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# A Study on Behaviour of Symmetrical I-Shaped Column Using Interaction Diagram

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**ABSTRACT**: There are many code books which explain design of compression member including SP-16, IS-456 etc. The information's also available in the form of interaction charts, but the negative point which says, the information's which can we get from these code books is only limited to conventional sectional columns such as circular or with 4 edged columns, and one cannot get the interaction charts regarding C, T, L or I-shaped columns. The present work explains about this lack of information. This work is an attempt which deals with the behavioural study and strength of I-shaped columns by varying its parameters like grade of steel and concrete, reinforcement spacing, the obtained results explains that, noticeable increase in moment and load carrying capacity of column by increasing in grade of materials.

**KEYWORDS**: I-shaped column, Interaction charts, Compression member, Grade of steel and concrete SP- 16, IS-456.

### I. INTRODUCTION

Since design of column is time consuming process compare to other designs and also it is an iterative process. Hence for simplification SP- 16 has given non-dimensional interaction charts for different d'/D ratios along with percentage of steel. The code book SP- 16 has mentioned the procedure for interaction diagram for any column having different criteria.

For a specified concrete grade, percentage of steel, pattern of reinforcement, shape and dimension of column, interaction charts could be obtained by calculating strain at different positions of the column, there by calculating stress which intern gives moment and load values, once this couple of values obtained for various positions it then can be graphically indicated as an interaction chart which refers the Failure Envelop indicating, any set of moment and load that falls inside the envelop is said to be Safe. Also any combination that matches to the boundary of envelop is said to be the maximum load moment carrying capacity of that column.

### II. SCOPE AND SIGNIFICANCE OF THE RESEARCH WORK

Columns are mainly subjected to axial load along with uni axial or bi axial bending, hence this compression member rather than a Pure Compression Member, actually reacts like a Beam-Column. IS 456-1978 and SP – 16 gives interaction charts only for conventional sections and not for special type of sections, and there is no information regarding the behavioural characteristics of this special type of sections by means of interaction diagram. Hence this study deals with one of specially shaped RC column i.e. I- shaped columns and study the affect of varying grade and reinforcement patterns on strength characteristics of I- shaped columns. A Spread Sheet has been developed which



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could give interaction charts for I-shaped and rectangular columns with which can vary the properties like grades, spacing of reinforcement etc... hence there by making plotting of interaction chart easier.

#### III. INTERACTION DIAGRAM

- 1. It is a graph which represents the strength characteristics of a column subjected to uni axial-eccentricity under given specification. Every column out there suffers from moments and axial load due to loading through eccentrically or some unbalanced residue moments at the corner of beam which supports on column as in below fig.
- 2. If a column having loaded less than limiting load axial capacity, it can resist considerable moment along with load. For various fluctuation of load, the systems can different values of moment. Likewise, the various combinations of moment and the load for each mode of failure for particular column section is determined and plotted the resulting curve is called Interaction Curve. This is useful either for checking or designing the section. Any set of combination of moment and load that falls outside the curve denotes failure, and any set of combination that falls inside the curve denotes Safe design.



Figure 1: Typical Interaction Diagram

- 3. There are 2 types of applications if this interaction diagrams are considered, it can be used to find the capacity of a particular column also if the combination of moment and loads are known, then the diagram could be efficiently used to design a column to resist against it.
- 4. To obtain interaction charts, neutral axis is positioned at various points, the corresponding moment and load capacity at that point is calculated, the total load carrying capacity of the section is formulated as the algebraic sum of all the axial loads of concrete at various points and that of steel. Similarly, total moment capacity of the column also evaluated.

#### IV. METHODOLOGY

Neutral axis is placed at different points on the section and the corresponding moment and load were calculated, by repeating this procedure for different positions of neutral axis it is called iterative, it involves the below procedure.



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### 1. Condition for Neutral Axis Case Located Inside the Column.

a) Concrete capacity to hold axially applied loads

 $\begin{array}{l} P_{uc} = 0.361 x f_{ck} x B x X u_{max} \\ \text{In which,} \end{array}$ 

 $f_{ck}$  = Characteristic compressive strength of concrete in (MPa).

B = Section width expressed in (mm).

 $X_{umax}$  = Depth of N.A from a high compression edge in (mm).

**b**) Strain in concrete or steel ( $\epsilon_c$ ) = 0.0035 x{  $\frac{Xumax-di'}{Xumax}$ }

In which,

di' = Distance of ith bars of the steel from edge where high compression exists (mm).

c) Stresses from steel  $(f_{sc})$ :

Table below shows the stress values for concrete grades from which stresses from steel could be extracted interpolation could also are carried.

Stress Level	$f_y = 415 MPa$		$f_y = 500 MPa$	
	Strain	Stress MPa	Strain	Stress MPa
0.80f	0.00144	288.7	0.00174	347.8
0.85f	0.00163	306.7	0.00195	369.6
0.90f	0.00192	324.8	0.00226	391.3
0.95f	0.00241	342.8	0.00277	413.0
0.975f	0.00276	351.8	0.00312	423.9
$1.0f_{yd}$	0.00380	360.9	0.00417	434.8

### Table 1: Stress-Strain Values for High Yield Strength bars

**d**) Stresses from concrete (fc):

If,

 $\varepsilon_{c} < 0.002, \ f_{cc} = 446 \ x \ f_{ck} \ x \ \varepsilon_{cx} \left\{ \begin{array}{c} _{1 \ \ ( \ 250 \ \ x} \ \varepsilon_{c \ )} \end{array} \right\}.$ 

 $\epsilon_c \ge 0.002, f_{cc} = \{ 0.446 f_{ck} \}$ 

e) Total Capacity of section to hold load  $\{P_u\}$ :

= 0.361 x fck x B x  $X_{umax} + \sum \{ [ Stress from steel. - Stress concrete ] x Ast \} \}$ 

f) Total Capacity of section to hold Moment {  $M_{uc}$  }:

=  $0.361 \text{ x } f_{ck} \text{ x } B \text{ x } X_{umax} \text{ x } \{ C.G - [0.416 \text{ x } X_{umax}] \}$ 

In which, C.G =Distance of Centroid from the high compression edge.



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g) Total capacity of section to carry out Moment {Mu}:

 $= 0.361 \text{ x } f_{ck}. \text{ B x } X_{umax} \text{ x } \{C.G - [0.416 \text{ x } X_{umax}]\} + \sum \{ [ ( \text{ Stress from steel-Stress in concrete}) \text{ x } \text{ Ast }] \text{ x lever } X_{ck} = 0.361 \text{ x } f_{ck}. \text{ B x } X_{umax} \text{ x } \{C.G - [0.416 \text{ x } X_{umax}]\} + \sum \{ [ ( \text{ Stress from steel-Stress in concrete}) \text{ x } \text{ Ast }] \text{ x lever } X_{ck} = 0.361 \text{ x } f_{ck}. \text{ B x } X_{umax} \text{ x } \{C.G - [0.416 \text{ x } X_{umax}]\} + \sum \{ ( ( \text{ Stress from steel-Stress in concrete}) \text{ x } \text{ Ast }] \text{ x lever } X_{ck} = 0.361 \text{ x } f_{ck} \text{ x } \text{$ arm}

In which,

Lever arm = [C.G - di']

### 2. Condition for Neutral Axis Case Located Outside the Column.

a) Concrete capacity to hold axially applied loads

 $P_{uc} = C_1 x f_{ck} x B x X_{umax}$ In which.  $f_{ck}$  = Characteristic compressive strength of concrete in (MPa). B = Section width expressed in (mm).  $X_{umax}$  = Depth of N.A from a high compression edge in (mm).  $C_{1^{\circ}}$  = Area of stress corresponding to stress block parameter.

**b**) Strain in concrete or steel  $\{\epsilon_c\}$ 

 $= \epsilon_{cu} \ge \{ \frac{Xumax - di'}{Xumax} \}$ 

In which,

 $\epsilon_{cu}$  = Ultimate strain = 0.0035 – 0.75 x  $\epsilon_{min}$  $\epsilon_{min}$  is evaluated as  $\frac{0.0035 - 0.75 \text{ x} \text{ emin}}{\text{P} \cdot (k + 1) \text{P}} = \frac{\epsilon_{min}}{(k + 1) \text{P}}$ 

evaluated as 
$$\frac{D+(k-1)D}{D+(k-1)D} = \frac{1}{(k-1)D}$$

- Using Table 4.1 Stress from steel { fsc } can be founded c)
- d) Stress from concrete { fc }

If,

 $\begin{array}{l} \varepsilon_{\rm c} < 0.002, \ f_{\rm cc} = 446 \ x \ f_{\rm ck} \ x \ \varepsilon_{\rm c} \ x \ \{ \ _{\rm l-} [250 \ _{\rm x} \ \varepsilon_{\rm c}] \ \} \\ \varepsilon_{\rm c} \ge 0.002, \ f_{\rm cc} = 0.446 \ f_{\rm ck} \end{array}$ 

Total capacity of section to hold load { Pu } e)

> =  $C_1$  x  $f_{ck}$  x B x D +  $\sum \{ [$  Stress from steel - Stress in concrete] x Ast  $\}$ Total capacity of concrete to hold moment { Muc }

 $= C_{1^{\text{``}}} x f_{ck} x B x X_{umax} x \{ C.G - [X_{umax} x C_{2^{\text{'}}}] \}$ In which,

C.G = distance of Centroid from high compression edge.

 $C_{2^{\circ}}$  = Coefficient related to centroid distance of Stress block from high compression edge.

Total moment controlling strength of member { Mu } g)

> =  $C_1 \times f_{ck} \times B \times X_{umax} \times \{ C.G - [C_2 \times X_{umax}] \} + \sum \{ [(Stress from steel - Stress from steel$ concrete) x Ast ] x lever arm distance} In which,

Distance of lever arm = { C.G - di' }

f)



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$\mathbf{k} = \frac{\mathbf{X}_{\mathbf{u}}}{\mathbf{D}}$	$Coefficient \\ C_1' = \frac{C_c}{f_{ck} \times B \times D}$	Coefficient C <sub>2</sub> ' = $\frac{Y_c}{f_{ck} \times B \times D}$
1	0.361	0.416
1.05	0.374	0.432
1.1	0.384	0.443
1.2	0.399	0.458
1.3	0.409	0.468
1.4	0.417	0.475
1.5	0.422	0.480
2.0	0.435	0.491
2.5	0.44	0.495
3.0	0.442	0.497
4.0	0.444	0.499

Table 2: Stress Block variables for N.A located outside the section

#### 1. Condition for Neutral Axis Case Located At Infinity [ 'e' = 0 ]

 $\begin{aligned} Pu &= 0.446 \ x \ f_{ck} \ x \ A_c \ + f_s \ x \ A_{sc} \\ In \ which, \\ Ac &= Total \ concrete \ area \ in \ (mm^2) \end{aligned}$ 

If grade of steel, 
$$f_y{=}\,415$$
 ( MPa, )  $f_s=0.79~x~f_y$ 

If grade of steel,  $f_y{=}~500$  ( MPa, )  $f_s = 0.75~x~f_y$  Asc = Total area of steel at compression provided

#### V. RESULTS AND DISCUSSIONS

Using Spread Sheet interaction diagrams were developed for I-shaped column, which also provides interaction diagram for rectangular column, the developed excel sheet will also has a capability of modifying grades, reinforcement spacing, size of sections etc.. SP -16 and IS: 456-2000, are the codes from which the methods are followed to develop interaction diagram, as mentioned in the methodology, the position of neutral axis is placed at different points on the section including K>1 as shown in table 2, Values of Mu and Pu were calculated, neutral axis position is also placed in between the reinforcement rows to get a continuous and smooth interaction diagram for particular I-shaped column.

Interaction diagrams are also potted by varying grade of steel and concrete for Fe 500 and Fe 415 along with M 20, M 25, M 30, M 35, M 40 grade concrete, also comparison is made with varying reinforcement spacing with that of constant spacing and interpreted using interaction diagrams.



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A. Effect of Fe415, Fe500 grade of steel with varying grade of concrete



Figure 2: Effect of Fe415 Grade steel with Varying Grade of Concrete

### 1) For Fe415 and M20 Grade

Problem Data:<br/> $f_{ck} = 20$  MPaWidth of Flange = 1200mmDiameter of bar = 25 mmTotal Depth = 1400mmEffective Cover = 50 mmSpacing of Bar = 100 mmThickness = 200mmfy = 415 MPaTables will be obtained in the same manner for other grades also

### 2) For Fe415 and M25 Grade

### 3) For Fe415 and M30 Grade

Problem Data:  $f_{ck} = 30$  MPa Diameter of bar = 25 mm Effective Cover = 50 mm Thickness = 200mm

Width of Flange = 1200mm Total Depth = 1400mm Spacing of Bar = 100 mm fy = 415 MPa

## 4) For Fe415 and M35 Grade

Problem Data:  $f_{ck} = 35 \text{ MPa}$  fy = 415 MPaDiameter of bar = 25 mm

Width of Flange = 1200mm Effective Cover = 50 mm

## 5) For Fe415 and M40 Grade

 $f_{ck} = 40 \text{ MPa}$ fy = 415 MPa Diameter of bar = 25 mm Width of Flange = 1200mm Effective Cover = 50 mm



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For the above cases, interaction diagrams has been developed and the following graph shows the evaluation of load and moment capacities of different grade of concrete with Fe415 grade of steel, same methods has been followed to obtain the interaction diagram for varying grade of concrete with Fe500 grade of steel, obtained results then compared with ETABS- 2015 and obtained results were then analysed.



Figure 3: Comparison of Interaction chart between Manual and ETABS for fy = 415 MPa dia = 25mm and  $f_{ck}$  = 25 MPa

N.A 'X <sub>U</sub> ' (mm)	AXIAL LOAD [P <sub>u</sub> ] (kN)	MOMENT VALUE [M <sub>U</sub> ] (kN-m)
$\infty$ (PURE COMPRESSION)	18153.2607	0.0000
5600	18265.4969	190.2672
4200	18171.6620	285.3401
3500	18069.3685	376.9470
2800	17809.5941	565.3094
2100	16885.1595	1163.3836
1960	16504.8768	1407.5181
1820	16016.9983	1702.8719
1680	15422.8151	2067.0916
1540	14636.5161	2527.7252
1470	14150.4223	2810.7101
1400	13569.2406	3150.2976
1350	12684.3533	3380.3603
1325	12225.0965	3494.8326
1300	11758.5298	3607.2800
1275	11284.2074	3717.9075
1250	10801.6448	3826.9358

#### Table 3: Moment and Load Capacity for M25 Grade Concrete and Fe 415 Steel



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1200	9799.7151	4046.6247
1150	9200.3605	4363.9354
1100	8552.7654	4700.3428
1050	7849.1716	5059.8031
1000	7078.2273	5445.0755
950	6242.9653	5851.0014
900	5500.3383	6177.7542
850	4858.7987	6428.8372
800	4287.6582	6610.7946
750	3746.1375	6755.2583
700	3224.3245	6867.0056
650	2731.8110	6930.9712
600	2276.1794	6948.7429
550	1820.7164	6940.7119
500	1361.6963	6906.4665
450	892.2766	6840.8024
400	398.0651	6742.5754
350	-132.2784	6605.0961
300	-675.4821	6431.1971
250	-1339.6938	6172.0051
200	-2426.3409	5662.8667
150	-4207.4967	4699.4761
125	-5371.3524	4046.8480
100	-6776.5315	3248.0610
75	-7788.5688	2623.7843
50	-9379.2548	1590.1655
0 (PURE TENSION)	-5933.738439	-78.53477345



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Figure 4: Interaction diagram for M40,M35,M30,M25 and M20 with Fe415 as Grade of Steel

### B. Effect of Fe415, Fe500 grade of steel with varying grade of concrete

Same steps were followed to develop interaction diagram with Fe500 as steel grade results were then compared with Fe415.



Figure 5: Interaction diagram for M40,M35,M30,M25 and M20 with Fe500 as Grade of Steel

### C. Effect of Variation of Diameter on Strength Capacity of I-shaped Column

# 1. Effect of 3 layers of bars Problem Data: $f_{ck} = 30 \text{ MPa}$

Diameter of bar = 16 mmEffective Cover = 50 mmThickness = 200mm

Width of Flange = 1200mmTotal Depth = 1400mm Spacing of Bar = 100 mmfy = 415 MPa



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#### 2. Effect of Absence of Diameter in the middle layer of bar

$f_{ck} = 30 \text{ MPa}$	Width of Flange $= 1200$ mm
Diameter of bar $= 16 \text{ mm}$	Total Depth = $1400$ mm
Effective Cover $= 50 \text{ mm}$	Spacing of $Bar = 100 \text{ mm}$
Thickness = 200mm	fy = 415  MPa

#### 3. Effect of absence of bars at the centre of web

$f_{ck} = 30 \text{ MPa}$	
Diameters took = $25 \text{ mm}$	
Spacing of $Bar = 100 \text{ mm}$	
Effective Cover $= 50 \text{ mm}$	

# Total Depth = 1400mm fy = 415 MPa

Width of Flange = 1200mm

### 4. Middle 4 Rows 20mm Diameter Remaining 25mm Diameter

$f_{ck} = 30 \text{ MPa}$	
Diameters took $= 23$	5 mm
=	20 mm
Effective Cover $= 5$	0 mm

Width of Flange = 1200mm Total Depth = 1400mm Spacing of Bar = 100 mm fy = 415 MPa

### 5. Middle 4 Rows 16mm Diameter Remaining 20mm Diameter

 $\begin{aligned} f_{ck} &= 30 \text{ MPa} \\ \text{Diameters took} &= 20 \text{ mm} \\ &= 16 \text{ mm} \\ \text{Effective Cover} &= 50 \text{ mm} \end{aligned}$ 

Width of Flange = 1200mm Total Depth = 1400mm Spacing of Bar = 100 mm fy = 415 MPa



Figure 6: Typical I-shaped column for Trial 5

Similar steps were followed for other variation in diameter case and results were plotted as seen in Figure 7. The obtained results were also compared with the results obtained from ETABS as shown in Figure 3.



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Figure 7: Interaction Diagram for Effect of Variation Diameter

#### VI. CONCLUSION

- a. Higher grades of concrete or steel will result in a consequent increase in moment and load carrying capacities of I-shaped RC columns.
- b. The study pertaining to the influence of variation in bar dia of reinforcement establishes that it would result in acknowledgeable increase in maximum load and moment carrying capacities as it is nothing but increase in percentage of steel.
- c. The deviation in load moment curve generated by ETABS with that developed by analytical procedure is mainly because of the application of Whitney's stress block parameters in ETABS and Parabolic Rectangular stress block as adopted by IS 456 in the present analytical procedure.
- d. A sudden reduction in load and moment carrying capacities was observed as depicted by the load moment interaction diagram at junction of flange and web. This behaviour is because of sudden reduction in Width of flange to a new value of width of web.

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