A CRITICAL COMPARATIVE STUDY OF IS:800-2007 AND IS:800-1984

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ABSTRACT

Now-a-days, the whole world is changing over to limit state method since it is more rational. The latest version of the Code of Practice for general construction in steel IS: 800-2007 is based on Limit State Method of design. The design concept is totally changed in comparison to earlier code IS 800:1984 which is based on Elastic method. In view of this, an effort has been made to highlight the critical comparison between the important clauses of IS:800-2007 and IS:800-1984. At a glance, the present study will provide the readers a quick and clear idea about the changes in the corresponding clauses of old (IS:800-1984) and new (IS:800-2007) codes of practice.


INTRODUCTION

Codes of practice provide the minimum requirements that a design has to satisfy. In India, Bureau of Indian Standards (B.I.S.) is the statutory body that publishes the codes of practice to be followed in the Indian Professional practice. Though the codes of practices issued by B.I.S. are revised after 20 to 25 years, the second revision of IS 800 was published in 1984. The third revision of the code was released after about 24 years, in December 2007, by the B.I.S. The material contained in the code reflects the state-of-the-art of knowledge and is based on the provisions in other international codes as well as other research publications. This version of the code is based on the Limit state method of design philosophy whereas the earlier version was based on Working stress method.

The revised Code IS:800-2007 will enhance the confidence of designers, engineers, contractors, technical institutions, professional bodies and the industry will open a new era in safe and economic construction in steel.
MAJOR MODIFICATIONS

In the latest revision of IS: 800, the following major modifications have taken place:

a) The standard is based on limit state method, reflecting the latest developments and the state of the art.
b) In view of the development and production of new varieties of medium and high tensile structural steels in the country, the scope of the standard has been modified permitting the use of any variety of structural steel provided the relevant provisions of the standard are satisfied.
c) The standard has made reference to the Indian Standards now available for rivets, bolts and other fasteners.

Codal Provisions

The code is divided into the following 17 Sections. It also contains seven appendices.

a) Contents
   8. Design of Members subjected to Bending 9. Member subjected to combined forces

b) It also includes the following Annexure
   A: List of referred Indian Standards B: Analysis and design methods
   C: Design against floor vibration D: Determination of effective length of columns
   E: Elastic lateral torsional buckling F: Connections
   G: General recommendations for Steelwork Tenders and Contracts
   H: Plastic properties of beams

c) General Design Requirement
   • The new edition of IS: 800 clearly classify cross sections as to, Plastic, Compact, Semi-Compact or Slender. Separate design procedures have been laid down for each type of classification.
   • The classification has been made based on each element of the section involved and depends on the ratio of the major and minor dimension of the element i.e., limiting width to thickness ratio.

d) Limit States Method of Design
   • Separate Partial Safety Factors for different loads and combinations are considered based on the probability of occurrence of the loads. Similarly different safety factors for materials are also considered depending on perfection in material characteristics and fabrication/erection tolerances.
   • Different permissible deflections considering different material of construction have also been proposed.

e) Tension Members
   • Tension members have been designed by considering not only failure of the net cross section (after taking Shear Lag) but also considering yielding of the gross cross section and rupture of the section at the joint.

f) Compression Members
   • Design of Compression members considers the appropriate buckling curve out of total four numbers depending on the type of section and the axis of buckling. Earlier version of the
Working Stress Method of design considered only one buckling curve for all types of members irrespective of the nature of buckling.

**g) Members Subjected To Bending**
- Reduction in Flexure capacity due to high Shear Force has been elaborated in detail.
- New version introduces tension field design of plated steel girders.

**h) Members Subjected To Combined Forces**
- Moment Gradient across a member / element considered in detail, while designing against combined action of axial force and bending moment in an element of a structure.

**i) Working Stress method of Design**
- Working Stress Method (WSM) of Design has been kept in a separate chapter with minor modifications (compared to the earlier code) and in tune with the specifications of the new code to ensure smooth transition from WSM to LSM for Practicing engineers and Academicians whosoever desires.

**j) Design Against Fatigue**
- Design against fatigue has been introduced for the first time. The state-of-art concept of stress range has been introduced for the first time in this code, this code automatically supersedes IS:1024 for steel structures which considered the stress–ratio method.

**k) Earthquake Resistance**
- Response Reduction factor has been introduced and elaborated in the new edition for the first time.
- Comparing the provisions of the 1984 version of the code with that of the present code, it is seen that the present code contains major revisions.

### Comparison of Critical Parameters/Clauses of IS:800-2007 and IS: 800-1984

In the newly revised IS: 800, stress is laid to make optimum utilisation of the structural member along with provision of making adequate checks for restricting local buckling. Comparison of the critical parameters/clauses of two versions of the code (i.e. IS: 800-2007 and IS: 800-1984) is as follows:

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Clause</th>
<th>IS:800-2007</th>
<th>IS:800-1984</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>Material</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>Structural Steels</td>
<td>Table-1(Pg.14)</td>
<td>Clause: 2.1(Pg.21)</td>
<td>No change.</td>
</tr>
</tbody>
</table>

All the structural steels used in general construction, coming under the purview of this standard shall conform to IS:2062 before fabrication. Structural steel other than that specified in IS 2062 can be used provided that the permissible stresses and other design provisions are suitably modified and the steel is also suitable for the type of fabrication adopted.
### 1.2 Fasteners / Rivets / Bolts / Nuts

<table>
<thead>
<tr>
<th>Fasteners / Rivets / Bolts / Nuts</th>
<th>Clause: 2.3, Pg. 12-15</th>
<th>Clause: 2.2.2.5, 2.6, Pg. 21-22</th>
</tr>
</thead>
</table>

### 2.0 General Design Requirements

#### 2.1 Load Combination

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Clause: 3.5, Pg. 16</th>
<th>Clause: 3.4.2, Pg. 24</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) DL + IL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2) DL + IL + WL or EL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3) DL + WL or EL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4) DL + ERL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DL-Dead Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>IL-Impose Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WL-Wind Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>EL-Earthquake Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ERL-Erection Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1) DL + IL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2) DL + IL + WL or EL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3) DL + WL or EL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DL-Dead Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>IL-Impose Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WL-Wind Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>EL-Earthquake Load</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Importance is also given to the erection loading in deciding critical load combination.

#### 2.2 Section Classification

<table>
<thead>
<tr>
<th>Section Classification</th>
<th>Clause: 3.7, Pg. 17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sections are classified based on its local buckling strength and the ability to allow rotation before failing. They are</td>
<td></td>
</tr>
<tr>
<td>a) Class 1 (Plastic)</td>
<td></td>
</tr>
<tr>
<td>b) Class 2 (Compact)</td>
<td></td>
</tr>
<tr>
<td>c) Class 3 (Semi-compact)</td>
<td></td>
</tr>
<tr>
<td>d) Class 4 (Slender)</td>
<td></td>
</tr>
</tbody>
</table>

No such classification has been made.

The Class of section governs its design.
### 2.3 Increase of Stresses

**Clause: 5.3.3 (Pg. 29)**
Partial safety factors have to be considered and no increase or decrease of stresses have to be considered for individual loads.

**Clause: 3.9 (Pg. 31)**
If Wind / Earthquake Loads are considered, permissible stresses in structural steel and steel castings shall be increased by 33.33%.
If Wind / Earthquake Loads are considered, permissible stresses in rivets, bolts and nuts shall be increased by 25%.

### 2.4 Stability

**Clause: 5.5.1 (Pg. 30)**
Structure should satisfy two limit states
1. Limit state of strength
2. Limit state of serviceability
The structure should adhere to:
(a) Stability against Overturning.
The loads and effects contributing to the resistance shall be multiplied with 0.9 and added together to get the design resistance (after multiplying with appropriate partial safety factor).
(b) Sway Stability.

**Clause: 3.12 (Pg. 34)**
Restoring moment > 1.2 \( x_{max} \).
Overturning moment (due to DL) + 1.4 times max. Overturning moment (due to IL and WL/EL).
In cases where DL provides the restoring moment, only 0.9 times DL shall be considered.

### 2.5 Limiting Deflection

**Clause: 5.6.1 (Pg. 31)**
Deflection limits have been provided separately for Industrial buildings and other buildings and separate limits have been mentioned for different members.

**Clause: 3.13 (Pg. 34)**
Max. Deflection for all applicable loads (Vertical / Horizontal) = \( \frac{l}{325} \) of the span.

### 3.0 Tension Members

#### 3.1 Axial Stresses

**Clause: 6.1 (Pg. 32-34)**
Design strength of a tension member should be least of:
- Strength due to yielding of gross c/s
- Strength due to rupture of critical c/s
- Strength due to block shear

**Clause: 4.1 (Pg. 37)**
Stress on the net effective area not to exceed \( \sigma_{net} = 0.6 f_y \) (MPa).

Importance is given to serviceability requirements for various members in a structure.

Additional provision for block shear has been incorporated.
### 3.2 Maximum Slenderness Ratio

**Clause: 3.8 (Pg. 20)**
- Tension member (other than pre-tensioned) = 400
- Tension Member = 180 (reversal of stress due to loads other than WL or EL)
- Tension member = 350 (reversal of stress due to WL or EL)

**Clause: 3.7, Table 3.1 (Pg. 30)**
- Tension member (other than pre-tensioned) = 400
- Tension Member = 180 (reversal of stress due to loads other than WL or EL)
- Tension member = 350 (reversal of stress due to WL or EL)

| No change. |

### 3.3 Design strength and net effective area

**Clause: 6.2, 6.3 & 6.4 (Pg. 32-34)**

Design strength shall be minimum of $T_{dg}, T_{dr}, T_{db}$.

Where,
- $T_{dg}$ = Strength in axial tension governed by yielding of gross section
  
  \[ = A_g f_y / \gamma_{m0} \]

- $T_{dr}$ = Strength of plate in axial tension governed by rupture of net cross sectional area at holes
  
  \[ = 0.9 A_n f_y / \gamma_{ml} \]

- $T_{db}$ = Rupture strength of an angle connected through one leg governed by rupture at net section
  
  \[ = 0.9 A_{net} f_y / \gamma_{ml} + \beta A_{go} f_y / \gamma_{ml} \]

- $T_{db}$ = Strength of connection governed by block shear at an end connection of plates and angles
  
  \[ = \left[ A_g f_y / (\sqrt{3} \gamma_{m0}) + 0.9 A_{net} f_y / \gamma_{ml} \right] \]

  \[ \text{(OR)} \]

  \[ = 0.9 A_{net} f_y / (\sqrt{3} \gamma_{m0}) + A_f f_y / \gamma_{ml} \]

Where,
- $f_y$ = yield stress of material
- $A_g$ = gross area of cross Section
- $\gamma_{m0}$ = partial safety factor for

**Clause: 4.2 (Pg. 37)**

Design strength $P = \sigma_d A_{net}$

**Single angle connected through one leg:**

\[ A_{net} = A_1 + A_2 k \]

\[ k = \frac{3 A_1}{3 A_1 + A_2} \]

**Pair of angles (single tee) back-to-back connected by one leg of each angle to the same side of a gusset:**

\[ A_{net} = A_1 + A_2 k \]

\[ k = \frac{5 A_1}{5 A_1 + A_2} \]

**Double angles or tees back-to-back connected to each side of a gusset:**

If the angles are connected by tacking rivets along their length at a pitch not exceeding 1.0m, then the effective area shall be taken equal to the gross area minus the deduction for holes.

**Double angles or tees back-to-back connected to each side of a gusset:**

If the angles are not tack riveted using a pitch not exceeding 1.0m, then each angle shall be designed as a single angle connected through one leg and effective sectional area.

**Partial safety factors have introduced.**
failure in tension by

yielding=$1.10$

$\gamma_{ml}$=partial safety factor for failure at ultimate stress=$1.25$

$f_u$ = ultimate stress of the material

$A_{nc}$ = net area of the connected leg

$A_{go}$ = gross area of the outstanding leg

$\beta = 1.40 \frac{w(t_w(t_s/L) + b_s)}{w_s}$

$\geq 0.7$

Where,

$w$ = outstanding leg width

$b_s$ = shear leg width

$L_e$ = length of the end connection(i.e., distance between outermost bolts in the end joint measured along the load direction or length of the weld along the load direction)

$A_g, A_m =$ minimum gross and net area in shear along bolt line parallel to external force respectively

$A_g, A_m =$ minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force respectively.

shall be calculated using

$A_{net} = A_1 + A_2 k$

Where, $A_1=$Effective cross sectional area of connected leg (or flange of tee)

$A_2 =$ Gross area of outstanding leg (or web of the tee)

<table>
<thead>
<tr>
<th>Clause: 7.1.2 (Pg.34)</th>
<th>Clause: 5.1.1, IS:800-2007</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design compressive strength, $P_d = A_c f_{cd}$</td>
<td>Design strength, $P_d = A_g \sigma_{ac} \leq 0.6 f_y$ nor the permissible stress $\sigma_{ac}$</td>
</tr>
<tr>
<td>Where, $A_c =$ effective sectional area (i.e., gross sectional area-deduction for rivets/bolts holes area)</td>
<td>Where, $A_g =$ gross sectional</td>
</tr>
<tr>
<td>$f_{cd} =$ design compressive Stress = $f_{cd} = \frac{f_y / \gamma_{ml}}{\phi + \left[\phi^2 - \lambda^2 \right]^{1/2}} = \chi f_y / \gamma$</td>
<td>$\sigma_{ac} = 0.6 \frac{f_{ac} f_y}{\left[\left(f_{ce}\right)^n + \left(f_y\right)^n\right]^{1/n}}$</td>
</tr>
<tr>
<td>Partial safety factors and imperfection factors (based on buckling class) have been introduced for design compressive stress.</td>
<td></td>
</tr>
</tbody>
</table>
Where,
\[ \phi = 0.5[1+\alpha(\lambda - 0.2) + \Lambda^2] \]
\[ \lambda = \text{non-dimensional effective slenderness ratio} \]
\[ = \sqrt{f_c/f_{cc}} = \sqrt{f_c(KL/r)^2/\pi^2 E} \]
\[ f_c = \text{Euler buckling stress} \]
\[ = \pi^2 E/(KL/r)^2 \]

Where,
\[ KL/r = \text{effective slenderness ratio or ratio of effective length, } KL \text{ to appropriate radius of gyration, } r \]
\[ \alpha = \text{imperfection factor} \]
\[ = 0.21 \text{ for buckling class } 'a' \]
\[ = 0.34 \text{ for buckling class } 'b' \]
\[ = 0.49 \text{ for buckling class } 'c' \]
\[ = 0.76 \text{ for buckling class } 'd' \]
\[ \chi = \text{stress reduction factor} \]
\[ (\text{Table 8, IS:800-2007}) \]

<table>
<thead>
<tr>
<th>4.2</th>
<th>Axial stresses</th>
<th>Clause:7.1.2.1(Pg.34)</th>
<th>Allowable axial stress or design compressive stress (f_{cd}) shall be calculated using the formulas given in the clause or can be calculated using Tables 9(a),9(b),9(c),9(d) on the basis of buckling class of the section.</th>
<th>Clause:5.1(Pg.38)</th>
<th>Direct stress in compression shall not exceed 0.6(f_y) or as calculated by equation given in Cl. 5.1.1. Permissible stress (\sigma_{ac}) shall be taken from Table-5.1(Pg.39) for corresponding slenderness ratio.</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.3</td>
<td>Effective Length ((l)), (l = KL)</td>
<td>Clause:7.2(Pg.35-45)</td>
<td>‘K’ values shall be taken appropriately based on degree of end restraint of member as given in Table-11(Pg.45).</td>
<td>Clause:5.2(Pg.38)</td>
<td>‘K’ values shall be taken appropriately based on degree of end restraint of member as given in Table-5.2 (Pg.41&amp;42) or follow ‘K’ values given in both the codes are same.</td>
</tr>
</tbody>
</table>

Concept of imperfection factor and buckling class of the section has been introduced.
Trusses and braced frames buckling in the plane of truss, effective length ‘1’ shall be taken as between 0.7 and 1.0 times the distance between the centres of intersections, depending on degree of end restraint.
For members buckling in the plane perpendicular to truss, ‘l’ shall be taken as distance between centres of intersection.

<table>
<thead>
<tr>
<th>4.4</th>
<th>Maximum Slenderness Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Table 3, Clause: 3.8 (Pg. 20)</strong></td>
<td></td>
</tr>
<tr>
<td>Compression member = 180 (subjected to DL and IL)</td>
<td></td>
</tr>
<tr>
<td>Compression member = 250 (subjected to WL and EL)</td>
<td></td>
</tr>
</tbody>
</table>

| **Clause: 3.7 (Pg. 30)** |
| Compression member = 180 (subjected to DL and IL) |
| Compression member = 250 (subjected to WL and EL) |
| **No change.** |

<table>
<thead>
<tr>
<th>4.5</th>
<th>Built up Members with Lacing</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Clause: 7.6 (Pg. 48-50)</strong></td>
<td></td>
</tr>
<tr>
<td>Lacing of compression member shall be designed for a transverse shear equal to at least 2.5% of the axial force in the member.</td>
<td></td>
</tr>
<tr>
<td>Slenderness ratio of the lacing bars shall not exceed 145.</td>
<td></td>
</tr>
<tr>
<td>Angle of inclination with the axis of 40° to 70° to the axis of built up section member (for both single &amp; double lacing).</td>
<td></td>
</tr>
<tr>
<td>Max. spacing of lacing shall be such that min. slenderness ratio (l/r) of the components of the member between consecutive connection is not greater than 50 or 0.7 times the most unfavourable (l/r) of the member as a whole, whichever is less.</td>
<td></td>
</tr>
</tbody>
</table>

| **Clause: 5.7.2 (Pg. 47)** |
| Lacing of compression member shall be designed for a transverse shear equal to at least 2.5% of the axial force in the member. |
| Slenderness ratio of the lacing bars shall not exceed 145. |
| Angle of inclination with the axis of member 40° to 70°. (for both single & double lacing) |
| Max. spacing of lacing shall be such that min. slenderness ratio (l/r) of the components of the member between consecutive connection is not greater than 50 or 0.7 times the most unfavourable (l/r) of the member as a whole, whichever is less. |
| **No change in the load calculations and basic design requirements.** |
### 4.6 Built up Members with Battens / Tie plates

**Clause: 7.7 (Pg. 50-52)** Battens shall be designed to carry the bending moment & shears arising from transverse shear force ‘V’ of 2.5% of the total axial force on the whole comp. member, at any point in the length of the member, divided equally between parallel planes of battens. Spacing of battens centre to centre of end fastenings shall be such that the slenderness ratio \((l/r)\) of the lesser main component over that distance shall be not greater than 50 or greater than 0.7 time the slenderness ratio of the member as a whole, about its x-x axis. (axis parallel to the battens)

No changes have been made in the load calculation and basic design requirements.

### 4.7 Column Base Plate

**Clause: 7.4 (Pg. 47)**

Minimum thickness of slab base under axial compression shall be

\[
t = \sqrt{2.5w(a^2 - 0.3b^2)\gamma_{m0}/f_y} > t_f
\]

Where,

- \(w\) = uniform pressure from below on the slab base under the factored load axial compression (MPa)
- \(a, b\) = larger and smaller projection, respectively of the slab base beyond the rectangle circumscribing the column (mm)
- \(t_f\) = flange thickness of compression member (mm)
- \(f_y\) = yield strength of steel.
- \(\gamma_{m0}\) = partial safety factor against yield stress and buckling.

**Clause: 5.4 (Pg. 44)**

Minimum thickness of slab base shall be

\[
t = \sqrt{3.0w(a^2 - 0.25b^2)/\sigma_{bs}}
\]

Where,

- \(w\) = pressure or loading on the underside of the base (MPa)
- \(a, b\) = larger and smaller projection, respectively of the slab base beyond the rectangle circumscribing the column (mm)
- \(t\) = slab thickness (mm)
- \(\sigma_{bs}\) = permissible bending stress in slab base (for all steels assumed as 185MPa)

The concept of effective area for load transfer has been introduced.
### 5.0 Members subjected to bending

#### 5.1 Bending Stress

**Laterally Supported Beams:** Design strength in bending shall be calculated as per the formulas given on the basis whether the section is susceptible to shear buckling before yielding. **Laterally Unsupported Beams:** Design strength in bending shall be calculated as per the formulas given and resistance to lateral torsional buckling should not be checked for:

- a) bending is about minor axis
- b) Section is hollow or a solid bar
- c) In case of major axis bending, the non-dimensional slenderness ratio is less than 0.4.

**Clause:** 8(Pg.52-59)

**Clause:** 6.2(Pg.55)  
Max. permissible stress, \( \sigma_{bc} \) or \( \sigma_{sc} \) = 0.66fy (For strong & weak axis bending)

Max. permissible stress \( \sigma_{bc} \) or \( \sigma_{sc} \) for I-beams & Channels (based on section properties and \( l/r_y \) ) shall be referred from Table-6.1A to 6.1F(Pg.57-62) as appropriate.

For beams & plate girders, max. permissible \( \sigma_{bc} \) shall be computed as per equation given in Cl. 6.2.3(Pg.56) or Table-6.2(Pg.64) may be referred for \( \sigma_{bc} \) calculated from \( f_{cb} \) for different values of fy. (All stresses in MPa)

**Clause:** 6.3(Pg.68)  
Max. permissible bearing stress on net area of contact, \( \sigma_p = 0.75 f_y \)

The partial safety factors are based on the values given in Euro code.

#### 5.2 Bearing Stress

**Clause:** 8.7.4(Pg.67)

Should be less than the yield stress of the steel divided by Partial safety factor i.e., \( f_y/1.1 \)

**Clause:** 6.3(Pg.68)

Max. permissible bearing stress on net area of contact, \( \sigma_p = 0.75 f_y \)

#### 5.3 Shear stresses

**Clause:** 8.4(Pg.59-60)

Nominal plastic shear resistance under pure shear should be calculated using the formulas

\[
V_n = V_p, \quad V_p = \frac{A_{yw} f_{yw}}{\sqrt{3}}
\]

shear are as specified for various sections. Resistance to shear buckling can be verified based on the value of ratio of depth to web thickness. Two methods have been specified for calculation of nominal shear strength. They are

**Clause:** 6.4(Pg.68)

Max. permissible Shear stress, \( \tau_{wm} = 0.45 f_y \)

Average shear stress in member calculated on the cross section of the web shall not exceed the limits as mentioned in Cl. No.6.4.2 (Pg.69). Also refer Table-6.6A, B, C (Pg.73-75) for stiffened webs.

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### 5.4 Effective length of compression flanges & max. slenderness ratio

Clause: 8.6.1.2 (Pg. 64)
No specific criteria are mentioned.
But in order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the criteria’s specified.

*Max. Slenderness ratio for Compression flange of beam = 300 Cl. 3.7, Pg. 3.1
*For Cantilever beams of projecting length ‘L’, refer Cl. No. 6.6.3, Pg. 77.

### 6.0 Combined stresses

#### 6.1 Axial Compression & Bending

Clause: 9.3.1 & 9.3.2.2 (Pg. 70-71)
Members subjected to combined axial compression and biaxial bending should satisfy the relationship:

\[
\left( \frac{\sigma_{ac}}{\sigma_{ac,cal}} \right) + \left( \frac{\sigma_{bc}}{\sigma_{bc,cal}} \right) \leq 1.0
\]

Where, \( C_m \), \( C_c \) = equivalent uniform moment factor as per Table 18
\( P \) = applied axial compression under factored load.
\( M_y, M_z \) = maximum factored applied bending moments about \( y \) and \( z \) axis of the member.
\( P_{ac}, P_{bc} \) = design strength under axial compression

Clause: 7.1.1 (Pg. 90)

\[
\left( \frac{\sigma_{ac,cal}}{\sigma_{ac}} \right) + \left( \frac{\sigma_{bc,cal}}{\sigma_{bc}} \right) \leq 1.0
\]

\( \sigma_{ac} \) = permissible axial
\( \sigma_{bc} \) = calculated bending compressive stress in extreme fibre.
governed by buckling about minor (y) and major (z) axis  
\[ M_{dy}, M_{dz} = \text{design bending strength about y (minor) or z (major) axis considering laterally unsupported length of the cross section.} \]

\[ K_y = 1 + (\lambda_y - 0.2) n_y \leq 1 + 0.8 n_y \]
\[ K_z = 1 + (\lambda_z - 0.2) n_z \leq 1 + 0.8 n_z \]

Where,
\[ n_y, n_z \] = ratio of the actual applied axial force to the design axial strength for buckling about y and z axis.

\[ C_{m,T} = \text{equivalent uniform moment factor for lateral torsional buckling as per Table 18 corresponding to the actual moment gradient between lateral supports against torsional deformation in the critical region under consideration.} \]

### 6.2 Axial Tension & Bending

Clause: 9.3.1 & 9.3.2.1 (Pg.70-71)

The reduced effective moment, \( M_{eff} \), under tension and bending should not exceed the bending strength due to lateral torsional buckling, \( M_d \).

\[ M_{eff} = \left[ M - \left( \psi TZ_{ec} / A \right) \right] \leq M_d \]

Where,
\[ M, T = \text{factored applied moment and tension} \]
\[ A = \text{area of cross section} \]
\[ Z_{ec} = \text{elastic section modulus of the section w.r.t. extreme compression fibre} \]
\[ \psi = 0.8, \text{if T and M can vary independently or otherwise}=1.0 \]

Clause: 7.1.2 (Pg.90)

Member should satisfy the following condition.

\[ \left( \frac{\sigma_{at,cal}}{0.6 f_c} \right) + \left( \frac{\sigma_{bt,cal}}{0.66 f_y} \right) + \left( \frac{\sigma_{ec,cal}}{0.66 f_y} \right) \leq 1.0 \]

Where,
\[ \sigma_{at,cal} = \text{calculated average axial tensile stress} \]
\[ \sigma_{bt,cal} = \text{calculated bending tensile stress in extreme fibre} \]
\[ x, y = \text{represent x-x and y-y planes} \]

Separate governing equations are specified for different types of sections.
### 6.3 Bending & Shear

| Clause: 9 (Pg. 69-70) | Moment carrying capacity of the section shall be reduced by the amount as specified in the code (for high shear force). No reduction is required for shear force value < 60% of allowable shear capacity of the section. |
| Clause: 7.1.4 (Pg. 91) | Equivalent stress calculated by the equation given in this clause shall not exceed the value, \( \sigma_e = 0.9 f_y \). |
| --- | The moment reduction is dictated by the percentage of shear force w.r.t. allowable shear force in the section. |

### 6.4 Bearing, Bending & Shear

| Clause: 7.1.5 (Pg. 92) | Equivalent stress calculated using the equation \( \sigma_{tot} = \sqrt{\sigma_{axial}^2 + \sigma_{tension}^2 + \sigma_{shear}^2 + 3\sigma_{bending}^2} \) (OR) \( \sigma_{tot} = \sqrt{\sigma_{axial}^2 + \sigma_{tension}^2 + \sigma_{shear}^2 + 3\sigma_{bending}^2} \) shall not exceed the value, \( \sigma_e = 0.9 f_y \). |

---

## 7.0 Connections

### 7.1 Bolted

#### 7.1.1 Permissible Stresses for bolts

| Clause: 10 (Pg. 73-77) | No specific value is prescribed. Procedure given for calculation of permissible loads (Axial Tension, Shear & Bearing). |
| Clause: 8.9.4 (Pg. 95), Table 8.1 | a) Axial tension 120 MPa b) Shear 80 MPa c) Bearing 250 MPa |

#### 7.1.2 Combined shear and tension in bolts

| Clause: 10.4.6 (Pg. 77) | No specific value is provided. Procedure given for calculation of permissible loads. |
| Clause: 8.9.4.5 (Pg. 96) | Individual stresses should not exceed allowable values and combined stress ratio should not exceed 1.40. |

#### 7.1.3 Minimum pitch

| Clause: 10.2.2 (Pg. 73) | Shall not be less than 2.5 times the nominal diameter of the fastener (Bolt/Rivet) |
| Clause: 8.10.1 (Pg. 96) | Shall not be less than 2.5 times the nominal diameter of the bolt. |
| No change has been made. |

#### 7.1.4 Minimum edge distance

<p>| Clause: 10.2.4.2 (Pg. 74) | Should be &gt;1.7 times hole dia. for sheared or hand-flame cut edges, &amp; &gt;1.5 times hole dia. for rolled, machine-flame cut, sawn and planed edges, from the centre of the hole. |
| Clause: 8.10.2 (Pg. 97), Table 8.2 | Distance from the centre of any hole to the edge of a plate shall not be less than that specified in Table 8.2. When two or more parts are connected together, a line of bolts shall be provided at a distance of |
| Not much variation is observed in the end results. |</p>
<table>
<thead>
<tr>
<th>7.1.5 Maximum pitch</th>
<th>Clause:10.2.3(Pg.74) Shall not exceed 32t or 300mm whichever is less, where t is the thickness of the thinner outside plate.</th>
<th>Clause:8.10.1(Pg.96) Shall not exceed 32t or 300mm whichever is less, where t is the thickness of the thinner outside plate.</th>
<th>No change has been made.</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1.6 Maximum edge distance</td>
<td>Clause:10.2.4.3(Pg.74) Shall not exceed 12tε , where t is the thickness of the thinner plate, and ε = (250/f_y)^1/2</td>
<td>No specific criteria are mentioned.</td>
<td>---</td>
</tr>
<tr>
<td>7.1.7 Clearance for fastener Holes</td>
<td>Clause:10.2.1(Pg.73) As given in Table 19</td>
<td>Clause:3.6.1.1(Pg.28) 1.5 mm in excess of the nominal diameter of the bolt irrespective of the diameter of the bolt, unless otherwise specified.</td>
<td>More practical aspect for clearance has been considered.</td>
</tr>
</tbody>
</table>

### 7.2 Welded

#### 7.2.1 Fillet welds

<table>
<thead>
<tr>
<th>7.2.1.1 Permissible stresses</th>
<th>Clause:10.5(Pg.78) Shear stress shall not exceed 110 MPa nor as calculated using clause 10.5.7(Pg.79-81)</th>
<th>Clause:8.9.4.7, IS:800-1984Clause:7.1.2 IS:816-1969(Pg.17) Shear stress shall not exceed 110 MPa.</th>
<th>---</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.2.1.2 Effective throat thickness</td>
<td>Clause:10.5.3(Pg.78) Shall not be &lt; 3mm and not &gt; 0.7t, where t is the thickness of the thinner plate. For stresses calculation in fillet welds joining faces inclined to each other, effective throat thickness shall be taken as K times the fillet size, where K is a constant.</td>
<td>Clause:8.9.4.7, IS:800-1984Clause:6.2.3, IS:816(Pg.10) Shall not be &lt; 3 mm and not &gt; 0.7t, where t is the thickness of the thinner plate. For stresses calculation, the effective throat thickness shall be taken as K times the fillet size, where K is a constant.</td>
<td>No Changes have been suggested</td>
</tr>
<tr>
<td>7.2.1.3 Effective length</td>
<td>Clause:10.5.4(Pg.78) Shall be the overall length of weld excluding end returns in case of Fillet welds and shall be the</td>
<td>Clause:8.9.4.7, IS:800-1984Clause:6.2.4,IS:816(Pg.11) Shall be the overall length of the weld plus twice the</td>
<td>---</td>
</tr>
<tr>
<td>Section</td>
<td>Description</td>
<td>Clause/Reference</td>
<td>Clause/Reference</td>
</tr>
<tr>
<td>---------</td>
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</tr>
<tr>
<td>7.2.1.4</td>
<td>Effective area of weld</td>
<td>Clause:6.2.3(Pg.10) Effective length times Effective throat thickness</td>
<td>Clause:8.9.4.7, IS:800-1984, Clause:6.2.3, IS:816(Pg.10) Effective length times effective throat thickness</td>
</tr>
<tr>
<td>7.2.1.5</td>
<td>Minimum length of weld</td>
<td>Clause:10.5.4(Pg.78)</td>
<td>Clause:8.9.4.7, IS:800-1984(Clause:6.2.4.1,IS:816 (Pg11) Shall not be less than four times the size of the weld.</td>
</tr>
<tr>
<td>7.2.1.6</td>
<td>Minimum size of the weld</td>
<td>Clause:10.5.2(Pg.78)</td>
<td>Clause:8.9.4.7, IS:800-1984, Clause:6.1.3, IS:816(Pg.5) Shall not be less than 3 mm nor more than the thickness of the thinner part joined. The minimum size of the first run or the single run weld shall be as given in Table 21(Pg 78).</td>
</tr>
<tr>
<td>7.2.2</td>
<td>Butt Welds</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.2.2.1</td>
<td>Permissible stresses.</td>
<td>Clause:10.5.7(Pg.79) Stresses in weld shall not exceed those permitted in the parent metal.</td>
<td>Clause:8.9.4.7, IS:800-1984, Clause:7.1.1, IS:816(Pg.16) Stresses in weld shall not exceed those permitted in the parent metal.</td>
</tr>
<tr>
<td>7.2.2.2</td>
<td>Minimum size of weld</td>
<td>Clause:10.5.2.4(Pg.78) Size of butt weld shall be specified by the effective throat thickness.</td>
<td>Clause:8.9.4.7, IS:800-1984, Clause:6.1.3, IS:816(Pg.5) Size of butt weld shall be specified by the effective throat thickness.</td>
</tr>
<tr>
<td>7.2.2.3</td>
<td>Effective area of weld</td>
<td>Clause:10.5.4(Pg.78-79) Effective length times the effective throat thickness</td>
<td>Clause:8.9.4.7, IS:800-1984, Clause:6.1.6, IS:816(Pg.7) Effective length times the effective throat thickness</td>
</tr>
<tr>
<td>7.2.2.3</td>
<td>Effective throat thickness</td>
<td>Clause:10.5.3.3.(Pg.78) For complete penetration, effective throat thickness</td>
<td>Clause:8.9.4.7, IS:800-1984 Clause:6.1.4, IS:816(Pg.6) For complete penetration,</td>
</tr>
</tbody>
</table>
shall be taken as thickness of thinner part joined.

For an incomplete penetration, effective throat thickness shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcement.

effective throat thickness shall be taken as the thickness of thinner part joined.

For incomplete penetration, effective throat thickness shall be taken as the thickness of the weld metal common to the parts joined, excluding reinforcement.

### 8.0 Gantry Girder

#### 8.1 Increase in stresses

| Clause:3.9.3(Pg.31) | While considering simultaneous effects of vertical & horizontal surge loads of cranes for the combination given in Cl. 3.4.2.3 & 3.4.2.4, the permissible stresses may be increased by 10 %. | Stresses are to be calculated using adequate Partial Safety factors. |

#### 8.2 Limiting deflection

| Clause:5.6.1(Pg.31) | Vertical deflection: Under DL and IL shall not exceed the following:
- i. L/500, where manually operated cranes are operated and for similar loads
- ii. L/750, where electric overhead travelling cranes operated up to 50 tonnes
- iii. L/1000, where electric overhead travelling cranes operated over 50 tonnes
- iv. L/600, other moving loads such as charging cars etc.

L=span of the crane runway girder. |

**Horizontal deflection:** At the caps of columns in single storey buildings, the horizontal deflection due to lateral forces should not exceed 1/325 of the actual length ‘L’ of the column. | --- |
### 9.0 Design and Detailing for Earthquake Loads

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Details</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| 9.1 | Load Combination | Clause: 12.2 (Pg. 87)  
Two more combinations have to be considered  
1) 1.2 DL + 0.5 LL ± 2.5 EL  
2) 0.9 DL ± 2.5 EL | No such criteria are given. |
| 9.2 | Lateral Load Resisting System | The Building has been classified as  
1) Braced Frame System  
a) Ordinary concentrically Braced Frames (OCBF)  
b) Special Concentrically Braced Frame (SCBF)  
c) Eccentrically Braced Frame (EBF)  
2) Moment Frame System  
a) Ordinary Moment Frame (OMF)  
b) Special Moment  
3) Frame (SMF)  
Various criteria for loads on members are specified for different lateral load resisting systems. | No such classification has been made |

### 10.0 Fatigue

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Details</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| 10.1 | Reference design Condition | Clause: 13.2.1 (Pg. 91)  
Conditions when fatigue design becomes necessary are mentioned along with a plot of standard S-N curve for each category.  
A capacity reduction factor \( \mu_r \) is to be applied when plates greater than 25 mm tk. Are joined by transverse fillet or butt welding. | No such criteria are mentioned. |
| 10.2 | Partial Safety Factors | Clause: 13.2.3 (Pg. 92)  
Based on consequences of fatigue failure, component details have been classified and Partial Safety Factors are given for each type. (Refer Table 25, Pg. 92) | No such criteria are mentioned. |
| 10.3 | Detail Category | Clause: 13.3 (Pg. 92-98)  
Tables 26 (a) to (d) indicate the classification of | No such criteria are mentioned. |
different details into various categories for the purpose of assessing fatigue strength.

<table>
<thead>
<tr>
<th>11.0</th>
<th>Fire Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.1</td>
<td>Clause: 13 (Pg. 105-110)</td>
</tr>
</tbody>
</table>

Following points have been discussed and relevant design standards have been mentioned:

- Fire Resistance Level
- Period of Structural adequacy
- Variation of mechanical properties of Steel with Temperature
- Limiting Steel Temperature
- Thermal Increase with Time in Protected members
- Temperature increase with Time in unprotected Members
- Determination of period of Structural adequacy from a single test
- Three-Sided Fire exposure condition.

No such criteria are mentioned.

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CONCLUSION

An explicit comparison of important clauses of IS:800-2007 and IS:800-1984 presented in this paper gives a quick insight to the readers about the changes made in corresponding clauses of the old and latest codes of practice.

REFERENCES