EXPERIMENTS ON BATTENED

COMPOSITE COLUMNS UNDER BIAXIAL BENDING

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by

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TABLE OF CONTENTS

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ABSTRACT

The subject of this thesis is concerned with the study of battened composite columns under biaxial bending .

In order to investigate the behaviour of the battened composite column under biaxial bending,four experiments were carried out .

In evaluating the experimental results,emphasis is placed on the load-carrying capacity,deflection,strains,and other features.

As expected,it was found that the load-carrying capacity of the tested columns is higher than that of the design ,also according to the deflections of the tested columns,it can be said that the columns behaved very well under loads .

Finally it was indicated that the steel channels have an excellent performance in resisting the applied stresses.

ACKNOWLEDGEMENTS

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LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

AN INTRODUCTION TO COMPOSITE COLUMNS

1.1 General :

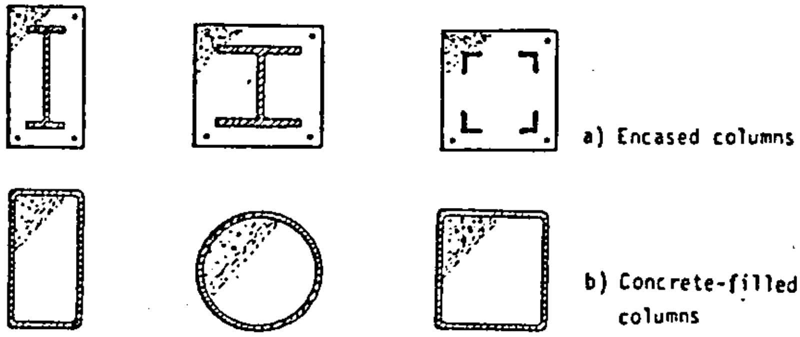
Composite columns of steel and concrete have been in use for several decades.

Most of the research in the behaviour of composite columns has dealt with both the concrete-encased and the concrete-filled columns .Two methods of design have now become available for the design of composite columns,namely the British Bridge Code BS(5400)[6],and the European recomendations for composite structures by the ECCS (EC4)[7], referred to hereafter as the Bridge Code and the ECCS method . These two methods of design are applicable to both the concrete-filled tubes and the concrete-encased steel sections .

In the case of concrete -encased steel sections,sufficient longitudinal and transverse reinforcement are required to ensure full composite action up to failure . In the case of concrete-filled hollow steel sections , no reinforcement is needed to ensure the composite action between the structural steel and the concrete elements which provides some economic and performance advantages.

1.2 The Battened Composite Column

The choice of the cross-section of the column is restricted in practice by the current experience in design and construction , and only two common types of composite columns are in use , one is the encased column , and the other is the concrete-filled tubular column . Figure (1.1) shows some conventional types of composite columns .



Figure(1.1): Typical cross-sections of composite columns

The battened composite column is shown in section in Figure (1.2). The structural steel work of the column consists of two channels joined together by means of welded batten plates to form the rectangular shaped cross-section . The interior of the column is then filled with in-situ concrete . This in-situ concrete combines with the in-situ concrete of the beam to join the whole into a rigid-jointed frame .

1. Steel channels
2. End and intermediate batten plates
3. In-Situ concrete

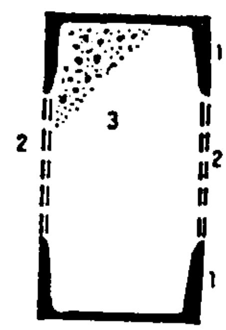


Figure 1.2: Section in battened composite columns

Compared to the traditional composite columns , the battened composite column offers the following advantages :

1. The shuttering required is of a simple and cheap form .
2. No reinforcement cage is required .
3. The structural steel is placed on the outside where it is most needed .
4. The spacing between the channels provides easy access to the inside core , and hence makes for easy beam-column connection . This space also allows for pipes (mechanical and electrical works) to go through the column in cases where such details are necessary preferable.

1.3 Previous Research

Early in the 1950's ,Faber[8], Stevens[15] and others carried out tests on encased stanchions to investigate the strengthening effect of the concrete encasement and this led to the cased-strut method of design which was incorporated in the 1959 edition of BS449[5].

Research on improved methods of design of composite columns has been undertaken since the early 1960's . Jones and Rizk [9] carried out a series of tests on encased stanchions .

The following conclusions were reported :

1) Steel strains observed in the building frame designed according to the current practice (BS449)indicated the conservative nature of the design practice for encased columns.

2)Under working loads the calculated strains based on the elastic theory for reinforced concrete could be adopted with the advantage of using a modular ratio m = 10 ;with the full considaration of the concrete section .

3)The calculated values for the ultimate load of encased columns when compared to test results showed that the reduced buckling coefficients calculated from BS449 recomendations were very conservative .

4)The slenderness ratios of the columns tested were in the range L/r = 100 and the columns showed no sign of buckling failure , while (BS 449) starts buckling reductions for L/r = zero . This supported the view of using a completely different buckling formula other than that given in (BS 449) .

Stevens [16] carried out further research to investigate the general behaviour, modes of failure, and the ultimate loads of encased stanchions . The following conclusions were reported :

1. The strength of a short encased stanchion in axial compression is safely given by the summation of the products of the steel section with its yield strength , and the cross-sectionl area of the concrete section with 0.67 fcu .

2. The reduction in strength of axially loaded encased stanchions with increasing slenderness is similar to that of reinforced concrete columns .

3. There is a simple interaction relationship between the strength of encased stanchions under axial and under eccentric loads . A simple empirical formula can be used to relate them .

In 1969 Basu and Sommerville [3] published a method based on the Perry-Roberston type of buckling curve, applicable to pin-ended composite columns with equal or unequal end eccentricities. The conclusions of this study were as follows :

1)In calculating the moment carrying capacity of the section , a constant compressive stress of 0.45 fcu acting over the area of concrete above the plastic neutral axis was shown to be jutified.

2) A long-term load was considered as a load of three day´s duration or longer .

3) An overall load factor of 1.7 was assumed to be adequate .

4) The strength of a composite column under biaxially eccentric load may be estimated from its failure loads under uniaxial bending about the major and minor axes with the help of an interaction formula proposed in the paper .

Moreover , Virdi and Dowling [18] carried out a study on encased composite columns to investigate the ultimate strength of these columns under biaxial bending . The following observations and conclusions were reported :

1) Initial imperfictions can be adeqately accounted for by considering residual stresses combined with a lack of straightness .

2)The test results showed predictable loss of strength with increasing lengths and applied eccentricities .

3) Recorded strains showed that there exists a continuity of strain between concrete and steel .

A search of literature indicats that no work had been carried out on in-filled battened composite columns , until an exploratory research was carried out at the University of Manchester by Taylor ,Shakir-Khalil and Yee [17,20,19] . Also research was carried out on the same topic at the University of Manchester by Hunaiti and Shakir Khalil[13].The conclusions of the above research were :

1. The ultimate loads of the columns were always well in excess of the values estimated using the design expressions.
2. The Bridge Code Method proved to be very conservative .
3. The first sign of damage to the concrete occured only at loads well in excess of loads corresponding to the service load .

1.4 Objectives of Study

It has been indicated before that no research (apart from that conducted at the university of Manchester)has been carried out to investigate the behaviour of the battened composite column considered in this study .

The aim of my research was to provide experimental data and to achieve a good understanding of the behaviour of the battened composite column . In this respect four tests were conducted (data is given in next chapter) , to investigate the behaviour of this column under biaxial bending . Emphasis will be placed on the load-carrying capacity , deflections , strains and other features .

CHAPTER 2

DESIGN OF COMPOSITE COLUMNS

Inelasticity of material and the P - Δ effects make both the mathematical solution and the method of design (assuming an exact analysis) of a composite column a complicated problem . Empirical methods of design have thus been used .

Continuous research to develop empirical methods of design yielded a method of design adopted by the British Bridge Code (BS 5400) [6], and the same approach was adopted by the European convention for construction steel work ECCS(EC4)[7] .

2.1 The Bridge Code Method

2.1.1 Columns under Axial Load

The load carrying capacity of a composite column is a function of its squash load ,N_u,its ultimate moment of resistance,M_u, and also of the concrete contribution factor , α . The squash load,N_u,is defined as the ultimate short-term axial load for a short column, and is given by:

$$N_u = A_s . f_{sd} + A_c . f_{cd} + A_r . f_{rd} \qquad (2.1)$$

in which

$$f_{sd} = f_{sk} / \gamma_{ms} \tag{2.2}$$

$$f_{cd} = f_{ck} / \gamma_{mc} \tag{2.3}$$

$$f_{rd} = f_{rk} / \gamma_{mr} \tag{2.4}$$

where

A_s, A_c, A_r = The area of the structural steel section, area of concrete core, and the area of reinforcement respectively .

f_{sd}, f_{cd}, f_{rd} = The design strengths of the structural steel, concrete and reinforcement respectively .

f_{sk}, f_{ck}, f_{rk} = The characteristic strengths of structural steel, concrete and reinforcement respectively .

γ_{ms}, γ_{mc}, γ_{mr} = Material partial safety factors of structural steel, concrete, and reinforcement respectively .

The value of fck in equation (2.3) is given as :

$$f_{ck} = 0.67 f_{cu} \tag{2.5}$$

where

f_{cu} = the characteristic 28-day cube strength of concrete .

It should be mentioned that for the purpose of comparison between short-term test results and calculated load-carrying capacities , the value of f_{ck} in equation (2.3) should be taken as : $f_{ck} = 0.83 f_{cu}$ (2.6)

In applying the material partial safety factors($\gamma ms=1.1$, $\gamma mc=1.5$, $\gamma mr=1.15$), the squash load ,N_u,is given by :

$$N_u = 0.91 A_s . f_{sk} + 0.45 A_c . f_{cu} + 0.87 A_r . f_{rk} \quad (2.7)$$

For a slender axially loaded composite column .Where either of the ratios L_x/h or L_y/b exceed 12(h and b are the greatest and least - lateral dimensions of the concrete in the cross section of the composite column,while $L_x$and L_y are the effective lengths in x,y directions) account should be taken of the eccentricity due to construction tolerances by considering the column in uniaxial bending about the minor axis,and the ultimate load carrying capacity should be calculated with a minimum eccentricity of e = 0.03B ,where B is the least lateral dimension , of the column .

For a short axially loaded composite column , where the ratios L_x/h and L_y/b do not exceed 12, the ultimate load carrying capacity of the column , N_{ay} , is given by :

$$N_{ay} = 0.85 K_{1y} . N_u \quad (2.8)$$

where

K_{1y} = Reduction factor which depends on the slenderness of the column as determined below , using the parameters appropriate to the minor axis .

N_{ay} = The ultimate load carrying capacity of the column about y-axis.

The value of K_1 is determined by using the appropriate buckling curve (or table)for bare steel column (Figure 2.1 from the ECCS recomendations) [7] . The choice of the curve to be used depends on the type of the structural steel section in the composite column . Table (2.1) shows the column buckling curve selection chart as adopted by the Bridge Code .

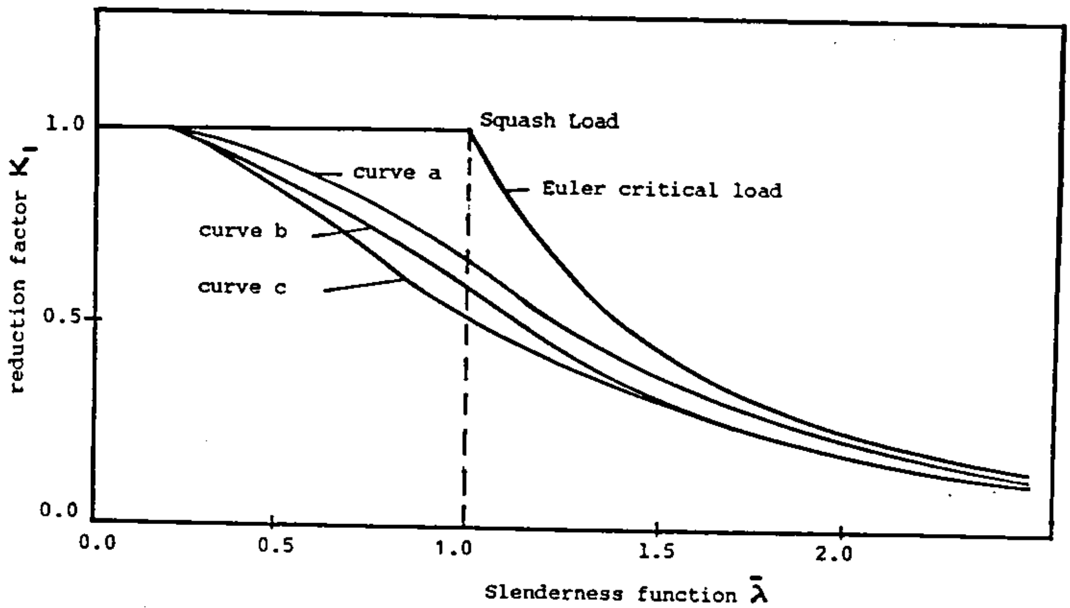
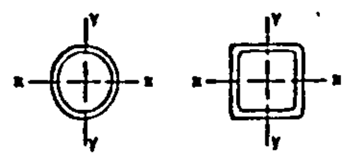
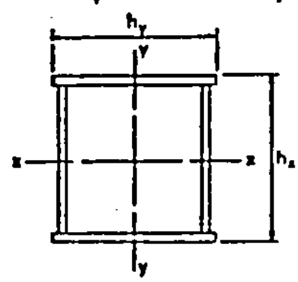
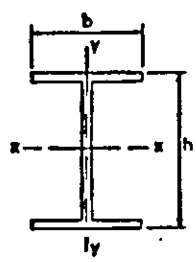
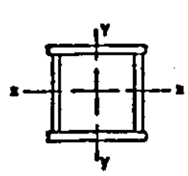
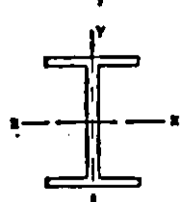


Figure 2.1 European Buckling curves for bare steel columns.

In addition to the curves and tables , the value of K_1 could be calculated from a formula . Before introducing this formula ,it is appropriate to introduce the main factor affecting the value of K_1 which is the slenderness of the column . The slenderness parameter,(also known as the slenderness function, $\bar{\lambda}$, or the equivelent slenderness ratio)of the column is given by the formula : $\bar{\lambda} = L_e/L_c$ (2.9)

Table 2.1 : Column Buckling Curve Selection Chart (Ref. BS 5400)

Shape of steel section		*Curve*	*Table*
	Rolled tubes Welded tubes (hot finished)	a	13.1
	Welded box sections x-x, h_x y-y, h_y	b	13.2
	Rolled sections* Buckling about x-x axis		
	$h/b > 1.2$	a	3.1
	$h/b \leq 1.2$	b	13.2
	Buckling about y-y axis		
	$h/b > 1.2$	b	13.2
	$h/b \leq 1.2$	c	13.3
	Welded sections Buckling about x-x axis		
	(a) flame cut flanges (b) rolled flanges	b	13.2
	Buckling about y-y axis		
	(a) flame cut flanges	b	13.2
	(b) rolled flanges	c	13.3
	Box sections stress relieved by heat treatment	a	13.1
	I and H sections, stress relieved by heat treatment		
	Buckling about x-x axis	a	13.1
	Buckling about y-y axis	b	13.2

* Not applicable to sections where the flange thickness exceeds 40 mm

where

L_e = The effective length of the actual column in the plane of bending considered depending on the end restraint.

L_c = The critical length of the column for which its squash load, N_u, equals its Euler critical load N_{cr} .

The critical load, N_{cr} , is given by :

$$N_{cr} = \pi^2.(E_c.I_c + E_s.I_s + E_r.I_r)/L_e^2 \quad (2.10)$$

Thus, the critical length, L_c , is given by :

$$L_c = \pi.\{(E_c.I_c + E_s.I_s + E_r.I_r)/N_u\}^{1/2} \quad (2.11)$$

where

E_s, E_c, E_r = Elastic modulus of structural steel, concrete and reinforcement respectively .

I_s, I_c, I_r = The second moment of area of structural steel, uncracked concrete and reinforcement respectively .

In calculating the value of the critical length, L_c , the modulus of elasticity of concrete Ec should be taken as :

$$E_c = 450f_{cu} \quad (2.12)$$

But ,for the purpose of comparison between short-term test results and calculated load-carrying capacities, the value of E_c in equation (2.12) should be taken as :

$$E_c = 5000.(f_{cu})^{1/2} \quad (2.12a)$$

where

f_{cu} = the characteristic 28-day cube strength of concrete (in formula 2.12a).

Having obtained the values of L_e and L_c, the equivalent slenderness ratio is then determined and the value of K_1 is thus given by :

$$K_1 = 1/2[1+(\eta+1)/\bar{\lambda}^2] - \{1/4.[1+(\eta+1)/\bar{\lambda}^2]^2 - 1/\bar{\lambda}^2\}^{1/2} \quad (2.13)$$

in which

$$\eta = \psi . \lambda_E (\bar{\lambda} - 0.2) \quad (2.14)$$

and

$$\lambda_E = \pi . \{E_s/f_{sd}\}^{1/2} \quad (2.15)$$

where

ψ = Imperfection factor which depends on the geometry of the steel section, the magnitude and disposition of residual stresses and the initial imperfections.

The value of the coefficient, ψ , is determined from table 2.2 . Excluding the value, ψ = 0.008, and comparing with Table 2.1, it seems that there is a specific value of ψ for each of the European buckling curves; i.e.

ψ = 0.0020 for curve "a"
= 0.0035 for curve "b"
= 0.0055 for curve "c"

Table 2.2. : Values of coefficient ψ

Type of steel section	Axis of buckling	Value of ψ
Rolled I (UB, Joist etc.)	x-x	0.0020
	y-y	0.0035
Rolled H (UC etc.) flanges up to 40 mm	x-x	0.0035
	y-y	0.0055
Rolled H (UC etc.) flanges over 40 mm	x-x	0.0055
	y-y	0.0080
Welded I or H flanges up to 40 mm	x-x	0.0035
	y-y	0.0055
Hot-rolled structural hollow sections	any	0.0020

* Retyped from the Bridge Code

2.1.2 Columns with end moments

The ultimate load-carrying capacity of an axially loaded column is given by :

$$N_k = K.N_u \quad (2.16)$$

where

K = Reduction factor which depends on K_1 for an axially loaded column , the material and section properties.

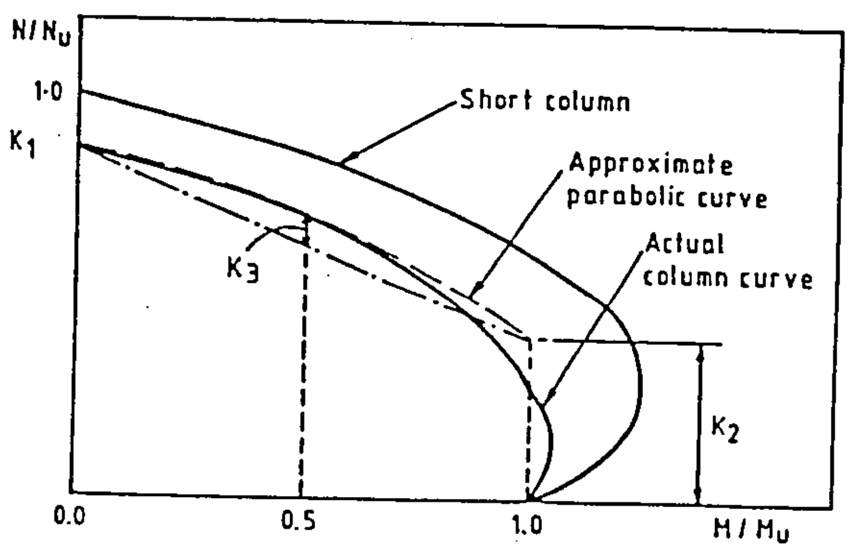
To compute K we must compute (α) ,(the concrete contribution factor) which is given by :

$$\alpha = A_c . f_{cd} / N_u \quad (2.17)$$

where A_c, f_{cd} = as defined in the previous section.

The values of the moment and axial compression load calculated from the stress block (for each position of the neutral axis) will give the points to construct the interaction curve .

The actual column curve, shown in Fig(2.2), can be obtained by inelastic analysis [1,2] . Basu and Sommerville [3] studied many interaction curves and suggested the use of an approximate parabolic curve .



Figure(2.2) : Typical interaction graphs for composite columns

2.1.2.1 Columns under uniaxial bending

The ultimate load-carrying capacity of a column under uniaxial bending about the appropriate axis is given by :

$$N_k = N_u \{K_1 - (K_1 - K_2 - 4K_3)M/M_u - 4K_3(M/M_u)^2\} \quad (2.18)$$

where

K_1 = As for axially loaded columns about the appropriate axis .

K_2, K_3 = As determined below .

M = The maximum applied moment acting about the appropriate axis .

M_u = The ultimate moment of resistance of the column section about the appropriate axis .

The values of the coefficient ,K_2,about the appropriate axis for concrete encased steel sections and concrete filled rectangular tubes are determined as follows :

$$K_2 = K_{20} \{[90-25(2\beta-1)(1.8-\alpha)-C_4 \cdot \bar{\lambda}]/30\ (2.5-\beta)\} \quad (2.19)$$

where

β = The ratio of the smaller to the larger end moments acting about the appropriate axis , — being positive for single curvature bending .

$\bar{\lambda}$, α = As defined in the previous sections .

C_4 = A constant taken as :

100 for columns designed to curve "a"

120 for columns designed to curve "b"

140 for columns designed to curve "c"

and

$$K_{20} = 0.9\,\alpha^2 + 0.2 \tag{2.20}$$

The value of K_2, as clculated above ,should lie between the following limits :

$$0 \leqslant K_2 \leqslant K_{20} \tag{2.21}$$

and $K_{20} \leqslant 0.75$

and if K_2 is negative ,it should be taken as zero.

The values of the coefficient ,K_3, are as follows :

For major axis bending :

$$K_{3x} = 0$$

For minor axis bending :

$$K_{3y} = 0.425 - 0.075\beta_y - 0.005\, C_4 . \bar{\lambda}_y \tag{2.22}$$

and should be taken between the limits :

$$(0.2-0.25\alpha) \geqslant K_{3y} \geqslant -0.03(1+\beta_y)$$

Equation (2.18) is only valid if :

$$1/K2 > N_u . e / M_u$$

If the above condition is not satisfied , equation (2.18) becomes :

$$N_k = M_u / e \tag{2.23}$$

where e = the eccentricity of the applied load about the appropriate axis

For a column subjected to uniaxial bending about the major axis and unrestrained against failure about the minor axis , the ultimate load-carrying capacity should be taken equal to that of a column subjected to biaxial bending with the minimum applied moment about the minor axis ; i.e : $M_y = 0.03\ B.N_k$.

2.1.2.2 Columns under biaxial bending

For this type of columns , the ultimate load-carrying capacity is :

$$N_{kxy} = 1/ \{1/N_{kx} + 1/N_{ky} - 1/N_{kax}\} \qquad (2.24)$$

where

N_{kx}, N_{ky} = The ultimate load-carrying capacities of the column under uniaxial bending about the major and the minor axis respectively,as calculated in section 2.1.2.1,

and $N_{kax} = K_{1x} . N_u$ (2.25)

where

K_{1x} = The K_1 coefficient as determined from section 2.1.1,with parameters appropriate to the major axis.

2.2 The ECCS Method

The method adopted by the ECCS(EC4), is almost identical to the Bridge Code Method,but there are few differences between the two methods :

1. For columns designed to curve "c", the value of the constant C_4 is taken as equal to 140 by the Bridge Code Method,and is equal to 135 by the ECCS(EC4) Method.

2. In the ECCS Method the value of the elastic modulus of concrete, E_c, is given by $E_c = 600 f_{ck}$, but by the Bridge Code, $E_c = 450 f_{cu}$.

3. The formula of the coefficient, K_1, by the ECCCS is given by :

$$K_1 = (1+\bar{a}(\bar{\lambda}^2 - 0.04)^{1/2} + \bar{\lambda}^2)/2.\bar{\lambda}^2 - \{[1+\bar{a}(\bar{\lambda}^2 - 0.04)^{1/2} + \bar{\lambda}^2] - 4\bar{\lambda}^2\}/2\bar{\lambda}^2 \quad (2.2ECCS)$$

where $\bar{a}$ = 0.158 for curve "a"

= 0.281 for curve "b"

= 0.384 for curve "c"

The values calculated by the Bridge Code Method are always very close to the values calculated by the ECCS Code due to above mentioned slight differences .

2.3 Design of the Battened Composite Column

2.3.1 Definitions

2.3.1.1 Axes of the column section

The axes of traditional types of composite columns are usally the same as those of the steel section .

The axes of the battened composite column will be as shown in Figure (2.3)

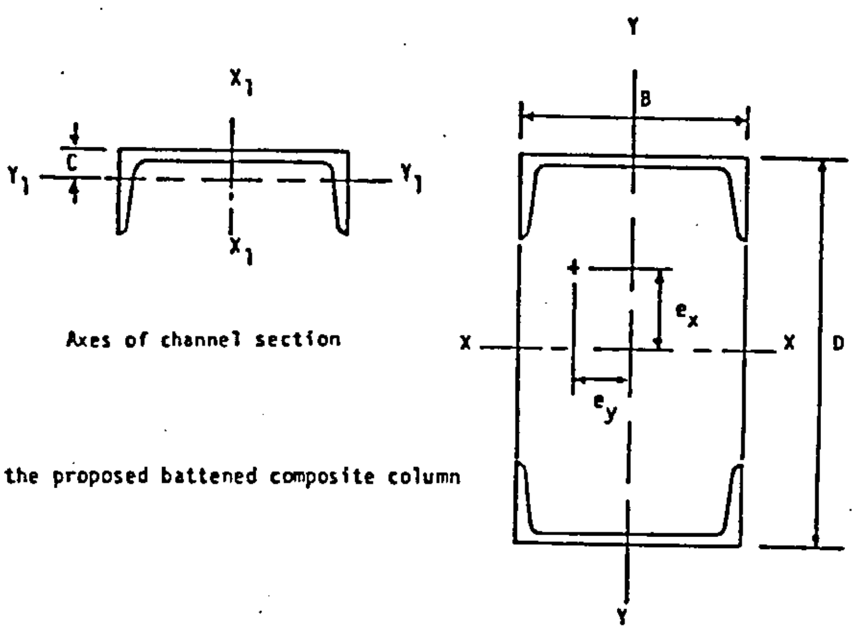


Figure 2.3 : Axes of the proposed battened composite column

2.3.1.2 Column buckling curve

To calculate coefficient ,K_1 , column buckling curve "a" is to be used . And equation (2.13) could be used with an imperfection factor, ψ equal to 0.002 .

2.3.1.3 Second moment of area

a. minor axis

The moment of inertia of the steel channels,I_{sy} ,is given by:

$$I_{sy} = 2I_{x_1x_1} \qquad (2.26)$$

where

$I_{x_1x_1}$ = The gross moment of inertia of each steel channel about its major axis .

The moment of inertia of the concrete section,I_{cy} ,is given by :

$$I_{cy} = \{D.B^3/12\} - I_{sy} \qquad (2.27)$$

where

B,D = The external dimensions of the column section .

b. major axis

The moment of inertia of the steel channels,I_{sx} ,is given by :

$$I_{sx} = 2\{I_{y_1y_1} + A_{ss}(D/2 - C)^2\} \qquad (2.28)$$

where

$I_{y_1y_1}$ = The moment of inertia of each channel about its minor axis .

A_{ss} = The cross-sctional area of each channel .

C = The centroidal depth of the channel , measured from the outer face of the web .

The moment of inertia of the concrete section , I_{cx} , is given by :

$$I_{cx} = (B.D^3/12) - I_{sx} \quad (2.29)$$

2.3.1.4 Ultimate moment of resistance

The ultimate moment of resistance of the column section is determined by considering equilibrium across a fully plastic section .

a. Minor axis

Figure (2.4) shows the stress block diagram for the battened composite column with idealised channel sections in bending about the minor axis .

The forces shown in Figure(2.4) are given as follows :

$$\begin{aligned} F_s &= 2A_{ss}.f_{sd} && (a) \\ F_c &= D.d.f_{cd} && (b) \\ F_1 &= 2d.t_w(2f_{sd} - f_{cd}) && (c) \\ F_2 &= 2T_1(b-t_w)(2f_{sd} - f_{cd}) && (d) \\ F_3 &= T_2(b-t_w)(2f_{sd} - f_{cd}) && (e) \end{aligned} \quad (2.30)$$

where

t_w = Thickness of the web of the steel channels .

T_1 , T_2 = As shown in Figure (2.4), The depth of the neutral axis ,d, is :

$$d = \{2A_{ss}.f_{sd} - [(b-t_w)(2T_1 + T_2)(2f_{sd} - f_{cd})]\}/\{D.f_{cd} + 2t_w.(2f_{sd} - f_{cd})\} \qquad (2.31)$$

The ultimate moment of resistance of the section is the sum of the moments due to the steel and concrete and is given by :

$$M_{uy} = A_{ss}.f_{sd}(B-d) + F_2(d-T_1) + F_3(d/2-T_1-T_2/3) \qquad (2.32)$$

where

F_2 = as given in equation (2.30) (d)

F_3 = as given in equation (2.30) (e)

b. Major axis

Figure (2.5) shows the stress block diagram for the battened column with idealised channel sections in bending about the major axis.

The distance, C_i ,in Figure(2.5) is the idealised value of $\bar{C}$ (equation(2.28)) .

The calculations of the ultimate moment of resistance, M_{ux} ,by neglecting the contribution of concrete is as :

$$M_{ux} = A_{ss}.f_{sd}(D-2C) \qquad (2.33)$$

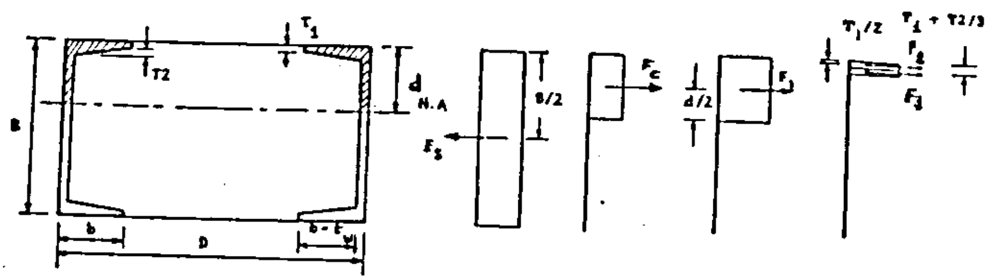


Figure 2.4: Stress Block Diagram of the battened composite column in minor axis bending.

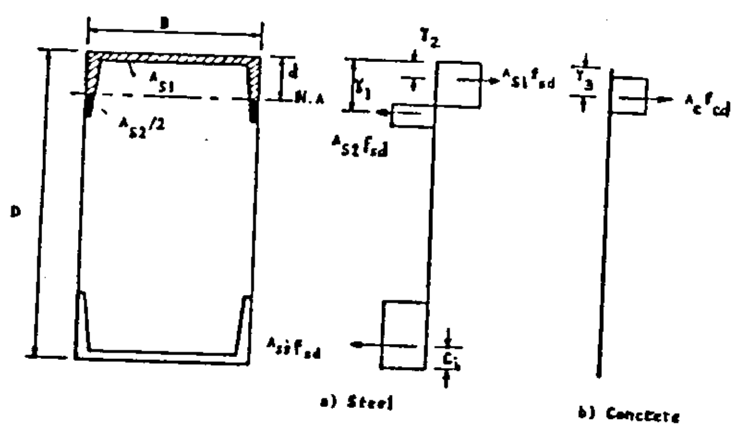


Figure 2.5: Stress Block Diagram of the battened composite column in major axis bending.

2.3.2 Design of the battened column in minor axis bending

For calculating the ultimate load, N_k, for an individual case, equation(2.18) (in which $M = N_k . e$) cannot be used as presnted and should be solved for N_k . The solution of equation (2.18) for the case of minor axis bending is given by :

$$N_{ky} = \{ - K_4 + [K_4^2 + 16K_{1y} . K_{3y} (N_u . e_y/M_{uy})^2]^{1/2} \} . N_u / \{8 \cdot K_{3y} (N_u \cdot e_y / M_{uy})^2 \} \quad (2.34)$$

in which

$$K_4 = 1 + (K_{1y} - K_{2y} - 4 \cdot K_{3y})(N_u . e_y/M_{uy}) \quad (2.35)$$

where

e_y = The eccentricity about the minor axis, as shown in Figure (2.3).

N_u, M_{uy} = As defined previously .

2.3.3 Design of the battened column in major axis bending

The solution of equation (2.18) for calculating the values of N_{kx} for individual cases becomes :

$$N_{kx} = K_{1x} . N_u / \{1 + (K_{1x} - K_{2x})(N_u . e_x/M_{ux})\} \quad (2.36)$$

where

e_x = The eccentricity about the major axis, as shown in Figure (2.3) .

2.3.4 Design of the battened column in biaxial bending

For columns subjected to biaxial bending, the ultimate load is calculated by using equations (2.24), (2.25) .

$$N_{kxy} = K_{xy} . N_u \tag{2.37}$$

in which

$$K_{xy} = 1/\{1/K_x + 1/K_y - 1/K_{1x}\} \tag{2.38}$$

where

K_x, K_y = As defined in equation (2.16) .

K_{1x} = As defined in equation (2.25) .

The interaction curves for a slender column in biaxial bending are obtained by considering the two uniaxial bending cases (i.e. minor and major axis bending) separately and then combining them by means of equation (2.24) or (2.38) .

The reduction factor, K, (equation(2.16)), is equal to K_{2x} and K_{2y} at M/M_u = 1.0 , for major and minor axis bending respectively . In the case of biaxial bending the reduction factor, K_{xy} at M/M_u = 1.0, will be K_{2xy}, and given by :

$$K_{2xy} = 1/\{1/K_{2x} + 1/K_{2y} - 1/K_{1x}\} \tag{2.39}$$

Figure (2.6) shows the three-dimensional diagram by which the biaxial interaction envelope is obtained .

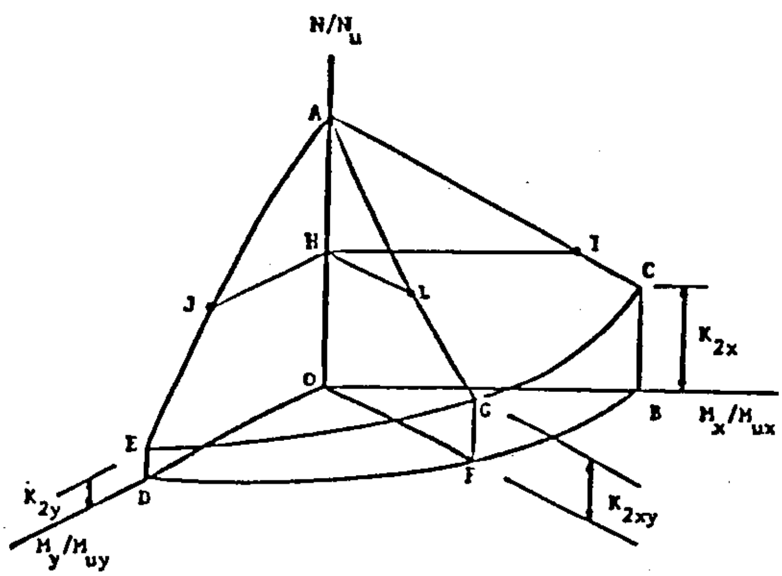


Figure 2.6 : Interaction surface for columns failing in biaxial bending.

The interaction curve, OAGF, for a column failing biaxially is obtained by rotating the curve, OACB, about the axis, OA, in a decreasing manner so that when the base, OB, coincides with OD, the curve, OACB, is reduced to OAED . It can be seen from the Figure that K_{1xy} is always equal to K_{1y} and K_{2xy} is greater than K_{2y} but less than K_{2x}. (equation(2.39)) .

To determine any point L (K_{xy}, $(M/M_u)_{xy}$) on the biaxial interaction curve, a similar procedure as for uniaxial bending can be used . Load ratio intervals are to be assumed and the uniaxial values of M/M_u , obtained then combined by means of equation (2.38) to give the ratio of $(M/M_u)_{xy}$, (i.e. line HL)

Figure (2.7) shows the interaction curve for a column subjected to biaxial bending, obtained by the above mentioned procedure .

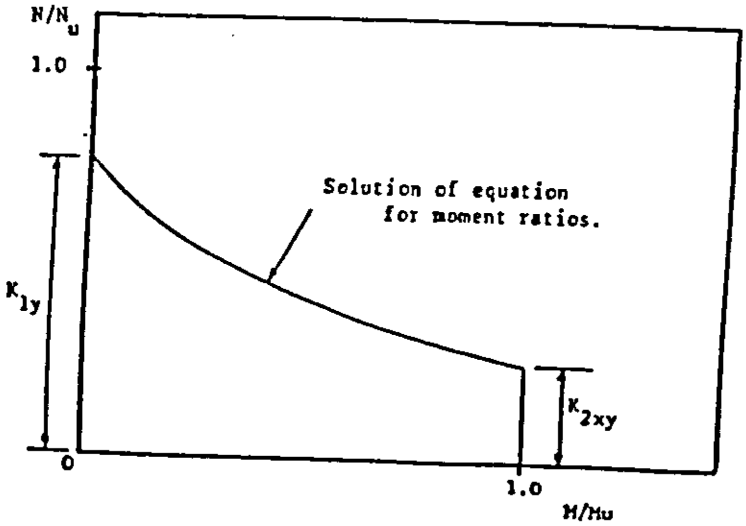


Figure 2.7 : Interaction curve for columns in biaxial bending.

* Interaction curves for short columns

Figures (2.8) and (2.9) show the interaction curves for short columns (using 254x76x8.5mm and 432x102x13.5mm steel channels).

Figure (2.10) is almost identical to the interaction curves for minor axis bending for concrete filled tubes , Figure (2.11) gives interaction curve for major axis bending (from Ref.[13]) .

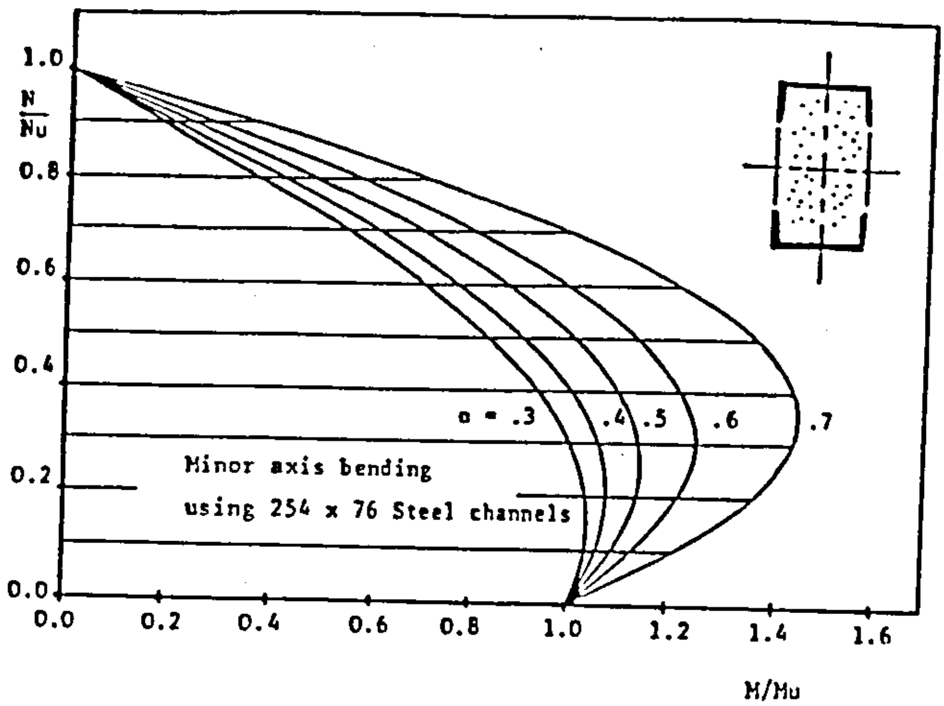


Figure 2.8: Interaction curves for short columns.

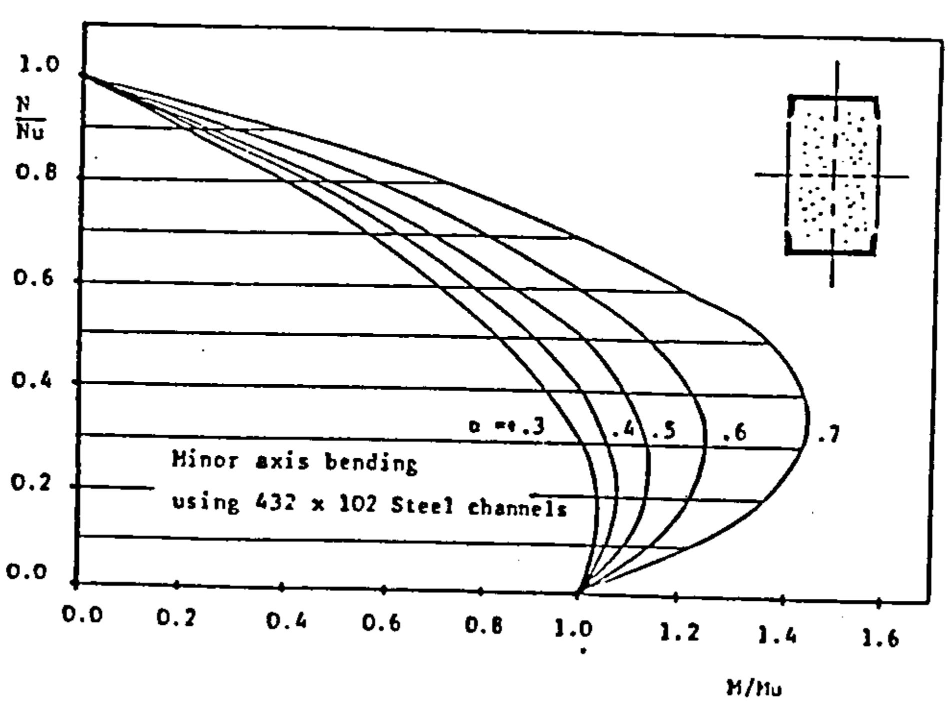


Figure 2.9: Interaction curves for short columns

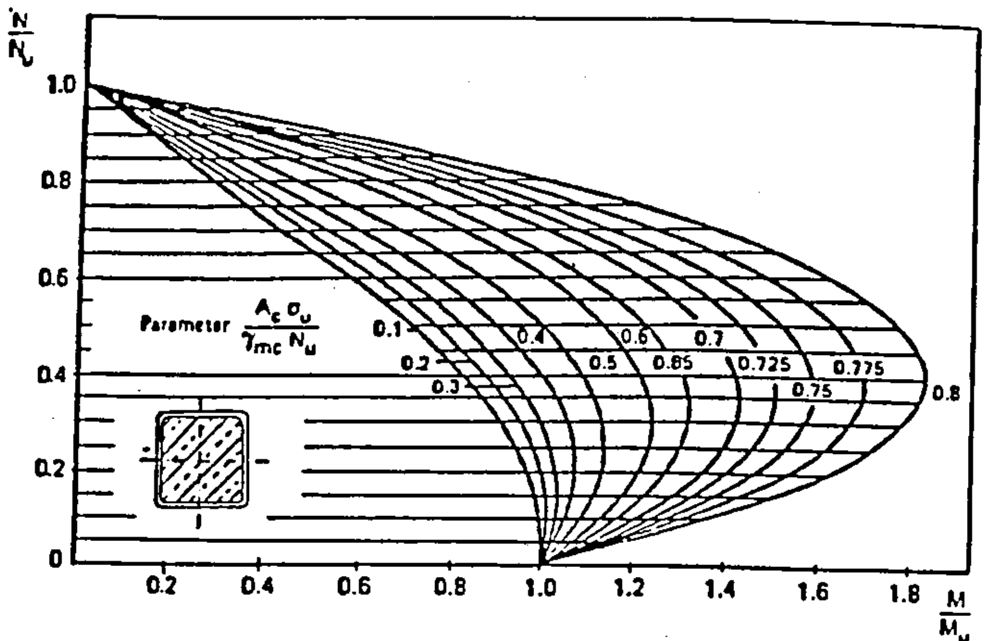


Figure 2.10 : Short column interaction curves for concrete filled tubes.

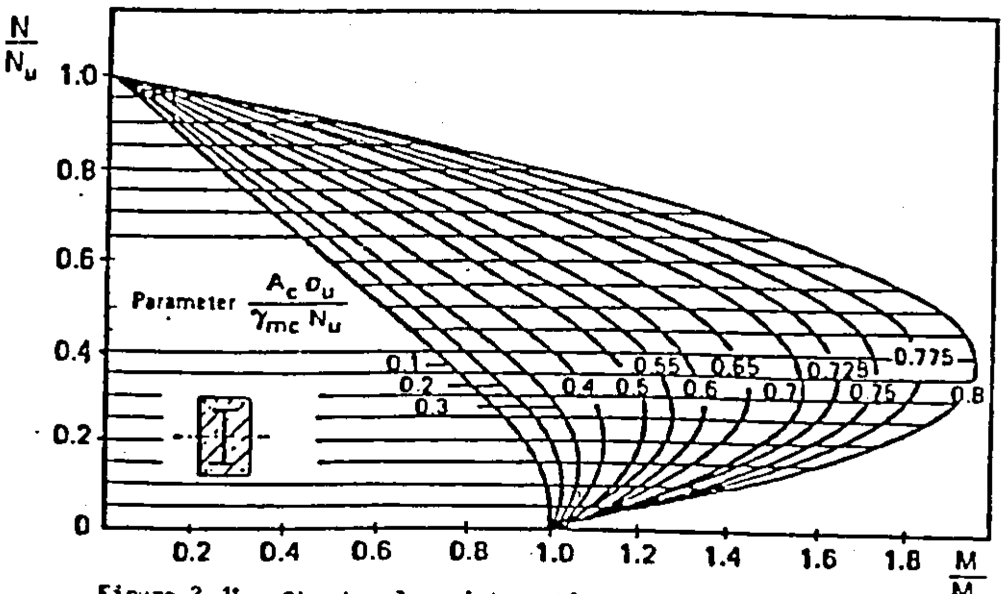


Figure 2.11 Short column interaction curves for encased section - major axis bending.

CHAPTER 3

DETAILS OF COLUMN TESTS

Relatively,little reasearch on the battened type of composite columns,can be found in the literature (Shakir Khalil,Yee,Taylor And Hunaiti)[13,17,20] . It should be mentioned that only two full scale columns were tested in biaxial bending . In view of this lack of information,tests on columns in biaxial bending were needed to confirm the validity of theoretically propsed models for this type of composite columns .

3.1 Test Rig

The column specimens were tested in a 400 kN-Test frame in the Heavy Structures Laboratory of the Civil Engineering Department of the University of Jordan.

The load was applied to the columns by way of a circular 25mm thick plate of 170mm diameter ,connected to a pair of 50mm thick plates at upper and lower ends .

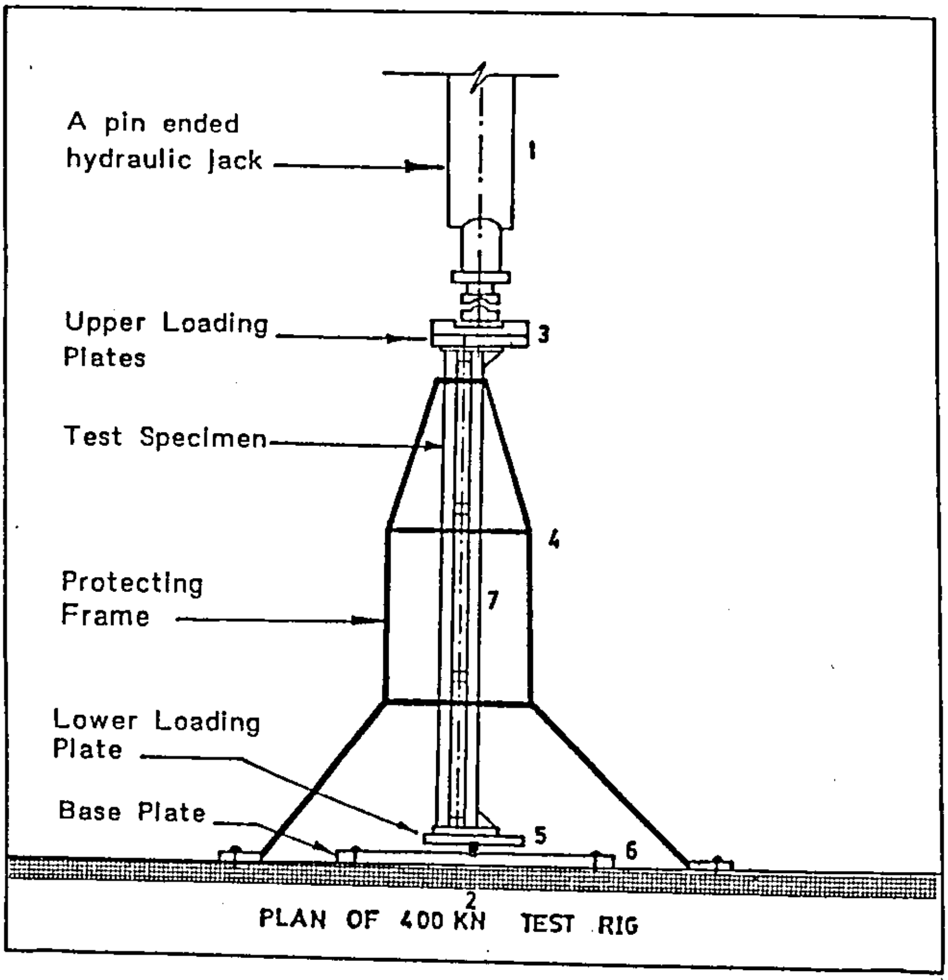
The loading system consisted of a pin-ended cylindrical hydraulic jack, with a 25mm thick,170mm diameter circular plate connected to the column loading plate through a 172mm-diameter circular groove in the upper loading plate . Figures (3.1) shows a section of the test rig , Figures (3.1.a) and (3.1.b) show general views of the test rig .

In addition to the end plates welded to the columns, 50mm-thick loading plates were used to reduce the effect of eccentricities involved in the tests apparatus.The column end plates were bolted to the loading plates by means of 17mm-diameter holes (see figure 3.1.c) .

A 50mm diameter steel ball was used and positioned in (18.5mm depth, 50mm diameter)spherical groove in the lower loading plate, and in to the 50mm-thick base plate . This arrangement was intended to give the column a rotational freedom in all directions (two pin ends). The base plate was fixed to the strong floor by 22mm -diameter tie rods of 850mm length. A protecting frame fixed to the strong floor by means of tie rods was used for safety purposes.The hydraulic jack is free to rotate in all directions and for this reason,a guide frame was used to prevent the horizontal sway .This guide frame was fixed on the protecting frame by welding. Black boltes of 16mm-diameter were used to connect the plates together . (see figure 3.1.d)

3.2 Test Specimens

Five columns were tested under eccentric biaxial bending. Figure (3.2.a) shows one of the columns placed in the test rig , also Figures (3.2.b) and (3.2.c) show the bottom plate connection and stiffening plates at the column ends .



1- The hydraulic Jack 400 KN Capacity.
2- Ground Lab. Floor 3,5-50mm thick Loading Plates.
4- Protecting Frame 6-50mm thick base plate.
7- Test Specimen.

Fig (3.1) A section of the test rig.

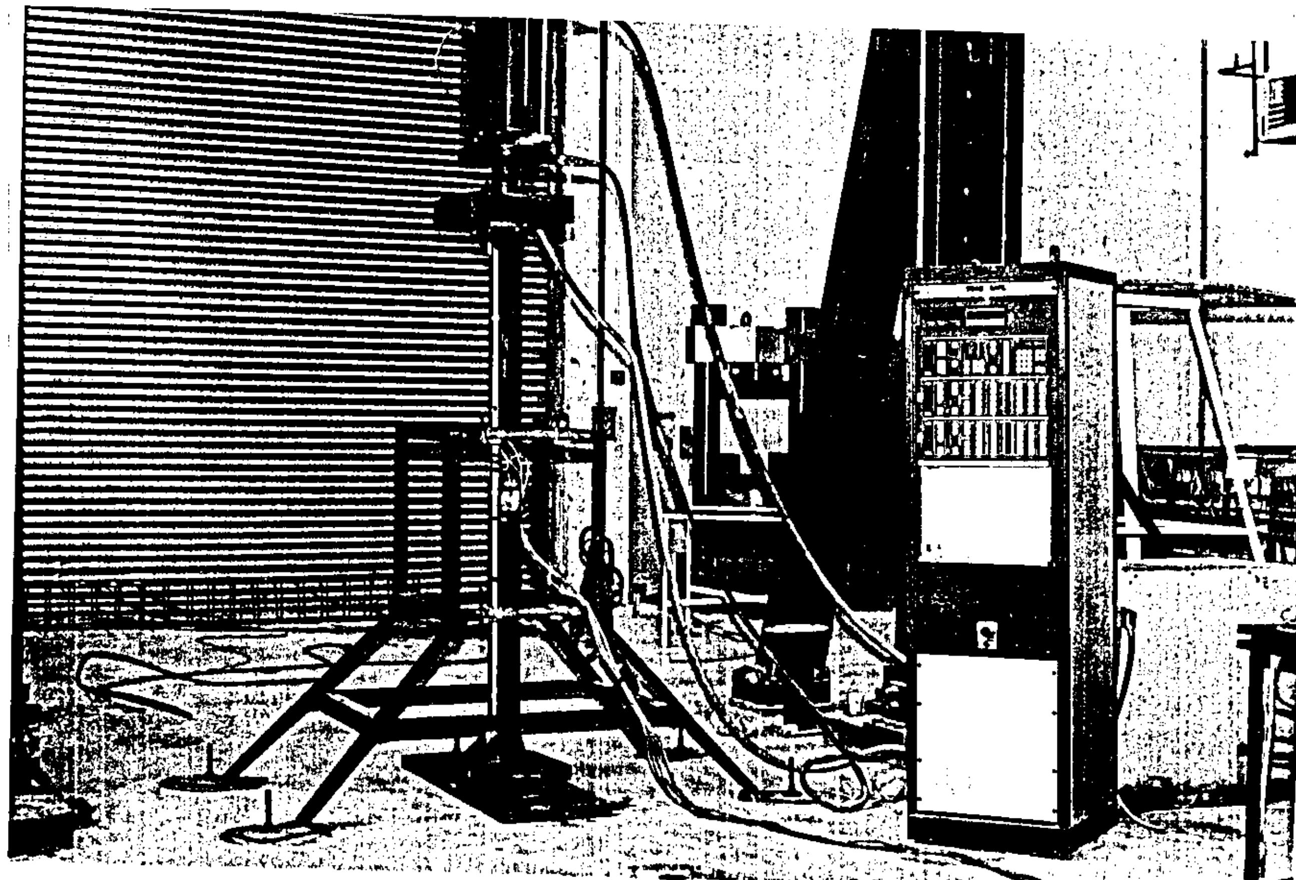


Figure (3.1.a) General View of Test Rig

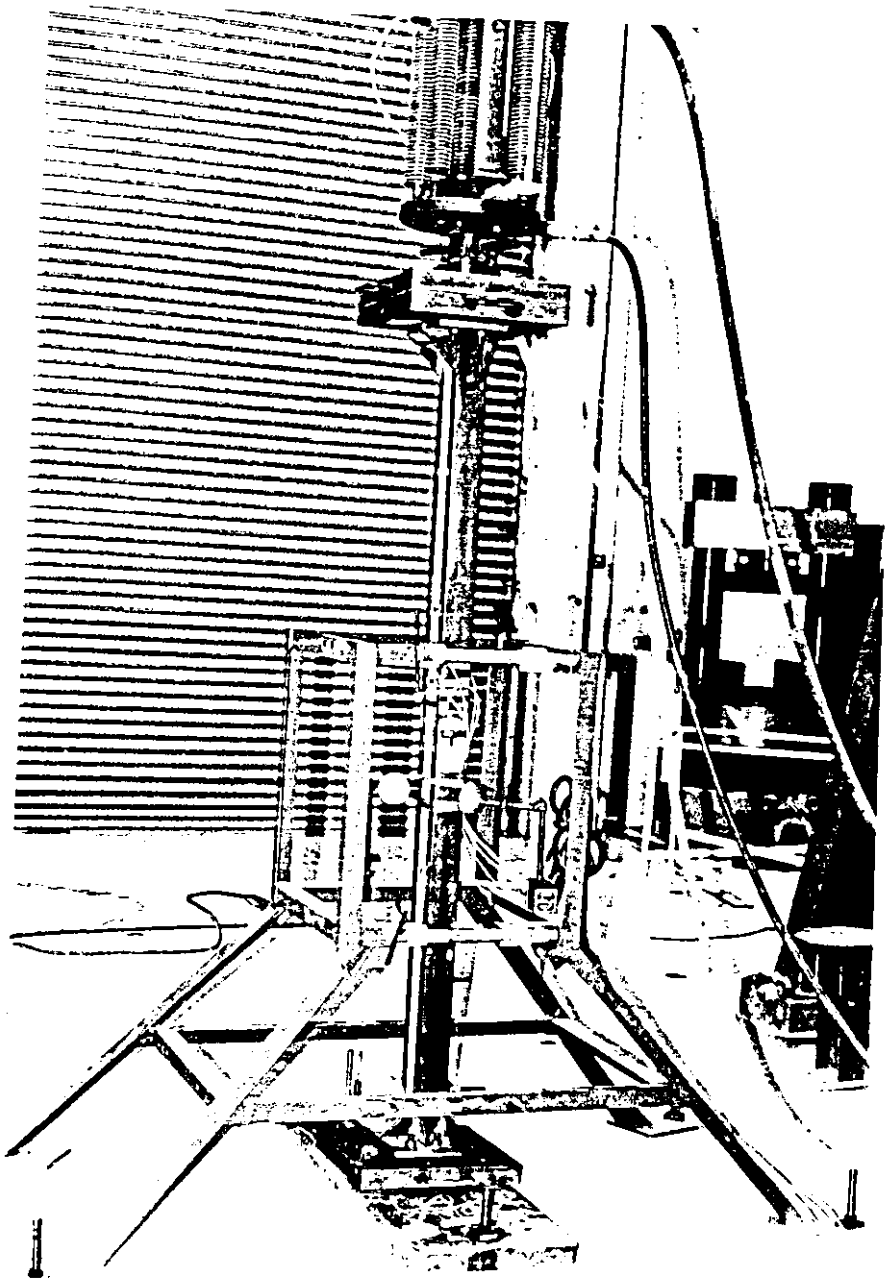


Figure (3.1.b) General View of Test Rig

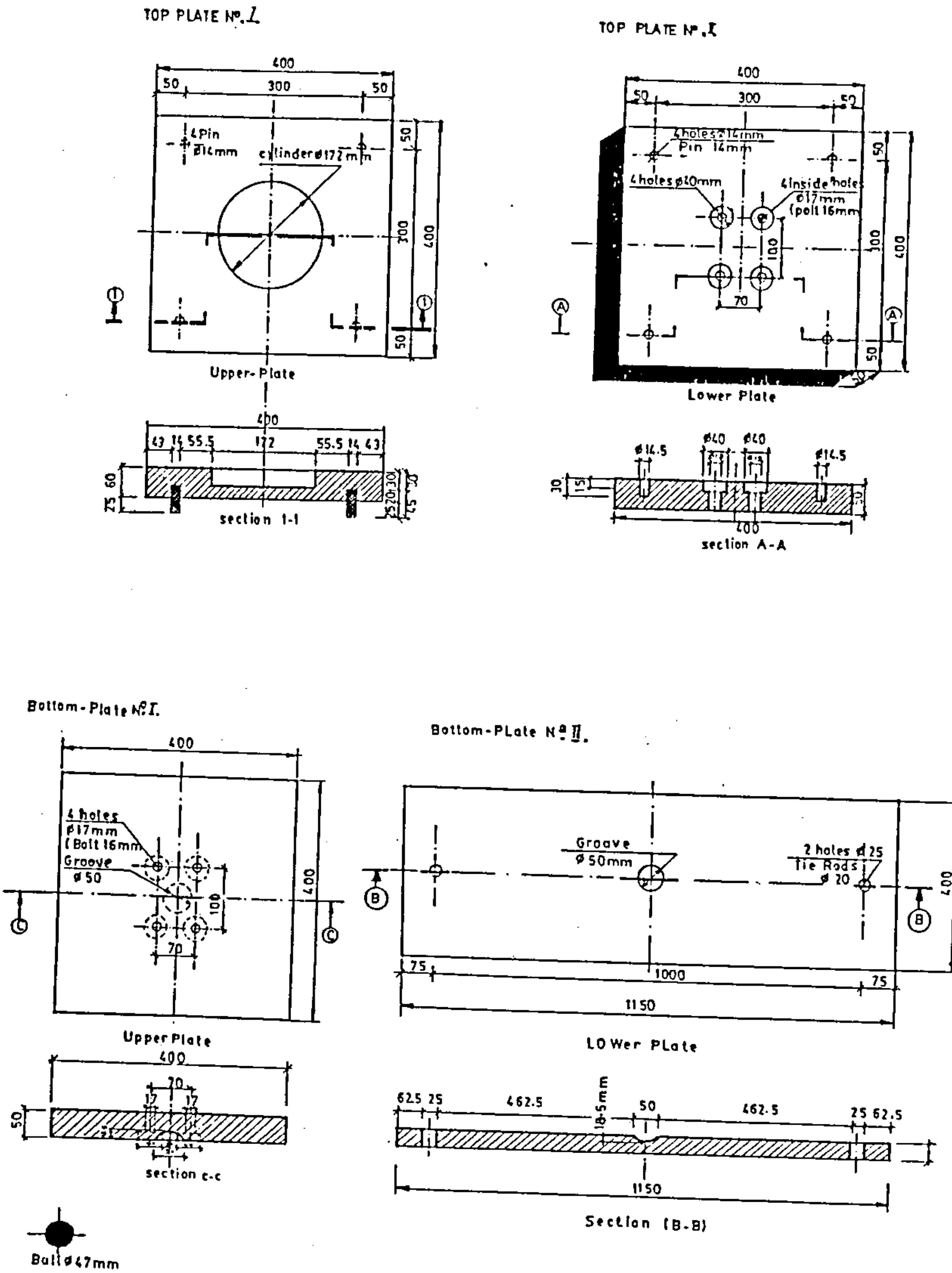


Figure (3.1.c)
loading plates

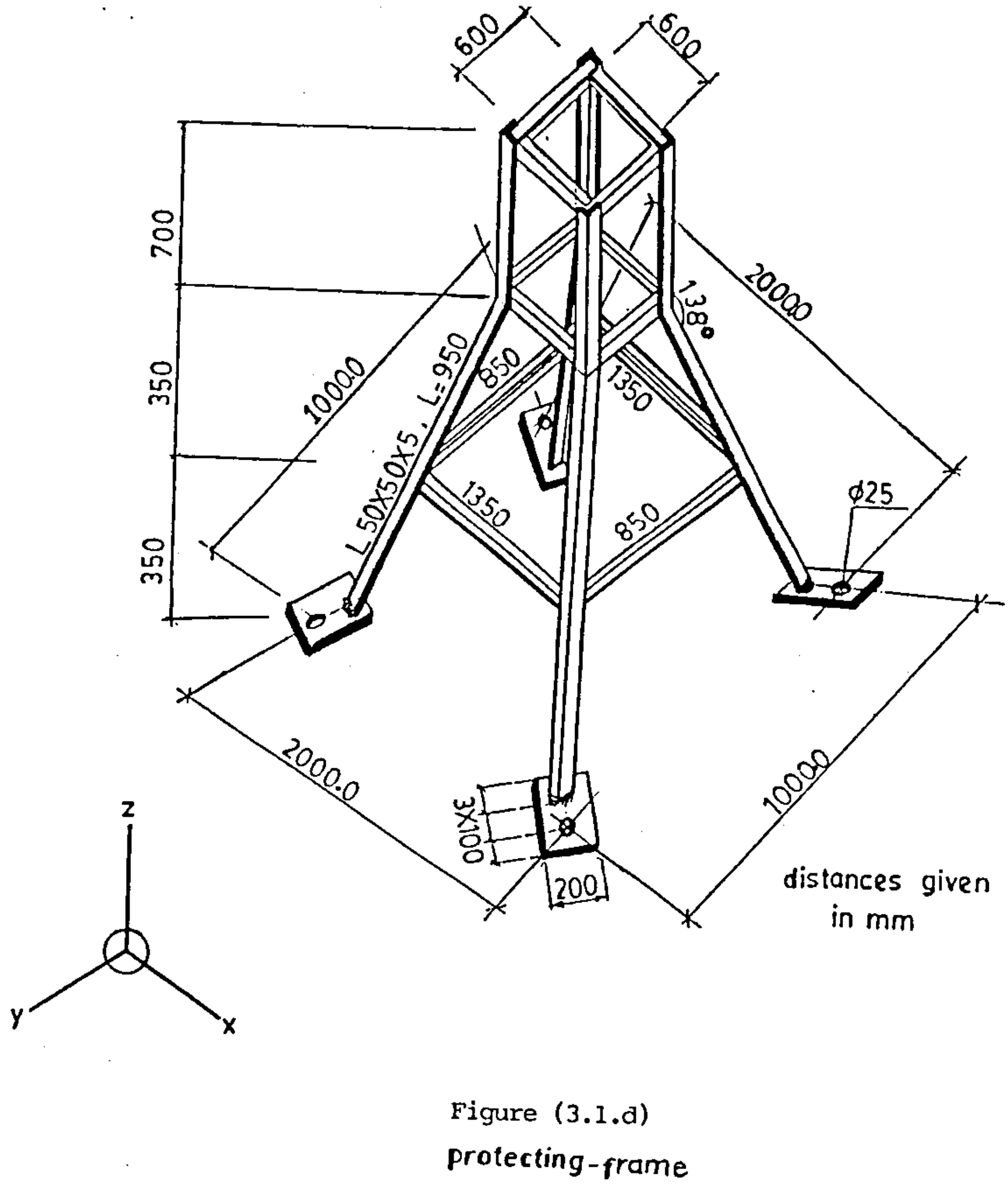


Figure (3.1.d)
protecting-frame

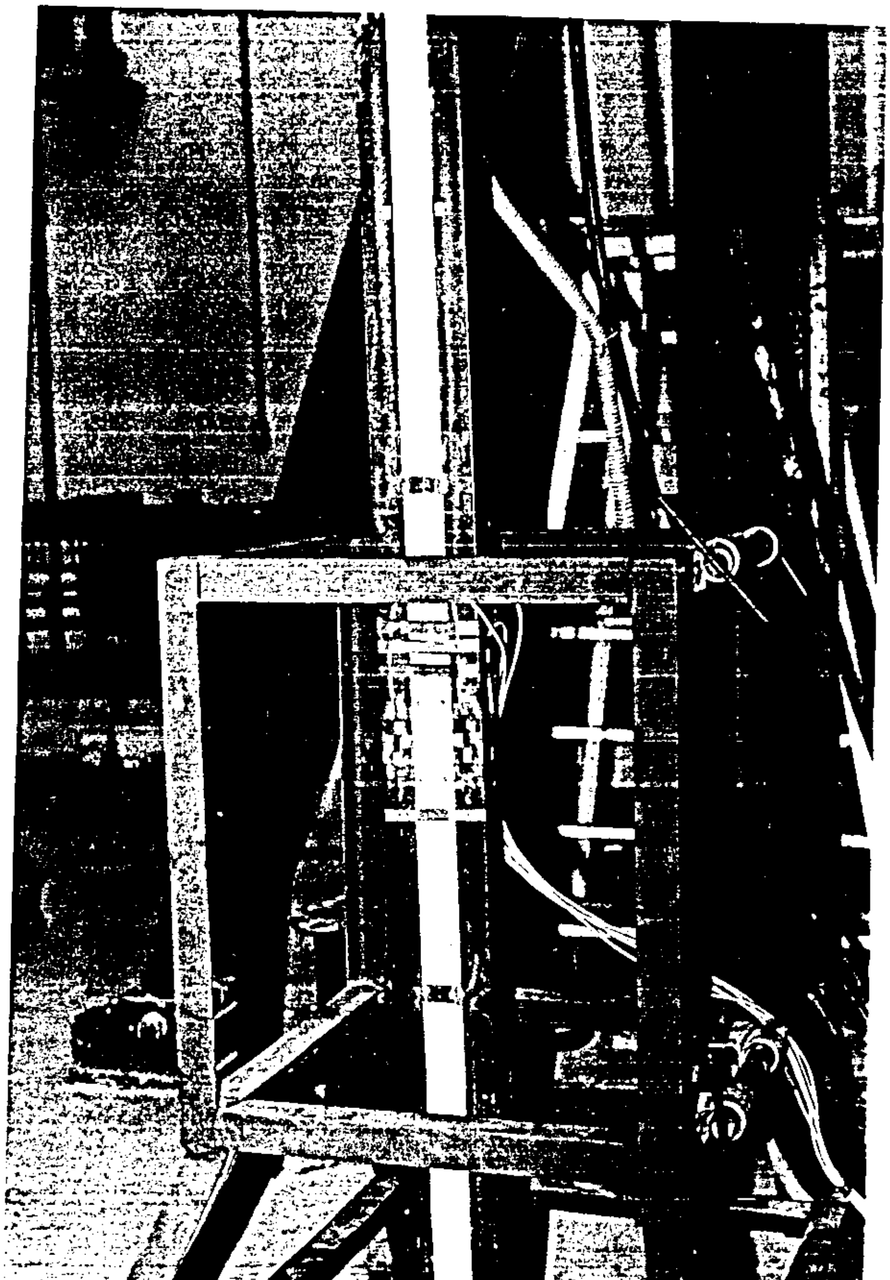


Figure (3.2.a) Test Specimen Placed in test rig

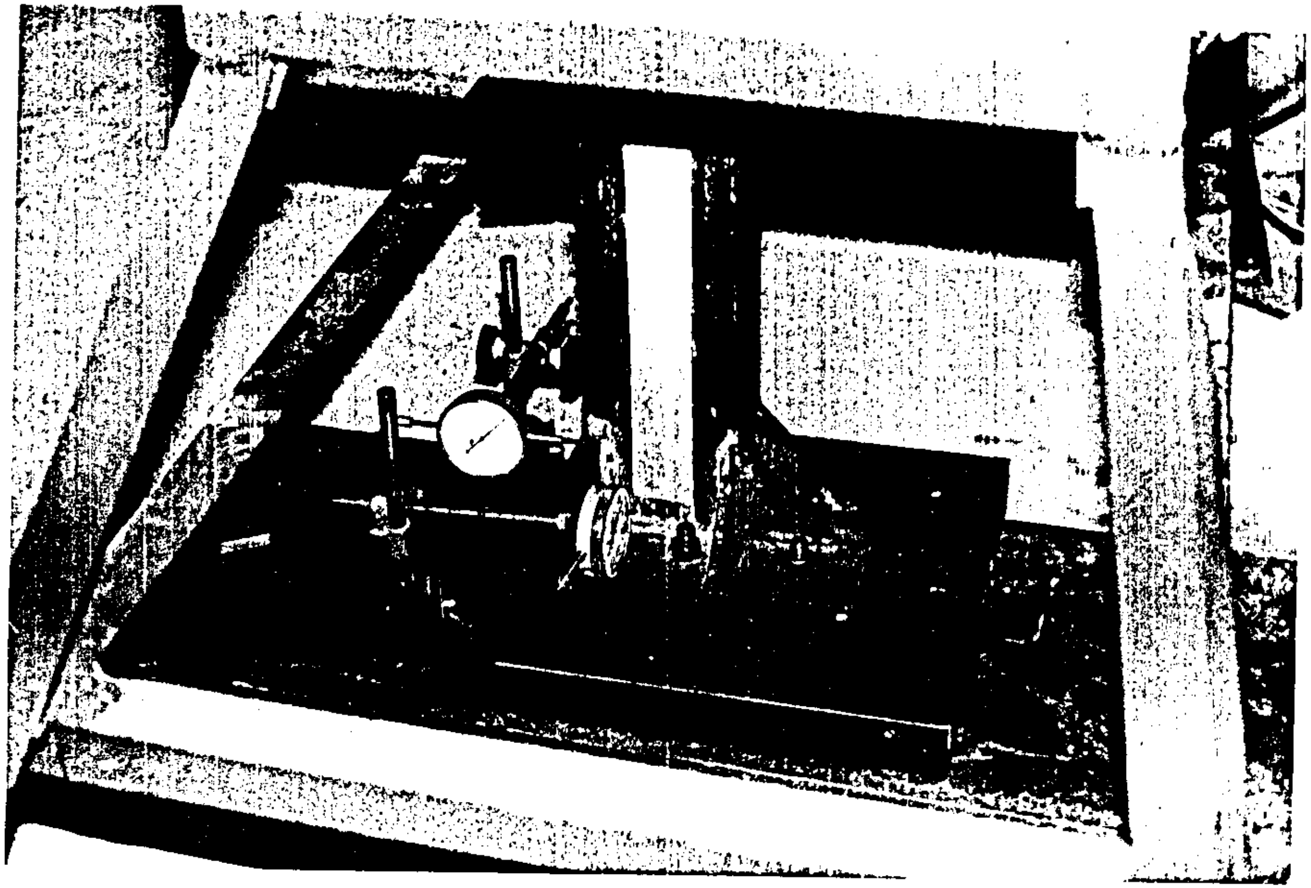


Figure (3.2.b) Bottom Plate Connection

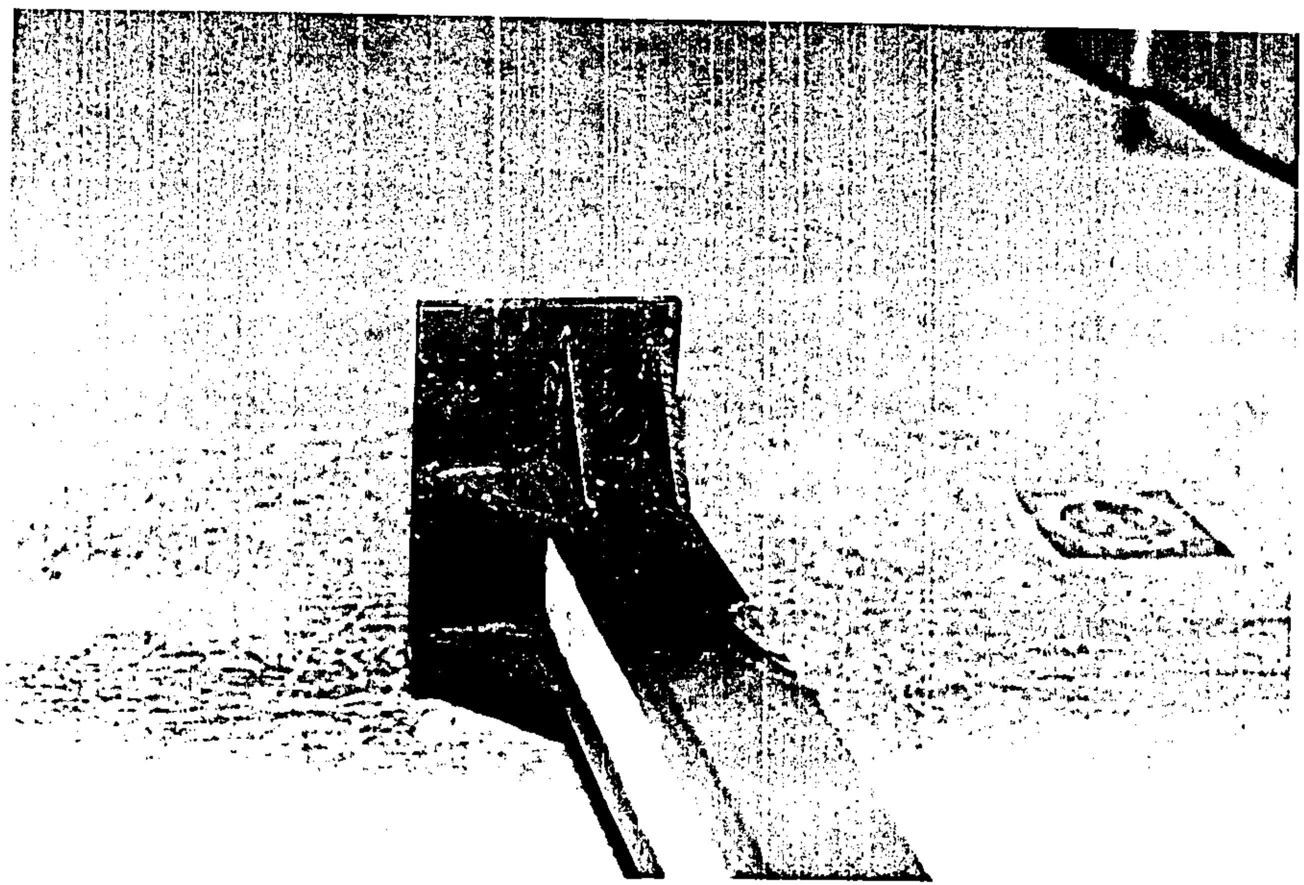
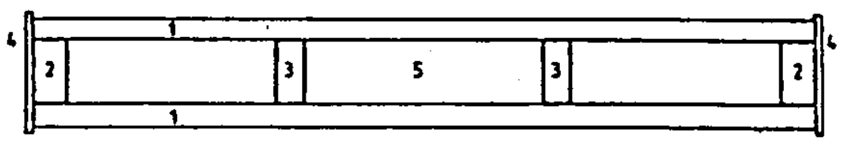


Figure (3.2.C) Triangular Stiffening Plates at Column end

3.2.1 Column Geometry

All the columns were of the same size and length, for comparison purposes. Two channels of 80x45mm size and 2.0m length, were battened to give a depth of 140mm. Four pairs of batten plates were welded to the flanges of the channels, two at the ends and two at the third points, using nominal welds. End batten plates 5mm thick were welded to the column ends using fillet welds. figure(3.2.1) shows the details of the column specimens .



1. Steel channels (2.0m long)
2. End batten plates
3. Intermediate batten plates
4. End plates
5. Plain concrete core

Fig.3.2.1 Details of Column Specimens

Triangular stiffening plates were welded to both end plates and column ends to ensure that premature failure of the columns would not occur as a result of local failure at the column ends . Figure (3.4) shows details of the column ends with extended end plates ,also figure (3.4.a) shows the eccentricity position of five tested columns .

3.2.2 Orientation of Columns in Test Rig

Columns were placed vertically in the test rig in such a way that the deflection of the column was measured in a horizontal plane as shown in Figure (3.5) .

3.2.3 Materials

Concrete of a mix (of 1(cement):2(sand):4(gravel)/0.55(w/c ratio) by weight) was used with maximum size of aggregate (crushed gravel) of 10mm. Ordinary portland cement and sieved sand were used. The columns were cast and compacted horizontally and then were kept moist by covering the top surface with wet hessian for 3 days . One 150x150x150mm cube was obtained from each column cast and stored in identical conditions . All columns were tested after 38 days from the casting date . The steel channels were originally 6.0m long , 4.0m were used for each column, and the remaining 2.0m were used to determine the material properties. The characteristics strength of steel was determined from tests on tension specimens machined from the remaining 2.0m long channel sections . One tension specimen cut from the flange was tested for each channel length,and another from the web . The elastic modulus of the steel was determined from tensile test results. Details and materials properties of the columns are shown in table (3.1) .

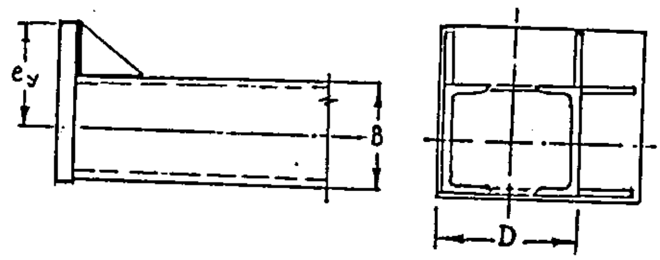


Fig.3.4 : Details of column end subjected to large eccentricities.

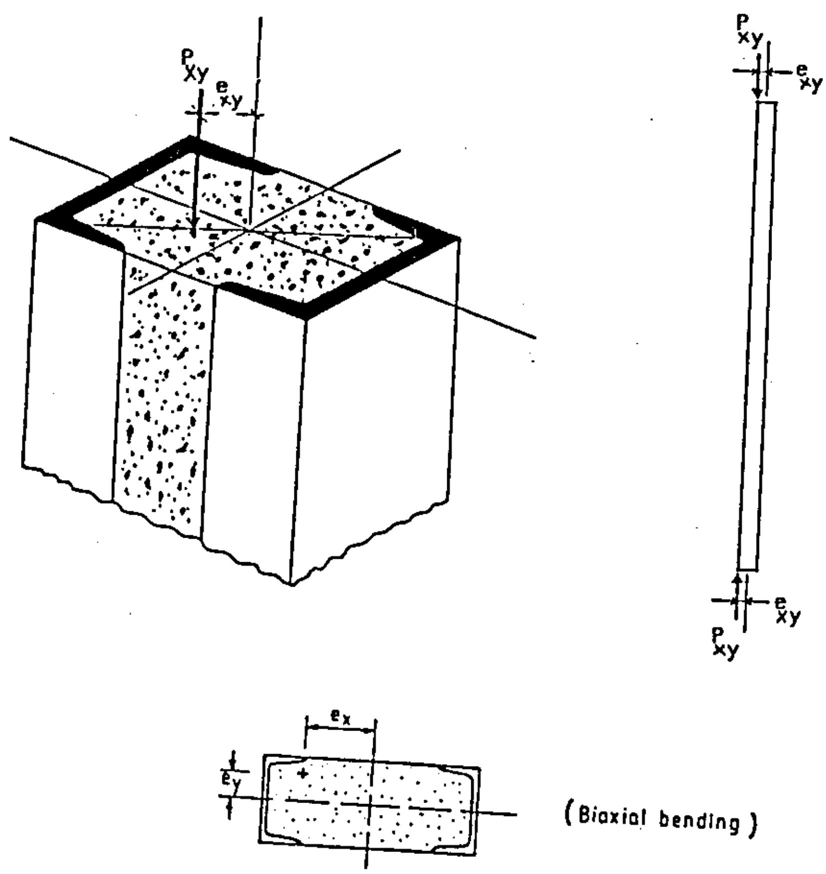


Fig. 3.5 : Orientation of column in test rig

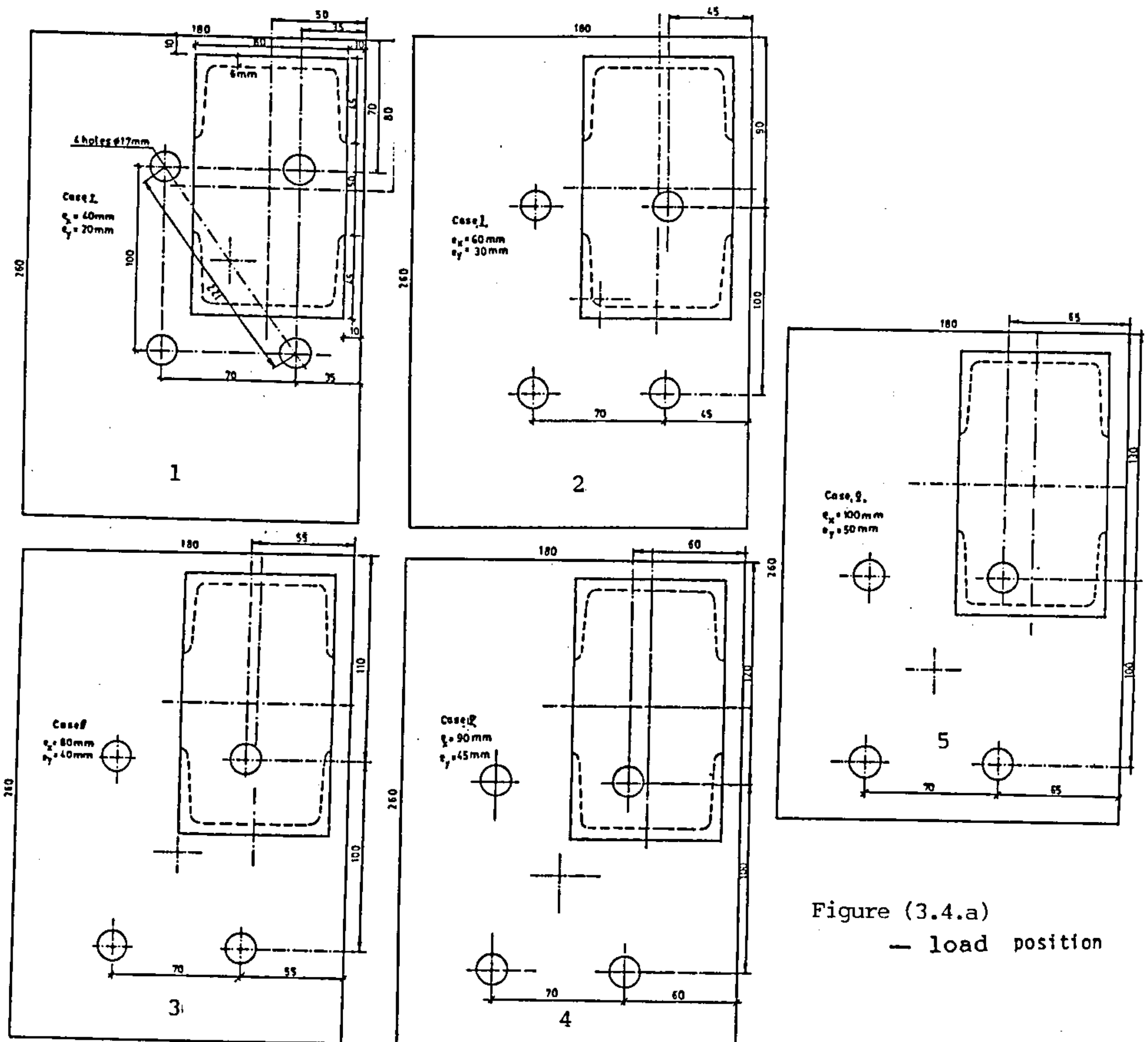


Figure (3.4.a)
— load position

Table No.: (3.1) Details and Properties of columns tested in biaxial bending

Col. No.	Size of Channel (mm X mm)	Depth of Col. D (mm)	Batten Plates			Steel Channels				Concrete Core			Specimens Length	
			Thick. (mm)	Length of end plates (mm)	Length of Intermediate plates. (mm)	Area (AS/2) (mm^2)	Characteristic strength F_{SK} (N/mm^2)	Yield Strain E_y %	Elastic Modulus E_s^{**} (KN/mm^2)	Area A_c (mm^2)	Cube Strength Fcu (N/mm^2)	Elastic Modulus (Bridge Code) Ec* (KN/mm^2)	Lex (mm)	Ley (mm)
1	80X45	140	5.0	35	25	1100	340	0.32	205.0	9000	30.9	27.8	2160	2160
2	80X45	140	5.0	35	25	1100	341	0.33	207.5	9000	33.3	28.9	2160	2160
3	80X45	140	5.0	35	25	1100	339	0.31	204.1	9000	30.9	27.8	2160	2160
4	80X45	140	5.0	35	25	1100	342	0.33	207.0	9000	30.9	27.8	2160	2160
5	80X45	140	5.0	35	25	1100	341	0.33	207.4	9000	30.2	27.6	2160	2160

* $E_c = 5000\sqrt{Fcu}$

**- Average Value.

3.3 Experimental Data Acquisition

The columns were connected to automatic measurement instrumens to measure loads, deflections and strains of steel as described in the following sections.Figure (3.3) shows the data acquisition setup .

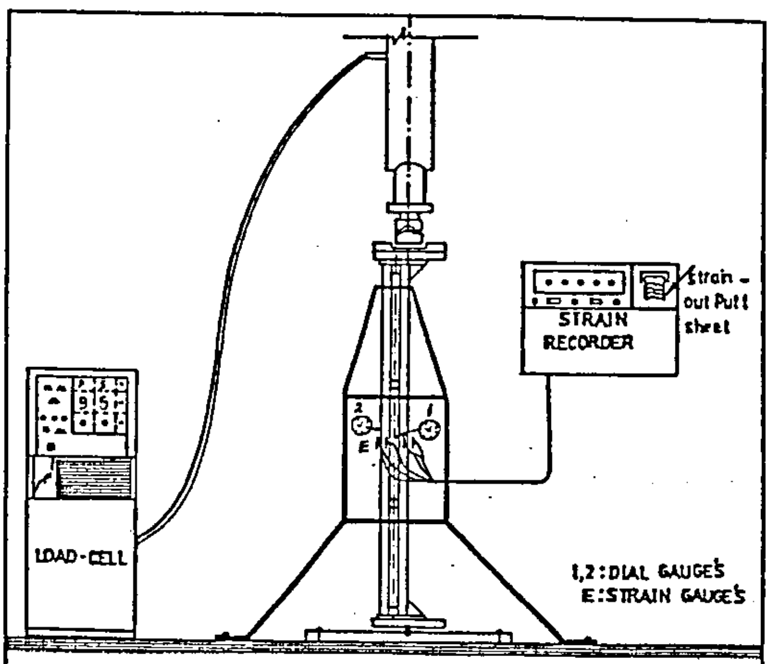


Figure 3·3 : DATA ACQUISITION SETUP

3.3.1 Load Measurements

Values of the applied load were given directly by the data recorder of the testing frame. Figure (3.3.1) shows the load application on column .

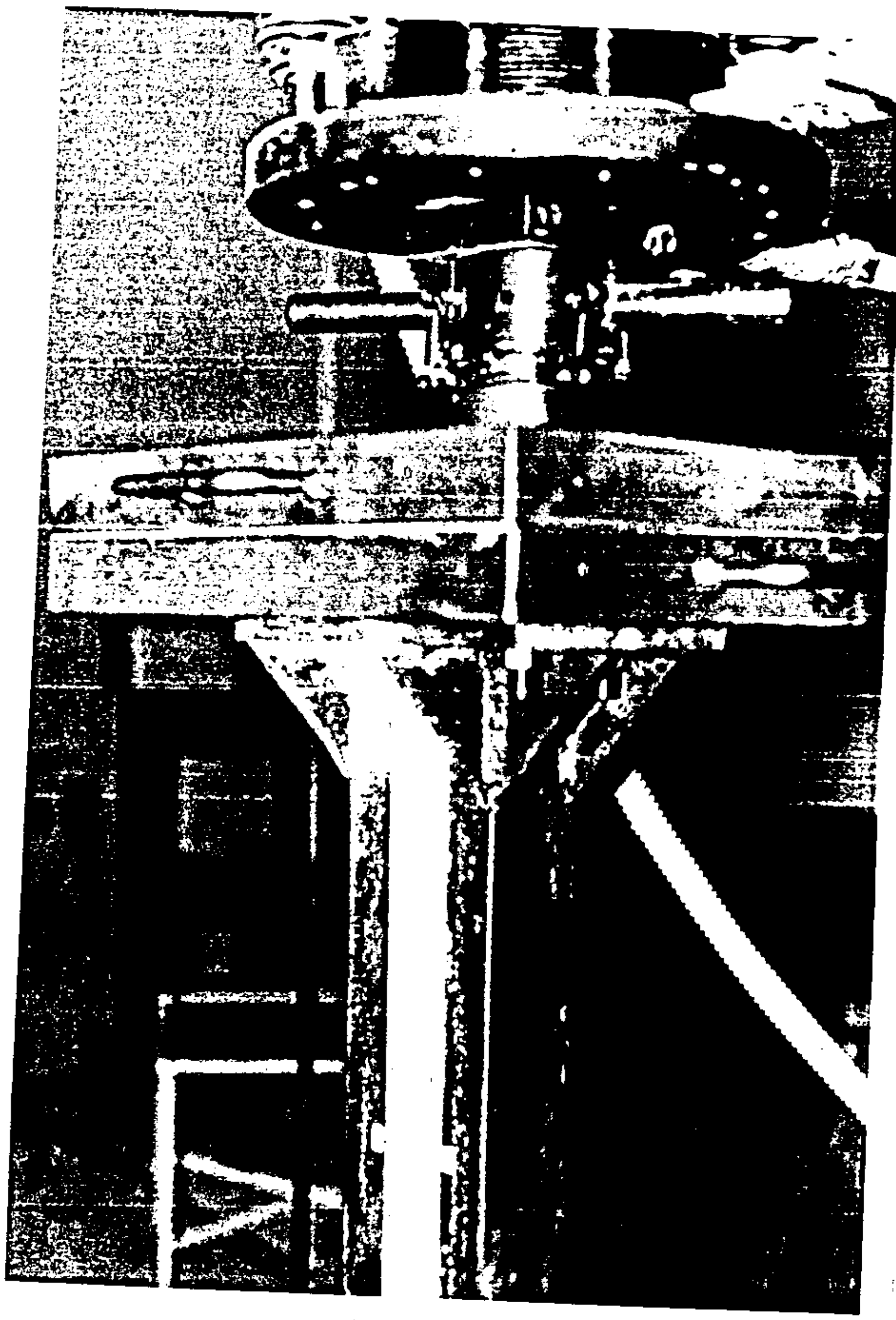


Figure (3.3.1) Load Application on Column

3.3.2 Deflection Measurements

The midspan deflection of the column specimens was measured by two dial gauges with accuracy of 0.01mm. The net midspan deflections were Δx in x direction , Δy in y direction . Table (3.3) shows values of lateral and vertical deflections for column No.2 .

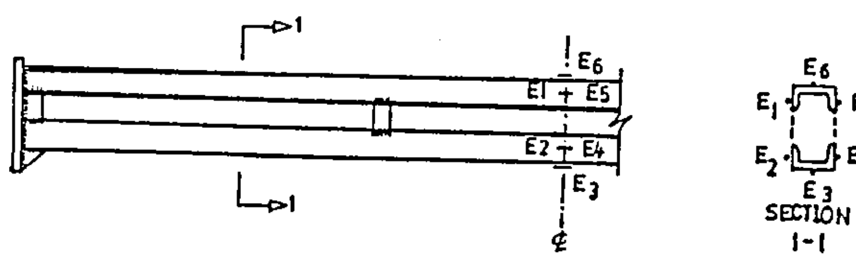
3.3.3 Strain Measurements

The strains in the steel channels were measured only for one column (column No.5)by electric strain gauges (gauge length 20 mm) placed in location as shown in Figures (3.6) and (3.6.a) . The strains in the concrete were not measured due to the difficulties associated with their measurement.

3.4 Experimental Procedure

When placed in the test rig , a load of about 5kN was applied to each test specimen and then released prior to testing . This was intened to ensure that no load loss would occur due to relaxation at the column ends .

In all the tests ,a sequence of increasing loads up to failure was applied . Different magnitudes of decreasing load increments were employed ,depending upon the eccentricity of the load . These load increments were chosen such that increments of about 5% of the failure load were employed towards the end of the test . This program of loading was used to determine the load at which deterioration of load, cracking and any other form of local instability occured .



E : Electrical resistance strain gauges

Distances given in mm

Fig. 3.6 : Typical position of strain gauges

Table (3.2)

Experiments on Battened Composite
Columns Under Biaxial
Bending

Column No. : 5 — Date of Casting: 20/11/88

Eccentricity : ex=100mm. — Date of Testing: 28/12/88
ey= 50mm.

Concrete Strength :- 30.22 N/mm^2. — Age of Concrete: 38 days

Applied Load (KN)	Vertical Deflection (mm)	Strain Gauge Readings	Applied Load (KN)	Vertical Deflection (mm)	Strain Gauge Readings (micro-strain)
0 10	0.0 0.2	0,0,0,0,0,0 E1= 40 E2= -40 E3= -110 E4= -50 E5= -10 E6= +30	150	6.3	E1= 540 E2=-530 E3= -1430 E4= -860 E5= + 80 E6= + 1030
50	1.8	+180 -190 -440 -300 -10 +280	160	6.8	590 -590 -1550 -920 +120 +1070
75	2.4	+270 -270 -660 -450 +10 +430	170	7.5	620 -630 -1730 -1020 130 1200
100	3.7	+350 -310 -930 -270 +30 +630	180	8.4	690 -750 -4300 -1240 200 +1380
115	4.7	+410 -420 -1060 -660 +50 +740	190	10.0	980, -1030 -8470,-2140 380,1750
			195	10.7	1140,-1130 -10180,-2940 + 470, +2380
125	5.1	+450 -420 -1180 -730 +60 +790	200	11.2	1250,-1210,-11020 -3000,530,2610
			206	11.9	1370,-1290 -12400,-4140 550,3070
140	5.8	510 -480 -1310 -820 +100 +930	209 Failure Load	12.8	6320,-2760 -12400,-10630 2400 over load
			200	18.9	-

Table No.: 3.3

Experiments on Battened Composite
Columns Under Biaxial
Bending

Column No...2....
Eccentricity : ex=...60...mm.
ey=..30.mm.

Concrete Strength : 33.3 N/mm^2.

Date of Casting:- 20/11/88
Date of Testing:- 28/12/88
Age of Concrete: 38 days.

Applied Load, KN.	Vertical (mm) Deflection.	Dial.Gauge Readings.(mm) ΔY	ΔX
50	1.8	1.5	0.5
100	3.5	2.6	0.8
150	4.6	4.0	1.2
200	4.9	5.6	1.9
250	6.4	7.5	2.6
273	8.2	8.7	3.1
280	8.7	9.0	3.2
290	9.9	10.1	3.5
293	10.5	10.6	3.6
295	10.8	10.9	3.7
297	11.0	11.2	3.8
300	11.1	11.3	3.9
301	11.2	11.4	3.95
303	11.3	11.5	4.0
305	11.5	11.7	4.05
307	11.7	11.9	4.1
309	11.9	12.0	4.2
311	12.1	12.2	4.3
313	12.3	12.4	4.5
316	12.8	12.9	4.6
P_f = 319	13.2	13.0	5.0
300	15.1	15.0	5.8
290	17.0	17.0	7.0
280	18.1	18.0	8.0

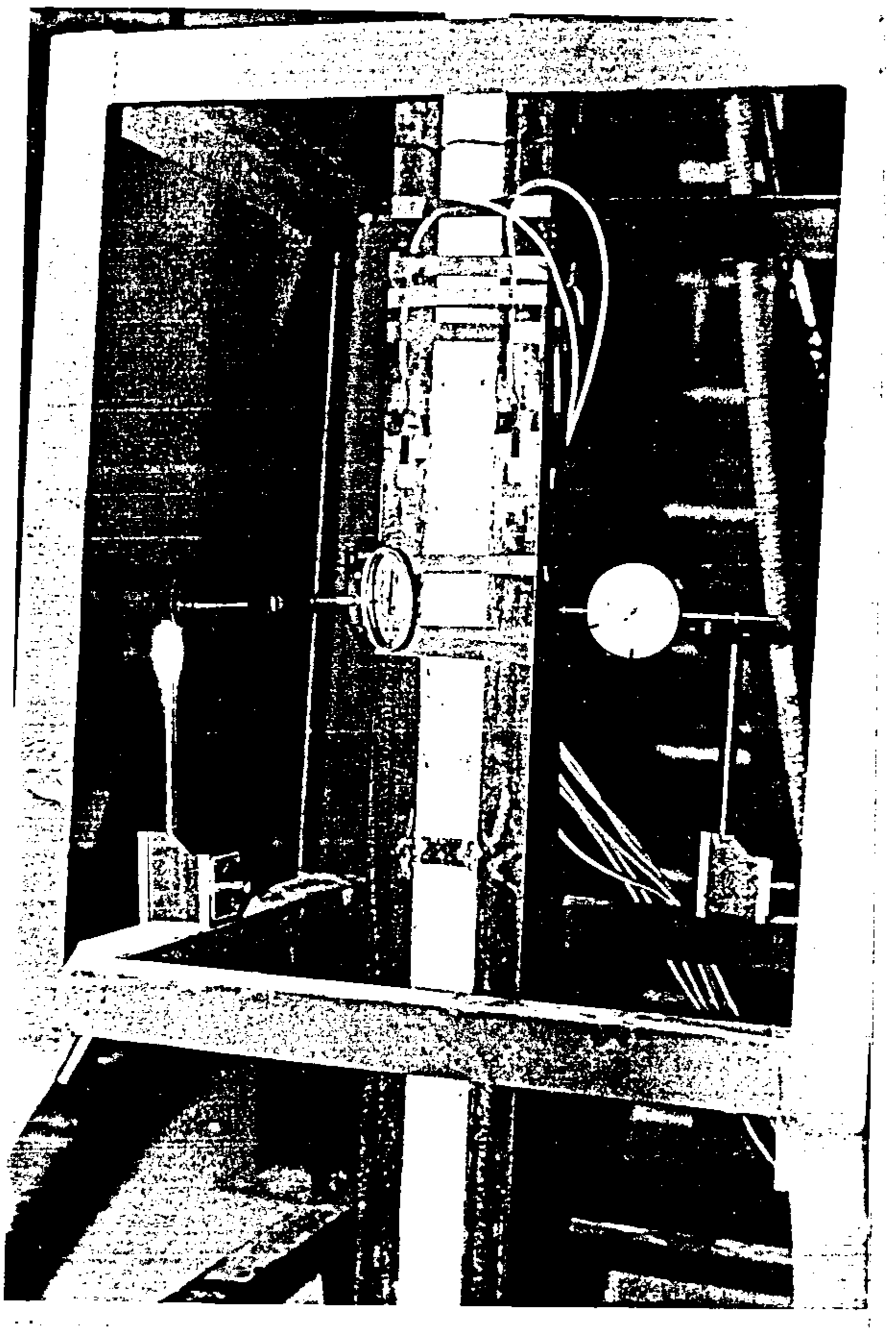


Figure (3.6.a) Position of dial and strain gages on column

CHAPTER 4

COLUMN BEHAVIOUR AND TEST RESULTS

Test results and the behaviour of the test specimens under load based on the recorded data and observations during testing are presented in this chapter .

4.1 Behaviour of Column Spcimens

As observed, all column specimens behaved very well under load and,as shown in the following section, the ultimate loads of all the columns were always well in excess of values estimated by the Bridge Code and the ECCS Method .

For all tested columns ,tension cracks in concrete were observed in the tests,especially when the increasing eccentricity implied that the load was applied outside the column cross section .

As all the columns were tested in single curvature (equal end moments;i.e equal eccentricities),it was expected that severe damage due to failure would occur at midspan,see Figures (4.1) to (4.5) and (4.1.a) .

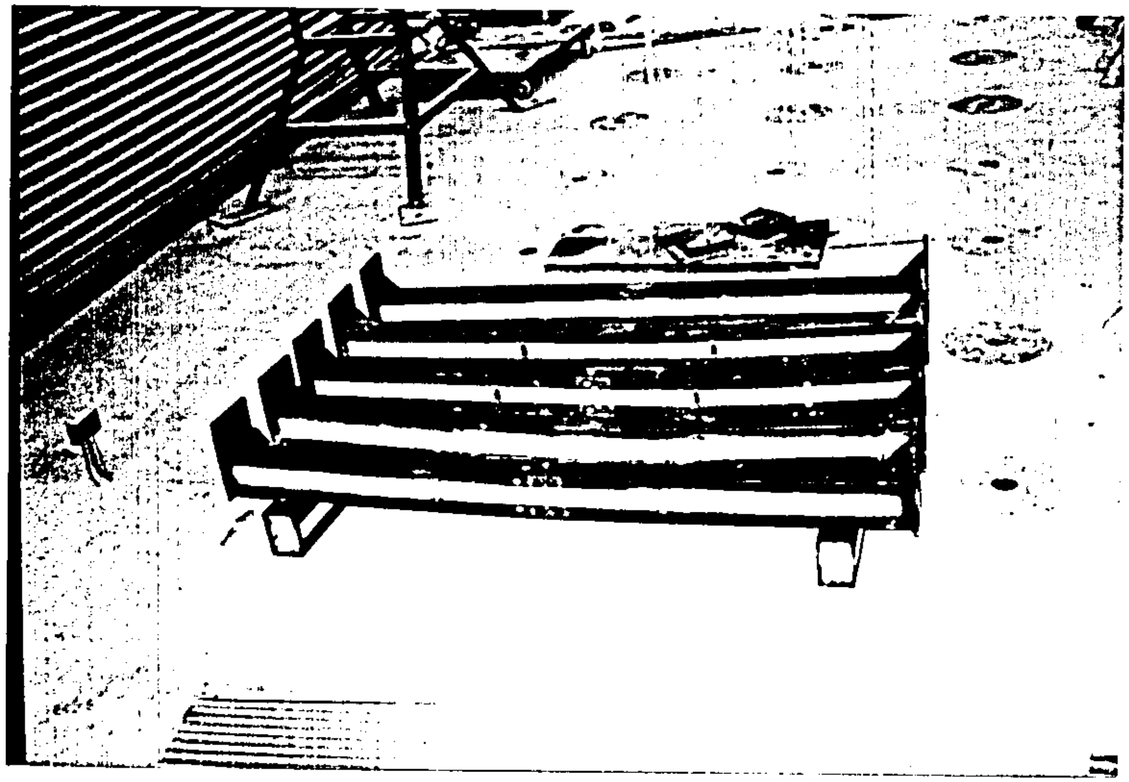


Fig (4.1) Tested columns after failure

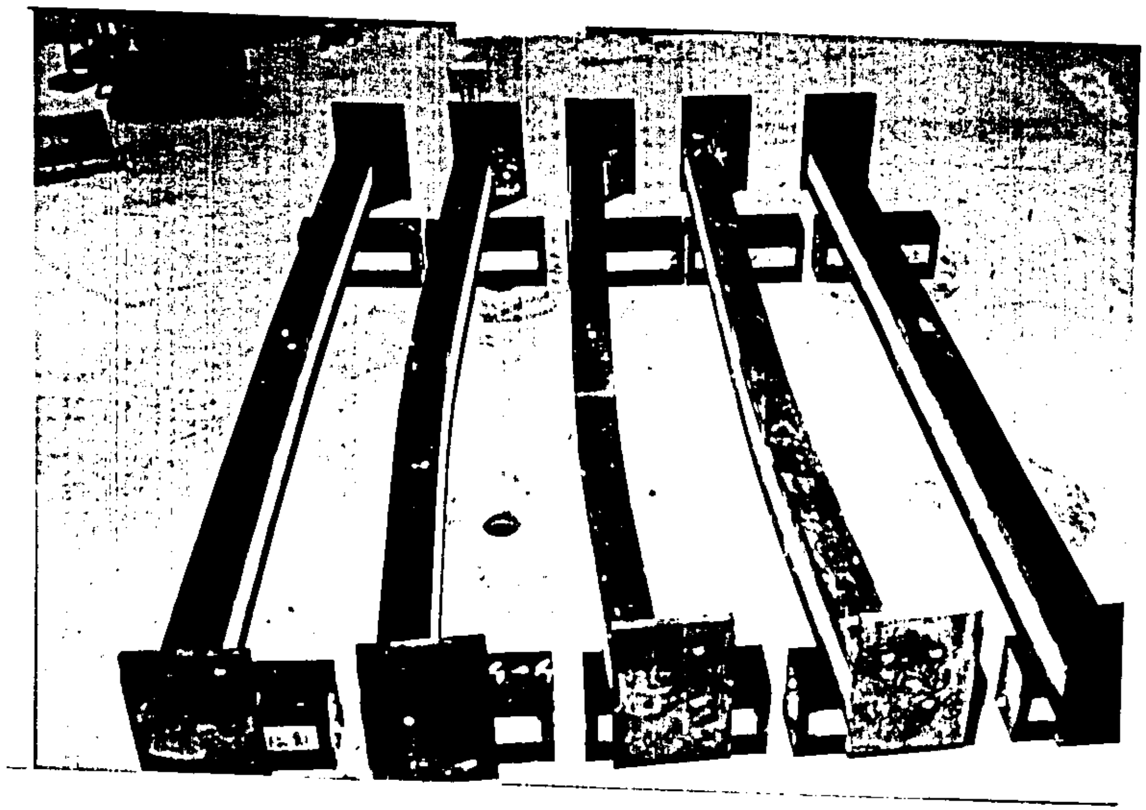


Fig (4.2) Damage due to failure for all tested columns

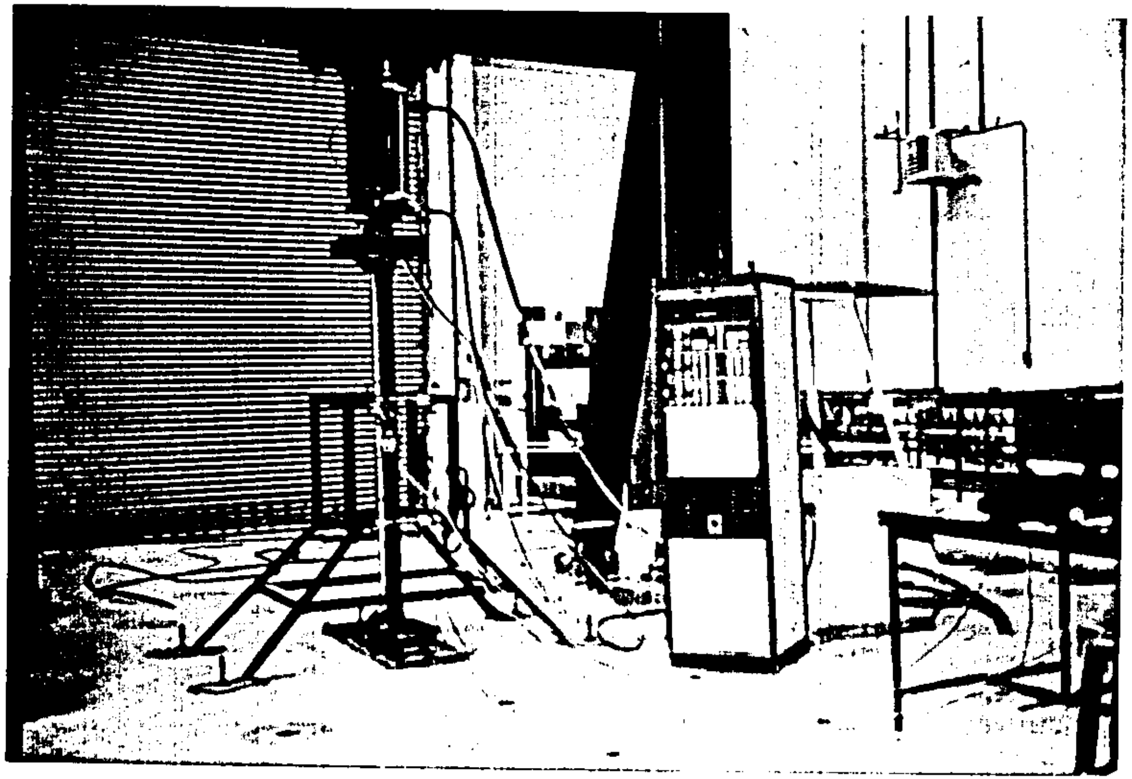


Fig (4.3) General view of test rig

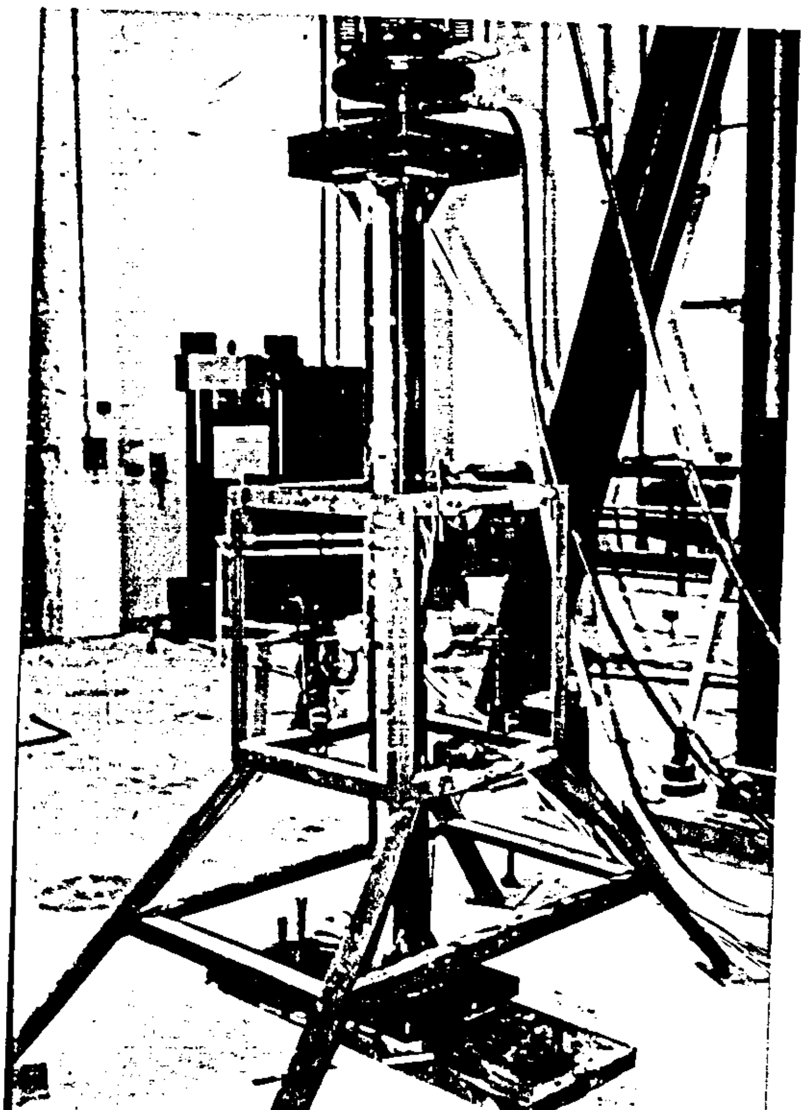


Fig (4.4) Test specimen under hydraulic jack

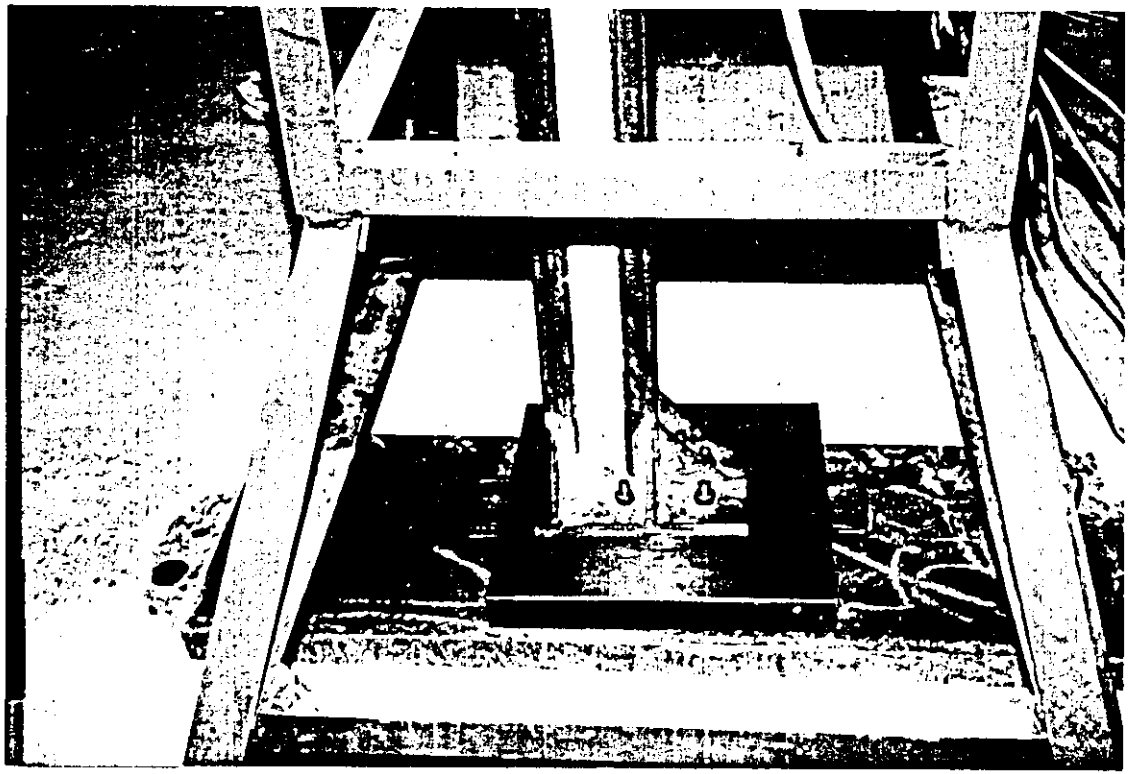


Fig (4.5) Lower end plate and its connections

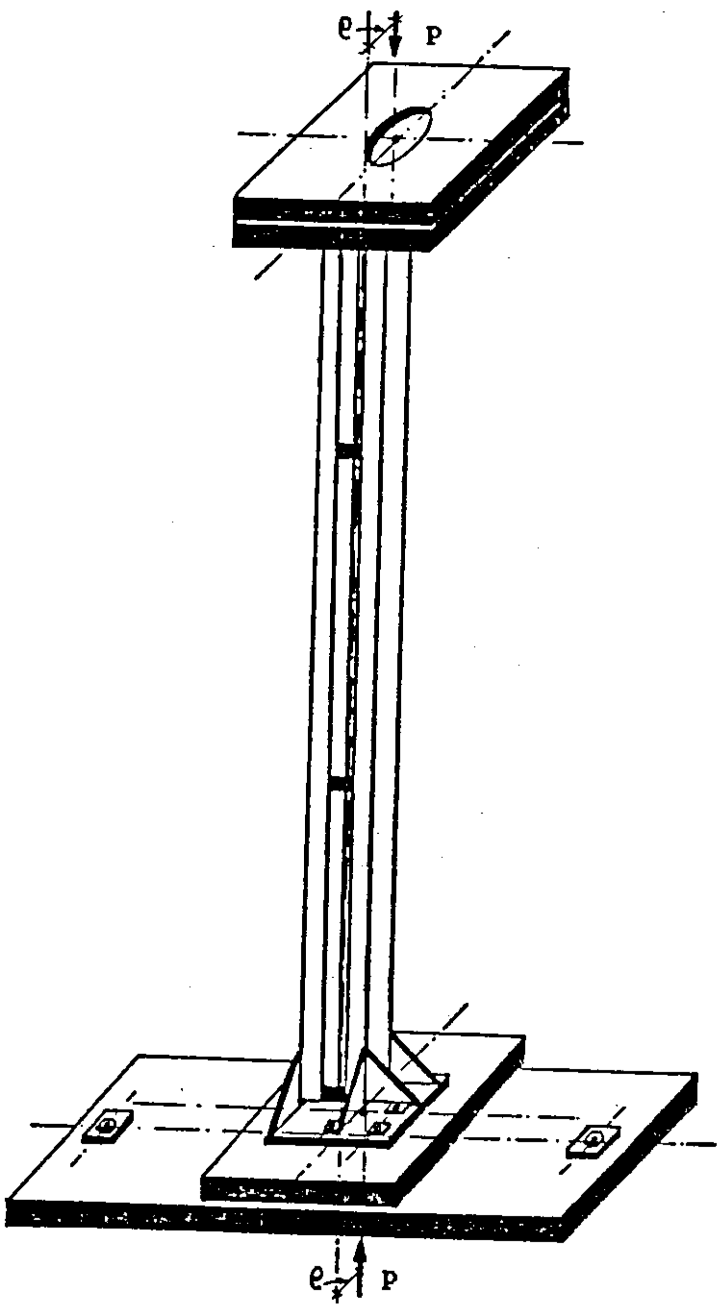


Figure (4.1.a) Test SETUP

4.2 Failure loads

The failure loads of the tested columns are given in Table (4.1) and are compared with the predicted loads ,as calculated by the Bridge Code . Although no material safety factors were taken into account in the calculations ,the failure loads are seen to be always well in excess of the calculated values .

The failure loads are shown graphically in Figure (4.6) as a function of the squash load ,Nu ,and are plotted against the eccentricity ratio $\bar{e} = e_{xy}/(D^2+0.77B^2)^{1/2}$. The Figure shows that the failure load is higher at low eccentricities ,decreasing steeply as eccentricity ratio increases .

The ratios of the failure loads to the predicted values against the eccentricity ratios are shown in Figure (4.7) .

4.3 Strains

Strains in steel obtained during tests were plotted against the load for column No.5 . It should be mentioned that the load-strain curves for all other columns will probably be similar to the curves presnted here .

The yield strains which were obtained from the similar tensile tests (Table 3.1) varied between 0.31% and 0.33% . It can be seen from the figures (strain results) that the steel channels yielded at failure since strains of the above magnitude were recorded in the tests .Figures (4.8) to (4.13) show the load-strain curves for column No.5 .

Table :(4.1)

Test results of columns

Col. No.	Eccentricity: about major axis e_x (mm)	Eccentricity: about minor axis e_y (mm)	Eccentricity ratio: about major axis e_x/D	Eccentricity ratio: about minor axis e_y/B	Eccentricity ratio: about biaxial $\bar{e} = \frac{e_{xy}}{\sqrt{D^2+0.77B^2}}$	Squash Load N_u (KN)	Concrete Contribution factor α	Ultimate Moment of resistance M_{ux} (KN.M)	Ultimate Moment of resistance M_{uy} (KN.M)	Exp. failure Load N_e (KN)	Mid Height deflection at failure V (mm)	Mid Height deflection at failure U (mm)	Mid Height Moment at failure * M_{ex} (KN.M)	Mid Height Moment at failure # M_{ey} (KN.M)	Bridge code Calculations: reduction factor K_B	Bridge code Calculations: Ult. Load N_{KB} (KN)	Bridge code Calculations: $\frac{N_e}{N_{KB}}$	$\frac{N_e}{N_u}$	ECCS $\frac{N_e}{N_{kE}}$
1.	40	20	0.29	0.25	0.29	911.5	0.253	37.44	21.2	N.R*	-	-	-	-	0.290	265	-	-	-
2.	60	30	0.43	0.38	0.43	931.4	0.267	37.54	21.3	319	5.0	13.0	20.70	13.70	0.223	208	1.53	0.342	1.52
3.	80	40	0.57	0.50	0.57	909.5	0.254	37.33	23.0	263	10.0	18.0	23.7	15.3	0.217	197	1.34	0.289	1.32
4.	90	45	0.64	0.56	0.64	913.5	0.253	37.58	20.9	231	13.0	23.0	23.8	15.7	0.209	191	1.21	0.253	1.20
5.	100	50	0.71	0.63	0.71	908.3	0.248	37.55	20.6	209	8.0	15.0	22.6	13.6	0.202	183	1.14	0.230	1.13

* $M_{ex} = N_e(e_x+V)$

\# $M_{ey} = N_e(e_y+U)$

* N.R---- The load capacity of the Jack is < Load capacity of column.

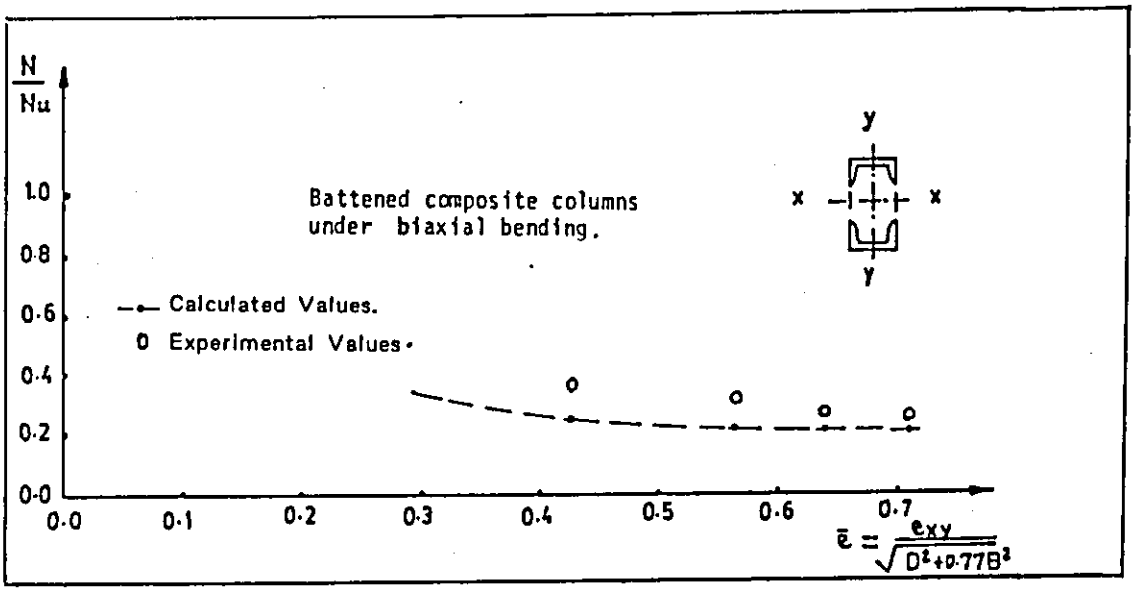


Figure 4.6 : Failure load v eccentricity for columns tested under biaxial axis bending

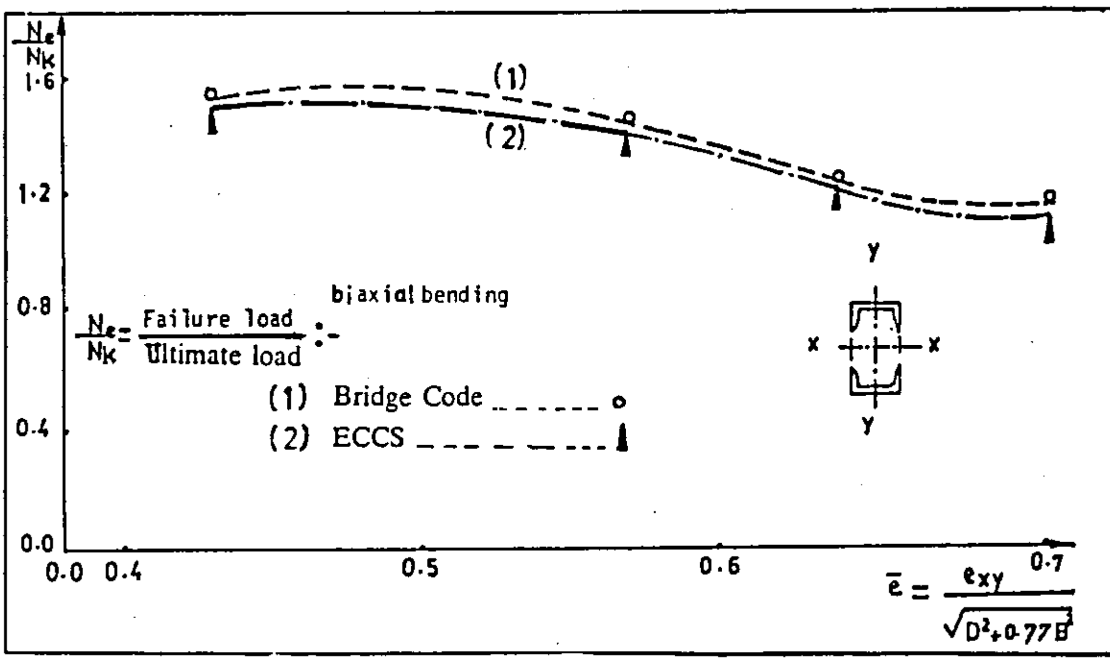


Figure 4.7 : Failure loads of columns tested under biaxial bending

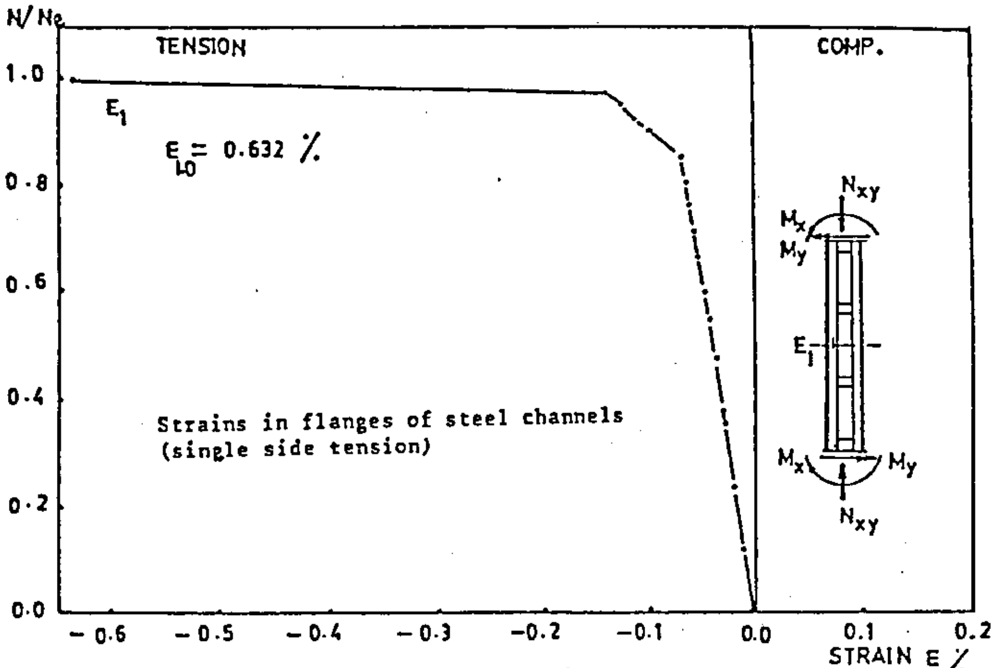


Figure 4.8 : Strain in Steel - Column No.5

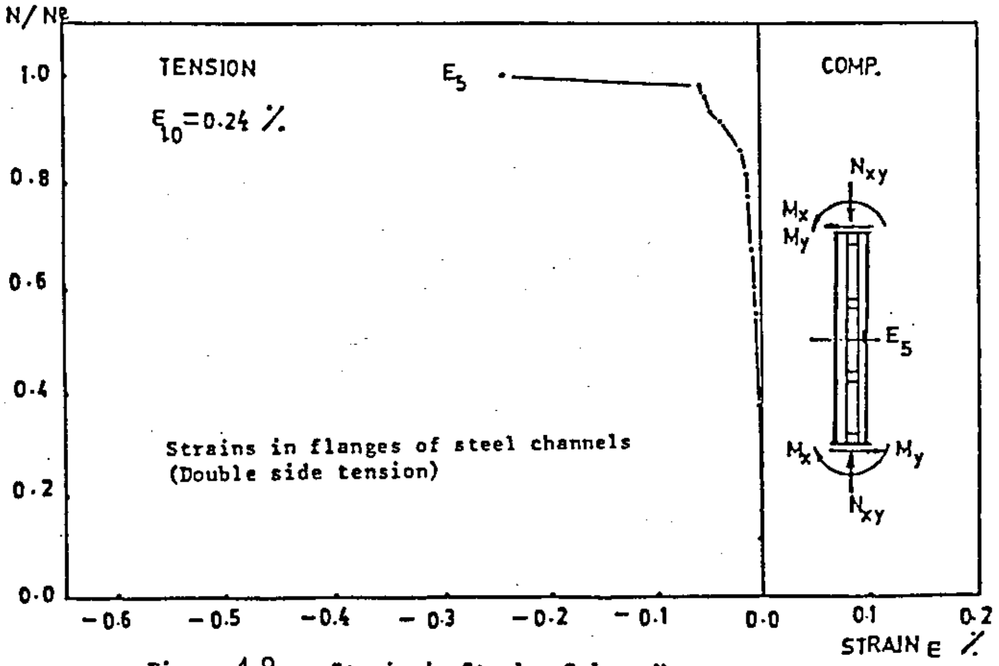


Figure 4.9 : Strain in Steel - Column No 5

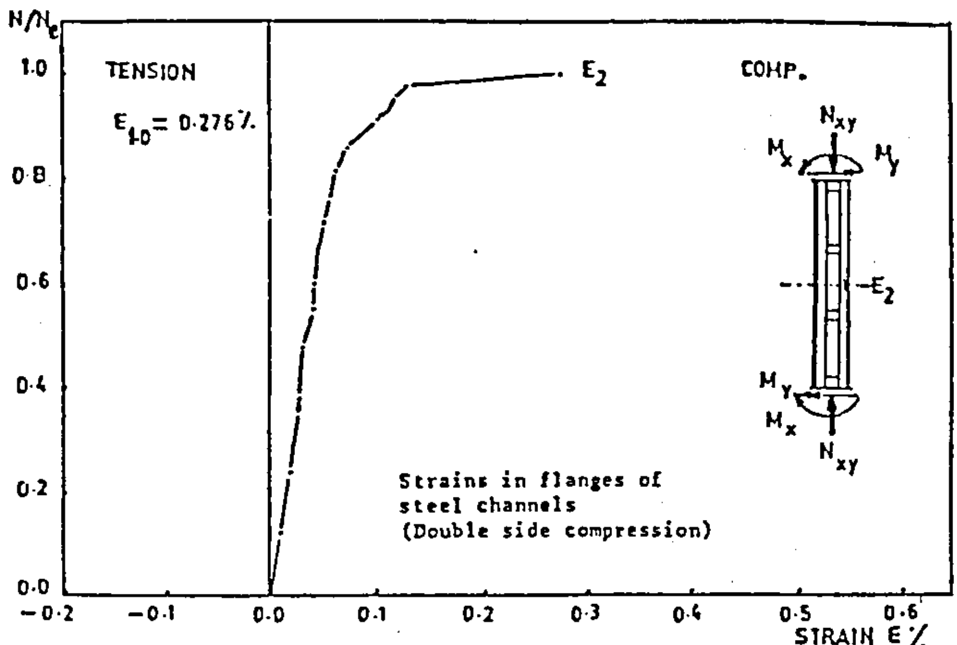


Figure 4.10 : Strain in Steel - Column No 5

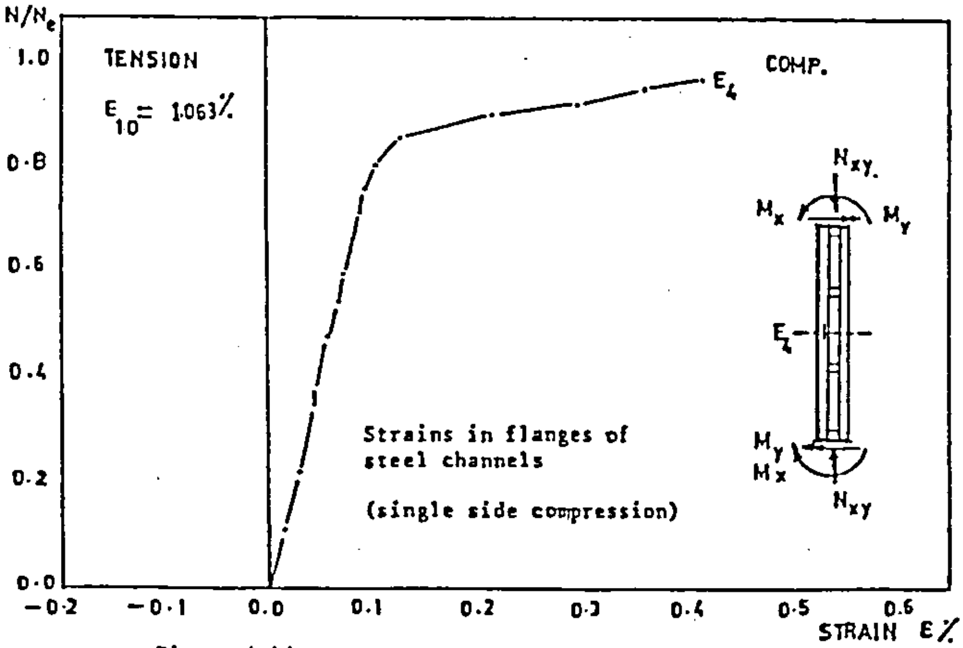


Figure 4.11 : Strain in Steel - Column No. 5

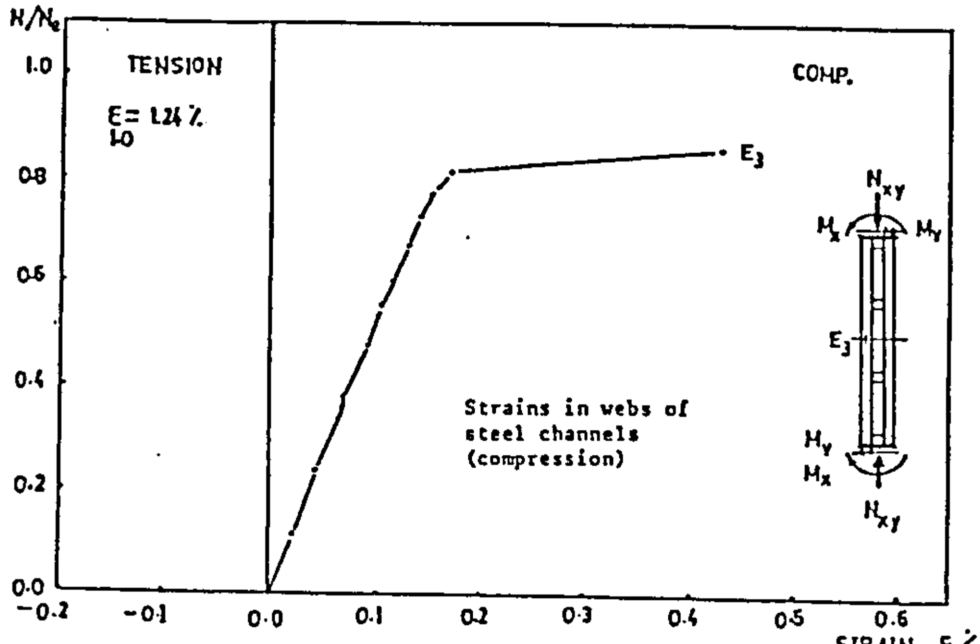


Figure 4.12 : Strains in Steel - Column No.5

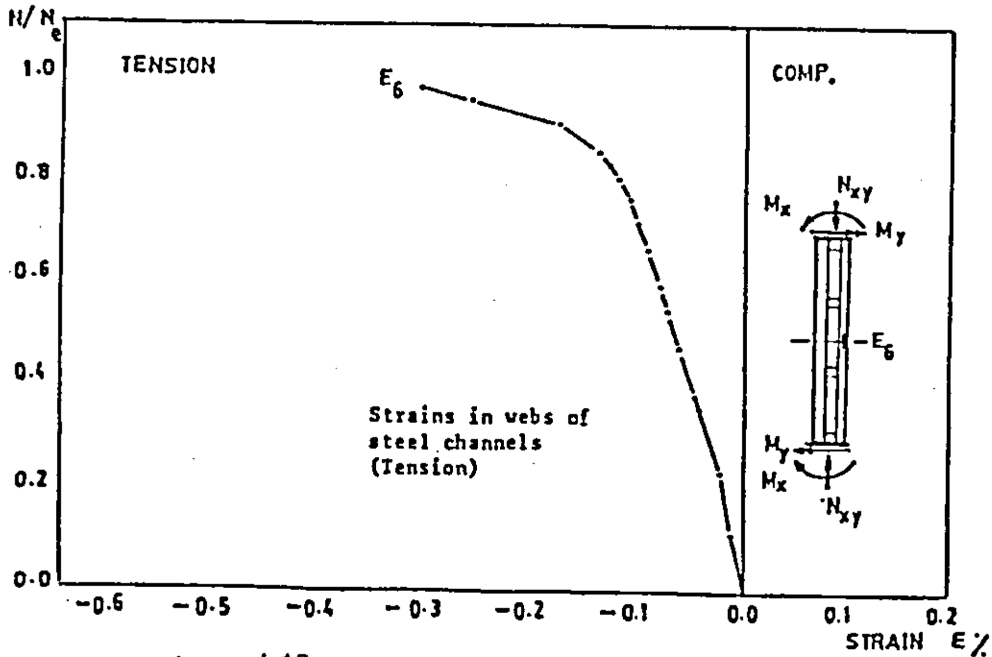


Figure 4.13: Strain in Steel - Column No.5

4.4 Deflections

The central deflections of column No.2 were plotted against the load . The load-deflection curve for column No.2 is shown in Figures (4.14) and (4.15) .

4.5 Cracks

Tension cracks were observed in the concrete during some of the tests . The tension side of the columns exhibited some cracking in concrete (one or two cracks in each panel) . The cracks were first observed between 60% and 75% of the failure load for all columns , and in all columns the crack width was measured to be less than 0.05mm when first observed , and increased to reach a maximum width around 0.2mm at failure (see Table No.4.2).

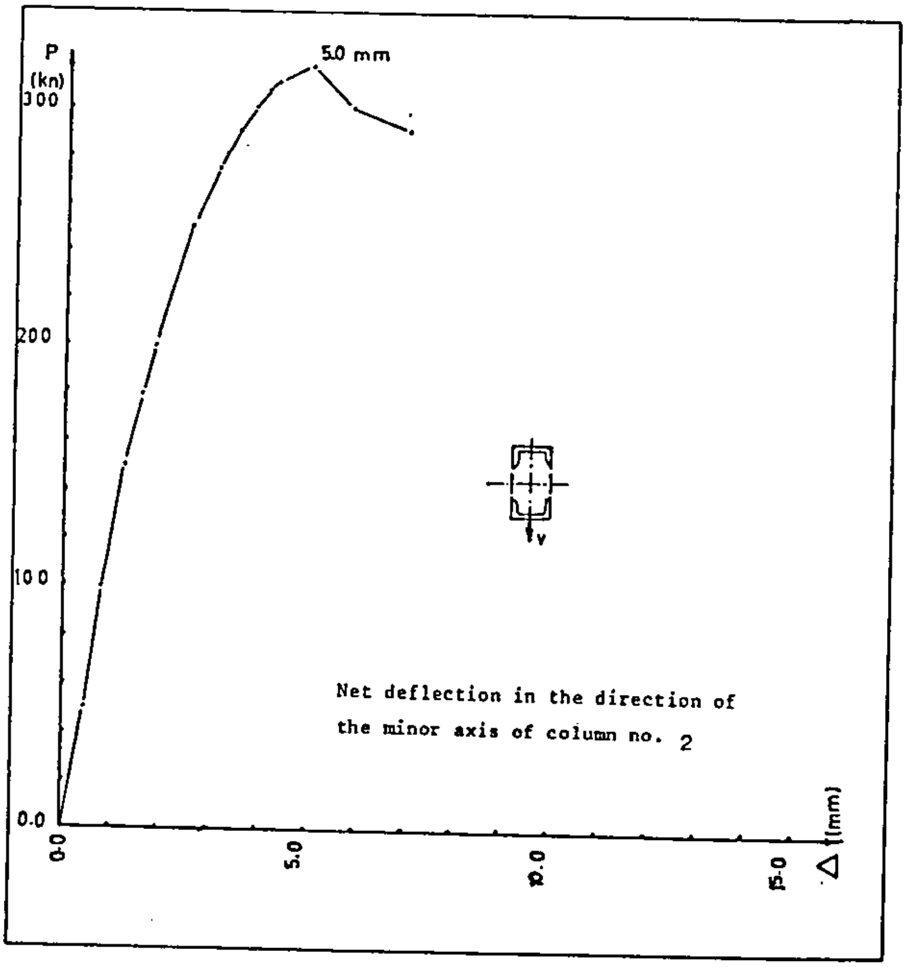


Figure 4.14 : Load-deflection curve - Column No. 2

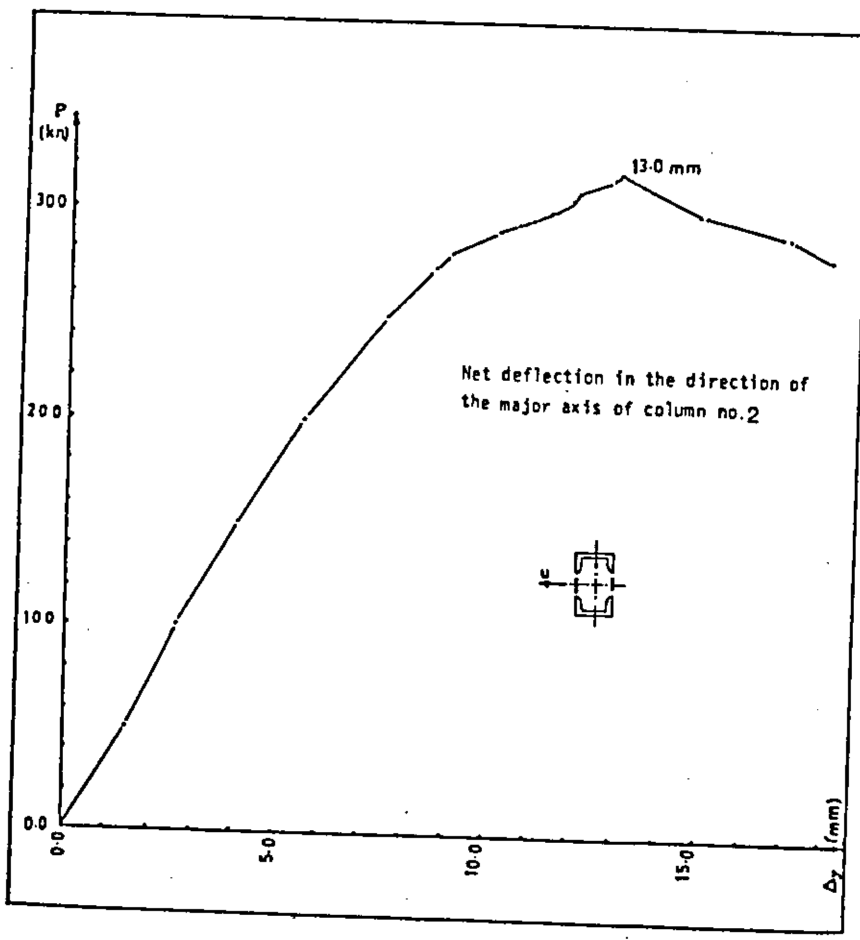


Figure 4.15 : Load-deflection curve - Column No .2

Table No.(4.2)

Column No.	e_{xy} (mm)	N_e (KN)	N_{cr} (KN) (cracking)	N_{cr}/N_e	Crack Development
1	44.7	N.R	-	-	--
2	67.0	319	240	0.75	-Two more cracks(0.05mm)
3	89.4	263	184	0.70	-Two more cracks(0.10mm)
4	100.6	231	139	0.60	-Cracks were about 0.1mm
5	111.8	209	124	0.61	-3 or 4 cracks (0.20mm)

$e_{xy} = (e_x^2 + e_y^2)^{1/2}$

N.R = No data recorded .

CHAPTER 5

DISCUSSION OF TEST RESULTS

5.1 Behaviour of Columns at Failure

As indicated in the prvious chapter , failure took place near midspan in most tests . Column No.3 showed different behaviour as the concrete crushed near the end . It was thought that the unstability of the column at the beginning of the test caused such behaviour , and to prevent such effect ,for the rest of the columns ,a small guide frame was welded to the top of the protecting frame. Another important observation must be recorded that the eccentric load must be measured with some correction due to the high stiffness of the loading plates which were used in the expeiments.A calculation made ,indicated that this correction is less than 5% of the actual value of the applied load . This shows that the actual eccentricity values would be smaller than the intended values by the applied load by a maximum of 5% .

5.2 Strains

Load-strain curves presented in the previous chapter clearly show that both the steel and the concrete act compositely up to high load levels . Compressive strains of up to 1.24% were measured in the steel channels . Compared to the yield strain of the steel (as measured from the test specimens) this value is almost four times the yield strain of the steel channels . Such a high strain value indicates an excellent performance of the steel in resisting the applied stresses ,and in reaching strain values which are much higher than the yield strain without buckling or suffering from any form of instability .Table (3.2) shows the strain gauge readings for column No.5.

These high strains occured as a result of the mutual restraint of both the concrete and the channels . The concrete is restrained from lateral expansion by the channels , whilst the channels are restrained from buckling by the contained concrete . Possible modes of buckling of steel channels are shown in Figure (5.1) . Such buckling modes occured so that the re-entrant angle of the channels remained at 90 degree .The contained concrete will prevent both types of buckling and , as a result ,enable the steel channels to sustain higher strains without buckling.

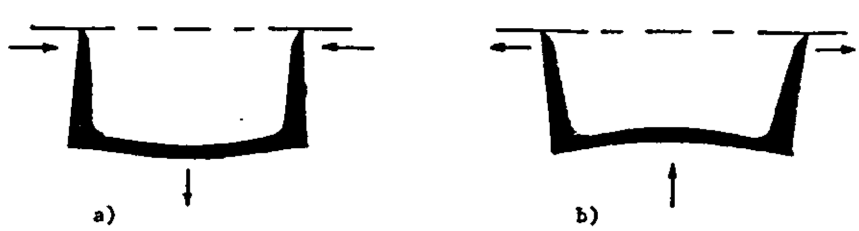


Figure 5.1: Buckling modes of steel channels

Strains measured in the batten plates in the previous tests indicate that the battens were subjected to very low tensile stresses . Strain gauge measurements (previous tests),show that the batten plates acted as ties between the steel channel flanges and that the batten plates were not subjected to significant bending stresses , as in the case of battened bare steel columns , which means that transverse shear across the column was taken by the concrete core . As seen from Table (3.1) , the length of the batten plates was 25mm and 35mm for intermediate and end battens , respectively . Complies with the recomendations of the previous studies [17,20,19],to reduce the dimensions of the batten plates used in the tested columns .

5.3 Tensile Cracking of Concrete

It has been recommended by the Bridge Code that no check for crack control need be made in

a) concrete-filled hollow steel sections , or

b) concrete-encased steel sections for which $N_{kB} < N_c$, where the value of N is given by :

$$N_c = 0.1\ f_{cu} \cdot A_c$$

in which

f_{cu} = the characteristic 28-day cube strength

A_c = the area of the concrete section .

However ,it must be remembered that the concrete subjected to tension in the cases of minor axis and biaxial bending is partially confined by the steel channels (which provide some enhancement to resist cracking) , and therefore a smaller value of N will be adequate in the case of battened composite columns . Needless to say , further data on cracks are required,especially on the effects of long term loading .

5.4 Remarks on the Test Results

a) Casting of columns :

As previously indicated , the columns were cast horizontally and therefore some air pockets can be formed beneath the top flange of the steel channels . But there is no indication of any ill-effect resulting from such voids , and it seems reasonable to ignore this possibility as would not occur in practice where the columns are cast vertically .

b) Local spalling of concrete adjacent to the battens :

The spalling always occurred at high load levels , and would not occur under serviceability conditions . Therefore , it seems advantageous to have the battens (as they were used in all the tests) flush with the outside of the steel channel flanges . This detail will enhance the composite action as the batten plates are embedded in the concrete core and function as shear connectors .

CHAPTER 6

CONCLUSIONS

The battened composite column may well have many advantages when compared to the traditional composite columns . As one aspect,the battened composite column has been found to behave well under load and the test results proved that it is a practical and viable type of composite column , thus this type should be included in the Bridge Code for strut curve selection .

Failure loads of the tested columns were always well in excess of the values predicted by the Bridge Code , and hence it is suggested that curve "a" should be used in the design of the battened composite columns . Observation during testing and the strain measurements confirm the fact that ,the steel channels and the concrete core acted compositly in resisting the applied load.Strains well beyond the yield strain of the steel channels were attained prior to failure . Also the load-deflection curves obtained confirm the visual observations that the type of failure the columns exhibited indicated no sign of either overall or local instability . The load carrying capacity of the tested columns is higher than the theoretical values . For column No.1

(with eccentricities : e_x = 40mm , e_y = 20mm) the load carrying capacity is greater than the capacity of the loading system .

The research on the battened composite columns in biaxial bending which has been carried out cannot be considered as exhaustive. Further tests might help establish a better understanding of the behaviour of the battened composite columns. For such tests the following suggestions are recommended :

1. Most of the tested columns were subjected to uniaxial simple curvature bending with equal eccentricities . It seems desirable to carry out more tests on column subjected to biaxial bending with unequal end eccentricities as well as double curvature bending .

2. All the tested columns had the same length and thus all of them had the same slenderness ratio . Tests on columns with higher senderness ratios covering awider range may provide useful information .

3. Tests on frames where the loading of the columns will be through the actual beam to column connection could be another topic of research . Such tests will allow for the investigation of the joint as well as the beam itself .

4.A finite element discretisation and subsequent computer analysis based on a satisfactory theoretical model are topics worthy of further research.

(A)

IDEALIZATION OF THE CHANNEL SECTIONS

The channel sectionshave been idealized as shown in Figure (A1.b) . The area of the idealized channel is the same as the area of the standard channel given in the Table [4] . The dimensions , T_1 and T_2 , (as shown in Figure A1.b) are given as follows :

$$T_1 = T -(b/2)\tan 5^\circ \qquad \ldots.A.1$$

$$T_2 = (A_{ss} - D_{ch}.t - 2b.T_1)/b \qquad \ldots.A.2$$

where

A_{ss} = the area of the channel section and all the dimensions are shown in Figure A.1

The idealised value of the centroid of the channel is given by : $C_{y_i} = t + (b^2 . T_1 + b^2 . T_2/3 - D_{ch} . t^2/2) / (D_{ch} . t + 2b.T_1 + b.T_2)$ A.3

The second moment of area about the major axis is given by :

$$I_{xx_i} = \{3t.D_{ch}^3 + 6b.T_1^3 + 18b.T_1 .(D_{ch} - T_1)^2 + 2b.T_2^3 + b.T_2 (3.D_{ch} - 6T_1 - 2T_2)^2\} / 36 \quad \text{.... A.4}$$

The second moment of area about the minor axis is given by :

$$I_{yy_i} = \{3 D_{ch} . t^3 + 9D_{ch} . t.(2C_{y_i} - t)^2 + 6T_1 . b^3 + 18T_1 . b(b + 2t - C_{y_i})^2 + 2T_2 . b^3 + 4.T_2 . b.(b + 3t - 3C_{y_i})^2\}/36 \quad \text{.... A.5}$$

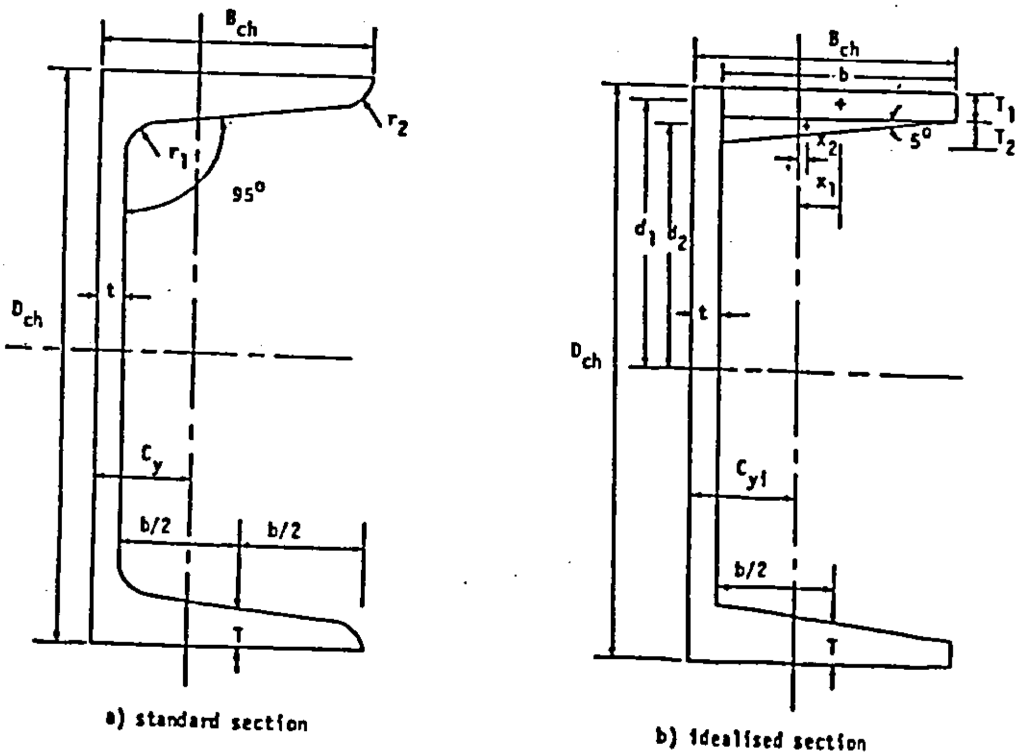


Figure A.1 : Idealisation of the channel sections

(B)

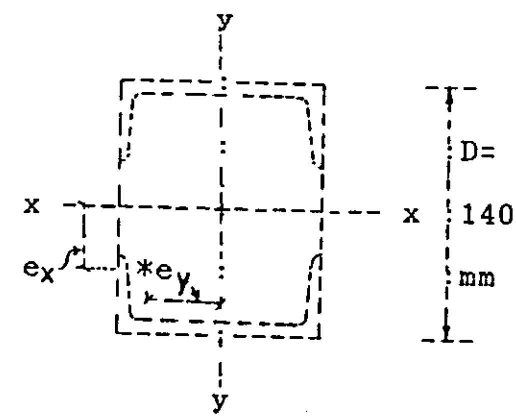
ILLUSTRATIVE EXAMPLES

Example B.1

Abattened composite column , 2m long (effective length is 2.16m), is subjected to biaxial bending with eccenticities of e_x = 40mm and e_y = 20mm at both ends . The column consists of two 80x45 steel channels spaced to give an overall depth of 140mm (Figure B.1)

Estimate the design failure load of the column , taking f_y = 340 N/mm2 , f_{cu} = 30.9 N/mm2 ,and A_{ss} = 1100 mm2 .

Figure(B.1)



(80x45mm)

steel channels

solution :

The squash load of the column :

$N_u = A_s \cdot f_{sd} + A_c \cdot f_{cd}$

$= 0.91 A_s \cdot f_y + 0.83 A_c \cdot f_{cu}$

(where $A_s = 2A_{ss}$,since two channels)

$N_u = 911503N = 911.5$ KN

The concrete contribution factor :

$\alpha = 0.83 A_c \cdot f_{cu} / N_u = 0.253$

The Bridge Code Method :

N_u = 911.5 KN , α = 0.253

The ultimate moment of resistance of the column section about the major axis bending :

$M_{ux} = 1/2 A_s \cdot f_{sd} \cdot (D-2C)$

in which C = 15.0mm , M_{ux} = 37437400 N.mm = 37.44 KN.m

The critical length :

$L_{cx} = \pi \cdot \{(E_c \cdot I_{cx} + E_s \cdot I_{sx}) / N_u\}^{1/2}$

where

$E_c = 5000(fcu)^{1/2} = 27.8$ KN/mm2

$E_s = 205$ KN/mm2

$I_{sx} = 2\{I_{yy} + 1/2 \cdot A_s \cdot (D/2-C)^2\}$

$= 199.67$ cm4

$I_{cx} = (BD^3/12) - I_{sx}$

$= 1629.7$ cm4

hence

$L_{cx} = 2.62$m

The slenderness function :

$\bar{\lambda}_x = L_{ex}/L_{cx} = 2.16/2.62 = 0.82$

From Table (13.1) curve "a" Bridge Code ,or from the formula of K_1 :

$K_{1x} = 0.73$ (from curve "a")

The design failure load of the column under major axis bending is :

$$N_{kx} = K_{1x} \cdot N_u / \{1 + (K_{1x} - K_{2x})(N_u \cdot e_x / M_{ux})\}$$

in which

$$K_{2x} = K_{20}\{[90 - 25(2\beta - 1)(1.8 - \alpha) - C_4 \cdot \bar{\lambda}_x]/30(2.5 - \beta)\}$$

where

$K_{20} = 0.9\alpha^2 + 0.2 = 0.258$

$\beta = 1.0$ (single curvature ,equal eccentricities)

$C_4 = 100$ (curve "a")

hence

$K_{2x} = -0.175$ take $K_{2x} = 0.0$

therefore

$N_{kx} = 489$ KN

The ultimate moment of resistance of the column section about minor axis bending :

$$M_{uy} = 1/2\ A_s \cdot f_{sd} \cdot (B - d) + 1/2 \cdot F_2 \cdot (d - T_1) + F_3(d/2 - T_1 - T_2/3)$$

where :

$$d = \{A_s \cdot f_{sd} - (b - t_w)(2T_1 + T_2)(2f_{sd} - f_{cd})\}/\{D \cdot f_{cd} + 2t_w(2f_{sd} - f_{cd})\}$$

$= 29.0$ mm

and

$F_2 = 2T_1 \cdot (b - t_w)(2f_{sd} - f_{cd})$

$F_3 = T_2' \cdot (b - t_w)(2f_{sd} - f_{cd})$

hence

$$M_{uy} = 21.24 \text{ KN.m}$$

The critical length :

$$L_{cy} = \pi \cdot \{(E_c \cdot I_{cy} + E_s \cdot I_{sy})/N_u\}^{1/2}$$

where

$$I_{sy} = 2I_{xx} = 213.2 \text{ cm4}$$

$$I_{cy} = D.B^3/12 - I_{sy} = 384.1 \text{ cm4}$$

therefore

$$L_{cy} = 2.3\text{m}$$

The slenderness function

$$\bar{\lambda}_y = L_{ey}/L_{cy} = 0.94$$

$$K_{1y} = 0.7 \text{ (curve "a" Figure (2.1))}$$

The design failure load of the column under minor axis bending is :

$$N_{ky} = \{ -K_4 + \{K_4^2 + 16K_{1y} \cdot K_{3y}(N_u \cdot e_y/M_{uy})^2\}\}^{1/2} . N_u / \{8K_{3y}(N_u \cdot e_y/M_{uy})^2\}$$

in which

$$K_4 = 1 + (K_{1y} - K_{2y} - 4 K_{3y})(N_u \cdot e_y/M_{uy})$$

$$K_{2y} = K_{20}\{[90 - 25 (2\beta - 1)(1.8 - \alpha) - C_4 \cdot \bar{\lambda}_y]/30(2.5 - \beta)\}$$

where

$$K_{20} = 0.9\alpha^2 + 0.2 = 0.258 , \quad \beta = 1.0$$

$K_{2y} = -0.24$ take $K_{2y} = 0.0$

(the values of K_{2y} and K_{20} are within the limits)

$$K_{3y} = 0.425 - 0.075\beta - 0.005C_4 = -0.115$$

The limits of K_{3y} are :

$$0.2 - 0.25\alpha = 0.137$$

$$-0.03(1+\beta) = -0.06$$

therefore

$$K_{3y} = -0.06$$

$$K_4 = 1.8$$

Therefore the design failure load under minor axis bending is :

$$N_{ky} = 369 \text{ KN}$$

The design failure load of the column under axial compression is :

$$N_{ax} = K_{1x} . N_u = 665.4 \text{ KN}$$

Hence the ultimate load carrying capacity of the column is :

$$N_{kxy} = 1/\{1/N_{kx} + 1/N_{ky} - 1/N_{ax}\}$$

$$= 265 \text{ KN}$$

========

Example B.2

Given data : e_x = 60mm, e_y = 30mm , f_y = 341N/mm2 , f_{cu} =33.3 N/mm2

E_c =5000(fcu)$^{1/2}$, E_s =207.5 KN/mm2

L_{ex} = L_{ey} = 2.16m (tow pin ended column sbjected under biaxial bending)

Required : Calculate the ultimate load carrying capacity of this column.

Solution : N_{kxy} = 208 KN.

Example B.3

Given Data : e_x = 80mm, e_y = 40mm, f_y = 339 N/mm2, f_{cu} = 30.9N/mm2

E_c = 5000(f_{cu})$^{1/2}$, E_s =204.1 KN/mm2

L_{ex} = L_{ey} = 2.16m

Required : As the previous example.

Solution : N_{kxy} = 197 KN.

Example B.4

Given Data : e_x = 90mm, e_y = 45mm, f_y = 342N/mm2 , f_{cu} =30.9 N/mm2

----------- E_c =5000$(f_{cu})^{1/2}$, E_s = 207 KN/mm2

L_{ex} = L_{ey} = 2.16m

Required : As the previous example .

Solution : N_{kxy} = 191 KN .

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Example B.5

Given Data : e_x = 100mm , e_y =50mm , f_y = 341N/mm2 , f_{cu} =30.2N/mm2

----------- E_c =5000$(f_{cu})^{1/2}$, E_s = 207.4KN/mm2

L_{ex} = L_{ey} = 2.16m

Required : As the previous example .

Solution : N_{kxy} = 183 KN.

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The ECCS Method

Example B.1*

Given Data : $e_x = 40mm$, $e_y = 20mm$, $f_y = 340 N/mm^2$, $f_{cu} = 30.9 N/mm^2$

$E_c = 5000(f_{cu})^{1/2}$, $E_s = 205 kN/mm^2$

$L_{ex} = L_{ey} = 2.16m$

Required : Calculate the ultimate load carrying capacity of this column .

Solution : $N_u = 911.5$ kN, $\alpha = 0.253$, $C = 15.0mm$, $M_{ux} = 37437.4 kN.mm$

$L_{cx} = 2.62m$; $\bar{\lambda}_x = 0.82$; $\bar{a} = 0.158$ (for curve a)

$K_{1x} = 0.79$ (from equation (2.2ECCS))

$K_{20} = 0.258$

$\beta = 1.0$

$C_4 = 100$

$K_{2x} = -0.175$ take $K_{2x} = 0.0$

then $N_{kx} = 406.9kN$

$d = 29.0mm$

$M_{uy} = 21240kN.mm$

$L_{cy} = 2.3m$,

$\bar{\lambda}_y = 0.94$,

$K_{1y} = 0.71$, ...(from equation (2.2ECCS))

, $K_{2y} = 0.038$

, $K_{3y} = -0.06$

, $K_4 = 1.782$

therefor $N_{ky} = 375.8kN$.

$N_{ax} = K_{1x} . N_u = 720kN$

finally $N_{kx\bar{y}} = 1/\{1/N_{kx} + 1/N_{ky} + 1/N_{ax}\} = 268kN$

Example B.2*

Given Data : (As in example B.2)

Required : As the previous example .

Solution : $N_{kx\bar{y}} = 210$ kN

Example B.3*

Given Data : (As in example B.3)

Required : As the previous example .

Solution : $N_{kx\bar{y}} = 199kN$

Example B.4*

Given Data : As in example B.4

Required : (As in the previous example)

Solution : N_{kxy} = 193kN .

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Example B.5*

Given Data : As in example B.5

Required : (As in the previous example)

Solution : N_{kxy} = 185kN

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(C)

LIST OF NOTATION

The following is a list of the most common and important notations only. All other symbols and other meanings of the symbols listed below are explained as and where they appear in the text.

A_c	Area of concrete section
A_r	Area of steel reinforcement
A_s	Area of structural steel section
A_{ss}	Area of each channel
B	Breadth of column section
B_{ch}	Depth of channel section
b	Width of flanges of channel section
C	Centroidal depth of channel from the outer face of the web
C_4	Constant
D	Depth of column section
d	Depth of neutral axis
E_c	Modulus of elasticity of concrete
E_r	Modulus of elasticity of reinforcement
E_s	Modulus of elasticity of structural steel
e	Eccentricity of load
$\bar{e}$	Eccentricity ratio
F_1	Reduction factor for axially loaded column,
F_2	Reduction factor for eccentrically loaded column
F_2	Reduction factor for eccentrically loaded column
f_{cd}	Design strength of concrete
f_{ck}	Characteristic strength of concrete

f_{cu}	Characteristic 28-day cube strength of concrete
f_{rd}	Design strength of reinforcements
f_{rk}	Characteristic strength of reinforcements
f_{sd}	Design strength of structural steel
f_{sk}	Characteristic strength of structural steel
I	Second moment of area
I_c	Second moment of area of uncracked concrete
I_r	Second moment of area of reinforcements
I_s	Second moment of area of structural steel
K	Reduction factor for a column subjected to end moments
K_1	Reduction factor for axially loaded columns
K_2	Factor to define the N-M interaction curve
K_3	Factor to define the N-M interaction curve
K_{20}	K_2 value for a column of zero length
L	Effective column length
L_c	Critical length of a column
L_e	Effective length of a column
M	Applied end moment
M_e	Experimental applied moment at failure
M_u	Ultimate moment of resistance of a column section
m	Modular ratio
N	Applied axial load on a column section

N_a	Load carrying capacity of an axially loaded column
N_{ck}	Axial load at initiation of cracks in concrete
N_{cr}	Euler critical load
N_e	Experimental failure load
N_k	Ultimate load of a column
T	Average flange thickness of channel
T_1	Flange thickness to calculate idealized properties of the channel section
T_2	Flange thickness to calculate idealized properties of the channel section
t_w	Web thickness of channel
u	Midspan deflection at failure in the direction of the x-axis
v	Midspan deflection at failure in the direction of the y-axis
α	Concrete contribution factor
β	Ratio of the smaller to the larger of the end moments with appropriate subscripts
γ_{mc}	Material partial safety factor of concrete
γ_{mr}	Material partial safety factor of reinforcements
γ_{ms}	Material partial safety factor of structural steel
Δ	Net midspan deflection.

N_u	Squash load of a column.
N_x	Design failure load of a column about the major axis.
N_{xy}	Design failure load of a column in biaxial bending.
N_y	Design failure load of a column about the minor axis.
ε	Measured strain.
ε_y	Yield strain.
η	Factor to calculate K_1.
$\bar{\lambda}$	Equivalent slenderness ratio.
λ_E	Factor to calculate K_1.
ϕ	Curvature.
ϕ_{cc}	Initial curvature of a composite column.
ϕ_{sc}	Initial curvature of bare steel column.
ψ	Imperfection factor to calculate K_1.

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VITAE

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