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Comparison Between Different Steel Codes

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ملخص البحث:
يهدف البحث الى دراسه مقارنه بين اكواد المنشآت المعدنيه المختلفه مثل الكود الامريكى والكود الاوربى التى يتم استخدامهم عالميا على نطاق واسع فى تصميم المنشآت المعدنيه مع الكود المصرى فى بعض الموضوعات الخاصه بالتصميم. تم عمل الدراسه بوضع معادلات التصميم و حدود التصميم القصوى فى ملف واحد لاستنتاج التشابهات و الإختلافات فى حسابات الاجهادات و تسهيل تعلم هذه الاكواد. تم عمل منحنيات و جداول للمقارنه بين معادلات التصميم, الاجهادات المسموحه و معاملات الامان. لكى تتم المقارنه تم عمل برنامج تصميم بإستخدام لغه البيسك المرئى لاستخدامه فى التصميم و الحصول على الجدول و المنحنيات لعمل المقارنه.

Abstract:

This paper presents a study for comparison between different steel codes American Institute of Steel Construction (AISC) and EUROCODE 3 (EC3), which widely used globally in design of steel structure, with E.C.P in many objects in design. The study has been undertaken to put together the expressions and limits presented in E.C.P, AISC and EC3 codes in a single document, to identify the similarities and the differences in calculations of strengths and to facilitate a rapid learning to these this codes .Design equations, allowable stress and safety factors are directly compared with each other wherever possible using clear tables and curves. To make this comparison, an overall program is performed with visual basic programming language a steel program was built to get results from design multi sections and drawing some charts.

1. Introduction:

The comparison is making on different objectives between the three codes such as Load factors and load combinations, material and grades of steel, design of tension member, design of compression member, design of beam and design of beam-column. In each objective there are charts and tables making by the built program to show the similarities and differences between the three codes. The comparison also shows the similarities and differences between the design equations, safety factors, allowable stresses and slenderness limitations.

2. Load Factors and Load Combinations:

Load factors and load combinations are the first step in design any sections, to calculate the design force. Load factors and load combinations will be discussed now for E.C.P, AISC and EC3 specifications to get the similarities and differences between codes.

For AISC, load factors and load combinations was defined by ASCE in its publication (ASCE Standard 7-05) not by AISC and the structures should be designed with this combinations, if they are designed according to AISC. But in E.C.P load factors and load combinations are defined in the same codes for ASD or LRFD. E.C.P and AISC take loads like dead, live, wind, earthquake in consideration but AISC take although rain, snow, flood load in its equation. Load factors and load combinations have the same equation form and some factors are the same except wind load it is 1.3 in E.C.P(LRFD)***[6]*** and 1.0 in AISC***[3]***.

For EC3, Load factors and load combinations was defined in the Basis of Structural Design (EN 1990) in EC3 not in the (EN 1993.1.1) specifications where the design equation founded. In EC3 load combinations generally referred as combinations of actions. EN 1990 deals with ultimate limit states and serviceability limit states, as ultimate limit states responsible of making people and the structure in safe, while other deals with the appearance of the structure and the comfort of people***[14]***. For ultimate limit states, the following checks should be carried out for the following, as relevant: EQU, STR, GEO and FAT as defined below.

• EQU: Loss of static equilibrium of the structure or any part of the structure.

• STR: Internal failure or excessive deformation of the structure or structural members.

• GEO: Failure or excessive deformation of the ground.

• FAT: Fatigue failure of the structure or structural members.

Combinations of actions are presented in EN 1990 for the four cases: persistent, transient, accidental and seismic. Combinations of actions defined the persistent and transient cases as the fundamental combinations. For combinations of actions for persistent or transient design cases (fundamental combinations) at ultimate limit states may be calculated by equations(1) or(2) or (3) ***[7]***. The National Annex has a vision to allow the use of these equations, though it should be noted that Equations (2) and (3) will provide more proper combinations of actions. Equation (3) is needed to be considered for strength (STR) verifications. For verifying equilibrium like sliding or over turning, only Equation (1) may be applied***[5]***.The load combination expressions, as they appear in Euro code, are provided below:

$$\sum_{j\geq 1}\gamma_{G,j}G_{K,j}\text{ ``}+\text{'' }\gamma_P\text{ P`` }+\text{ ``}\gamma_{Q,1}\,Q_{K,1}\text{````}\sum_{j>1}\gamma_{Q,j}\Psi_{0,1}Q_{K,i} \tag{1}$$

$$\sum_{j\geq 1}\gamma_{G,j}G_{K,j}\text{ ``}+\text{'' }\gamma_P\text{ P`` }+\text{ ``}\gamma_{Q,1}\,\Psi_{0,1}\,Q_{K,1}\text{````}\sum_{j>1}\gamma_{Q,j}\Psi_{0,1}Q_{K,i} \tag{2}$$

$$\sum_{j\geq 1}\xi\,\gamma_{G,j}G_{K,j}\text{ ``}+\text{'' }\gamma_P\text{ P`` }+\text{ ``}\gamma_{Q,1}\,Q_{K,1}\text{````}\sum_{j>1}\gamma_{Q,j}\Psi_{0,1}Q_{K,i} \tag{3}$$

3. Steel grades in Accordance with E.C.P, EC3 and AISC Provisions:

Varieties of steel grades utilized by EC3 include S235, S275, S355 and S450***[7]***. Those utilized by AISC are A36,A 529 (Gr.50 and Gr. 55), A 572 (Gr.42, Gr. 50, Gr.55 and Gr.60) and A992***[4]***. For E.C.P St37, St44 and St52 are utilized***[9]***.The most widely used in the construction industry of these are S235, S275, S355,A36,A 529 (Gr.50 and Gr. 55), A 992, St37 and St44.

It is important to mention that in AISC and EC3 steel grades which used in hot rolled sections like I-beam and angels aren`t allowed to use in Hollow Square Sections and Pipes***[4]***, where in E.C.P all sections has the same grades.

From the steel grades mentioned above, table (1) listed the equivalences steel grades in AISC and EC3 to E.C.P.

EC3	AISC	E.C.P
S235	A 36	St 37
S275	A572 Gr. 42	St 44
S355	A992	St 52

Table (1) Steel Grade Equivalences

4. Cross-Section Classification:

4.1 Member axis for EC3:

It's usual to use axis x-x as the major axis (parallel to the flanges), axis y-y is the minor axis(perpendicular to the flange) and axis z-z is the axis along the member. But in EC3 the matter is different because EC3 consider the y-y is the major axis(parallel to the flanges), z-z is the minor axis (perpendicular to the flange) and x-x is the axis along the member***[7]***. For angle sections, the y-y axis is parallel to the smaller leg, and the z-z axis is perpendicular to the smaller leg. For cross section where the major and minor principal axes do not coincide with the y-y and z-z axis, such as for angle sections, then these axes should be referred to as u-u and v-respectively as shown in Fig.(1).

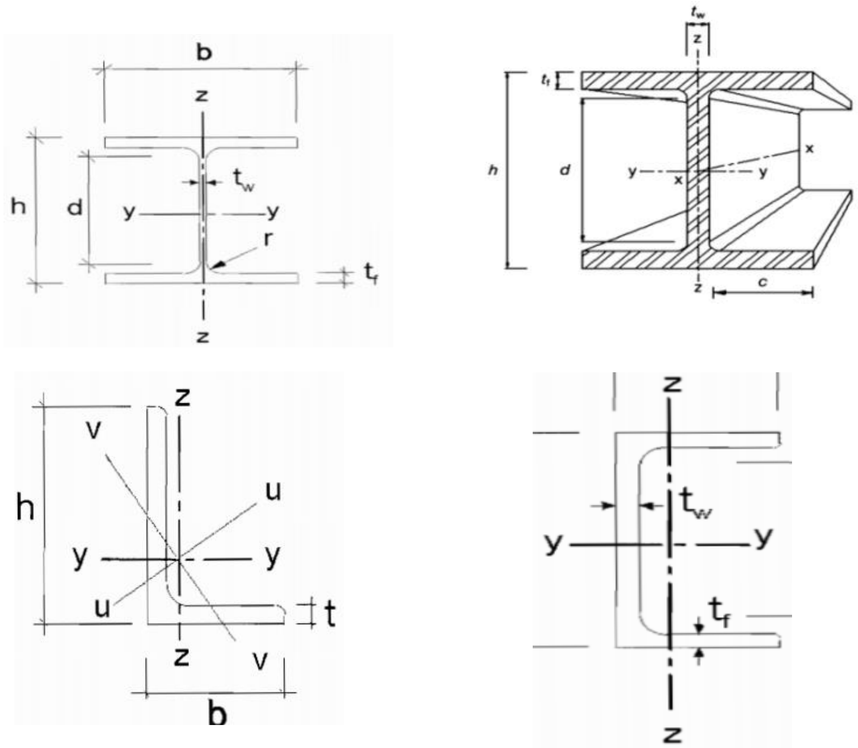


Fig.(1) Member axis for EC3

4.2 Cross-Section Classification Definition:

In E.C.P (ASD, LRFD) and AISC sections are classified as compact, non-compact, and slender. But EC3 sections are classified cross-section as class 1,class 2,class 3and class 4***[7]***. For AISC requirements, the Seismic Provisions for Structural Steel Buildings (AISC-341) mentioned an additional classification called(seismically compact) until year 2010 but in year 2016 the classification name became two classifications (Moderately Ductile and Highly Ductile Members)***[1]***. The Difference in capacity of sections is shown in Fig. (2).

E.C.P	AISC	EC3	Description
Compact	Compact	Class 1	These are sections which can develop their plastic moment capacity but still have quite an amount of rotation capacity.
		Class 2	These are sections which can develop their plastic moment capacity but still have rather limited amount of rotation capacity due to local buckling.
Non-compact	Non-compact	Class 3	These are sections where the plastic moment capacity's development may be prevented by local buckling when extreme compression fiber stresses reach yield strength.
Slender	Slender	Class 4	These are sections where a member's (plate) local buckling will occur before the yield stress attained.

Table (2) Cross-Section Classification Definition

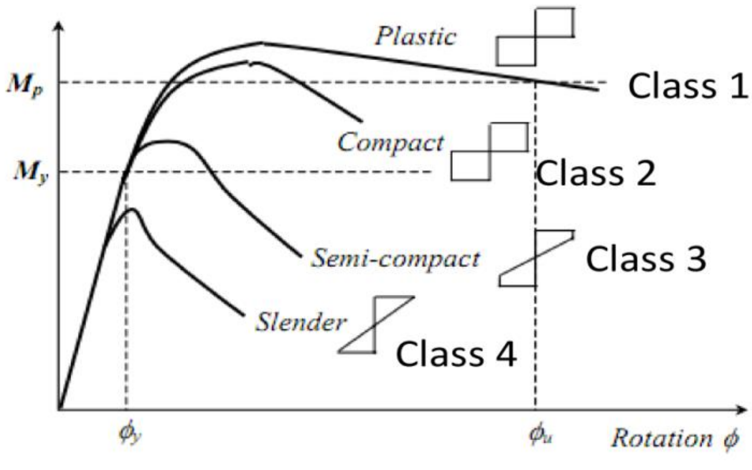


Fig.(2) Difference in capacity of section

4.3 Widths-to-Thickness Ratio:

The width-to-thickness ratios in AISC differ from E.C.P and EC3 as listed in table (2)

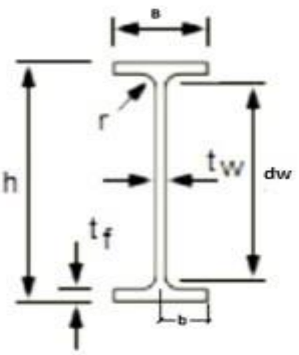
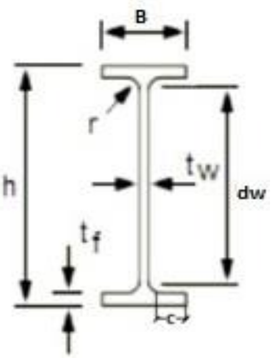
	AISC	E.C.P and EC3
		
Outstand Flange	$b=\frac{B}{2}$	$c=\frac{B-t_w-2r}{2}$
Internal Compression Part	$d_w = h - 2t_f - 2r$	

Table (3) width- thickness ratios

4.4 Comparison between classification ranges:

In this section we will compare between ranges of Classification to show the differences between E.C.P, EC3 and AISC equations. There are two ways to compare; the first is a table collecting all codes equations to compare it as shown in table (3) and the second are drawn charts which show the difference between ranges for our compared codes as shown in Figures (3) to (7)

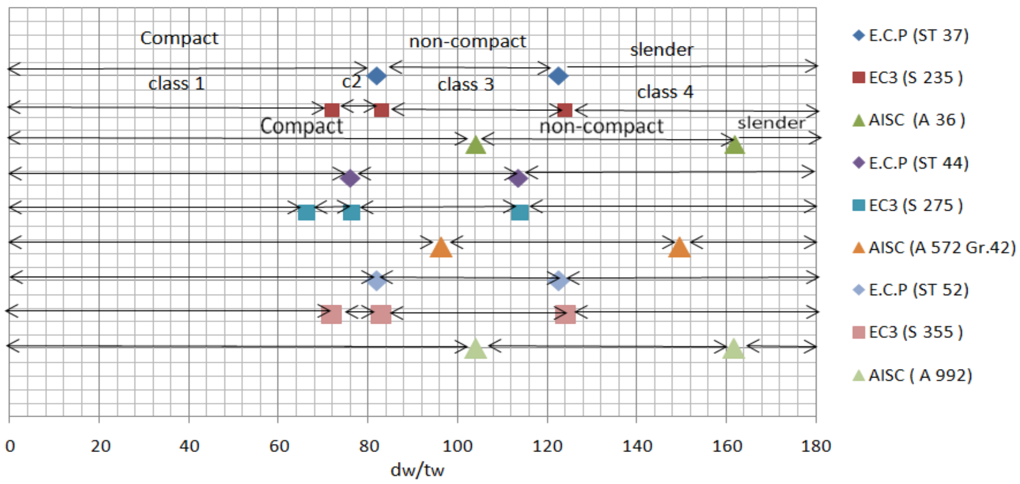
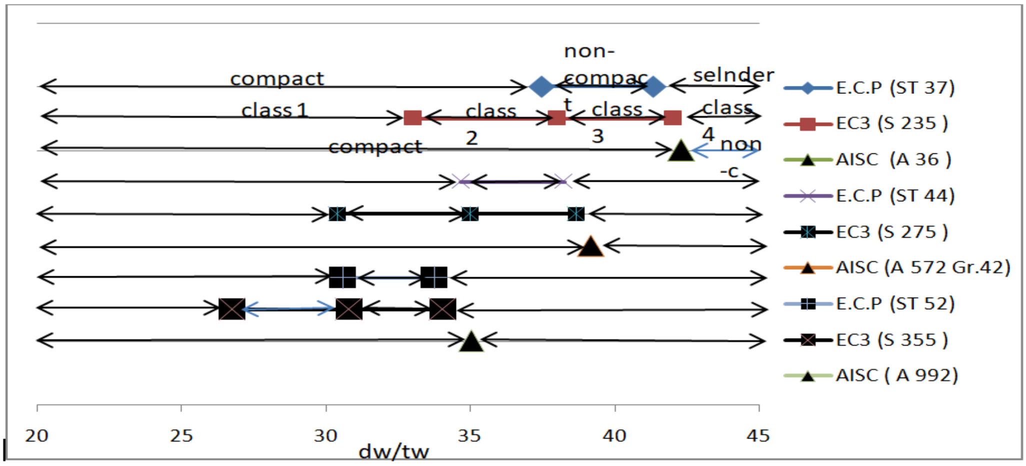
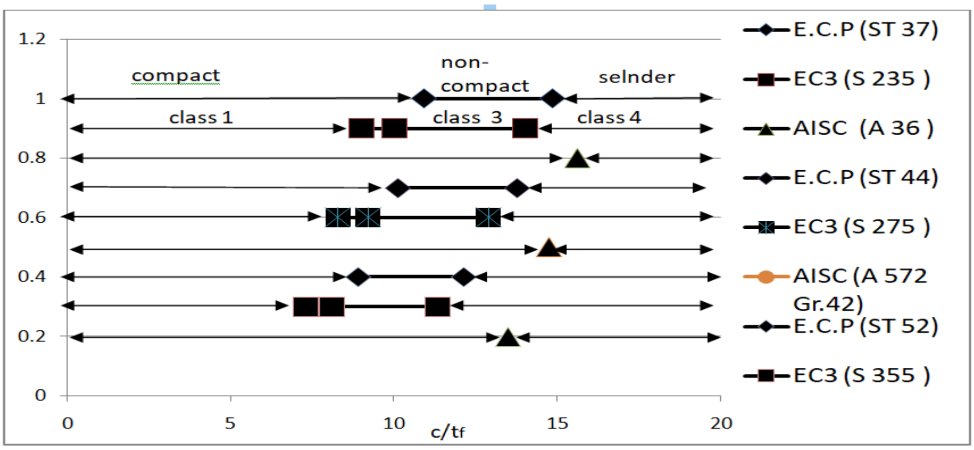


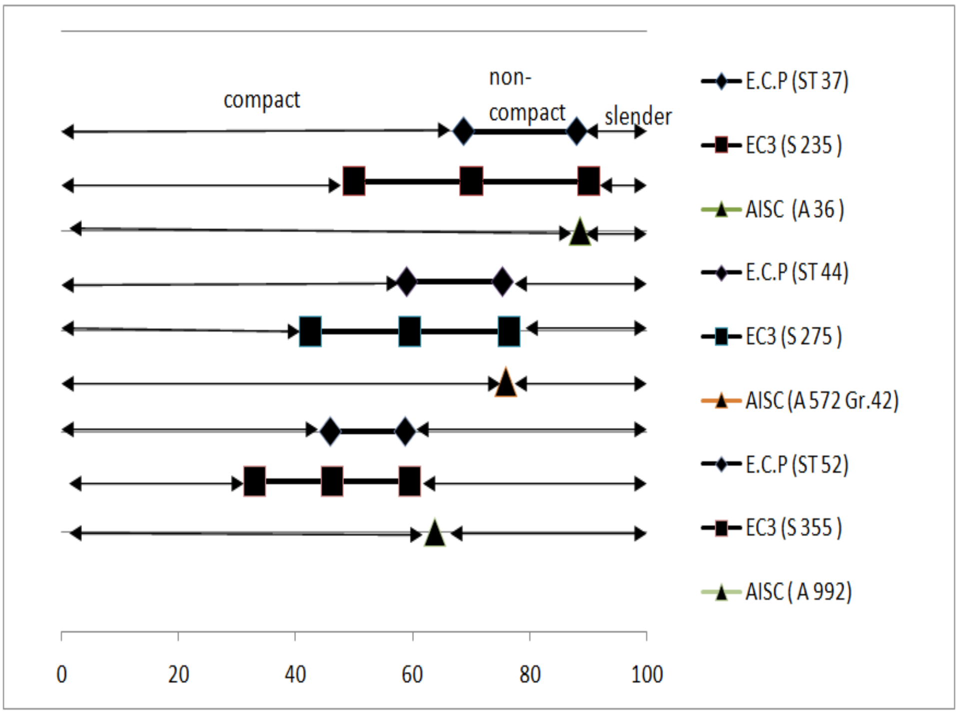
Fig.(3) Web subjected to bending ranges



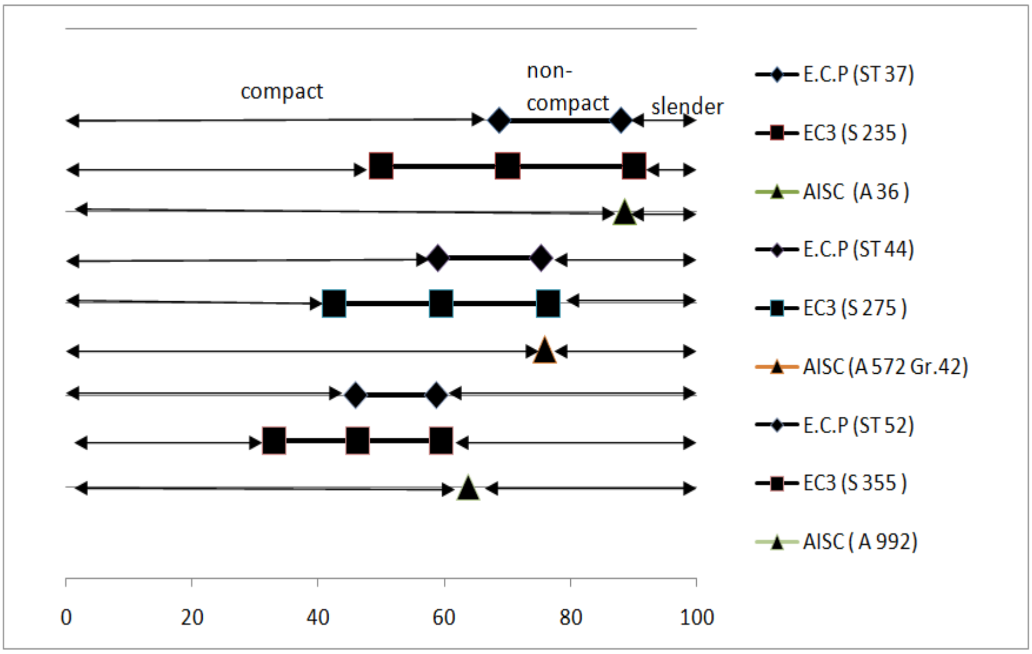
Fig(4) Web subjected to compression ranges



Fig(5) Flange subjected to compression



Fig(6)Tube subjected to compression ranges



Fig(7)Angel subjected to compression

	Description of Element	Maximum Width-Thickness Ratios for Compression Parts					
		Limiting Width-Thickness Ratio					
		EC3	AISC	E.C.P	EC3	AISC	E.C.P
		class 2	λ_p	Compact	class 3	λ_r	Non-Compact
Stiffened Elements	Flexure in webs of doubly symmetric I-shapes (rolled or built-up)	127/√Fy	169/√Fy	127/√Fy	190/√Fy	257/√Fy	190/√Fy
	Uniform compression in webs of doubly symmetric I shapes (rolled or built-up)	58/√Fy	N.A	58/√Fy	64/√Fy	67/√Fy	64/√Fy
	Cicular Hollow Sections In uniform compressi	164.5/√Fy	142/√Fy	165/√Fy	211/√Fy	629/√Fy	211/√Fy
	In flexure	164.5/√Fy	142/√Fy	165/√Fy	211/√Fy	223/√Fy	211/√Fy
Unstiffened Elements	Uniform compression in flanges of rolled I-shaped	15.3/√Fy	N.A	16.9/√Fy	21/√Fy	25/√Fy	23/√Fy
	Uniform compression in flanges of rolled I-shaped	15.3/√Fy	N.A	15.3/√Fy	21/√Fy	17.3/√Fy	21/√Fy
	angle	______	______	______	23/√Fy	20/√Fy	23/√Fy

Table (4) Limiting Width-Thickness Ratio

After the last table and all this figures that show the ranges between the three codes, it's noted that AISC give ranges higher than E.C.P and EC3 for all limits except when Angel subjected to compression although, E.C.P and EC3 have the same ranges. There is inverse relationship between the limit of section and the steel grade as shown in figures.

5. Tension members:

5.1 Comparison between AISC, EC3 and E.C.P design equations:

The three codes in specifications consider tensile yielding in the gross section and tensile rupture in the net section as the two primary limit states for tension members but E.C.P consider tensile yielding only in ASD***[9]***. The following equation which used in codes to calculate the nominal resistance of members to these limit states are as follow without resistance factors:

$P_n = A_g F_y$(Yielding){AISC, EC3andE.C.P(ASD and LRFD)} (4)

$P_n = U A_n F_u$(Fracture){AISC and E.C.P (LRFD)} (5)

$P_n = 0.9\ A_n F_u$(Fracture)(EC3) (6)

$U = 1 - \frac{\bar{X}}{L}$ (7)

The fundamental difference between these equations is how to calculate the shear lag factor U.In AISC and E.C.P the shear lag factor U equal 1.0 is using if the tension load is transmitted directly to each of the cross sectional elements. An elaborate treatment is tabulated in the AISC and E.C.P specification for bolted and welded connections and both codes has the same factors nearly***[2],[6]***.Separate rules are presented for I-section, L-shaped, and HSS members as defined in tables in codes. Shortly, shear lag factor is ranging between 0.6 and 0.9 are found based on the recommended procedure in AISC and E.C.P. For equation (5)of shear lag factor the E.C.P has a limit that value of U should not exceed 0.9 but AISC there is no limit.

On the other hand, a less elaborate treatment for shear lag is given in EC3.In general, a 10 percent reduction in tensile fracture capacity is considered even if all cross sectional elements are connected as shown in equ.(6).For EC3 there are parameters β_2,β_3 for single angles connected by one leg are given in Part 1.8 Section 3.10.3 of EC3.According to these parametersβ_2,β_3 the 0.9 coefficient was replaced. Parameters β_2 and β_3 are reduction factors which are depended on number of bolts in member and the pitch between holes, values for these parameters ranged between 0.4 to 0.7***[8]***.

5.2Comparison between resistance factors:

Code / Stress	AISC		EC3	E.C.P	
	LRFD Φ	ASD 1/Ω	LRFD only	LRFD Φ	ASD 1/Ω
Yield	0.9	1.67	1.0	0.85	1.72
Fracture	0.75	2.0	0.8	0.7	NO design equ.

Table (5) Comparison between resistance factors

From table (4) we can note that LRFD of E.C.P is less than LRFD of AISC but in ASD is bigger.

5.3 Comparison between AISC, EC3 and E.C.P slenderness limitations:

There is no maximum slenderness limit for members in tension in AISC and EC3 but in E.C.P it L/r, should not exceed 300.AISC see that for members designed on the basis of tension, the slenderness ratio, L/r, preferably should not exceed 300***[2]***. This suggestion does not apply to rods or hangers in tension.

6 Compression members:

6.1 Comparison between AISC, EC3 and E.C.P equations:

Capacity of compression members depends on the use of non-dimensional slenderness for flexural buckling (λ= KL/r) in all codes. Ever code has a unified approach adopted on for various forms of member buckling. In other words, flexural buckling and flexural-torsional buckling are treated using a unified set of reduction factors.

The critical non-dimensional slenderness for flexural buckling, (λ_c), can be calculated as follows:

$$\lambda_c = \frac{KL/r}{\pi}\sqrt{\frac{F_y}{E}} \tag{8}$$

6.2 The nominal axial strength for flexural buckling:

All codes have the same form to calculate the nominal axial strength for flexural buckling as follows:

$$p_n = \chi F_y \, A_g \quad \text{\{AISC, EC3 and E.C.P(LRFD)\}} \tag{9}$$

Every code has a different way to calculate the reduction factor χ as follow:

$$\chi = \frac{1}{\phi+\sqrt{\phi^2+\lambda^2}} \text{Where, } \phi = 0.5(1+\alpha(\lambda-0.2)+\lambda^2) \quad \text{For EC3} \tag{10}$$

$$\chi = \left(1 - 0.384\,{\lambda_c}^2\right) \text{ for } \lambda_c \le 1.1 \quad \chi = \frac{0.648}{{\lambda_c}^2} \text{ for } \lambda_c > 1.1 \text{ For E.C.P (LRFD)} \tag{11}$$

$$\chi = 0.658^{{\lambda_c}^2} \text{ for } \lambda_c \le 1.5 \quad \chi = \frac{0.877}{{\lambda_c}^2} \text{ for } \lambda_c > 1.5 \quad \text{For AISC} \tag{12}$$

The E.C.P (ASD) has different equations for flexural buckling resistance as follow:

$$F_c = 0.58\,F_y - \frac{(0.58F_y - 0.75)}{10^4}\lambda^2 \text{ For } \lambda < 100 \quad F_c = \frac{7500}{\lambda^2} \text{For } \lambda \ge 100 \tag{13}$$

In EC3 equations we note there is a factor (α), it is an imperfection coefficient to separate between different column strength curves. Value of factor (α), may be one five values termed as a_o, a, b, c, d are mentioned in EC3***[7]***.The choice of value is dependent upon the properties, steel grade of the cross section and upon the axis of buckling. The rules for choosing the value of factor (α), are tabulated in EC3.

Fig.(8) is showing a comparison of reduction factors for the three codes. According to this chart, buckling curve of AISC is similar to buckling curve "a" of EC3 and the curve of E.C.P (LRFD) is similar to them until non-dimensional slenderness λ_c equal 1.1 but for $\lambda_c > 1.1$ gives a reduction factor less than AISC and EC3 and the capacity of member will decreased as will be shown in next curve for capacity of members.

6.3 The nominal axial strength for flexural-torsional buckling:

AISC and E.C.P (LRFD) only take the flexural-torsional buckling effect in their consideration and take the minimum of flexural buckling and flexural-torsional buckling as the capacity of member. They have the same equation in the both codes to calculate the effect of flexural-torsional buckling. Flexural-torsional buckling applies to singly symmetric and unsymmetric section, and doubly symmetric members, applied when the torsional unbraced length is bigger than the lateral unbraced length, all without slender elements. These provisions also apply to single angles with $b/t > 0.71\sqrt{E/F_y}$, where b is the width of the longest leg and t is the thickness ***[2]***.

6.4 Comparison between AISC, EC3 and E.C.P SLENDERNESS LIMITATIONS:

EC3 has no maximum slenderness limit for members in compression in but in E.C.P, L/r should not exceed 180 for compression members ,180 for bracing systems and secondary members***[9]*** but in AISC, L/r should not exceed 200 all types***[2]***.

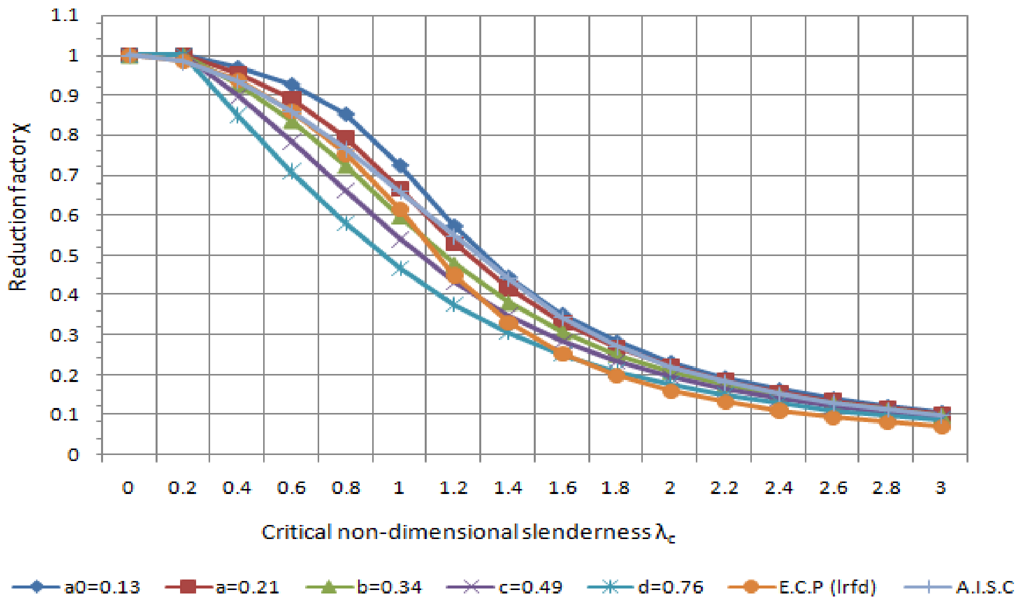


Fig.(8)Reduction factor comparison

5.6.6 Comparison between AISC, EC3 and E.C.P capacities:

To calculate the difference in capacities between the three codes, the present program was used. H.E.B (200) was selected to calculate the capacities of compression column with lengths from 4m to 8m, the steel grade used was ST.37 and design the column with the three codes to calculate the difference. The results were putting in curves shown in Fig.(9) as LRFD method and ASD method, respectively. We can note that AISC has high capacity more than others. E.C.P has the lowest capacity in LRFD method and ASD method.

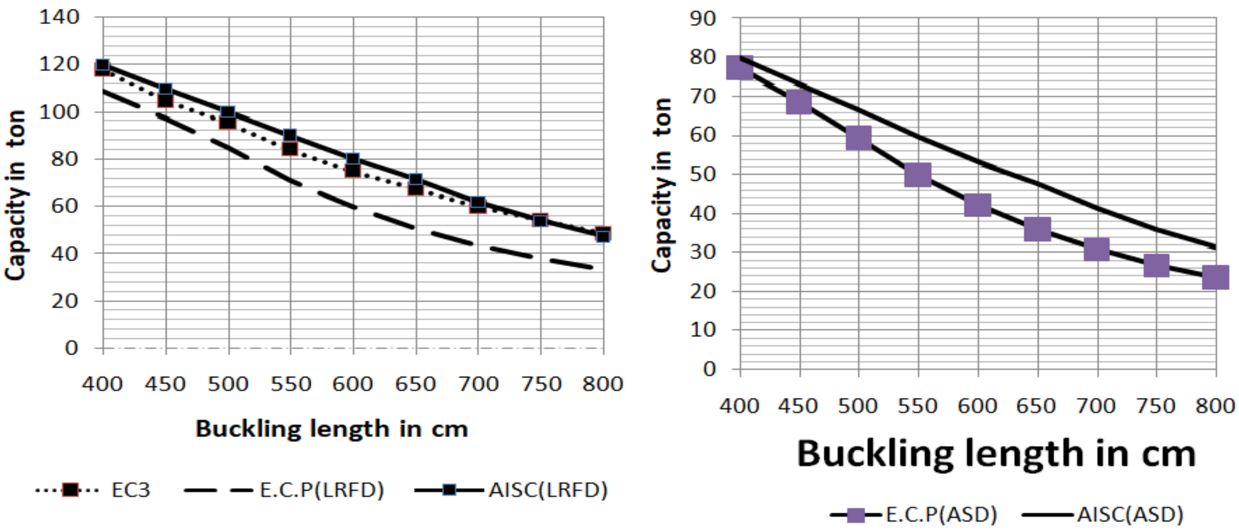


Fig.(9) Column capacity comparison

7. Flexure member:

7.1 Design of members for flexure:

According to E.C.P, AISC and EC3 specifications, yielding and lateral torsional buckling are consider to be the two limit states for flexural members. These two limits will be treated separately for clarity of comparisons.

7. 2 Limit State of Yielding:

For limit state of yielding it's assumed that section is laterally supported beams and it's a rarely case. For this case the moment capacity (M_n) of a section is depending onits plastic section bending modulus and grade of steel as shown in equation (14). This case in EC3 called as laterally restrained beam ***[13]***.

$$M_n = Z\, F_y \tag{14}$$

7.3 Lateral Torsional Buckling of Compact I-shaped Members:

AISC and E.C.P (LRFD) have the same way for the treatment of lateral torsional buckling but EC3 have differences in its method. AISC and E.C.P (LRFD) specification identifies three limits of buckling which defined for the member by the value of unbraced length of the member (L_b).Two threshold values for unbraced length namely L_p and L_r are found in AISC***[2]*** and E.C.P (LRFD)***[6]***.The L_p value draw the limit between plastic and inelastic buckling behavior. Similarly, the L_r value draws the limit between inelastic and elastic buckling behavior. According to AISC and E.C.P, the section is in its plastic moment capacity of as s compact member where the unbraced length is less than L_p. The member's capacity reduces linearly between M_p and (0.7-0.75) M_y if the unbraced length is between L_p and L_r. When the unbraced length is bigger than limit of L_r, the section is in elastic buckling and the capacity is calculated by elastic critical buckling moment (M_{cr}).AISC and E.C.P (LRFD) have difference equations form but near in results for calculating L_P and L_r in this section. The following equations showing the capacity for lateral torsional buckling as founded in AISC and E.C.P (LRFD) specification:

$$M_n = M_p = Z_x F_y \quad when\ L_b \leq L_p \tag{15}$$

$$M_n = c_b\left[M_p - (M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \quad when\ \ l_p < L_b \leq L_r \tag{16}$$

$$M_n = C_b\, M_{cr} \quad when\ \ L_b > L_r \tag{17}$$

$$M_{cr} = \frac{\mathrm{S_x}\pi^2 E}{\left(\frac{L_b}{r_{st}}\right)^2}\sqrt{1 + 0.078\ \frac{J\,C}{S_x h_0}\left(\frac{L_b}{r_{st}}\right)^2} \qquad \text{For AISC} \tag{18}$$

$$M_{cr} = \mathrm{S_x}\sqrt{\left(\frac{1380\ \mathrm{A_f}}{\mathrm{d\ L_b}}\right)^2 + \left(\frac{20700}{(\mathrm{L_b/r_t})^2}\right)^2} \qquad \text{For E.C.P (LRFD)} \tag{19}$$

Note form equations (18) and (19) there are difference between AISC and E.C.P (LRFD) in calculating M_{cr} for case $L_b > L_r$. The two codes have the same equation to calculate the modification factor C_b, but E.C.P (LRFD) has another equation for straight line moment diagrams within the unbraced length***[6]***.

For E.C.P (ASD) laterally unsupported length (L_u) is calculated and compared with the actual unsupported length $L_{u,act}$, if $L_{u,act} < L_u$ the section called supported and the allowable bending equal 0.64 F_y but if $L_{u,act} > L_u$ the section called unsupported and the allowable bending is calculated by some long equations but not exceed 0.58 F_y***[9]*** .

For EC3, it has a different way to calculate capacityM_n of section***[13]*** as seen in equation (20). Value of Z in equation (15) is the plastic modulus for class 1, 2 and the elastic modulus for class 3. As mentioned before in the compression members section, EC3 assumed a reduction factor to solve any buckling problems. Also there is reduction factor (χ_{LT}) expression for lateral torsional buckling is developed to design these flexure members. This reduction factor (χ_{LT}) for can be calculated by two methods mentioned in EC3 code a general method that can be applied to any type of cross section (more conservative) and an alternative method that can be applied to rolled cross sections or equivalent welded sections.

$$M_n = \chi_{LT}\, Z\, F_y \tag{20}$$

The reduction factor (χ_{LT}) is defined as:

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{\,2} - \bar{\lambda}_{LT}^{\,2}}} \qquad \chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^{\,2} - \beta\,\bar{\lambda}_{LT}^{\,2}}} \tag{21}$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^{\,2}\right] \qquad \phi_{LT} = 0.5\left[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^{\,2}\right] \tag{22}$$

i)General method *ii)Alternative method*

$$\bar{\lambda}_{LT} = \sqrt{\frac{M_p}{M_{cr}}} \tag{23}$$

$$M_{cr} = C_1\frac{\pi^2\, E\, I_Z}{L^2}\left(\sqrt{\frac{I_w}{I_z} + \frac{L^2\, G\, I_T}{\pi^2\, E\, I_z} + (C_2 Z_g - C_3 Z_j)^2}\right) - (C_2 Z_g - C_3 Z_j) \tag{24}$$

First of all, no M_{cr} expression is recommended in EC3 like AISC and E.C.P (LRFD). Any rational analysis to determine M_{cr} is acceptable. In this study, the elastic critical moment expression equation (24) was considered to be used in the EC3 expressions and the present program ***[13]***.Although, we can note the difference between the two method, in the second method (*Alternative method)* there are two factors$\overline{\lambda}_{LT,0}$andβ which not found in first one.EC3 recommended the following values for rolled sections or equivalent welded sections***[13]***:

$\overline{\lambda}_{LT,0} = 0.4$ (Maximum value) $\beta = 0.75$ (Minimum value)

The factor α_{LT} is dependent on the imperfections and its value is identical to the α factors given in the compression members section. The appropriate buckling curve as the EC3 recommend is based on the depth to width ratio (d/b_f) of the member. For rolled I-sections, curve "b" is set for $d/b_f<2$ and curve "c" for others. Similarly, for welded I sections, curve "c" is set for $d/b_f<2$ and curve "d" for others.

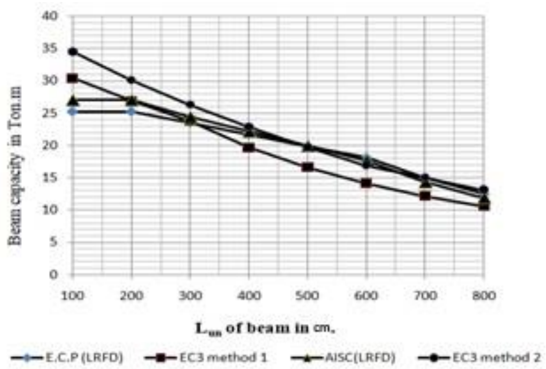
According to this second method, there is ability to take shape of the bending moment diagram, between braced sections, into account by using the modified reduction factor $\chi_{LT,mod}$ from equation (25). The parameter f can be calculated from equation (26).Where k_c is a correction factor, defined by choosing the shape of moment from three shapes of bending moment diagrams tabulated in EC3***[7]***.

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \tag{25}$$

$$f = 1 - 0.5(1 - K_c)\left[1 - 0.2\left(\overline{\lambda}_{LT} - 0.8\right)^2\right] \tag{26}$$

7.4 Comparison between AISC, EC3 and E.C.P flexure capacities:

To calculate the difference in capacities between the three codes, the present program was used. I.P.E (400) was selected to calculate the capacities for flexure member with lengths from 4m to 8m, the steel grade used was ST.37 and design the member with the three codes to calculate the difference. The results were putting in curves shown in Fig. (10) as LRFD method and ASD method, respectively. For LRFD method as shown in Fig. (10) EC3 second method give the higher capacity where E.C.P (LRFD) give the lowest capacity. For ASD method as shown in Fig. (10) AISC and E.C.P (ASD) are very close in capacities. But from length 300 cm to 500 the capacity is constant due to the allowable of bending is constant at value 0.58 F_y because the section became unsupported and the allowable bending should not exceed 0.58 F_y .



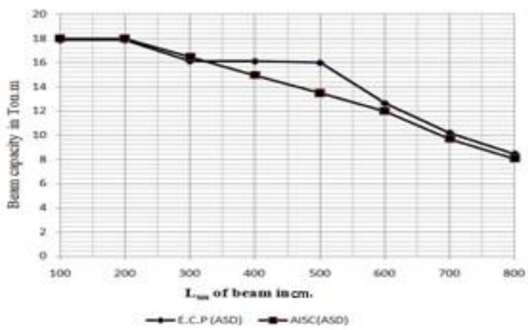


Fig.(10) Beam capacity comparison

8 Combined axial compression and bending (beam-column):

8.1 Design equations of beam-column member:

Beam-column member equations are a mixed of compression equations and Flexure equations in the three codes. The general form for all equations is that ratio between the applied loads and the resistance of member not exceeds 1.0 as shown in the following equations:

For AISC:

$$\frac{P_u}{\emptyset P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\emptyset M_{nx}} + \frac{M_{uy}}{\emptyset M_{ny}}\right) \leq 1.0 \text{ For } \frac{P_u}{\emptyset P_n} \geq 0.2 \quad (27)$$

$$\frac{P_u}{2\emptyset P_n} + \left(\frac{M_{ux}}{\emptyset M_{nx}} + \frac{M_{uy}}{\emptyset M_{ny}}\right) \leq 1.0 \text{ For } \frac{P_u}{\emptyset P_n} < 0.2 \quad (28)$$

For EC3:

$$\frac{N_{Ed}}{\chi_y \frac{N_{RK}}{\gamma_{M1}}} + K_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,RK}}{\gamma_{M1}}} + K_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,RK}}{\gamma_{M1}}} \leq 1.0$$
(29)

$$\frac{N_{Ed}}{\chi_z \frac{N_{RK}}{\gamma_{M1}}} + K_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,RK}}{\gamma_{M1}}} + K_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,RK}}{\gamma_{M1}}} \leq 1.0$$
(30)

For E.C.P (LRFD):

$$\frac{P_u}{\emptyset P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\emptyset_b M_{nx}} + \frac{M_{uy}}{\emptyset_b M_{ny}}\right) \leq 1.0 \text{ IF } \frac{P_u}{\emptyset P_n} \geq 0.2 \quad (31)$$

$$\frac{P_u}{2\emptyset P_n} + \left(\frac{M_{ux}}{\emptyset_b M_{nx}} + \frac{M_{uy}}{\emptyset_b M_{ny}}\right) \leq 1.0 \text{ IF } \frac{P_u}{\emptyset P_n} < 0.2 \quad (32)$$

For E.C.P(ASD):

$$\frac{f_{ca}}{F_c} + \frac{f_{bcx}}{F_{bcx}}A_1 + \frac{f_{bcy}}{F_{bcy}}A_2 \leq 1.0 \quad (33)$$

From the design equations we can note that, the equation of AISC and E.C.P(LRFD) and have the same parameters .The only difference is the resistance factor value$\emptyset$, it`s 0.9 in compression and flexure for AISC***[2]***but value is 0.8 in compression , 0.85 in flexure for E.C.P(LRFD) and A_1,A_2 are bending modification factor for E.C.P(ASD) equation***[6]*** .On the other hand, EC3 has two equations (28) and (29) to design the beam column section and engineers use the two equations not one of them. Factors K_{yy} ,K_{yz},K_{zy} and K_{zz} which defined as the interaction factors and there is two methods in EC3 code to calculate this factors***[7]***. In EC3 equations still using χ_y, χ_zas the reduction factors due to flexural buckling about y and z, respectively. $\chi_{LT}$is the reduction factor due to lateral-torsional buckling.

8.2 Comparison between AISC, EC3 and E.C.P beam-column capacities:

Using the presented program to do this comparison there was a note, EC3 equations for design have some parameters depends on the applied loads on member so the comparison was done on AISC and E.C.P as ASD method and LRFD method as shown in tables (5) and (6). Using st.37 and taking section B.F.I (500) for this comparison.

From this comparison it`s noted that AISC give higher capacity more than E.C.P in ASD and LRFD method that because the difference between resistance factor ∅as it is high in AISC.

L_{un} (cm)	M(t,m),N(t)	E.C.P (LRFD)	AISC (LRFD)
100	M	95.46	102.3
	N	454.26	517.61
200	M	95.46	102.3
	N	442.68	502.76
300	M	95.46	102.3
	N	426.39	479.05
400	M	94.74	101.2
	N	396.38	447.84
500	M	91.83	97.87
	N	361.66	410.64
600	M	88.93	94.48
	N	319.22	369.23
700	M	86.02	91.08
	N	269.07	325.72
800	M	83.1	87.69
	N	211.5	281.76

Table (6) Beam-Column capacity as LRFD

L_{un} (cm)	M(t,m),N(t)	E.C.P (ASD)	AISC (ASD)
100	M	66.07	68.07
	N	331.6	344.38
200	M	66.07	68.07
	N	322.6	334.5
300	M	66.07	68.07
	N	308.88	318.76
400	M	59.63	67.38
	N	288	297.97
500	M	59.63	65.12
	N	261.7	273.21
600	M	59.63	62.86
	N	229	245.66
700	M	59.63	60.6
	N	193.3	216.72
800	M	59.63	58.34
	N	148.1	187.47

Table (7) Beam-Column capacity as ASD

9. Conclusion:

Based on the information discussed in the previous sections, the following conclusions were drawn:

1. The AISC and E.C.P use the both methods ASD and LRFD where EC3 use LRFD only. The E.C.P has different equations for ASD and LRFD but AISC has the same equation the difference is the safety factor in the resistance of member.
2. For load combinations, AISC and E.C.P (LRFD) have the same equations and factors but EC3 has different complex equations and factors.
3. For steel grade, the three codes have equal strength but with different names.
4. For cross-section classification, E.C.P and EC3 have the same limits for all types of sections and have three categories to classify the section but AISC have higher limits and have four categories to classify the section.

5. For tension member, the main difference in design equations between all specifications is the calculation of shear lag factor U. An elaborate treatment is tabulated in AISC and E.C.P. However, a less elaborate treatment is given in EC3. EC3 has high factor of safety more than AISC and E.C.P. There is no maximum slenderness limit in AISC and EC3 but in E.C.P it L/r, should not exceed 300.
6. For compression member, the difference in design equations found in the reduction factor where EC3 has five curves but AISC and E.C.P have one curve.
   AISC has high capacity more than EC3 and E.C.P. There is no maximum slenderness limit in EC3 but in E.C.P it L/r, should not exceed 180 and for AISC
   should not exceed 200.
7. For flexure member, for laterally unsupported flexural members, AISC, E.C.P and EC3 have different treatments. AISC and E.CP identifies three regimes of buckling depending on the unbraced length of the member (L_b). However,EC3 utilizes a reduction factor,(χ_{LT}), approach which calculated by two methods to treat lateral torsional buckling problem. EC3 *alternative method* gives high capacity more than AISC and E.C.P
8. For beam-column, for design equations AISC and E.C.P(LRFD) and have the same parameters the only difference is the resistance factors. EC3 has two equations with interaction factors and there is two methods to calculate this factors. AISC has high capacity more than E.C.P where EC3 is out of comparison because some parameters depends on the applied loads.
9. The present program can achieve immediately the best economic sections, making all necessary checks.
10. Putting code equations in form of computer programs help to get the similarities and differences between codes.
11. Engineers can design Tension member, Compression member, Beam and Beam-Column, get economic design using simple and clear tables without using the present program.

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