edge 2	85	73	58	46		
	Mean	84	70.5	57.5	48	0.000	0.000
	Edge 1	100	89	84	80		
B11	Edge 2	89	87	83	75		
	Mean	94.5	88	83.5	77.5	4.794	4.794
	Edge 1	114	123	109	113		
B10	Edge 2	114	110	112	106		
	Mean	114	116.5	110.5	109.5	4.788	9.582
	Edge 1	127	132	133	141		
B9	Edge 2	122	125	131	143		
	Mean	124.5	128.5	132	142	4.790	14.372
	Edge 1	129	123	127	127		
B8	Edge 2	127	136	132	140		
	Mean	128	129.5	129.5	133.5	4.760	19.132
	Edge 1	164	153	157	166		(total col
B7	Edge 2	160	150	152	158		spacing Y_2=)
	Mean	162	151.5	154.5	162	4.815	23.947

Table 2.23 Construction errors along row B (site 3)

Col. No.	Ht above floor level(m)	Offsets (mm)			Constructn error e_1 (=C-B)mm
		Measured A	ReducedB (=A-A')	Read C (=y.X) Y	
B6	0.00	102.5=A'_1	0.0	0.0	0.0
	0.70	90	-12.5	0.0	12.5
	1.40	80	-22.5	0.0	22.5
	2.10	77.5	-25.0	0.00	25.0
B5	0.00	83	-19.5	-1.3	18.2
	0.70	87.5	-15.0	-1.3	13.7
	1.40	93.5	- 9.0	-1.3	7.7
	2.10	102.5	0.0	-1.3	-1.30
B4	0.00	86.5	-16.0	-2.6	13.4
	0.70	89	-13.5	-2.6	10.9
	1.40	92.5	-10.0	-2.6	7.4
	2.10	96	-6.5	-2.6	3.9
B3	0.00	89	-13.5	-3.9	9.6
	0.70	77.5	-25	-3.9	21.1
	1.40	76	-26.5	-3.9	22.6
	2.10	73	-29.5	-3.9	25.6
B2	0.00	95.5	-7.0	-5.2	1.8
	0.70	87.5	-15.0	-5.2	9.8
	1.40	82	-20.5	-5.2	15.3
	2.10	76.5	-26.0	-5.2	20.8
B1	0.00	96	-6.5(=X_1)	-6.5	0.0
	0.70	86	-16.5	-6.5	10.0
	1.40	81	-21.5	-6.5	15.0
	2.10	77	-25.5	-6.5	19.0

B12	0.00	84=A'_2	0.0	0.0	0.0
	0.70	70.5	-13.5	0.0	13.5
	1.40	57.5	-26.5	0.0	26.5
	2.10	48	-36.0	0.0	36.0
B11	0.00	94.5	10.5	15.6	5.1
	0.70	88	4.0	15.6	11.6
	1.40	83.5	-0.5	15.6	16.1
	2.10	77.5	-6.5	15.6	22.1
B10	0.00	114	30.0	31.2	1.2
	0.70	116.5	32.5	31.2	-1.3
	1.40	110.5	26.5	31.2	4.7
	2.10	109.5	25.5	31.2	5.7
B9	0.00	124.5	40.5	46.8	6.3
	0.70	128.5	44.5	46.8	2.3
	1.40	132	48.0	46.8	-1.2
	2.10	142	58.0	46.8	-11.2
B8	0.00	128	44.0	62.3	18.3
	0.70	129.5	45.5	62.3	16.8
	1.40	129.5	45.5	62.3	16.8
	2.10	133.5	49.5	62.3	12.8
B7	0.00	162	78(=X_2)	78.0	0.0
	0.70	151.5	67.5	78.0	10.5
	1.40	154.5	70.5	78.0	7.5
	2.10	162	78.0	78.0	0.0

Table 2.24 Eccentricity errors along row B, site 3 (absolute values)

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)
0.0	0.00	10.0	100.00
12.5	156.25	15.0	225.00
22.5	506.25	19.0	361.00
25.0	625.00	0.0	0.00
18.2	331.24	13.5	182.25
13.7	187.69	26.5	702.25
7.7	59.29	36.0	1296.00
1.30	1.69	5.1	26.01
13.4	179.56	11.6	134.56
10.9	118.81	16.1	259.21
7.4	54.76	22.1	488.41
3.9	15.21	1.2	1.44
9.6	92.16	1.3	1.69
21.1	445.21	4.7	22.09
22.6	510.76	5.7	32.49
25.6	655.36	6.3	39.69
1.8	3.24	2.3	5.29
9.8	96.04	1.2	1.44
15.3	234.09	11.2	125.44
20.8	432.64	18.3	334.89
0.0	0.00	16.8	282.24

16.8	282.24
12.8	163.84
0.0	0.00
10.5	110.25
7.5	56.25
0.0	0.00
Σ=554.6	Σ=9939.22

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{554.6}{48} = 11.554\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{9939.22 - \frac{(554.6)^2}{48}}{48}} = 8.577\text{mm}$$

$$e = \bar{e} + (1.64\sigma)$$

$$= 11.554 + (1.64 \times 8.577)$$

$$= 25.620\text{mm}$$

Table 2.25 Offsets along row C, from station 3 site 3

Col No	Ruler Position	Offsets at col. edge (mm) Ruler ht above floor level (m) 0.00	0.70	1.40	2.10	Col. spacing (m)	Cummulative col.spacing $y=\Sigma y_i$ (m)
	Edge 1	172	183	190	210		
C6	Edge 2	172	176	183	200		
	Mean	172	179.5	186.5	205	0.000	0.000
	Edge 1	181	171	175	196		
C5	Edge 2	190	180	178	194		
	Mean	185.5	175.5	176.5	195	4.800	4.800
	Edge 1	221	213	210	204		
C4	Edge 2	221	214	209	213		
	Mean	221	213.5	209.5	208.5	4.795	9.595
	Edge 1	221	207	192	174		
C3	Edge 2	224	209	194	185		
	Mean	222.5	208	193	179.5	4.790	14.385
	Edge 1	236	231	227	220		
C2	Edge 2	231	234	233	229		
	Mean	233.5	232.5	230	224.5	4.795	19.180
	Edge 1	218	219	224	224		(total col
C1	Edge 2	220	222	225	229		spacing Y_1=)
	Mean	219	220.5	224.5	226.5	4.790	23.970

Table 2.26 Offsets along row C, from station 4 site 3

Col. No	Ruler Position	Offsets at col edge (mm) Ruler ht above floor level (m) 0.00	0.70	1.40	2.10	Column spacing (m)	Cummulative col.spacing $y=\Sigma y_i$ (m)
	Edge 1	75	96	100	106		
C12	Edge 2	79	89	95	102		
	Mean	77	92.5	97.5	104	0.000	0.000
	Edge 1	94	99	110	120		
C11	Edge 2	95	94	106	116		
	Mean	94.5	96.5	108	118	4.800	4.800
	Edge 1	98	91	86	83		
C10	Edge 2	103	96	94	85		
	Mean	100.5	93.5	90	84	4.784	9.584
	Edge 1	102	113	114	116		
C9	Edge 2	102	108	105	108		
	Mean	102	110.5	109.5	112	4.780	14.364
	Edge 1	121	120	120	118		
C8	Edge 2	112	112	107	107		
	Mean	116.5	116	113.5	112.5	4.779	19.143
	Edge 1	132	133	132	123		(total col
C7	Edge 2	119	127	125	137		spacing Y_2 =
	Mean	125.5	130	128.5	130	4.790	23.933

Table 2.27 Construction errors along row C site 3

Col No.	Ht above floor level (m)	Offsets (mm)			Constructn error e_1 (=C-B)mm
		Measured A	Reduced B (=A-A')	Reqd C (=$\frac{Y}{Y}$.X)	
C6	0.00	172=A'_1	0.0	0.0	0.0
	0.70	179.5	7.5	0.0	-7.5
	1.40	186.5	14.5	0.0	-14.5
	2.10	205	33.0	0.0	-33.0
C5	0.00	185.5	13.5	9.4	-4.1
	0.70	175.5	3.5	9.4	5.9
	1.40	176.5	4.5	9.4	4.9
	2.10	195	23.0	9.4	-13.6
C4	0.00	221	49	18.8	-30.2
	0.70	213.5	41.5	18.8	-22.7
	1.40	209.5	37.5	18.8	-18.7
	2.10	208.5	36.5	18.8	-17.7
C3	0.00	222.5	50.5	28.2	-22.3
	0.70	208	36.0	28.2	-7.8
	1.40	193	21.0	28.2	7.2
	2.10	179.5	7.5	28.2	20.7
C2	0.00	233.5	61.5	37.6	-23.9
	0.70	232.5	60.5	37.6	-22.9
	1.40	230	58.0	37.6	-20.4
	2.10	224.5	52.5	37.6	-14.9
C1	0.00	219	47.0=X_1	47.0	0.0
	0.70	220.5	48.5	47.0	-1.5
	1.40	224.5	52.5	47.0	-5.5
	2.10	226.5	54.5	47.0	-7.5

C12	0.00	77=Λ_2'	0.0	0.0	0.0
	0.70	92.5	15.5	0.0	-15.5
	1.40	97.5	20.5	0.0	-20.5
	2.10	104	27.0	0.0	-27.0
C11	0.00	94.5	17.5	9.7	-7.8
	0.70	96.5	19.5	9.7	-9.8
	1.40	108	31.0	9.7	-21.3
	2.10	118	41.0	9.7	-31.3
C10	0.00	100.5	23.5	19.4	-4.1
	0.70	93.5	16.5	19.4	2.9
	1.40	90	13.0	19.4	6.4
	2.10	84	7.0	19.4	12.4
C9	0.00	102	25.0	29.1	4.1
	0.70	110.5	33.5	29.1	-4.4
	1.40	109.5	32.5	29.1	-3.4
	2.10	112	35.0	29.1	-5.9
C8	0.00	116.5	39.5	38.8	-0.7
	0.70	116	39.0	38.8	-0.2
	1.40	113.5	36.5	38.8	2.3
	2.10	112.5	35.5	38.8	3.3
C7	0.00	125.5	48.5=X_2	48.5	0.0
	0.70	130	53.0	48.5	-4.5
	1.40	128.5	51.5	48.5	-3.0
	2.10	130	53.0	48.5	-4.5

Table 2.28 Eccentricity errors along row C, Site 3 (absolute values)

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)
0.0	0.00	1.5	2.25
7.5	56.25	5.5	30.25
14.5	210.25	7.5	56.25
33.0	1089.00	0.0	0.00
4.1	16.81	15.5	240.25
5.9	34.81	20.5	420.25
4.9	24.01	27.0	729.00
13.6	184.96	7.8	60.84
30.2	912.04	9.8	96.04
22.7	515.29	21.3	453.69
18.7	349.69	31.3	979.69
17.7	313.29	4.1	16.81
22.3	497.29	2.9	8.41
7.8	60.84	6.4	40.96
7.2	51.84	12.4	153.76
20.7	428.49	4.1	16.81
23.9	571.21	4.4	19.36
22.9	524.41	3.4	11.56
20.4	416.16	5.9	34.81
14.9	222.01	0.7	0.49
0.0	0.00	0.2	0.04

2.3	5.29
3.3	10.89
0.0	0.00
4.5	20.25
3.0	9.00
4.5	20.25
Σ =522.7	Σ=9915.85

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{522.7}{48} = 10.890\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{9915.85 - \frac{(522.7)^2}{48}}{48}} = 9.381\text{mm}$$

$$e = \bar{e} + 1.645$$

$$= 10.89 + (1.64 \times 9.381)$$

$$= 26.275\text{mm}$$

Table 2.29 Eccentricity errors at site 3

Site/row	Σe_1 (mm)	Σe_1^2 (mm^2)	No. of readings n
3B	554.6	9939.22	48
3C	522.7	9915.85	48
3	1077.3	19855.07	96

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{1077.3}{96} = 11.222\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{19855.07 - \frac{(1077.3)^2}{96}}{96}} = 8.994\text{mm}$$

$$e = \bar{e} + 1.64\sigma$$

$$= 11.222 + (1.64 \times 8.994)$$

$$= 25.972\text{mm}$$

2.3 Floor to floor (transfer) alignment

The errors in the floor to floor alignment of the columns were measured at sites 2 and 3 using a builder's spirit level and a scale rule. At site 3 where the columns have direct contact (on the outer half of the column width), the builder's level was placed along one of the edges of the columns, and with the help of the spirit level bubble, the builder's level was made to lie vertically along the column. The offset of the second column from the builder's level's edge was then measured at the column-to-column interface using a scale rule (see fig 2.5). This procedure was then repeated on the opposite side of the column, (see fig 2.6). The column widths at the column-to-column interface were also measured. Figure 2.7 shows the alignment error at site 3 in detail.

At site 2 where there is no direct contact between the columns, the edge-lines of the lower and upper columns were first transfered and marked on the slab edge (see fig 2.8) with the aid of the builder's level, then vertical lines were drawn through these marks, (see fig 2.9). The distance between the two lines drawn on the same edge of the lower and upper columns gives the relative displacement on that edge. The column widths at the junction with the floor slab were also measured. Figure 2.10 shows the alignment error at site 2 in greater detail.

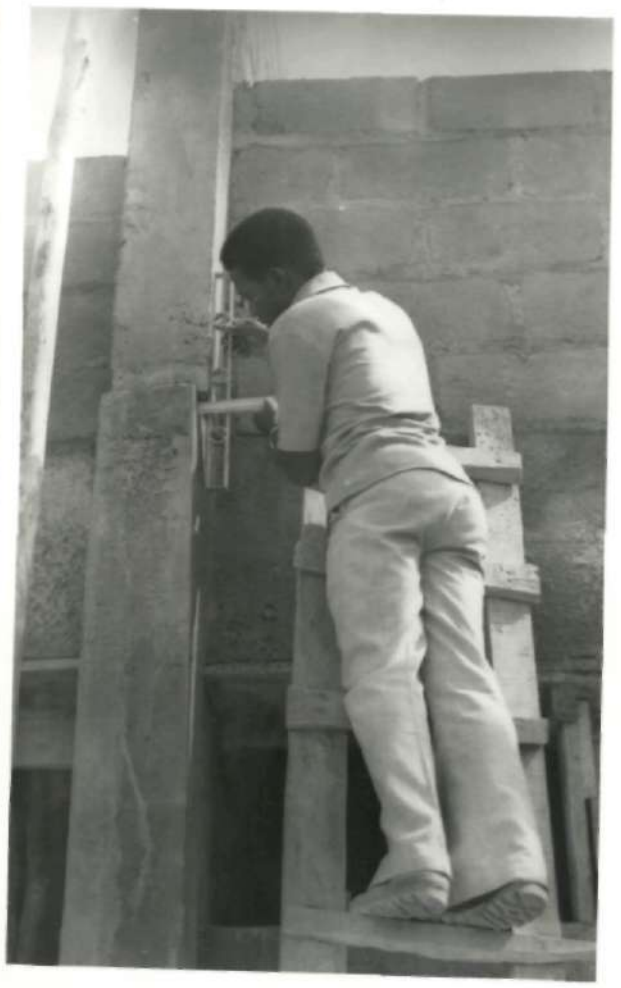


FIG 2.5 TRANSFER ALIGNMENT ERROR AT SITE 3



FIG 2-6 TRANSFER AT ALIGNMENT SITE ERROR 3

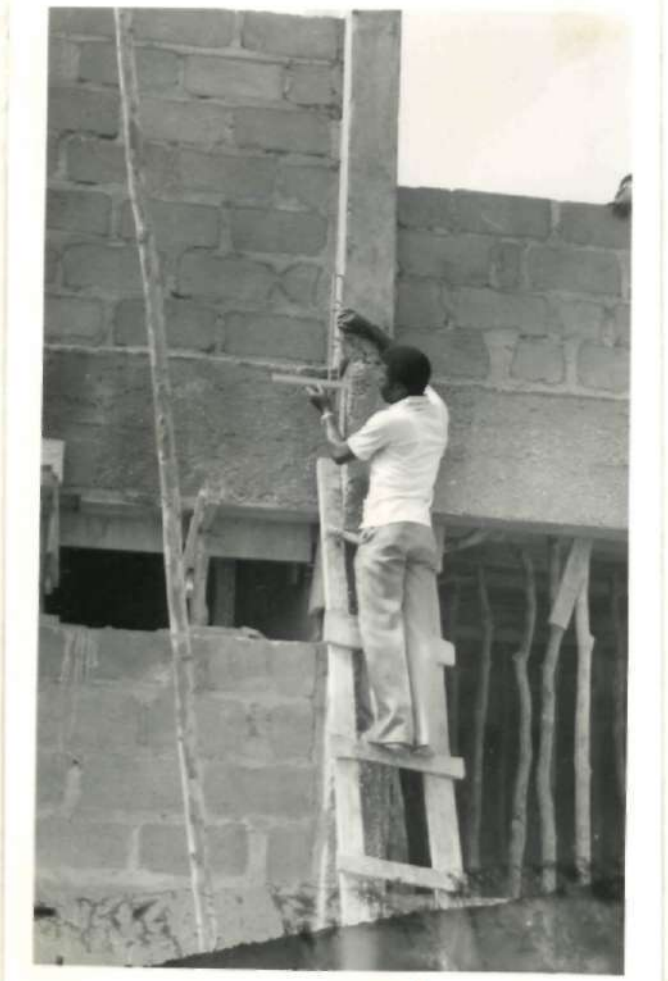


FIG 2-6 TRANSFER AT ALIGNMENT SITE ERROR 3



FIG 2·7 TRANSFER ALIGNMENT ERROR AT SITE 3 (IN DETAIL)



FIG 2.8 MARKING COLUMN-EDGE ON SLAB-FACE AT SITE 2



FIG 29 MARKING COLUMN-EDGE ON SLAB-FACE AT SITE 2



FIG 2.10 TRANSFER ALIGNMENT ERROR AT SITE 2
(IN DETAIL)

In all the readings below, Edge 1 and Edge 2 are respectively the edges further away from, and nearer to, the theodolite station (see fig 2.11). If an edge of the upper column is displaced further away from the theodolite station than the corresponding edge of the lower column, (as at edge 2 in fig 2.11), this displacement is booked as positive, and vice versa for negative displacement.

The construction or alignment error e_1 between the columns is given, (with reference to fig 2.11), by:

$$e_1 = 0.5h_1 + x_1 - 0.5h_2$$

$$= 0.5(h_1 - h_2) + x_1 \qquad (2.6)$$

It was not possible to take measurements on all the edge columns on sites 2 and 3 due to obstructions such as expansion joints, progressing construction work , etc, and measurements were not taken at site 1 because most of the edge columns along which readings could have been taken had already been rendered with sandcrete mortar, and access to the others restricted by the time the transfer alignment readings were to be taken.

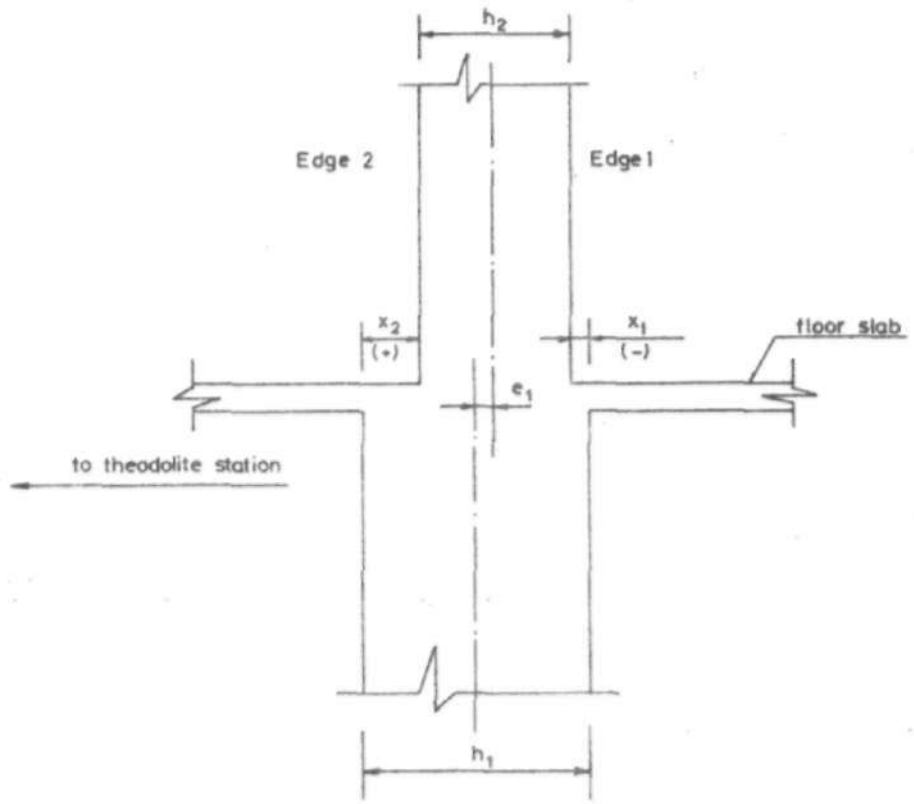


FIG. 2.11 ERRORS IN FLOOR TO FLOOR ALIGNMENT.

Table 2.30 Offsets between the ground and first floor columns at site 2

Col No.	Error X at		Col width at junction with floor slab, (mm)		Error $e_1 = 0.5(h_1 - h_2) + X_1$ (mm)
	Edge 1 (mm)	Edge 2 (mm)	Lower col h_1	Upper col h_2	
B8	+20	+ 3	223	240	11.5
B7	+7	-10	222	239	-1.5
B6	-5	- 4	236	235	-4.5
B5	-8	-18	228	238	-13.0
B4	-10	0	238	228	-5.0
B3	+4	-7	225	236	-1.5
B2	0	-8	232	240	-4.0
B1	15	0	228	243	7.5
B12	-8	-15	230	237	-11.5
B11	-14	+14	231	203	0.0
A1	+6	0	228	234	3.0
A2	+2	-4	229	235	-1.0
A3	-8	-11	233	236	-9.5
A4	+6	-12	232	250	-3.0
A5	+15	-9	219	243	3.0
A6	+29	-4	211	244	12.5
A7	0	-13	233	246	-6.5
A8	+4	-24	234	262	-10.0
A11	+11	-17	215	243	-3.0
A12	-9	-27	221	239	-18.0

D9	0	-6	230	236	-3.0
D8	-2	-6	231	235	-4.0
D7	+2	-11	222	235	-4.5
D6	-3	-27	232	256	-15.0
D5	+6	0	222	228	3.0
D4	+9	-11	223	243	-1.0
D3	+20	-9	220	249	5.5
D2	+19	+6	223	236	12.5
D1	0	-35	225	260	-17.5
C1	0	-2	234	236	-1.0
C2	+11	-8	225	244	1.5
C3	-5	0	233	228	-2.5
C4	-18	-7	224	213	-12.5

Table 2.31 Eccentricity errors between grond floor and first floor columns at site 2 (absolute values)

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)
11.5	132.25	3.0	9.0
1.5	2.25	18.0	324.0
4.5	20.25	3.0	9.0
13.0	169.0	4.0	16.0
5.0	25.0	4.5	20.25
1.5	2.25	15.0	225.0
4.0	16.0	3.0	9.0
7.5	56.25	1.0	1.0
11.5	132.25	5.5	30.25
0.0	0.0	12.5	156.25
3.0	9.0	17.5	306.25
1.0	1.0	1.0	1.0
9.5	90.25	1.5	2.25
3.0	9.0	2.5	6.25
3.0	9.0	12.5	156.25
12.5	156.25	Σ=213.0	Σ=2244.0
6.5	42.25		
10.0	100.0		

Sample mean eccentricity $\bar{e} = \frac{\Sigma e_1}{n}$

$= \frac{213.0}{33}$

$= 6.455\text{mm}$

standard deviation $\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$

$= \sqrt{\frac{2244 - \frac{(213)^2}{33}}{33}}$

$= 5.132\text{mm}$

Characteristic mean eccentricity $e = \bar{e} + 1.64\sigma$

$= 6.455 + (1.64 \times 5.132)$

$= 14.871\text{mm}$

Table 2.32 Offsets between first and second floor Columns at site 2.

Col No.	Error X at Edge 1 (mm)	Error X at Edge 2 (mm)	Col width at junction with floor slab (mm) lower col h_1	Col width at junction with floor slab (mm) Uper Col h_2	Error $e_1 = 0.5(h1-h2)+X_1$ (mm)
A1	+30	0	205	235	15
A6	+8	+3	230	235	5.5
A7	-10	+36	246	200	13
A8	-14	-43	222	251	-28.5
B1	+13	0	221	234	6.5
B2	-5	-9	241	245	-7
B3	+20	+17	221	224	18.5
B4	+30	-7	223	260	11.5
B5	+12	-18	223	253	-3
B6	-18	-24	228	234	-21
B7	-8	-9	229	230	-8.5
B8	+4	-29	207	240	-12.5
B12	+8	-25	215	248	-8.5
B13	+15	+6	228	237	10.5
C1	0	-20	222	242	-10
C2	+23	+5	224	242	14
C3	+12	-7	217	236	2.5
C4	+13	+5	218	226	9
C5	+9	-5	214	228	2
D1	0	-16	214	230	-8
D2	-5	-31	227	253	-18
D3	+4	-12	226	242	-4
D4	+3	-20	207	230	-8.5
D5	+5	-24	220	249	-9.5

D6	+7	-23	214	244	-8
D7	+19	-15	231	265	2
D8	+11	-5	238	254	3
D9	0	-14	221	235	-7

Table 2.33 Eccentricity errors between first and second floor columns at site 2 (absolute values).

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)	(e_1 (mm)	e_1^2 (mm^2)
15	225	8.5	72.25	18	324
5.5	30.25	12.5	156.25	4	16
13	169	8.5	72.25	8.5	72.25
28.5	812.25	10.5	110.25	9.5	90.25
6.5	42.25	10	100	8	64
7	49	14	196	2	4
18.5	342.25	2.5	6.25	3	9
11.5	132.25	9	81	7	49
3	9	2	4	Σ=275	Σ=3743
21	441	8	64		

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{275}{28} = 9.821\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{3743 - \frac{(275)^2}{28}}{28}}$$

$$= 6.101\text{mm}$$

$$e = \bar{e} + 1.64\sigma$$

$$= 9.821 + (1.64 \times 6.101)$$

$$= 19.827\text{mm}$$

Table 2.34 Eccentricity errors between the floor at site 2

Floor	Σe_1 (mm)	Σe_1^2 (mm^2)	No. of readings n
0 to 1	213	2244	33
1 to 2	275	3743	28
total	488	5987	61

$$\bar{e} = \frac{\Sigma e_1}{n}$$

$$= \frac{488}{61} = 8.000\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{5987 - \frac{(488)^2}{61}}{61}} = 5.843\text{mm}$$

$$e = \bar{e} + 1.64\sigma$$

$$= 8 + 1.64 \times 5.843) = 17.583\text{mm}$$

Table 2.35 Offsets between ground and first floor columns at site 3.

Col No.	Error X at		Col width at junction with floor slab (mm)		Error e_1 $=0.5(h_1-h_2)+x_1$ (mm)
	Edge 1 (mm)	Edge 2 (mm)	lower col. h_1	Upper col h_2	
A2	-15	-33	220	238	-24
A3	-6	+4	240	230	-1
A4	+20	+12	232	240	16
A5	+24	+19	228	233	21.5
A8	+16	+8	231	239	12
A9	+16	+15	233	234	15.5
A10	+6	+12	245	239	9
A11	+42	+37	226	231	39.5
D11	-21	-32	225	236	-26.5
D10	-12	0	243	231	-6
D9	-3	-11	225	233	-7
D8	-10	0	228	218	-5
D5	-16	-11	225	220	-13.5

Table 2.36 Eccentricity errors between ground and first floor columns at site 3 (absolute values)

e_1 (mm)	e_1^2 (mm^2)	e_1 (mm)	e_1^2 (mm^2)
24	576	39.5	1560.25
1	1	26.5	702.25
16	256	6	36
21.5	462.25	7	49
12	144	5	25
15.5	240.25	13.5	182.25
9	81	Σ=196.5	Σ=4315.25

$$\bar{e} = \frac{\Sigma e_1}{n} = \frac{196.5}{13}$$

$$= 15.115\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{4315.25 - \frac{(196.5)^2}{13}}{13}}$$

$$= 10.172\text{mm}$$

$$e = \bar{e} + 1.64\sigma$$

$$= 15.115 + (1.64 \times 10.172)$$

$$= 31.797\text{mm}$$

2.4 Vertical alignment

The verticality of the columns from the ground floor to the top floor was measured at sites 2 and 3 using a steel plumb-bob and nylon string. The plumb-bob was suspended from the top of the top-floor column and offset from it so that the bob suspends freely. To counteract the effects of wind, the steel plumb-bob was made heavy (5kg) and it was also shielded using a hollow rectangular box, /2.4/.

After the bob has settled, the offsets from the string to the columns were measured at various heights (see fig 2.12). The diference between each of these offsets and the ground floor floor-level offset(reference level) gives the relative errors in vertical alignment.

Edge-walls 0.9m high had already been built round the ground floor columns at site 2 by the time the vertical alignment readings were to be taken thereby hindering measurements to be taken at the ground floor floor-level (see fig 2.12); the reference level at this site was therefore taken at 1m above the ground floor level. Measurements were not taken at site 1 because the few external colums along which measurements could have been taken had either been rendered with mortar, or access for measurements along them impaired.



FIG 2.12 MEASURING OFFSETS IN VERTICAL ALIGNMENT AT SITE 2

Table 2.37 Plum-bob offsets at site 2

Col No.	Ht above ground floor level (m)	Offset from string (mm)	Error e_1 (= x_i - offset) (mm)	e_1^2 (mm^2)
A1	1	198=X_1	0	0
	2	199	-1	1
	4	199	-1	1
	5	203.5	-5.5	30.25
	7	220	-22	484
	8	226	-28	784
A2	1	174=x_2	0	0
	2	174	0	0
	4	178	-4	16
	5	179	-5	25
	7	198	-24	576
	8	218	-44	1936
A3	1	148=x_3	0	0
	2	149	-1	1
	4	145	3	9
	5	148	0	0
	7	177	-29	841
	8	194	-46	2116
A4	1	147=x_4	0	0
	2	146	1	1
	4	152	-5	25
	5	160	-13	169
	7	183	-36	1296
	8	124	23	529

A5	1	191	0	0
	2	201	-10	100
	4	175	16	256
	5	175	16	256
	7	206	-15	225
	8	224	-33	1089
A6	1	165	0	0
	2	174	-9	81
	4	175	-10	100
	5	174	-9	81
	7	160	5	25
	8	162	3	9
A7	1	204	0	0
	2	210	-6	36
	4	217	-13	169
	5	213	-9	81
	7	220	-16	256
	8	215	-11	121
A8	1	142	0	0
	2	146	-4	16
	4	163	-21	441
	5	173	-31	961

	7	175	-33	1089
	8	180	-38	1444
B1	1	171	0	0
	2	175	-4	16
	4	170	1	1
	5	161	10	100
	7	157	14	196
	8	165	6	36
B2	1	172	0	0
	2	180	-8	64
	4	159	13	169
	5	157.5	14.5	210.25
	7	156	16	256
	8	155.5	16.5	272.25
B3	1	249	0	0
	2	246.5	2.5	6.25
	4	245	4	16
	5	245	4	16
	7	243	6	36
	8	247.5	1.5	2.25
B4	1	136.5	0	0
	2	133.5	3	9
	4	154.5	-18	324
	5	159	-22.5	506.25
	7	162	-25.5	650.25
	8	155.5	-19	361
B5	1	140	0	0
	2	146	-6	36
	4	139	1	1
	5	142.5	-2.5	6.25
	7	138	2	4
	8	141	-1	1

B6	1	152.5	0	0
	2	150	2.5	6.25
	4	161	-8.5	72.25
	5	152	0.5	0.25
	7	147.5	5	25
	8	153	-0.5	0.25
B7	1	136	0	0
	2	134	2	4
	4	136	0	0
	5	144	-8	64
	7	129	7	49
	8	135.5	0.5	0.25
B8	1	126	0	0
	2	132	-6	36
	4	130.5	-4.5	20.25
	5	129	-3	9
	7	121.5	4.5	20.25
	8	125	1	1
B11	1	170	0	0
	2	175	-5	25
	4	156	14	196
	5	149	21	441
	7	140	30	900
	8	127	43	1849
B12	1	156	0	0
	2	156	0	0
	4	176.5	-20.5	420.25
	5	171	-15	225
	7	152.5	3.5	12.25
	8	149	7	49

C1	1	157	0	0
	2	160	-3	9
	4	164	-7	49
	5	162	-5	25
	7	162	-5	25
	8	162	-5	25
C2	1	135	0	0
	2	123	12	144
	4	131	4	16
	5	132	3	9
	7	134	1	1
	8	133	2	4
C3	1	162	0	0
	2	163	-1	1
	4	178	-16	256
	5	177	-15	225
	7	149	13	169
	8	144	18	324
C4	1	172	0	0
	2	173	-1	1
	4	152	20	400
	5	165	7	49
	7	169	3	9
	8	158	14	196
D1	1	85	0	0
	2	89	-4	16
	4	102	-17	289
	5	104	-19	361
	7	90	-5	25
	8	93	-8	64

D2	1	147	0	0
	2	158	-11	121
	4	156	-9	81
	5	141	6	36
	7	149	-2	4
	8	156	-9	81
D3	1	167	0	0
	2	157	10	100
	4	171	-4	16
	5	161	6	36
	7	159	8	64
	8	161	6	36
D4	1	151	0	0
	2	142	9	81
	4	153	-2	4
	5	157	-6	36
	7	168	-17	289
	8	172	-21	441
D5	1	157	0	0
	2	157	0	0
	4	163	-6	36
	5	167	-10	100
	7	153	4	16
	8	151	6	36
D6	1	175	0	0
	2	181	-6	36
	4	161	14	196
	5	159	16	256
	7	155	20	400
	8	164	11	121

D7	1	191	0	0
	2	184	7	49
	4	175	16	256
	5	186	5	25
	7	156	35	1225
	8	152	39	1521
D8	1	148	0	0
	2	150	-2	4
	4	143	5	25
	5	154	-6	36
	7	175	-27	729
	8	179	-31	961
D9	1	68	0	0
	2	69	-1	1
	4	79	-11	121
	5	90	-22	484
	7	94	-26	676
	8	106	-38	1444
			$\Sigma\|e_1\| = 1731.5$	$\Sigma = 36269.25$

$$\bar{e} = \frac{\Sigma|e_1|}{n}$$

$$= \frac{1731.5}{186} = 9.309\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{36269.25 - \frac{(1731.5)^2}{186}}{186}}$$

$$= 10.408\text{mm}$$

$$e = \bar{e} + 1.64\sigma$$

$$= 9.309 + (1.64 \times 10.408)$$

$$= 26.378\text{mm}$$

Table 2.38 Plumb-bob offsets at site 3

Col No.	Ht above ground floor level (m)	offset from string (mm)	Error e_1 ($=x_1$ - offset) (mm)	e_1^2 (mm^2)
A2	0	$69=x_1$	0	0
	1	71	-2	4
	2	65	4	16
	3	88	-19	361
	4	85.5	-16.5	272.25
	5	81	-12	144
A3	0	$70=x_2$	0	0
	1	52	18	324
	2	39	31	961
	3	55.5	14.5	210.25
	4	67	3	9
	5	65	5	25
A4	0	$55=x_3$	0	0
	1	54	1	1
	2	61	-6	36
	3	66	-11	121
	4	59	-4	16
	5	57	-2	4
A5	0	121.5	0	0
	1	119	2.5	6.25
	2	120	1.5	2.25
	3	141.5	-20	400
	4	124	-2.5	6.25
	5	120	1.5	2.25

A8	0	78	0	0
	1	73	5	25
	2	75	3	9
	3	84	-6	36
	4	83	-5	25
	5	85	-7	49
A9	0	61	0	0
	1	68	-7	49
	2	69.5	-8.5	72.25
	3	85.5	-24.5	600.25
	4	89.5	-28.5	812.25
	5	81.5	-20.5	420.25
A10	0	77	0	0
	1	81	-4	16
	2	84	-7	49
	3	95	-18	324
	4	121.5	-44.5	1980.25
	5	111	-34	1156
D5	0	55	0	0
	1	52	3	9
	2	45	10	100
	3	61	-6	36
	4	77	-22	484
	5	75	-20	400
D8	0	70	0	0
	1	56	14	196
	2	44	26	676
	3	35.5	34.5	1190.25
	4	83.5	-13.5	182.25
	5	78	-8	64

D9	0	64	0	0
	1	66	-2	4
	2	64	0	0
	3	84	-20	400
	4	62	2	4
	5	60	4	16
D10	0	65	0	0
	1	57	8	64
	2	56	9	81
	3	45	20	400
	4	86	-21	441
	5	87	-22	484
D11	0	63	0	0
	1	73	-10	100
	2	83.5	-20.5	420.25
	3	79	-16	256
	4	76	-13	169
	5	68	-5	25
			$\Sigma/e_1/=729$	$\Sigma=14746.5$

$$\bar{e} = \frac{\Sigma/e_1/}{n}$$

$$= \frac{729}{72} = 10.125\text{mm}$$

$$\sigma = \sqrt{\frac{\Sigma e_1^2 - \frac{(\Sigma e_1)^2}{n}}{n}}$$

$$= \sqrt{\frac{14746.5 - \frac{(729)^2}{72}}{72}}$$

$$= 10.114 \text{ mm}$$

$$e = \bar{e} + 1.64\sigma$$

$$= 10.125 + (1.64 \times 10.114)$$

$$= 26.712 \text{mm}$$

2.5 Discussion of results obtained from field measurements

Table 2.39 Summary of results from field measurements

Data Measured	Site	No. of Columns Surveyed	Col. size mm	Mean $\bar{e}$ mm	δ mm	Construction error specified					
						charact. ratio e_n (=e/h)	mean $\bar{e}$ (mm)	Proposed $(0.5d_2)_{mm}$	ACI318-77 $(0.03h + 15.2)_{mm}$	CP110 $(0.05h)_{mm}$	CEB h/30mm
Ground	1	18	250x400	12.3	9.5	0.070	27.9	32.8	27.2	20.0	13.3
floor	2	32	230x700	8.8	9.0	0.034	23.6	26.3	36.2	35.0	23.3
align-	3	24	230x230	11.2	9.0	0.113	26.0	26.3	22.1	11.5	7.7
ment											
Floor to floor	2	32	230x700	8.0	5.8	0.076	17.6	26.3	22.1	11.5	7.7
align-	3	13	230x450	15.1	10.2	0.133	31.3	26.3	22.1	11.5	7.7
ment											
Vertical	2	31	230x700	9.3	10.4	0.038	26.4	26.3	36.2	35.0	23.3
align-	3	12	230x450	10.1	10.1	0.059	26.7	26.3	22.1	22.5	15.0
ment											

NB: The underlined values in the fourth column above represent the depth of section h in the plane of bending.

2.5.1 Ground floor alignment

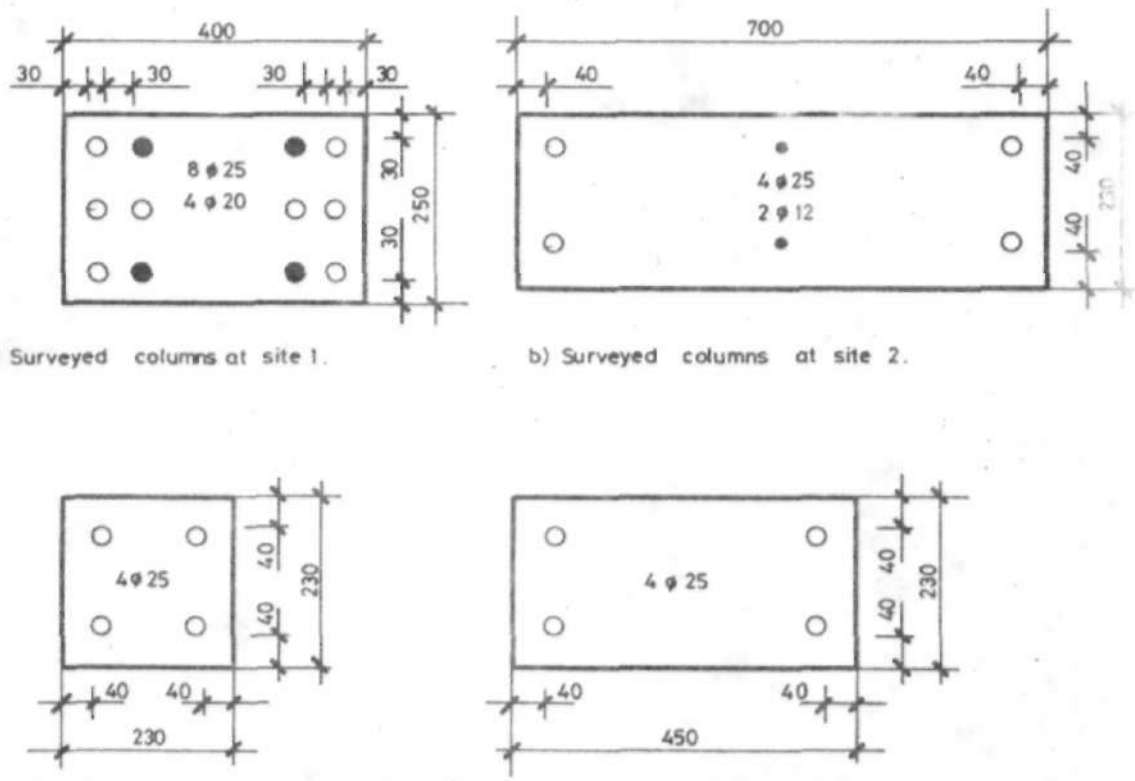
With a minimum eccentricity e equal to 0.03h or 0.05h as specified in the CEB and CP codes respectively, one would expect the eccentricity error to vary proportionally with the section size so that the largest error should occur on the largest column, and the smallest error on the smallest column. The site results as summarised in Table 2.39 however show no such correlation, hence suggesting that the CEB and CP codes' construction tolerances as presently specified have little relation with the column size.

The ACI code which allows a tolerance of (15.2 + 0.03h)mm shows a better correlation with the site results than the CEB and CP codes' specifications, although it looks as if for small-sized columns the specified tolerance in the ACI code is rather small, while for large-sized columns the specified tolerance is larger than that obtained from the site measurements. For medium-sized columns a fairly good correlation is obtained between the specified and the actual tolerances.

It may be notted from Table 2.39 that the CEB code's specification of 0.03h shows the least correlation with the site results, but by allowing an additional 15.2mm to this value on all sections irrespective of their sizes (as done in

the ACI code) the degree of correlation has been greatly improved upon. However, even in this case the problem of discontinuity between sections with computed eccentricities less than the minimum eccentricity and those with computed eccentricities greater than or equal to the minimum eccentricity, as discussed in section 1.2, still remains unsolved.

Consider now a proposal to specify $0.5d_2$ as the minimum eccentricity, d_2 being the depth to reinforcement. With a tolerance of $0.5d_2$ good correlation is obtained (see Table 2.39), not only for the small- and big-sized columns but also for the medium-sized columns, and in some cases the tolerance of $0.5d_2$ is slightly greater than the actual error measured at the sites. This observation suggests that the construction error is a function of the ease encountered while fixing the reinforcing bars and placing the concrete in the columns, in otherwords the ease of constructing the columns, (assuming that factors such as the experience of the workmen, and the type of equipment used by the three firms, are identical). Due to the difference in column size, it will be easier to fix the reinforcement and vibrate the concrete in the columns at site 2 than in the columns at site 3, the reinforcement in both columns being relatively uncongested, (see fig. 2.13), so the construction error at site 2 will be expected to be smaller than that at site 3; the site results show that this appears to be the case.



a) Surveyed columns at site 1.

b) Surveyed columns at site 2.

c) Ground floor alignment columns at site 3.

d) Floor to floor, and vertical alignment columns at site 3.

FIG. 2.13 CROSS-SECTIONS OF COLUMNS SURVEYED.

The reinforcement in the columns at site 1 on the otherhand, is relatively congested, (see fig. 2.13), the column size being approximately midway between those at sites 2 and 3, so one will expect more difficulties to be encountered, and hence more errors made, while constructing the columns at this site. The construction error therefore will be expected to be greatest at this site, and this in fact, is what the site results show.

The more congested the reinforcement is, the more reinforcing bars will need to be placed, and in more layers too, thereby increasing the depth to reinforcement d_2, which in turn leads to an increase in the tolerance $0.5d_2$. The value $0.5d_2$ is thus a fair index of the construction errors.

It may be noted that irrespective of their depths, columns in normal exposure conditions have concrete cover to all reinforcement, including links, of about 25mm; if we allow 10mm for the diameter of the links, the cover to the main bars will be 35mm. Since columns are usually reinforced with bars of between 16mm and 25mm diameter, a proposed construction tolerance (ie $0.5d_2$) value of between 21.5mm and 23.8mm will result; these are comparable with the observed site errors of between 23.6mm and 27.9mm. For larger diameter bars and more severe exposure conditions the proposed construction

tolerance of $0.5d_2$ will be greater than the afore-quoted values

2.5.2 Vertical Alignment

The observations made on the ground floor alignment also hold when the vertical alignment of the columns in all the floors are considered, as the results in Table 2.39 show.

It may also be noted that the errors in the vertical alignment are of identical magnitudes to those in the ground floor alignment.

2.5.3 Floor to floor (transfer) alignment

The results of the floor to floor alignment are rather difficult to interprete. For one, the magnitude of the construction error at site 2 is smaller than, while that at site 3 is larger than, the errors in either the ground floor alignment or the vertical alignment. Ordinarily, one will expect less error in the transfer alignment than in the vertical or the ground floor alignments because of the relative ease of aligning the kicker or stater columns above each column.

Secondly, one would have expected that since there is column-to-column contact on the outer half of the columns at site 3 (see figs 2.5, 2.6 and 2.7), less difficulties should have

been encountered, and hence less errors committed, in the transfer alignment of these columns than those at site 2 where the columns on adjacent floors have no contact at all because of the floor slab which is sandwiched between them, (see figs 2.8, 2.9 and 2.10).

The results in Table 2.39 show the reverse of what was expected. Investigations conducted to find the possible cause of this anomaly suggest that it might have been due to the different methods of construction adopted at the two sites during the transfer alignment of the columns. The supervisor at site 2 explained that what they did while placing the kickers of each column was to transfer the edge-lines of the lower columns to form the inner edge-lines of the kicker-columns on the next floor. Such edge-line transfer was not applied to all the columns at site 3; instead, the supervisor on this site explained. it was applied only to the four edge columns of the building (that is, columns 1 and 12 in rows A and D of fig. 2.3). After this the kicker column formwork for the other columns were aligned from these four columns so that the kickers were correctly spaced centre-to-centre along each row and between rows, as specified in the structural drawings, thereby attempting to avoid, on the upper-floor columns, any alignment errors that might have occurred on the lower-floor columns; unfortunately this procedure magnified the error below the floor slab, as can

be seen on figs 2.5 and 2.7. In anycase, the proposed tolerance of $0.5d_2$ appears even in this case to give values more on the safer side than do the tolerances of the other codes.

2.6 From the discussion in section 2.5 we may conclude that, irrespective of the section depths, much the same magnitudes of errors are made while constructing these sections, and that the proposed tolerance of $0.5d_2$ gives a more accurate index of this error than do the ACI, CP, or CEB codes' specifications.

It should be noted however, that the level of skill of the workers, experience of the contractors, and the type of equipment used on the sites where my field measurements were taken are below the average standards obtainable in the advanced countries where the three codes above were drafted. It is not surprising therefore that higher tolerances than the codes predicted were obtained; but in my view the site errors I obtained can be adequately related to developing countries like Nigeria where the level of skill is very low and only very few contractors measure up to the standards assumed in the ACI, CP or CEB codes.

In the next chapter, the proposed tolerance of $0.5d_2$ will be utilized to propose some modifications to the CP110 design approach.

References

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CHAPTER THREE

APPLICATION OF THE RESEACH FINDING

3.1 Having established that $0.5d_2$ is a good index of construction tolerances, we are now faced with the task of introducing this tolerance into the design process, bearing in mind firstly, the shortcomings of the current tolerance specifications (as discussed in section 1.2), and secondly, that the intention of the CP110 Code Drafting Committee was not to produce a document that is a repository for all the latest, most complex and fashionable methods of calculation; instead the CP code is intended to define that design effort which is necessary and sufficient for the satisfactory design of normal structures, /3.1/. In otherwords, any proposal put forward should aim at attempting to simplify the design process and not to make it more complex.

A design method which attempts to do this will now be presented. This proposal was put forward by Oleszkiewicz, /3.2, 3.3/, at a seminar in the Department of Civil Engineering, Ahmadu Bello University, Zaria.

3.2 A uniform approach to the design of concrete structures (as proposed by Oleszkiewicz, /3.2, /3.3/).

3.2.1 The strength of a rectangular concrete cross-section under a combined action of bending moment M and axial load N may be assessed on the basis of generally accepted assumptions

such as those given in section 3.3.5.1 of the CP code. For simplicity, the simplified rectangular concrete stress block and the bilinear stress-strain relation for reinforcing steel, as given by the CP and CEB codes respectively, will be assumed. These simplifications are approximations of the actual relations, and both of them err on the safer side, /1.1, 3.1/.

When a concrete section is loaded by a longitudinal force N acting at an eccentricity e, this loading arrangement is usually taken from the principle of superposition, as equivalent to the force N acting axially (ie at 0.5h) in conjuction with a bending moment M equal to N.e, as shown in fig. 3.1a. The stress conditions that may arise from this principle of superposition are shown in fig. 3.1b. When the eccentricity is small and the line of action of the force N lies within the middle third of the section, a condition of no-tensile stress is developed in the section, as shown in fig. 3.1b(i). The whole of the concrete section is in compression, and the point of zero stress lies outside the section. When the bending moment is large, the effect of the combination of axial and flexural force is such as to develop tensile and compressive stresses in the section, as shown in fig. 3.1b(ii). Here the neutral axis or the point of zero strain lies within the section, its position being dependent on the magnitude of the force N.

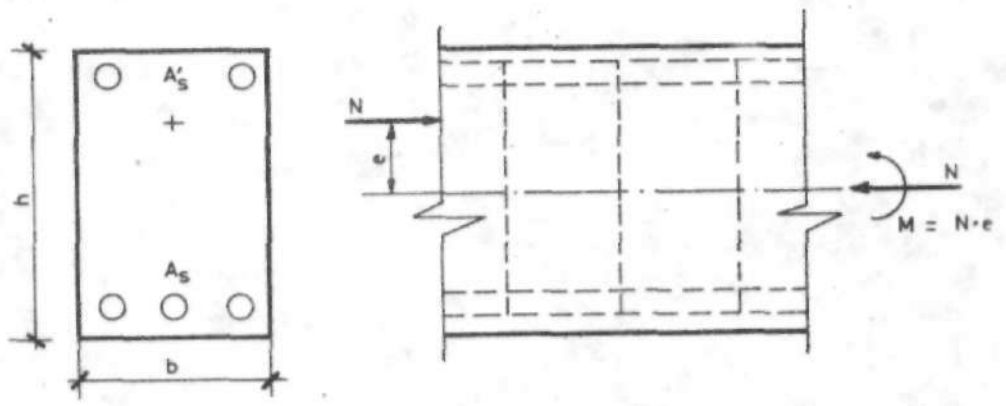


FIG. 3.1a, EQUIVALENT LOADING ARRANGEMENTS.

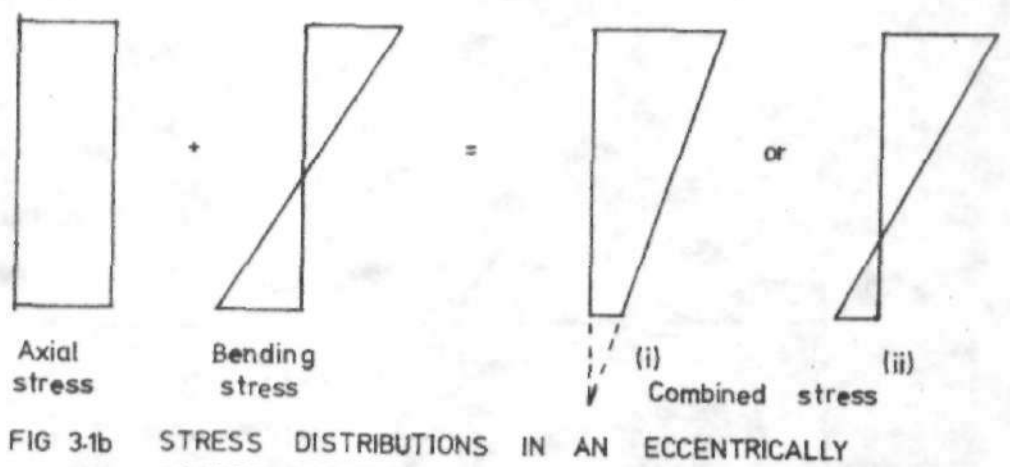
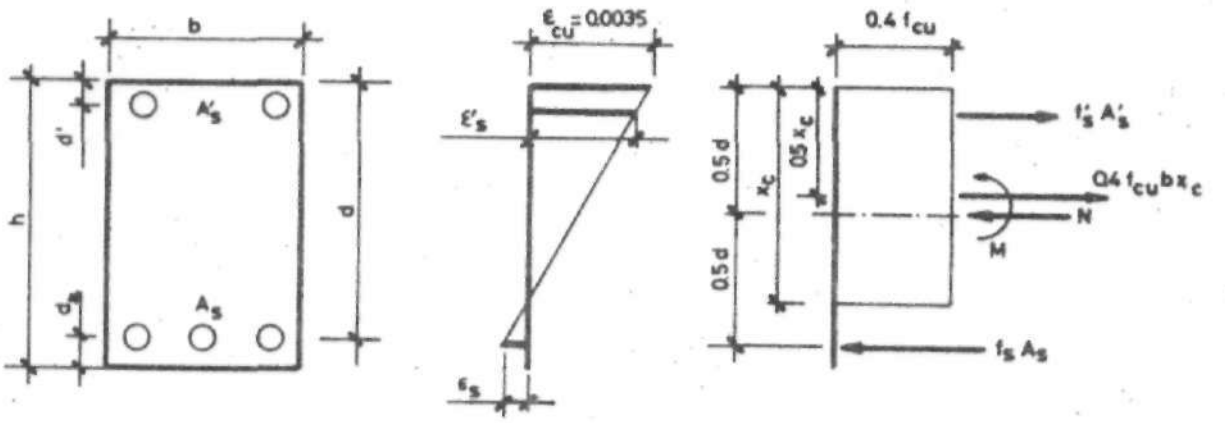


FIG 3.1b STRESS DISTRIBUTIONS IN AN ECCENTRICALLY LOADED SECTION.

In order to allow for eccentricity due to construction tolerances, Oleszkiewicz proposed that the concrete cover to the less compressed reinforcement be neglected when analysing the section. The nominal eccentricity so introduced is equal to $0.5d_2$ (as will be shown in section 3.3), which corresponds with the value of construction errors that I obtained from site measurements. My research findings may therefore be applied in line with Oleszkiewicz's theory, as done below.

With the above assumptions the stress and strain distrubution at failure will be as shown in fig. 3.2. Because the cover to the less compressed reinforcement has now been neglected, it may be noticed from fig. 3.2c that the force N is now acting at 0.5d instead of at 0.5h as was the case in fig. 3.1a.



a) Section b) Strain c) Stress.

FIG. 3.2 STRAIN AND STRESS DISTRIBUTION AT FAILURE.

From the geometry of fig 3.2b, the strains in the compression and tension reinforcements may be expressed as

$$\frac{\varepsilon'_s}{\varepsilon_{cu}} = \frac{x_c - d'}{x_c}$$

$$\text{ie } \varepsilon'_s = \frac{\varepsilon_{cu}\,(x_c/d - \frac{d'}{d})}{x_c/d} \qquad (3.1)$$

$$\text{and } \frac{\varepsilon_s}{\varepsilon_{cu}} = \frac{d - x_c}{x_c}$$

$$\text{ie } \varepsilon_s = \varepsilon_{cu}\,\frac{(1 - x_c/d)}{x_c/d} \qquad (3.2)$$

From eqns 3.1 and 3.2, the stresses f'_s and f_s in the compression and tension reinforcements may be derived from the appropriate stress-strain curves.

By equating longitudinal forces and taking moments about the level of the axial force N, we obtain

$$N = 0.4f_{cu}bx_c + f'_sA'_s - f_sA_s \qquad (3.3)$$

$$M = 0.4f_{cu}bx_c(0.5d - 0.5x_c) + f_s'A'_s(0.5d - d') + 0.5f_sA_sd \qquad (3.4)$$

or, in dimensionless form,

$$\alpha = \frac{N}{0.4f_{cu}bd} = \frac{x_c}{d} + \frac{f'_s}{0.4f_{cu}} \cdot \frac{A'_s}{bd} - \frac{f_s}{0.4f_{cu}} \cdot \frac{A_s}{bd}$$

$$= \alpha_c + \alpha'_s - \alpha_s \qquad (3.5)$$

$$\beta = \frac{M}{0.4f_{cu}bd^2} = 0.5\alpha_c(1-\alpha_c) + \alpha'_s\,(0.5-d'/d) +$$

$$+ 0.5\alpha_s$$

$$= \beta_c + \beta'_s + \beta_s \qquad (3.6)$$

where the strength ratios

$$\alpha_c = \frac{x_c}{d} \qquad (3.7)$$

$$\alpha'_s = \frac{f'_s A'_s}{0.4f_{cu}bd} \qquad (3.8)$$

$$\alpha_s = \frac{f_s A_s}{0.4f_{cu}bd} \qquad (3.9)$$

Denoting the stress ratios by

$$\varphi' = \frac{f'_s}{0.4f_{cu}} \quad \text{and} \quad \varphi = \frac{f_s}{0.4f_{cu}}$$

and the reinforcement ratios by

$$\rho' = \frac{A'_s}{bd} \quad \text{and} \quad \rho = \frac{A_s}{bd}$$

the strength ratios may be written as

$$\alpha'_s = \varphi' \cdot \rho' \text{ and } \alpha_s = \varphi \cdot \rho \qquad (3.10)$$

In eqn 3.6, the moment ratios

$$\beta_c = 0.5\,\alpha_c\,(1-\alpha_c) \qquad (3.11)$$

$$\beta'_s = \alpha'_s\,(0.5 - \frac{d'}{d}) \qquad (3.12)$$

$$\beta_s = 0.5\,\alpha_s \qquad (3.13)$$

The ultimate resistance of the section to axial load is now represented by α and that to bending moment by β where the contribution of the concrete compression zone is represented by (α_c, β_c) and that of the tension and compression reinforcements by (α_s, β_s) and (α'_s , β'_s) respectively.

3.2.2 Interaction diagrams

For a given section, the quantities b, d, d', A'_s , A_s , f_{cu} and f_y are known, hence the values of α and β may be

calculated for a range of $\frac{x_c}{d}$ values and a complete failure curve (or interaction diagram) constructed. As an illustration, the construction of an interaction diagram is now discussed for

$$f_{cu} = 30N/nn^2, \quad \rho' = \rho = 0.01$$

$$f_y = 410N/mm^2, \quad d' = 0.1765d \text{ (or } d = 0.85h)$$

The designed yield stresses in tension and compression are respectively,

$$f_{sd} = 0.87 \times 410$$

$$= 356.7N/mm^2$$

$$\text{and } f'_{sd} = 0.72 \times 410$$

$$= 295.2N/mm^2$$

The compression stress $f'_s = \varepsilon'_s E_s$

$$= \varepsilon_{cu} E_s \frac{(x_c/d - \frac{d'}{d})}{\frac{x_c}{d}} \text{ from eqn 3.1}$$

$$= \frac{700\ (x_c/d - \frac{d'}{d})}{\frac{x_c}{d}} \leq 295.2$$

and the designed stress f'_{sd} is reached at

$$\frac{x_c}{d} = \frac{d'}{d} \cdot \frac{\varepsilon_{cu} E_s}{\varepsilon_{cu} E_s - f'_{sd}} \qquad (3.14)$$

$$= 0.1765 \cdot \frac{700}{(700-295.2)}$$

$$= 0.3052$$

or the designed yield strength of this reinforcement being in tension:

$$\frac{x_c}{d} = \frac{d'}{d} \cdot \frac{\varepsilon_{cu} E_s}{(\varepsilon_{cu} E_s + f_{sd})} \qquad (3.15)$$

$$= 0.1765 \cdot \frac{700}{(700 + 356.7)}$$

$$= 0.1169$$

Similarly the tensile stress

$$f_s = \varepsilon_s E_s$$

$$= \varepsilon_{cu} E_s \frac{(1 - \frac{x_c}{d})}{\frac{x_c}{d}} \text{ from eqn 3.2}$$

$$= 700 \frac{(1 - \frac{x_c}{d})}{\frac{x_c}{d}} \leq 356.7$$

and the designed stress f_{sd} occurs at

$$x_c/d = \frac{\varepsilon_{cu}E_s}{\varepsilon_{cu}E_s + f_{sd}} \qquad (3.16)$$

$$= \frac{700}{(700 + 356.7)}$$

$$= 0.6624$$

or the designed yield strength of this reinforcement being in compression:

$$x_c/d = \frac{\varepsilon_{cu}E_s}{\varepsilon_{cu}E_s - f'_{sd}} \qquad (3.17)$$

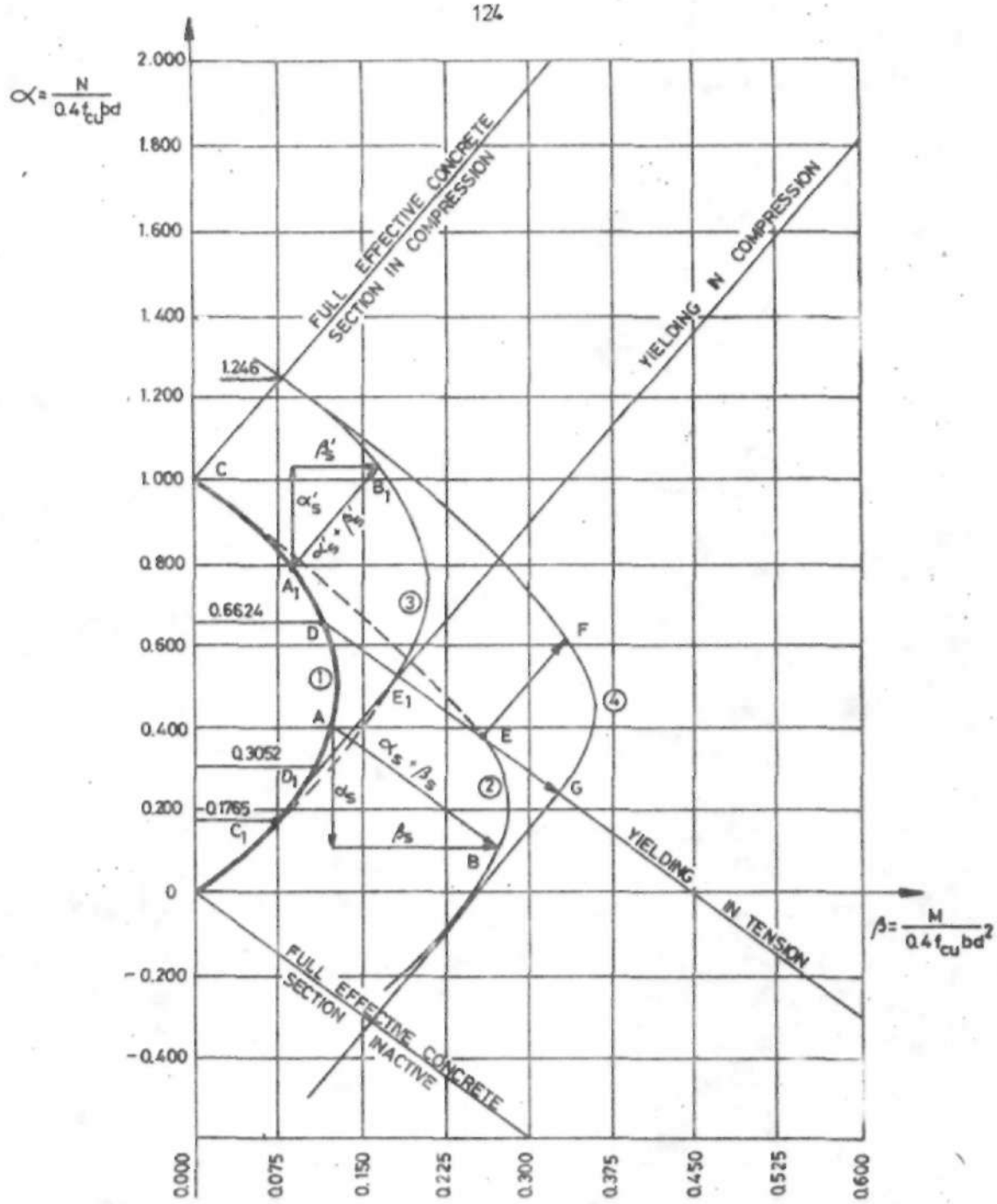
$$= \frac{700}{(700-295.2)}$$

$$= 1.7292$$

The values of α and β given by eqns 3.5 and 3.6 are now calculated and tabulated in Table 3.1 for x_c/d values varying from 0.10 to 1.00 including the characteristic points at which the reinforcement reaches the designed yeild stresses. These values are then plotted for various reinforcement combinations to obtain the failure curves shown in fig. 3.3. The principles governing the construction of the curves in fig. 3.3 will now be briefly discussed.

TABLE 31

$\alpha_c = \frac{x_u}{d}$	β_c	f_s'	α_s'	$\alpha_c + \alpha_s'$	β_s'	$\beta_c + \beta_s'$	f_s	α_s	$\alpha_c - \alpha_s$	β_s	$\beta_c + \beta_s$	$\alpha_c + \alpha_s' - \alpha_s$	$\beta_c + \beta_s' + \beta_s$
0.1000	0.0450	-356.7	-0.2973	-0.1973	-0.0962	-0.0512	356.7	0.2973	-0.1973	0.1487	0.1937	-0.4946	0.0975
0.1765	0.0727	0.0	0.00	0.1765	0.00	0.0727	356.7	0.2973	-0.1208	0.1487	0.2214	-0.1208	0.2214
0.2000	0.0800	82.3	0.0686	0.2686	0.0222	0.1022	356.7	0.2973	-0.0973	0.1487	0.2287	-0.0287	0.2509
0.3052	0.1060	295.2	0.2460	0.5512	0.0796	0.1856	356.7	0.2973	0.0079	0.1487	0.2547	0.2539	0.3343
0.4000	0.1200	295.2	0.2460	0.6460	0.0796	0.1996	356.7	0.2973	0.1027	0.1487	0.2687	0.3487	0.3483
0.5000	0.1250	295.2	0.2460	0.7460	0.0796	0.2046	356.7	0.2973	0.2027	0.1487	0.2733	0.4487	0.3533
0.6000	0.1200	295.2	0.2460	0.8460	0.0796	0.1996	356.7	0.2973	0.3027	0.1487	0.2687	0.5487	0.3483
0.6624	0.1118	295.2	0.2460	0.9084	0.0796	0.1914	356.7	0.2973	0.3652	0.1487	0.2605	0.6111	0.3401
0.7000	0.1050	295.2	0.2460	0.9460	0.0796	0.1846	300.0	0.2500	0.4500	0.1250	0.2300	0.6960	0.3096
0.8000	0.0800	295.2	0.2460	1.0460	0.0796	0.1596	175.0	0.1458	0.6542	0.0729	0.1529	0.9002	0.2325
0.9000	0.0450	295.2	0.2460	1.1460	0.0796	0.1246	77.8	0.0648	0.8352	0.0324	0.0774	1.0812	0.1570
1.0000	0.000	295.2	0.2460	1.2460	0.0796	0.0796	0.0	0.00	1.0000	0.0000	0.00	1.2460	0.0796



CURVE 1 = Failure curve for plane concrete
CURVE 2 = " " " concrete & 1 % tension reinforcement
CURVE 3 = " " " concrete & 1 % compression reinforcement
CURVE 4 = " " " concrete & 2 % symmetrical reinforcement

FIG. 3.3 INTERACTION DIAGRAMS (FAILURE CURVES).

3.2.3 Failure curve for plain concrete (curve 1).

For a plain concrete section, the contribution of the tension and compression reinforcements (α_s, β_s), (α'_s, β'_s) to the resistance of the section is neglected, so eqns 3.5 and 3.6 reduce to

$$\alpha = \alpha_c \qquad \beta = \beta_c$$

$$= x_c/d \qquad = 0.5\, x_c/d\,(1-x_c/d)$$

By successively assigning different values to x_c/d as done in table 3.1, α and β are made to take various values which can then be plotted to obtain the complete failure curve 1 in fig. 3.3. Each point on this curve represents a state of incipient failure while points inside and outside it represent safe and unsafe combinations of α and β respectively.

3.2.4 Concrete and tension reinforcement, (curve 2)

If tension reinforcement is added to the above plain concrete, equations 3.5 and 3.6 become

$$\alpha = \alpha_c - \alpha_s \qquad (3.18)$$

$$\beta = \beta_c + \beta_s \qquad (3.19)$$

For a given section, α_s and β_s are completely defined by the steel stress f_s (see eqns 3.9 and 3.13) which, for a given steel depends only on the strain ε_s (since $f_s = \varepsilon_s E_s$). Eqn 3.2 shows that this strain, and hence α_s and β_s, is completely defined by the x_c/d ratio. For a given section therefore, the α_s and β_s values can be computed for a range of x_c/d values as shown in table 3.1. When these values are superimposed

(vectorially) on curve 1, we obtain curve 2. From this curve it is seen that for any specified value of x_c/d, the effect of the tension reinforcement is represented by the vector $(\overline{\alpha_s} + \overline{\beta_s})$ where

$$\text{length of vector} = \sqrt{(\alpha_s^2 + \beta_s^2)}, \qquad (3.20)$$

and inclination of vector to the α- axis, $\theta = \tan^{-1}\left(\frac{\beta_s}{\alpha_s}\right)$

$$= \tan^{-1}(-0.5) \text{ from eqn } 3.13 \qquad (3.21)$$

where the negative sign in eqn 3.21 is due to the direction of $\overline{\alpha s}$ to satisfy eqn 3.18. Thus point A is moved to point B (fig.3.3) and now represents the N and M combination that will produce a state of incipient failure. Similarly point D is moved to point E.

From Table 3.1 and fig 3.3 it may be observed that for x_c/d equal to 1, (point C), the steel is inactive ($f_s = 0$), while for lower values the reinforcement is in tension. At x_c/d equal to 0.6624, (point D), the reinforcement reaches its designed yield strength in tension. The length of the vector $(\overline{\alpha s} + \overline{\beta s})$ now reaches a maximum value (DE) and remains unchanged as x_c/d is further decreased. As x_c/d is increased beyond 0.6624 the vector length shortens until it vanishes at point C. Irrespective of the x_c/d value however, the vector inclination remains constant at the value given by equation 3.21.

3.2.5 Concrete and compression reinforcement, (curve 3)

If we had compression reinforcement instead of tension reinforcement as was the case in section 3.2.4 above, then eqns 3.5 and 3.6 would have reduced to

$$\alpha = \alpha_c + \alpha'_s \qquad (3.22)$$

$$\beta = \beta_c + \beta'_s \qquad (3.23)$$

Similar situations as discussed in section 3.2.4 take place on curve 3, with the modification that:

$$\text{length of vector} = \sqrt{(\alpha_s'^2 + \beta_s'^2)} \qquad (3.24)$$

$$\text{vector inclination } \theta' = \tan^{-1}\left(\frac{\beta'_s}{\alpha'_s}\right)$$

$$= \tan^{-1}(0.5 - \frac{d'}{d}) \text{ from eqn 3.12} \qquad (3.25)$$

Points A_1, B_1, etc on fig 3.3 are the corresponding points to A, B, etc.

3.2.6 Concrete and symmetrical reinforcement, (curve 4)

When α'_s and β'_s are superimposed on curve 2, or α_s and β_s superimposed on curve 3, curve 4 (on fig 3.3) is obtained which gives the failure curve for a section reinforced on both faces. Hence the α and β for curve 4 are as given by eqns 3.5 and 3.6, namely

$\alpha = \alpha_c + \alpha'_s - \alpha_s$ from eqn 3.5

$\beta = \beta_c + \beta'_s + \beta_s$ from eqn 3.6

In section 3.2.4 it was explained how point D on curve 1 was moved to point E on curve 2. If the vector $\overline{A_1B_1}$ (fig 3.3) is superimposed on point E, this point moves to point F on curve 4. Similarly, it was explained in section 3.2.5 how point D_1 was moved to E_1. Superimposing the vector $\overline{AB}$ on point E_1 moves the latter to point G on curve 4.

3.2.7 Systems of reinforcement (ultimate strength fields)

Good as the interaction diagram of fig 3.3 may be, it has an undesirable setback in that it was drawn for, and is valid only for the particular section of given parameters f_{cu}, f_y, ρ, ρ', and d'/d. When these parameters change, the curves will change. Hence if our design charts are to be given in the form of interaction diagrams a considerable number will be required. Advantage can however be taken of some properties of the interaction diagrams, namely that curve 1 and the slope of the line for yield in tension are independent of the above parameters, (see eqns 3.7, 3.13 and 3.21), and the slope line for yeild in compression depends only on the cover ratio d'/d, (eqn 3.25). Moreover, the slope lines sub-divide the interaction diagram into areas of specific systems of

reinforcement. These areas we may call Strength Fields. It will be more convenient therefore, if the design charts are presented in the form of strength fields instead of in the form of interaction diagrams. This point will now be illustrated with a discussion on how the strength fields (fig. 3.4) are constructed.

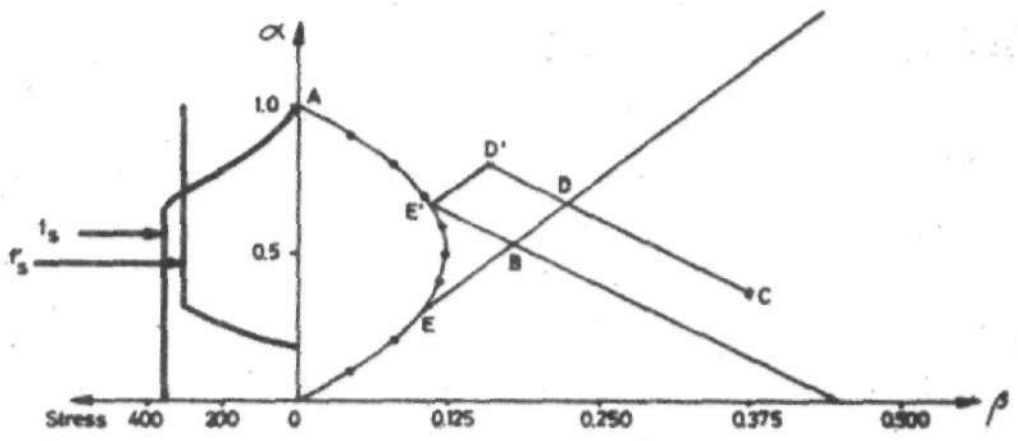


FIG. 3.4 ULTIMATE STRENGTH FIELD (d' = 0.1765 d)

For economic reasons reinforcement, if provided, should be used to maximum advantage, so it should only be provided in areas where its designed strength can be reached. Therefore in the area defined by the parabola

$$\beta_c = 0.5\alpha_c(1-\alpha_c) \text{ ie eqn 3.11,}$$

theoretically no reinforcement is required (except for small nominal amounts to provide a good vertical tie force for

stability, /3.4/). For the same reason compression reinforcement is to be used in the area above the line ED in fig. 3.4 starting from the plain concrete parabola at the point where the designed strength is first reached, that is at

$$\frac{x_c}{d} = \frac{d'}{d} \cdot \frac{\varepsilon_{cu} E_s}{(\varepsilon_{cu} E_s - f'_{sd})}$$ from eqn 3.14, (which in figs 3.3 and 3.4 is equal to 0.3052), and inclined at the angle

$\beta' = \tan^{-1}(0.5 - \frac{d'}{d})$ from eqn 3.25 (which in figs 3.3 and 3.4 is equal to $\tan^{-1}$ 0.3235). The equations above show that x_c/d and β' are both dependent on the cover ratio d'/d.

Similarly, the area below the line E'B in fig 3.4, starting from the parabola at

$$\frac{x_c}{d} = \frac{\varepsilon_{cu} E_s}{\varepsilon_{cu} E_s + f_{sd}}$$ from eqn 3.16 (which in figs 3.3 and 3.4 is equal to 0.6624), and inclined at the angle

$\beta = \tan^{-1}(-0.5)$ from eqn 3.21 requires tensile reinforcement only.

Within the area limited by these two lines both tensile and compression reinforcements are required, although not necessarily of equal amounts. For instance, when load conditions are represented by point C, (fig 3.4), the required reinforcement may be represented by CDE or CD'E' or any other line drawn between these two, limited only by yield conditions in both reinforcements. The segment DC or D'C, inclined at the angle θ to the α- axis, represents the tension reinforcement while ED or E'D', inclined at the angle θ' to the α- axis, represents the compression reinforcement, and their vertical components must satisfy the condition

$$\alpha = \alpha_c + \alpha'_s - \alpha_s, \text{ from eqn 3.5.}$$

The maximum value of x_c/d determined from eqn 3.14 is dependent on the values of d'/d and f'_{sd}. The steel bars in current use have characteristic strengths f_y in the range of, /3.5/:

$$250 \leq f_y \leq 485 N/mm^2 \qquad (3.26)$$

Also, it can be shown /2.1/ that as long as

$$\frac{d'}{d} \leq 0.21 \qquad (3.27)$$

the compression reinforcement in a doubly reinforced section reaches its designed yield strength 0.72 f_y at the ultimate

limit state. Hence x_c/d in eqn 3.14 may reach a value as high as

$$(x_c/d)max = (d'/d)max \cdot \frac{\varepsilon_{cu} E_s}{\varepsilon_{cu} E_s - (f'_{sd})max}$$

$$= 0.21 \cdot \frac{700}{700-(0.72x485)}, \text{ from eqns 3.26 and 3.27}$$

$$= 0.42$$

above which all the compression reinforcement reach the designed yield stress.

Similarly, x_c/d determined from eqn 3.16 depends on f_{sd}, and may be as low as

$$(x_c/d)min = \frac{\varepsilon_{cu} E_s}{\varepsilon_{cu} E_s + (f_{sd})max}$$

$$= \frac{700}{700+(0.87x485)}, \text{ from eqn 3.26}$$

$$= 0.62$$

below which all the tension reinforcement reach their designed yield stress.

We may therefore recommend as a safe value

$$x_c/d = 0.5$$

at which both tension and compression reinforcements reach their designed stresses, as shown in fig 3.5.

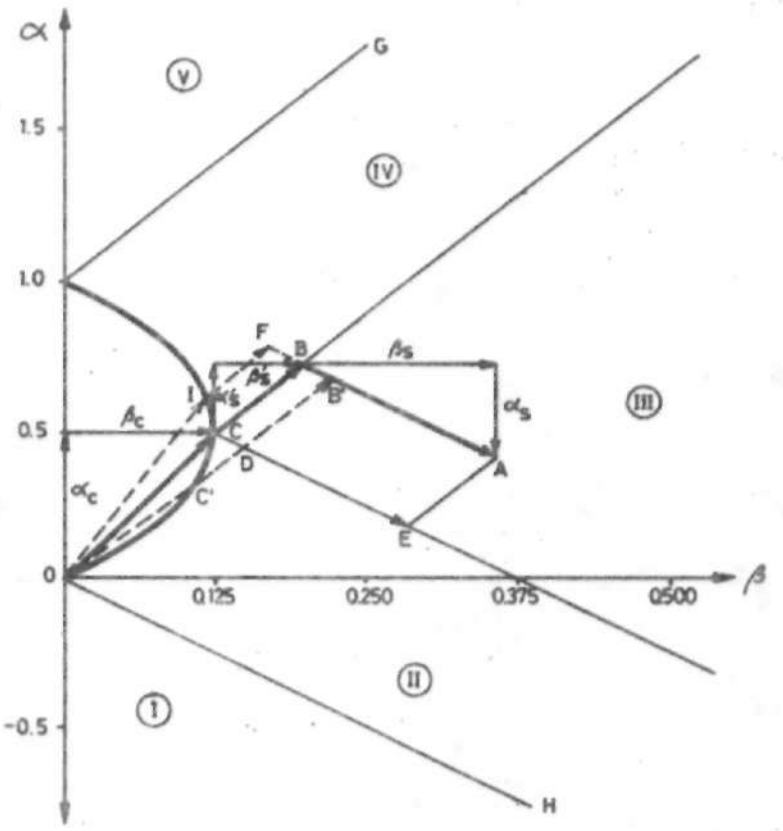


FIG. 3.5 RECOMMENDED ULTIMATE STRENGTH FIELD (d' = 0.1765 d)

2.2.8 The load condition represented by point A (fig 3.5) may be resisted by the concrete section with different reinforcements, for example that represented by the vectors $\overline{AB}$, $\overline{BC}$ and $\overline{CO}$, or that represented by vectors $\overline{AB'}$, $\overline{B'C'}$ and $\overline{C'O}$, as explained earlier. As can be seen from the figure however, the second option requires more steel than the first one, since the concrete contribution has been reduced.

Since the maximum concrete contribution to the bending resistance is reached at $x_c/d = 0.5$, there is no need to apply compression reinforcement for load conditions represented by points lying below the line CE. For these conditions, tension reinforcement is sufficient and the compression reinforcement may only reduce the concrete contribution to the moment of resistance. For instance, consider point E as lying just below the line CE. The required resistance of the section may be reached by applying tension reinforcement only, represented by vector $\overline{CE}$, or by applying both tension and compression reinforcement represented by vectors $\overline{DE}$ and $\overline{C'D}$ respectively. In the second option however, the tension reinforcement has been reduced by $\overline{CD}$ but compression reinforcement of $\overline{C'D}$ has been added.

As $\overline{C'D}$ is greater than $\overline{CD}$, the total reinforcement required in the second option is greater than in the first one to cater for the reduction of the concrete contribution to the moment of resistance.

Similarly, there is no need to apply tension reinforcement for load conditions represented by points lying above the line CB, which may increase x_c/d above 0.5 and thus decrease the concrete contribution to the moment of resistance, as may be seen by considering point B (like point E) as reinforced by $\overline{CB}$ or by $(\overline{IF} + \overline{FB})$.

Load conditions represented by points lying between the lines CB and CE requre both tension and compression reinforcement, (as was shown for point A above), with a constant $\alpha c = 0.5$.

Load conditions represented by points lying above the line 1G (which is inclined at the angle β' as the compression reinforcement line CB) require compression reinforcement on both faces of the concrete section, as the total effective concrete area is already fully exploited, ($\alpha c = 1$).
Similarly, for points lying below the line OH (which is inclined at the angle β as the tension reinforcement line CE), tension reinforcement is required on both faces of the section as the concrete takes no part, ($\alpha c = 0$) in resisting these tensile loads.

Hence, in relation to load conditions all reinforced concrete sections of linear elements can be distinguished into five groups viz:

Group 1, doubly reinforced sections in tension

Group 2, singly reinforced sections in tension

Group 3, sections with tension and compression reinforcements

Group 4, singly reinforced sections in compression

Group 5, doubly reinforced sections in compression.

These groups are shown in fig. 3.5.

3.2.9 Proposed design approach

3.2.9.1 For a given section, the strength and moment ratios (α and β) are first computed from the equations

$$\alpha = \frac{N}{0.4f_{cu}bd} \text{ and } \beta = \frac{M}{0.4f_{cu}bd^2}$$

afterwhich the section may be placed within one of the five groups in the ultimate strength field. Vectors are then drawn parallel to the boundary lines CE and/or CB (fig 3.5) to intersect the concrete parabola, and in such a way as to take maximum advantage from the concrete while at the same time using the least amount of reinforcement required, (as discussed in section 2.2.7). The vertical component of the vector parallel to the tension boundary line CE, and that

parallel to the compression boundary line CB, give the contribution of the tension and compression reinforcements (α_s and α'_s) respectively, to the axial load resistance.

The reinforcement ratios are then computed (from eqn 3.10) as

$$\rho = \frac{\alpha_s}{\varphi} \quad \text{and} \quad \rho' = \frac{\alpha'_s}{\varphi'},$$

or, since $\rho = \frac{A_s}{bd}$ and $\rho' = \frac{A'_s}{bd}$,

the amount of reinforcement is given as

$$A_s = \frac{\alpha_s bd}{\varphi} \quad \text{and} \quad A'_s = \frac{\alpha'_s bd}{\varphi'}, \text{ where}$$

$$\varphi = \frac{0.87 f_y}{0.4 f_{cu}} \quad \text{and} \quad \varphi' = \frac{0.72 f_y}{0.4 f_{cu}}$$

3.2.9.2 As an alternative to the use of the design charts above, design formulae can be derived for use for each of the five areas on the ultimate strength field as shown in ref 3.3.

3.3 Conclusions

3.3.1 When the ultimate strength is related to the effective depth by neglecting the concrete cover to the less compressed reinforcement, a nominal eccentricity e_1 equal to $0.5d_2$ is applied in the section, as shown below.

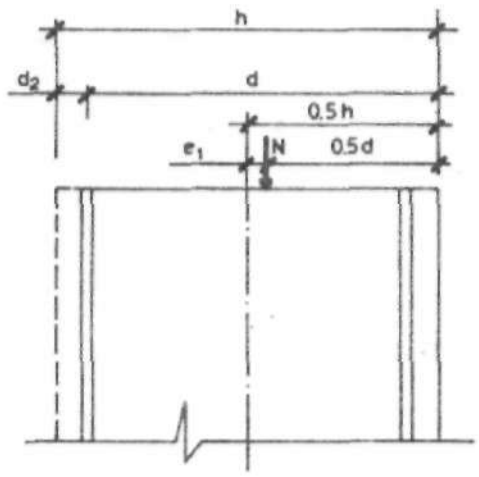


FIG. 3.6 NOMINAL ECCENTRICITY (PROPOSED)

From fig 3.6, $e_1 = 0.5(h-d)$

$= 0.5d_2$

Moreover, this nominal eccentricity is built-in into the design formulae and thus affects all the moments acting on the section. Hence the question of discontinuity discussed in section 1.2i, between sections which have computed eccentricities less than the minimum eccentricity, and those with computed eccentricities greater than or equal to the minimum eccentricity, no longer arises. Furthermore, the allowance of $0.5\ d_2$ corresponds to test results of nominal eccentricities.

3.3.2 The proposed design procedure is valid for concrete sections under combined bending and axial load, including pure flexure and pure axial tension as well as pure axial

compression. Furthermore, the ultimate strength field chart in fig 3.5 is valid for all strength ratios, and only the cover ratio d'/d restricts the use of each chart (see eqn 3.25). The CP code provides design charts using four values of cover ratios: 0.05, 0.10, 0.15 and 0.20. It we follow this example and utilize these values on the strength fields, and in addition specify the same cover to the tension and compression reinforcements, ie $d_2 = d'$, only four strength field charts need be provided (as given in figs 3.7 - 3.10), as against the 90 charts presently provided in the CP110 code for rectangular beams and columns.

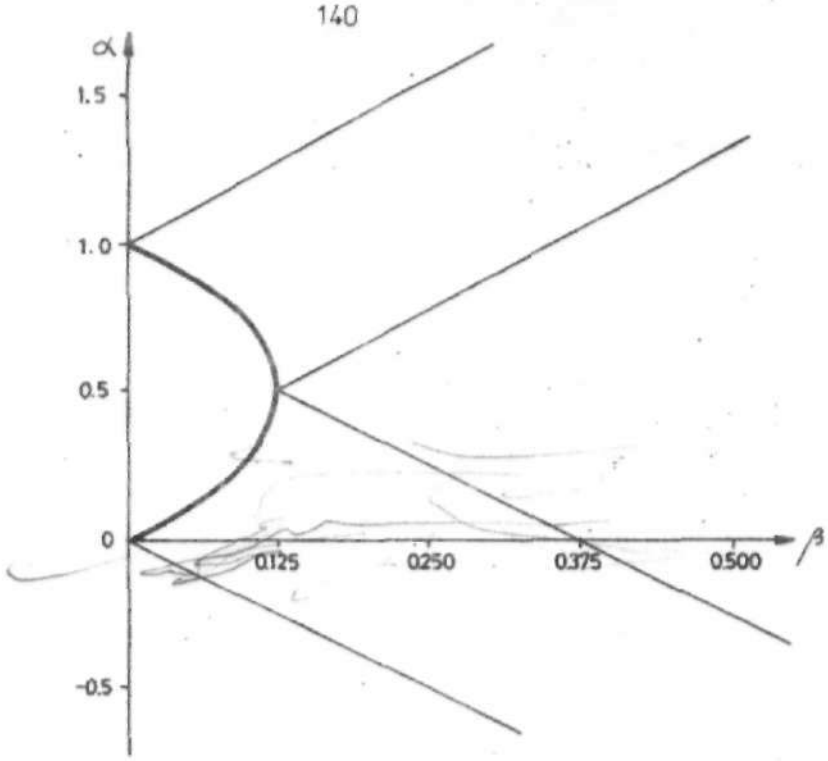


FIG. 3.7 RECOMMENDED ULTIMATE STRENGTH FIELD (d' = 0.05 d)

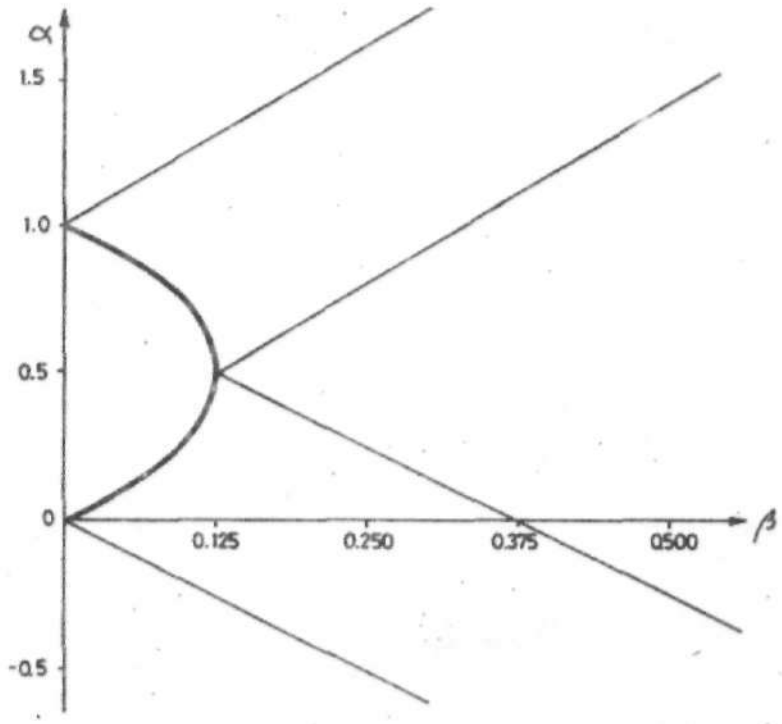


FIG. 3.8 RECOMMENDED ULTIMATE STRENGTH FIELD (d' = 0.10 d)

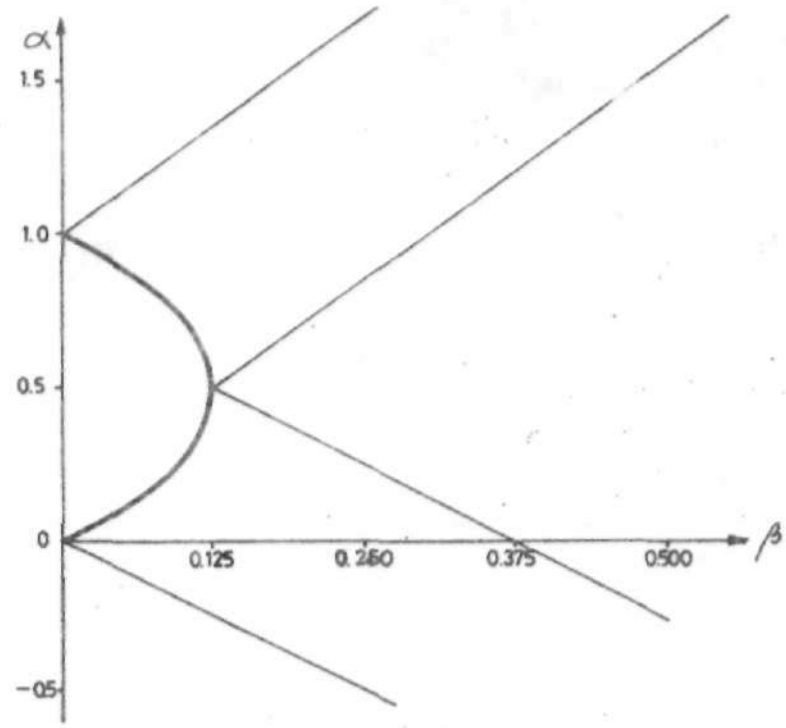


FIG. 3.9 RECOMMENDED ULTIMATE STRENGTH FIELD (d' = 0.15 d)

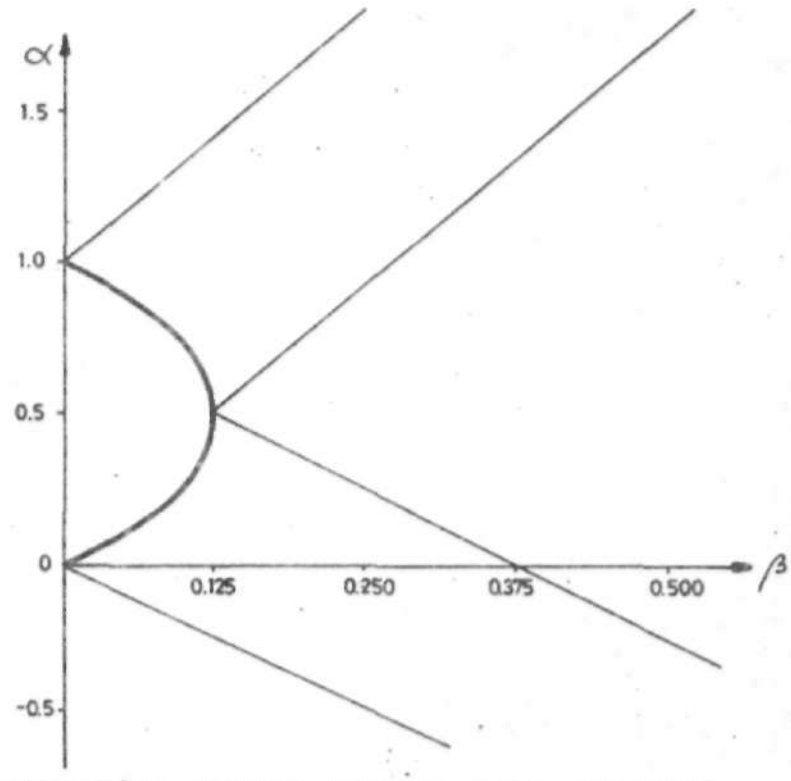


FIG. 3.10 RECOMMENDED ULTIMATE STRENGTH FIELD (d' = 0.20 d)

3.3.3 Once the α and β values are computed, the section can easily be placed within one of the five groups in the ultimate strength field, and the system of reinforcement required is immediately known. Moreover, for any system of reinforcement that is found applicable, the ultimate strength field chart enables us to see at a glance the various design options available, as well as telling us which option is the most economic, even before the section is designed.

3.3.4 In order to take maximum advantage of the concrete resistance, the maximum depth of compression zone in sections with tensile reinforcement is assumed as $x_c = 0.5d$ for both beams and columns, but this value can easily be changed if other values are required.

3.3.5 As can be observed from the table of the summary of site results, (Table 2.39), the proposed construction tolerance of $0.5d_2$ is in most cases greater than the CP110 allowance of 0.05h; in those cases where $0.5d_2$ is less than 0.05h, the site results show that the CP110 allowance is greater than that to be expected on the construction sites. The proposed tolerance therefore not only duplicates a closer representation of the actual construction errors, it also provides a greater degree of safety than does the CP110 tolerance.

3.3.6 Since a code should not be a repository for all the latest, most complex and fashionable methods of calculation, but should include methods which give sufficiently simple, realistic accurate representations of the actual behaviour of the structure to give a satisfactory balance between economy and design effort while ensuring safe and serviceable designs /3.1/, it is my belief that if the proposed nominal eccentricity is adopted, it will be a step forward towards achieving this objective.

3.3.7 As stated earlier, the field measurements were conducted at Zaria, Nigeria. It will be desirable to conduct similar measurements, firstly in other parts of Nigeria, and secondly in other developing countries. Comparison can then be made of the results to see whether there are any appreciable variations within this country, and between developing countries. The data so obtained should help towards the recommendation of suitable values for the minimum eccentricity when these countries draft their own Codes of Practice.

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