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COLUMN BEHAVIOUR SUBJECT TO COMPRESSION

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Signature	: ……………………………..
Name of Supervisor	: Prof. Dr. Shahrin Mohammad
Date	: 11th June 2008

COLUMN BEHAVIOUR SUBJECT TO COMPRESSION

SHARIZA MAT ARIS

A project report submitted in partial fulfilment of
the requirements for the award of the degree of
Master of Engineering (Civil-Structure)

Faculty of Civil Engineering
Universiti Teknologi Malaysia

June 2008

I declare that this project report entitled "Column Behaviour Subject to Compression" is the results of my own research except as cited in the references. The report has not been accepted for any degree and is not concurrently submitted in candidature of any other degree.

Signature	:	…………………….
Name	:	Shariza binti Mat Aris
Date	:	9th June 2008

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ABSTRACT

This project studies only the static non-linear behaviour of a CHS as a column member with four (4) type of end conditions i.e pinned end, fixed end, pin fixed and free fixed with applied vertical load at top of the column. OASYS software is used for the non-linear buckling analysis of the column. Beams element models are considered in this study. Geometrical non-linearities are modelled by introduced imperfection of L/1000 to perform the deflection. The influences of the end conditions under the vertical point loads are investigated. It can be concluded that from the analysis results, the compression capacity of the column section capacity reduced due to the slenderness. No additional reduction to the section capacity is required.

ABSTRAK

Projek ini hanya mengkaji kelakuan static bukan linear keluli bulat gerongang sebagai angota tiang dengan empat jenis keadaan hujung iaitu ; hujung cemat, hujung terikat, hujung terikat cemat dan hujung terikat bebas dengan beban tegak kenaan di hujung atas tiang. Perisian Oasys telah digunakan untuk analisis lengkokan bukan-linear bagi tiang. Model unsur rasuk adalah diambil kira dalam kajian ini. Geometri bukan-linear dimodelkan dengan mengenakan ketaksempurnaan sebanyak L/1000untuk membentuk pesongan. Pengaruh keadaan hujung disebabkan beban titik tegak adalah dikaji. Boleh disimpulkan bahawa keputusan kajian menunjukkan keupayaan mampatan kepada keupayaan keratan tiang adalah berkurangan disebabkan oleh kelangsian tiang. Pengurangan tambahan kepada keupayaan keratan adalah tidak diperlukan.

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LIST OF SYMBOLS

A	cross-sectional area
A_e	sum of effective net area
A_{eff}	effective cross-sectional area
A_g	gross cross-sectional area
A_n	total net areas
A_v	shear area
a_e	net areas
D	diameter
F_c	axial compression at critical location/compression force due to axial load
F_t	axial tension at critical location
F_v	shear force
L_E	effective length
M_b	buckling resistance moment
M_{bs}	buckling resistance moment for simple columns
M_{cx}	moment capacity about major axis
M_{cy}	moment capacity about minor axis
M_{LT}	maximum major axis moment in segment L governing M_b
M_x	nominal moment about major axis at critical location
M_y	nominal moment about minor axis at critical location
M_{rx}	major axis reduced plastic moment capacity in presence of axial load
M_{ry}	minor axis reduced plastic moment capacity in presence of axial load
p_b	bending strength
P_c	compression resistance smaller of P_{cx} and P_{cy}
p_c	compressive strength
P_{cx}	compression resistance, buckling about major axis
P_{cy}	compression resistance, buckling about minor axis

LIST OF SYMBOLS

P_t	tension capacity
P_v	shear capacity
p_{cs}	compressive strength with reduced slenderness
p_y	design strength of steel
r	radius of gyration
S	plastic modulus
S_{eff}	effective plastic modulus
S_x	plastic modulus about major axis
$S_{x.eff}$	effective plastic modulus about major axis
t	thickness
Z	elastic modulus
Z_{eff}	effective section modulus
Z_x	section modulus about major axis
$Z_{x.eff}$	effective section modulus about major axis
Z_y	section modulus about minor axis

m_{LT}	* factors for lateral torsional buckling
m_x	* factors for major axis flexural buckling
m_y	* factors for minor axis flexural buckling

*	equivalent uniform moment

LIST OF APPENDICES

CHAPTER 1

INTRODUCTION

Circular Hollow Section (CHS) are frequently used as columns and rafters or trusses member in both commercial and residential construction. The cross-sectional properties around the longitudinal axis of the CHS are uniform to distribute load. The structural capacity and integrity of the member may be degraded during fabrication, due to erection and fire protection. In practices, structural element should be design to ultimate design load. Slender column will reduce the section capacity of the CHS.

These studies only look into CHS as a column member under compression. This report presented the study of non-linear buckling analysis of a simulation using computer programme *Oasys* – GSA8.2 to study the behaviour of CHS column under compression. Varying magnitudes of axial force and fixities are tested with this programme to a various diameter and thickness of the CHS. The finding is compared with the compression resistance, P_c from equation employed by the British Standard (BS 5950-1:2000) which provide the strength prediction. The task of this study is to find the influence of the column end condition with compression capacity and also to quantify the degrading effect of the CHS in relation to the column slenderness.

Figure 1.1 showing the relationship between the short column with compression and the slender column reduced strength due to buckling. The short column failed under crushing or squashing as shown in Figure 1.1 (a). The squash load, P_y is :

$P_y = p_y\,A$ Eqn. 1.1 (from page 189 of ref [4])

where A = area of cross section

p_y = design strength

Figure 1.1 (b) shows the column failed due to buckling and depends on the degree of the slenderness. The compression resistance P_c, is :

$P_c = p_c\,A_g$ Eqn. 1.2 (from cl. 4.7.4 a) of ref [8])

where A_g = gross sectional area

p_c = compressive strength (degree of slenderness)

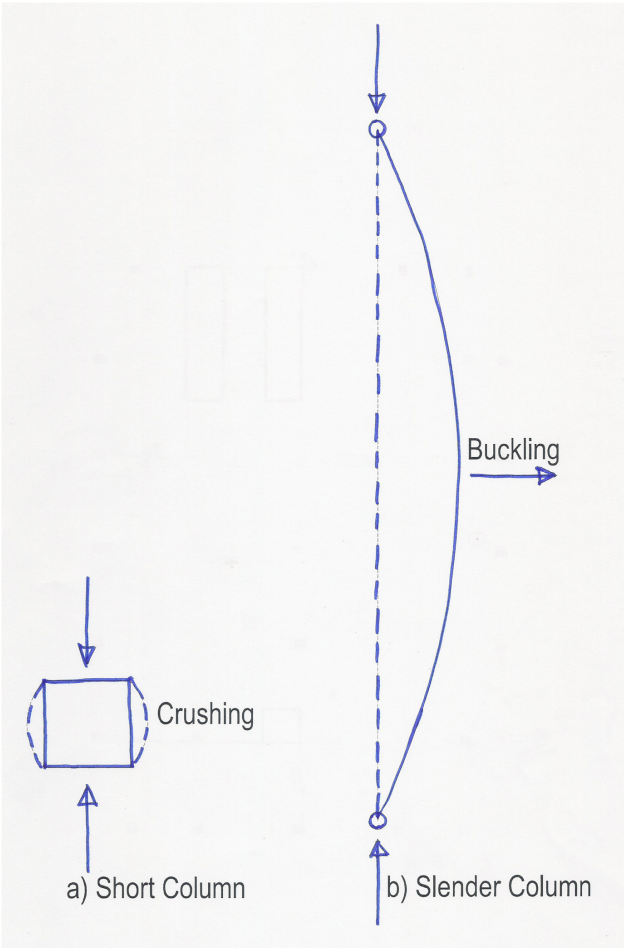

Figure 1.1 Behaviour of column under loading

(from figure 8.6 of ref [4])

1.1 Background and Statement of the Problem

The problem presented is geometrically non-linear with linear material behaviour and displacement is symmetrical at mid length of column. Basis of the method of analysis presented is non-linear beam element formulation under static load condition. The analysis is of structures behaviour under static instability of column (under prescribe non-linear static analysis) but more basic investigation of the effect on the behaviour in element due to the slenderness. The analysis is limited to axial load under compression only with geometrical imperfection.

A column considered in this numerical analysis has a uniform cross section and the support condition are pinned end, fixed end, pin fixed and fixed free at each end and subjected to axial load compression only at the top of the column. Columns are initially straight but an initial geometric imperfection at mid column of L/1000 is given to performed non-linear analysis. The results show that the behaviour of the columns under axial load P can be significantly affected by the column slenderness.

1.2 Objective of The Study

The objective of this study is to find out the effect of the CHS with respect to capacity and compression resistance in the member. The factored which will be considered in the study are :

- The size of the CHS.
- The slenderness ratio of the CHS.
- Type of the end conditions / support conditions.
- Ratio of thickness to the diameter of CHS (D/t).
- Vertical load applied to the member.

The study are confined to the computer modelling using non-linear buckling analysis of the CHS as a column member (beam element) under compression from buckling with various diameter and thickness. This study also to review the design based on the existing Code of Practice i.e BS 5950-1:2000 of the CHS, and identifying if there is any downgrade of the section capacity base on simulation compared to the code allowed for.

1.3 Scope of The Study

This study is focusing on the CHS as a column with beam element. The scope of work including

- Review the compression resistance based on Code of Practice BS 5950-1:2000.
- Computer simulation investigation using buckling non-linear static analysis to find out the ultimate capacity of the column from axial load and horizontal deflection plot.
- Evaluation the simulation results.
- Comparing the simulation results with the Code of Practice calculated capacity.

CHAPTER 2

LITERATURE REVIEW

Structures behave static under normal type of loading i.e live load, self weight and super imposed dead load. Structures remained linear and static under condition of small deflection and occur no yielding. Columns form part of practical structure with axial stiffness. The less the stiffness of the section, the less the ultimate capacity of the column. The behaviour of pin-ended steel column is analysed for the situation when the lateral deflection can be large enough to be of the order of the cross section depth (but small compared to length) with stress remain elastic [6].

2.1 Buckling and Deformation Behaviour

The deformation behaviour is an important factor for defining the buckling behaviour and buckling loading Shanley's inelastic buckling theory. The strain level at buckling stage and the slenderness ratio were two keys factors that affected the buckling load. The classical approach of simply using certain effective modulus in Euler's formula to define the buckling load is not adequate for column models with small slenderness ratio [1].

Circular hollow sections with flattened edges, fail under compressive loading, with excessive plastic deformation near the area of flattened edges and cannot reach nominal buckling strength, i.e elastic buckling failure mode, where new failure mode is found due to lower steel quality and elements with low slenderness value [2].

2.2 Bending Behaviour

Web elements with openings subject to bending, in compact and slender, having circular, elliptical or rectangular openings located at mid depth of the section could reduced the plastic moment capacity up to 40%. For cold formed steel beam [3], determined that local buckling were influence by web opening and presence of web punch out would result in decrease the structural performance of the web.

2.3 Shear Behaviour

Shear buckling coefficients and approximate methods for computing the ultimate shear capacity proposed due to influence of holes on the shear behaviour in flat plates. Nominal shear strength determined by applying a strength reduction factor to strength calculation for a cross section of web punch outs [3].

2.4 Bending and Shear Behaviour

Behaviour of channels with web openings subject to combined bending moment and shear force find that the current AISI specification interaction equation adequately predicts the web capacity if the nominal shear and bending strength are appropriately modified to account the presence of a web opening. The design recommendation is limited to beams having geometric and material properties for the study only. Figure 2.1 – Fig. 6 from [3] indicate the AISI specification for solid web does not provide good relationship between bending and shear for beam webs with opening. Figure 2.1 – Fig. 7 from [3] presents a better correlation between bending moment and shear force when compared with the AISI design approach [3].

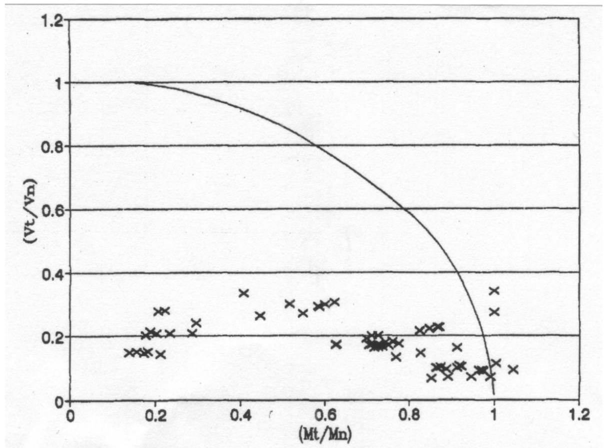

FIG. 6. Interaction Diagram for V_t/V_n and M_t/M_n

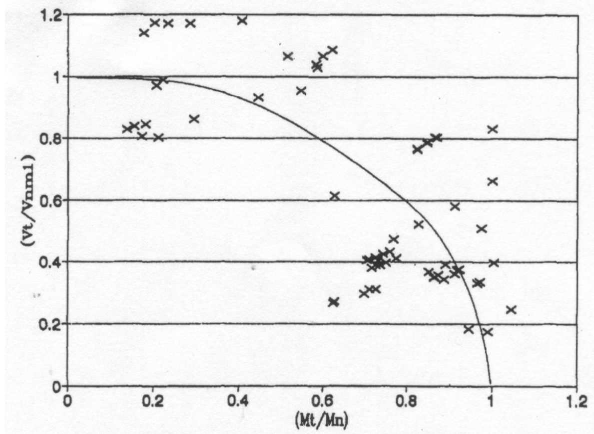

FIG. 7. Interaction Diagram for V_t/V_{nm1} and M_t/M_n

Figure 2.1 Plot Relationship between Bending and Shear from [3]

2.5 Euler Buckling

Element which is subject to compression must be checked against buckling with relation of [5]:

$$P/A \leq \sigma_u \qquad \text{eqn. 2.1}$$

where P = Factored axial compression.

A = Cross Section

σ_u = Buckling failure stress

Euler produced a first solution to the problem of column stability for pin ended at both ends in 1750. Euler Critical load [6] :

$$Pcr = \pi^2 EI/ L^2 \qquad \text{eqn 2.2}$$

where E = Modulus of elasticity

I = Moment of inertia

L = Column Length

Euler Critical Stress [4] :

$$\sigma_E = Pcr / A \qquad \text{eqn 2.3}$$

$$= \pi^2 EI / AL^2$$

$$= \pi^2 E / (L/r)^2$$

$$\sigma_E = \pi^2 E / \lambda^2 \qquad \text{eqn 2.4}$$

where λ = slenderness ratio = L/r

r = radius of gyration

The slenderness λ, is the only variable affecting the critical stress. At the critical load the column is in neutral equilibrium. The central deflection is not defined and may be in unlimited extend. [4]. Euler critical load does not take into account of the imperfections to be found in actual column, geometrical or structural. Due to this the actual column failure load is lesser than the Euler critical load [5].

2.6 Design Review of Circular Hollow Section to BS 5950-1:2000

This design review is to present the interaction equation for bending moment and compression currently BS 5950-1:2000 specified.

2.6.1 Section Properties Materials

General section properties for CHS are presented in Appendix B. Holes for larger opening other than for bolts should be deducted during determined gross cross-section properties. Cross-section subject to compression due to bending moment or an axial force for circular hollow sections should be classified separately for axial compression and for bending. Limiting width to thickness ratio D/t for CHS in compression due to bending are $40\varepsilon^2$ for class 1 plastic, $50\varepsilon^2$ class 2 compact and $140\varepsilon^2$ class 3 semi-compact. For CHS in axial compression D/t are $80\varepsilon^2$ for class 3 semi-compact. Effective plastic modulus for CHS, for class 3 semi-compact, should be obtained from

$$S_{eff} = Z + 1.485\left[\left(\left(\frac{140}{D/t}\right)\left(\frac{275}{py}\right)\right)^{0.5} - 1\right](S\text{-}Z) \qquad \text{eqn. 2.5 [13]}$$

Calculating resistance to local buckling in the design should be made for possible effect of any shift of the centroid of the effective cross-section compared to gross cross-section. CHS with cross-section of internal element wider than 80ε times thickness should check for possible effect of local buckling on serviceability when member stressed by axial compression.

Effective cross-sectional area A_{eff} and effective section modulus Z_{eff} of class 4 slender CHS of thickness t can be determined from :

$$\frac{A_{eff}}{A} = \left[\left[\frac{80}{D/t}\right]\left[\frac{275}{p_y}\right]\right]^{0.5} \qquad \text{eqn. 2.6 [13]}$$

$$\frac{Z_{eff}}{Z} = \left[\frac{140}{D/t}\right]\left[\frac{275}{p_y}\right]^{0.25} \qquad \text{eqn.2.7 [13]}$$

Provided that overall diameter D does not exceed 240 $t\varepsilon^2$.

The elastic properties of steel are:

- Modulus of elasticity $E = 205\ 000\ N/mm^2$
- Shear Modulus $G = \dfrac{E}{[2(1 + \upsilon)]}$
- Poison's ratio $\upsilon = 0.3$

2.6.2 Design of Structural Member

Members subject to bending should meet the following conditions.

1. Combination of maximum moment and co-existent shear and combination of maximum shear and co-existent moment at critical points.
2. Deflection criteria.
3. Resistance to lateral-torsional buckling should be check unless member is fully restrained.
4. Local buckling check for slender sections.

Shear force Fv should not be greater than shear capacity P_v given by $P_v = 0.6\ p_yA_v$, for CHS $A_v = 0.6A$, should be assumed to be located adjacent to the neutral axis. Peak value of shear stress distribution should not exceed 0.75 f_y for linear elastic behaviour. For cross-section with larger opening should refer to web opening.

Generally moment capacity determined from allowing for the effects of co-existing shear.