

EXPLANATORY MATERIALS TO

CODE OF PRACTICE FOR THE STRUCTURAL USE OF STEEL 2011



EXECUTIVE SUMMARY

The Explanatory Materials (EM) contains background information and considerations reviewed in the preparation of the Code of Practice for the Structural Use of Steel 2011 (the 'Code'), and should be read in conjunction with the Code.

Elaborations on robustness of structures, steel material classification, maximum thickness for prevention of brittle fracture, second-order analysis, limitation of material strengths used in composite design, reduction of Young's modulus of steel at elevated temperatures, and cold-formed steel, etc are given in the EM. In addition, numerous worked examples in using the Code to demonstrate second-order effects, section classification, structural analysis and design, composite beams and columns, cold-formed profiled sheet and purlin, etc are incorporated in the EM for readers' reference.

The EM aims to provide a concise guidance on the design of steel and steel-concrete component structures with their theoretical backgrounds and original assumptions, sources of reference, limitations and worked examples, where the application of the provisions in the Code may require special attention.

ACKNOWLEDGMENT

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CONTENTS

Executive Summary	i
Acknowledgment	ii
Contents	iii
E1	GENERAL1
E2	LIMIT STATE DESIGN PHILOSOPHY5
E3	MATERIALS20
E4	LOAD FACTORS AND MATERIAL FACTORS27
E5	SERVICEABILITY LIMIT STATES31
E6	DESIGN METHODS AND ANALYSIS36
E7	SECTION CLASSIFICATION66
E8	DESIGN OF STRUCTURAL MEMBERS70
E9	CONNECTIONS82
E10	COMPOSITE CONSTRUCTION89
E11	DESIGN OF COLD-FORMED STEEL SECTIONS AND SHEET PROFILES114
E12	FIRE RESISTANT DESIGN130
E13	PERFORMANCE-BASED DESIGN GUIDANCE FOR PARTICULAR TYPES OF STRUCTURES, INCLUDING GUIDANCE ON GENERAL MAINTENANCE OF STEEL STRUCTURES133
E14	FABRICATION AND ERECTION151
E15	ACCURACY OF FABRICATION AND ERECTION159
E16	LOADING TESTS161
E17	GUIDANCE FOR EVALUATION AND MODIFICATION OF EXISTING STRUCTURES163
ANNEXES170

E1 GENERAL

E1.1 SCOPE

The Code of Practice for the Structural Use of Steel 2011 is hereafter referred to as the Code in this Explanatory Materials (EM).

The Code adopts a similar approach to that of the Australian and UK codes rather than the Eurocodes or North American codes. It includes in one volume all those topics which are generally required for the design of building structures. It contains 17 Sections and 4 Annexes in order to provide a concise single document containing guidance and requirements for the design of buildings and related structures. In particular, it provides guidance on tall building design including appropriate comfort criteria, composite design of beams and columns, long span structures, stability issues making reference to the use of second order analysis and a wide range of steel grades and qualities. It also provides more detailed specifications for materials and workmanship than many other codes.

The Code addresses fundamental principles of overall stability, robustness, and the behaviour of the structure as a whole. It proposes an advanced philosophy and a number of methods for design against strength, ductility, robustness and stiffness under ultimate and serviceability limit states. Both manual and computer-based stability design methods are provided.

Section 1 of the Code contains general design requirements including the scope of the Code. Short clauses are provided on the overall design process and requirements for structures. Brief descriptions of limit state design philosophy, structural systems and integrity are included. These are expanded in subsequent sections of the Code.

Hong Kong does not produce structural steel and the intention of the Code is to allow use of steel and steel materials, such as nuts and bolts, from the major worldwide suppliers on a "level playing field" basis. **Section 3** covers the use of hot rolled steel sections, flats, plates, hot finished and cold formed structural hollow sections and cold formed sections conforming to acceptable international steel product standards from Australia, China, Japan, United States of America and United Kingdom versions of European Union standards. In addition to covering normally available steel with yield stresses in the range from 190 N/mm² to 460 N/mm², this section gives design recommendations on the use of high strength steel with yield stresses between 460 and 690 N/mm², and uncertified steel, whereby the design strength is limited to 170 N/mm². The use of steel with yield strength greater than 690 N/mm² is not covered in the Code.

Recommendations for the practical direct application of "second order" methods of global analysis are provided in **Section 6**.

Section 10 of the Code does not cover structures made from fibre composites, such as carbon or glass fibre.

Section 1.1 of the Code points out the limitations of its scope, i.e. it does not cover special types of steel structure such as rail or road bridges, articulated access walkways, nuclear power stations or pressure vessels. These are all specialist areas and it is essential that the designers of such structures should use the particular relevant design codes and specialist literatures which are available. Naturally, the Code contains general principles of steel design which can be applied to the preliminary design of some special types of structure.

Design of slender structures including tall buildings is specifically considered in the Code. It recommends that for stability analysis, when a frame has an elastic critical load factor less than 5, manual methods should not be used and a non linear second-order analysis, which includes consideration of P-Δ and P-δ effects and member and frame imperfections, should be adopted. This will take account of the amplification of moment due to second-order effect for sway and non-sway frames. New clauses on cold formed hollow section and cold formed steel sheet pile section are incorporated in **Section 11**.

E1.2 DESIGN PHILOSOPHY

E1.2.1 Aims of Structural Design

Structural design should aim to provide an economical structure capable of fulfilling its intended function and sustaining the specified loads for its intended working life. The design should avoid disproportional collapse and should facilitate safe fabrication, transport, handling and erection. It should also take account of the needs of future maintenance, final demolition, recycling and reuse of materials.

E1.2.2 Design Responsibility and Assumptions

In Hong Kong, the Responsible Engineer (i.e. the responsible engineer for the structural design) of private building development projects should be a Registered Structural Engineer or **RSE**.

The design documents, i.e. design statement and loading, drawings, specifications and justification calculations, should contain sufficient information to enable the design to be detailed and the structure fabricated and erected. The design assumptions, structural system, and whether loads or reactions are factored or not, should be clearly stated.

It is assumed that construction is carried out and supervised by qualified and competent persons having the appropriate levels of knowledge, skill and experience.

The structure is also assumed for use as intended by the design brief and will be properly maintained.

E1.2.3 Structural System, Integrity and Robustness

Clause 1.2.3 of the Code is self-explanatory. See also clauses E2.3.4 and E2.5.9 in this EM.

E1.2.4 Overall stability

Clause 1.2.4 of the Code is self-explanatory.

E1.2.5 Limit State Design

Clause 1.2.5 of the Code is self-explanatory.

E1.2.6 Economy

Clause 1.2.6 of the Code is self-explanatory.

E1.2.7 Design working life

The Code assumes a design working life of 50 years which is a widely accepted value for normal buildings and other common structures.

The concept of a longer design life for buildings, which society considers more important, is logical and similar to the idea of differing values of Importance Factors in American codes such as UBC 1997 and IBC 2000.

For example, for buildings providing essential emergency services (such as Hospitals, Police Stations, Fire Stations), or buildings of high economic or civic importance (such as Government Headquarters, Power Stations, Fuel Depots), the Responsible Engineer should consider discussing the adoption of a longer design working life with the client. Various bridge design codes use a 120 year working life.

E1.3 REFERENCES

Lists of acceptable standards and references for use in conjunction with the Code are given in Annex A in order to make the body of the Code easier to read. Other informative references provide more detailed guidance on particular aspects of design.

Annex D of the Code contains abstracted essentials of some standards where appropriate and where required, references are short and their contents are straightforward. The abstracted essentials are for guidance and ease of use of the Code; however, compliance with the acceptable standards and references is mandatory and takes precedence over guidance given in the abstracted essentials.

Thus, the required (or acceptable) standards and references underpin the abstracted essences and take precedence in any dispute in order to avoid ambiguity. This is also necessary for quality assurance purposes to avoid the risk of error because an abstracted essential omits some information.

The Code accepts materials, that is, hot rolled steel plates and sections, cold formed steel plates and sections, forgings, castings, bolts, shear studs, welding consumables to acceptable international steel product standards from the five regions. These are Australia, China, Japan, United States of America and United Kingdom versions of European Union standards.

Thus, the required, deemed to satisfy or normative standards and references for materials and fastenings include manufacturing standards from a wide range of countries in order not to restrict designers and suppliers to products from one region. The term "required" shall be considered to have the same meaning as the term "normative" used, for example, by Euro codes.

In the normal design office situation, it is unlikely that designers would need to refer to these standards and references, their main purpose is to provide standards for materials, with which suppliers must comply. However, it has been considered useful to abstract some essential guidance, where possible and appropriate, from some references in order to make the code more self-contained and user friendly.

Where relevant Hong Kong codes exist, such as the Code of Practice on Wind Effects in Hong Kong and the Code of Practice for the Structural Use of Concrete, they are given as the required references.

All required standards and references have been dated. This means that any revised required standards and references can be reviewed by the Buildings Department prior to its acceptance for use with this Code.

In order to provide a single consistent set of standards for workmanship, testing of materials which may be required in Hong Kong, testing and qualification of workers and quality assurance procedures, such tests and procedures shall generally be defined in the Code or as given in the references in Annex A which are acceptable to the Building Authority.

Weld testing and workmanship

For the sake of consistency, standards and references on workmanship and testing of welds and on qualification of welders and weld testing personnel are based either on UK versions of European Union standards or on American standards in order to avoid ambiguity. This follows from current local practice. These standards and references are given in Annex A1.4.

References of various other design guides are given in Annex A2, for example, the UK Steel Construction Institute guides on Simple and Moment connection design and on castings.

E1.4 GLOSSARY OF TERMS AND DEFINITIONS

Clause 1.4 of the Code contains general terms and definitions which are used throughout the Code. In the Code, these are organized in generic groups whilst definitions of more specialized terms are given in relevant sections. Most definitions are self explanatory while some further clarification of definitions and newer concepts are given below:-

An acceptable quality assurance (QA) system is a QA system which is acceptable to the Buildings Department and conforms to the requirements stipulated in ISO 9001 and any other requirements under the Hong Kong Accreditation Service. Under a QA system, the primary responsibility for testing of steel materials and products and ensuring its compliance with the Code and relevant acceptable references lies with the steel material or product manufacturer. A system of third party certification of the manufacturer to the quality standards of ISO 9002 is designed to ensure that this is carried out properly.

E1.5 MAJOR SYMBOLS

Clause 1.5 of the Code contains a list of the major symbols used and is generally self explanatory. The symbols are generally as used in BS 5950 since Hong Kong engineers are familiar with them. Those additional symbols for specialized applications are given in relevant sections of the Code for easy reading. Diagrams of typical welding symbols are given in Annex C.

E2 LIMIT STATE DESIGN PHILOSOPHY

E2.1 GENERAL

E2.1.1 Introduction

Clause 2.1.1 of the Code introduces the design methods allowed in the following clauses 2.1.2 to 2.1.6. It highlights the importance of the assumptions made on joint design for structural steelwork, which may be simple (i.e. effectively pin joints carrying no moment); continuous (i.e. capable of carrying full moments applied to them); and semi-continuous or semi-rigid, only capable of carrying limited moments. The assumptions in clauses 2.1.2 to 2.1.4 of the Code apply both to bolted and welded connections.

E2.1.2 Simple design

Simple design is most commonly used for relatively low rise steel structures and often provides an economical structural solution. The distribution of forces may be determined assuming that members intersecting at a joint are pin connected, thus beams are typically designed as simply supported and columns are designed for axial forces and only those moments which arise from eccentricities of reactions at beam ends.

Simple design allows a straightforward manual analysis of the structure.

Joints are assumed not to develop moments adversely affecting either the members or the structure as a whole. In reality some moments will occur at typical multi-bolted connections and the necessary flexibility in the connections, other than the bolts, may result in some non-elastic deformation of the materials. These deformations are assumed to be acceptable and will generally be so if simple connection details are used, for example a flexible endplate or bolted finplate connection. Examples of simple connections may be found in the publication of Steel Construction Institute “Joints in Steel Construction – Simple Connections” given in the Informative Reference Annex A2.2 of the Code.

A separate structural system is required to provide lateral restraint in-plane and out-of-plane, to provide sway stability and to resist horizontal forces. This system may take the form of diagonal steel bracing or concrete core or shear walls. Clauses 2.5.3 and 2.5.8 of the Code discuss and summarise minimum lateral loads and notional horizontal forces.

E2.1.3 Continuous design

Continuous design is where the connections are capable of sustaining the moments which actually occur as the structure deforms to carry the various load combinations which are applied.

Elastic or plastic analysis may be used. In elastic analysis, the joints should have sufficient rotational stiffness to justify analysis based on full continuity. The joints should also be capable of resisting the moments and forces resulting from the analysis.

In plastic analysis, the joints should have sufficient moment capacity to justify analysis assuming that plastic hinges occur in the members adjacent to the joints. They should also have sufficient rotational stiffness for in-plane stability.

In continuous design, the frame itself, rather than a separate structural system, will generally provide overall resistance to lateral loads and thus stability should be properly considered in all analyses. The frame is thus defined as a moment resisting frame (MRF).

E2.1.4 Semi-continuous design

Semi-continuous design may be used where the joints have some degree of strength and stiffness which is insufficient to develop full continuity.

Relative rotation at a joint may occur from bolt slip in normal clearance holes and the amount of slip is difficult to predict analytically. Or it may occur from limited elastic or plastic deformation of plates forming the joint.

Either elastic or plastic analysis may be used. The moment capacity, rotational stiffness and rotation capacity of the joints shall be based on experimental evidence or advanced elasto-plastic analysis calibrated against tests. This may permit some limited plasticity, provided that the capacity of the bolts or welds is not the failure criterion. On this basis, the design should satisfy the strength, stiffness and in-plane stability requirements of all parts of the structure when partial continuity at the joints is taken into account in determining the moments and forces in the members.

The Steel Construction Institute (UK) Publication P183 gives guidance and a design method for semi-continuous braced frames.

A particular application of the semi-continuous method is the Wind-Moment method for unbraced frames. This is applicable to structures where wind loads are relatively low and allow the beams and columns to be designed for gravity loads assuming simple connections. The method recognises that the “simple” joints will actually have some moment strength and allows this to be used for resisting lateral loads. Thus the “simple” joint moment capacity must be justified as being sufficient for the applied wind framing moments. The Steel Construction Institute (UK) Publication P263 gives guidance on the method for wind-moment design.

E2.1.5 Design justification by tests

Clause 2.1.5 of the Code is self-explanatory.

E2.1.6 Performance based design

Clause 2.1.6 of the Code allows new and alternative methods of design which are not explicitly covered in the Code to be used. It notes that the Responsible Engineer must provide adequate design justification (which must be acceptable to the Building Authority) that it meets the requirements of the aims of design given in clause 1.2.1 of the Code.

The term “performance based design” needs some clarification. Generally, codes are a mix of performance based and ruled based design. For example, calculations to justify that a beam will not collapse under load are calculations about the performance of the beam and a code based design will achieve this. This may be contrasted with a code with “rule based design” whereby a masonry wall shall not have a height to thickness ratio exceeding “N”.

In some building sub-contracts, for example for cladding design, the term means that a performance specification is given by the client to the designer/contractor who is then required to achieve the stated performance, typically by designing to normal codes of practice. Typically, for example, the performance specification might state:- “The design must comply with the Code of practice for the Structural use of Steel 2011”.

When used in the Code, the term “performance based design” is either taken to mean that the design does not of itself comply with the Code but is justified by engineering arguments and calculations, for example, the Code requires deflections at the top of a building not to exceed Height/500 but will allow performance based justification of a marginally higher value of deflection.

Alternatively, calculations may be done to justify an aspect of a design on which the Code does not have specific provisions, such as differential shortening between core and perimeter columns.

Owing to the rapid development of technology in materials and in design concept, performance-based design is allowed as an alternative to the prescriptive approach in various sections of the Code. These include fire engineering, floor vibration, comfort analysis of tall buildings and non linear analysis and design.

E2.1.7 Calculation accuracy

Clause 2.1.7 of the Code acknowledges that engineering design is not a precise science and is self-explanatory.

E2.1.8 Foundation design

Clause 2.1.8 of the Code is generally self-explanatory. The clause notes the importance of stating whether or not the forces and moments given on foundations result from factored or unfactored loads. Any tension connection, for example from wind uplift, between foundation and structure, must be designed to safely carry the required tension with the appropriate factor for the ultimate stability case.

E2.2 LIMIT STATE PHILOSOPHY

Clause 2.2 of the Code gives a brief description of the philosophy of limit state design, i.e. design loads, design load effects, design resistance and verification of adequacy. This is expanded in the EM as the concepts may be less familiar to those used to permissible stress codes.

Furthermore, an understanding of the philosophy of the various partial load factors is important when applying engineering judgment to particular situations, such as the assessment of existing structures and considerations of extreme events.

Further descriptions of the method may be found in BS5400 part 1 and BS5950 annex A.

Limit state design considers the functional limits in the aspects of strength, stability and serviceability of both single elements of the structure and the structure as a whole. This contrasts with allowable stress design which considers permissible upper limits of stress in the cross-sections of single members. It is generally considered that the main weakness of the allowable stress design method is the over-simplistic use of a single material factor of safety applied to the material yield strength to control the safety margin of a structure.

The weakness of the permissible stress approach was highlighted in the collapse of the Ferrybridge power station cooling towers in U.K. Structural instability is often critical in long and slender members and structures under high applied loads, and it is more common in steel and composite structures than in concrete structures.

In limit state design, both cross section capacity and member resistance are checked against material yielding and structural instability respectively, and various load and material partial safety factors are incorporated for different modes of failure and limit states. Limit state design will normally lead to more economical and safer designs. Limit state design methods accord more logically with the performance-based design approach.

Examples of limit states relevant to steel structures are given in Table 2.1 of the Code.

Differential settlement or rotation of foundations may be a serviceability or a strength issue, depending on magnitudes.

E2.3 ULTIMATE LIMIT STATES (ULS)

Clause 2.3 of the Code is self-explanatory. Ultimate limit states consider the strength and stability of structures and structural members against failure.

E2.3.1 Limit state of strength

Clause 2.3.1 of the Code is self-explanatory.

E2.3.2 Stability limit states

Clause 2.3.2 of the Code is generally self-explanatory and the principles are restated here for clarity.

General

Stability includes global stability or equilibrium of the structure, for example against overturning or sliding caused by lateral forces or against uplift caused by water pressure.

Static equilibrium

Clause 2.3.2.2 of the Code states that factored loads (as given in section 4 of the Code) should be used for overall stability checks on sliding, overturning and uplift since stability failure is an ultimate limit state. The last sentence of the clause goes on to say that the design should **also** comply with the requirements of the Building (Construction) Regulation (the 'B(C)R') for stability. The current requirements of the B(C)R are more onerous than the Code and thus will prevail. For example, when considering stability against overturning, the Code combination 2 uses 1.0 Dead +/- 1.4 Wind compared with 1.0 Dead +/-1.5 Wind given in the B(C)R.

Resistance to horizontal forces

Where required by the overall structural system, floor and roof slabs should have adequate strength and be properly fixed to the structural framework so as to provide diaphragm action and transmit all horizontal forces to the lateral load resisting elements (collector points). The Code also notes that cladding elements must be strong enough to transmit wind loads to the supporting structure.

Sway stiffness and resistance to overall lateral or torsional buckling

A large error may often be made in assumptions of buckling length, effective length or the K-factor. In an example of a portal frame, an error larger than the load factor can result if the Responsible Engineer assumes an effective length equal to the distance between nodes. When this error is larger than the load factor, the structure will collapse.

Non linear advanced analysis can be used as a performance-based design method for strength and stability since the design codes buckling curves and formulae are not used at all and the structure is only required to be checked against the criteria of equilibrium, strength, stability and ductility under ultimate or service loads. The criteria for using the non linear design method can be set for the magnitude of notional forces, imperfection mode, frame and member imperfections. Updated Eurocode 3 (2005) gives detailed information on all these values and the Code will extend the criteria with allowance for local conditions and use of eigen-buckling modes as imperfection modes.

The performance-based non linear analysis can be used as a good example to demonstrate the deficiency of the prescriptive design in which most engineers give largely varied assumption of effective length. In overseas and local practice, engineers assume the effective length normally as distance between nodes which can be erroneous by more than the margin of load factors whilst non-linear analysis gives a close estimation of load capacity when compared with hand calculation methods.

E2.3.3 Fatigue

Clause 2.3.3 of the Code gives a general introduction to the principles of fatigue design. It notes that design for fatigue is not normally required for buildings and that fatigue need not be considered unless a structure or element is subjected to numerous significant fluctuations of stress. Stress changes due to normal fluctuations in wind loading need not be considered.

However, there are some situations where fatigue design is required, examples of these which may occur in buildings are: steel masts which can be subjected to cross wind vibration at relatively low wind speeds by vortices, steelwork supporting vibrating machinery etc. Clause 13.6.3.3 of the Code gives a method for fatigue assessment of footbridges.

The introduction to the design method given in the Code is similar to that given in Section 9 of the Australian steel Code AS 4100 or Clause 9 of Eurocode 3 (BS EN 1993-1-1: 2005). These codes are in turn similar to the very comprehensive fatigue code:- the Code of Practice for Fatigue Design and Assessment of Structures, BS 7608 – 1993. In addition, an alternative method based on a translation of the China Code GB 50017–2003 is given in the Code.

Fatigue design procedure based on Appendix E of GB50017 - 2003

The design method given here is directly based on a translation of GB 50017– 2003. Alternative methods are given in Section 9 of AS 4100, BS EN 1993: Part 1-9: 2005 or BS 7608: 1993, the Code of Practice for Fatigue Design and Assessment of Structures, which provides a very comprehensive reference guide.

Fatigue Design

- (a) For steel members and their connections that are directly subjected to repeated dynamic loading: once the number of stress cycles 'n' equals or exceeds 5×10^4 , a fatigue calculation should be carried out.
- (b) This section is not applicable to fatigue calculations of structural members and their connections under special conditions such as:-
 - 1) Members with a surface temperature higher than 150°C.
 - 2) Members exposed to corrosive sea water.
 - 3) Residual stresses which have been eliminated after welding and heat treatment.
 - 4) Low period – high strain loading.
- (c) A permissible stress amplitude method should be used for fatigue calculations (in which the stresses are derived from elastic analysis). The number of stress cycles and the type of member and connection, the detail category, determine the permissible stress amplitude. When no tension stress exists in a stress cycle, the fatigue calculation need not be carried out.

Fatigue Calculation

(a) Constant amplitude fatigue

For constant amplitude fatigue (with constant stress amplitude during every stress cycle), the following formula below should be used:

$$\Delta\sigma \leq [\Delta\sigma] \quad (1 - 1)$$

where:

$\Delta\sigma$ – stress amplitude of welded area, $\Delta\sigma = \sigma_{\max} - \sigma_{\min}$; stress amplitude of non - welded area, $\Delta\sigma = \sigma_{\max} - 0.7 * \sigma_{\min}$

σ_{\max} – the maximum tension stress of every stress cycle (take the positive value)

σ_{\min} – the minimum tension stress (take the positive value), or compression stress (take the negative value) of every stress cycle.

$[\Delta\sigma]$ – when calculating permissible stress amplitude (N/mm^2) of constant amplitude fatigue, the following formula below should be used:

$$[\Delta\sigma] = (C / n)^{1/\beta} \quad (1 - 2)$$

where:

n is the number of stress cycles,

C and β are factors which are determined from Table E2.1 and the member and connection detail categories given in Table E2.4.

Table E2.1 - C and β factors for various detail categories

Detail Category of Member and Connection	1	2	3	4	5	6	7	8
C	1940×10^{12}	861×10^{12}	3.26×10^{12}	2.18×10^{12}	1.47×10^{12}	0.96×10^{12}	0.65×10^{12}	0.41×10^{12}
β	4	4	3	3	3	3	3	3

(b) Varying amplitude fatigue

This is the case where stress amplitude varies stochastically during stress cycles. During the service life of a structure, if the following:

- 1) Different loading frequency distribution
- 2) Stress amplitude level
- 3) Sum of frequency distribution
- 4) Design stress spectrum

can be predicted, then resolved (1 – 4) to effective constant fatigue by using the following formula:

$$\Delta\sigma_e \leq [\Delta\sigma] \quad (1-3)$$

where

$\Delta\sigma_e$ – effective stress amplitude of varying amplitude fatigue, using the following method:

$$\Delta\sigma_e = [\sum n_i * (\Delta\sigma_i)^\beta / \sum n_i]^{1/\beta} \quad (1-4)$$

$\sum n_i$ – anticipated service life of structure, which is determined by stress cycle number

n_i – stress cycle number, which is determined by the stress amplitude level matches $\Delta\sigma_i$ during the anticipated service life.

(c) Fatigue of heavy duty crane beams and trusses

The fatigue of heavy duty crane beams and trusses of medium to heavy cranes may be calculated by using the formula:

$$\alpha_f * \Delta\sigma \leq [\Delta\sigma]_{2*10^6} \quad (1-5)$$

where

α_f – effective factor under no load effect, refer to Table E2.2.

$[\Delta\sigma]_{2*10^6}$ is the permissible stress amplitude with cycle number $n = 2 \times 10^6$, refer to Table E2.3.

Table E2.2 - Effective Factor α_f for Crane Beam or Truss Under No Load Effect

Type of Crane	α_f
Heavy Duty Crane With Hard Hook	1.0
Heavy Duty Crane With Soft Hook	0.8
Middle Duty Crane	0.5

Table E2.3 - Permissible Stress Amplitude (N/mm²) with Cycle Number $n = 2 \times 10^6$

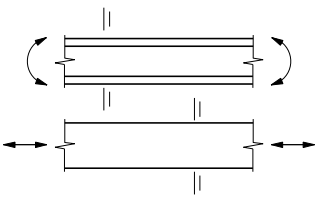
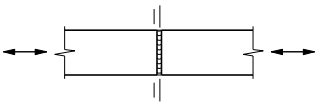
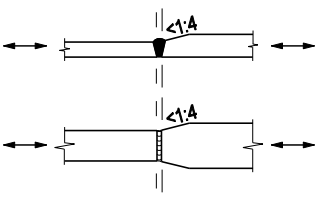
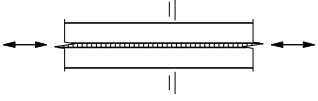
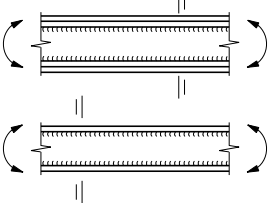
Detail Category of Member and Connection	1	2	3	4	5	6	7	8
$[\Delta\sigma]_{2*10^6}$	176	144	118	103	90	78	69	59

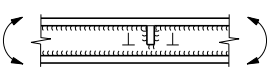
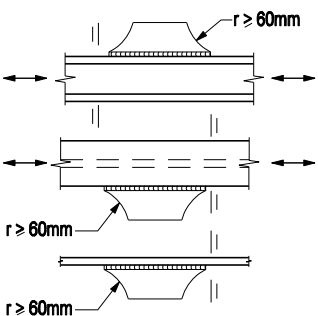
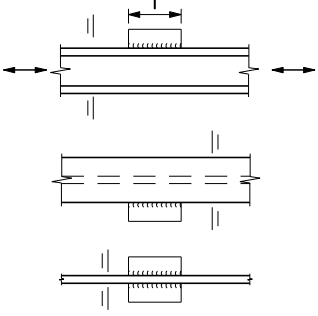
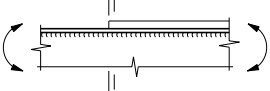

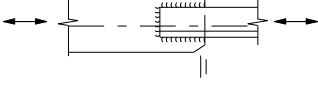
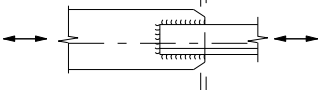
Note: Permissible Stress Amplitude in the above table has been calculated using the formula 1-2.

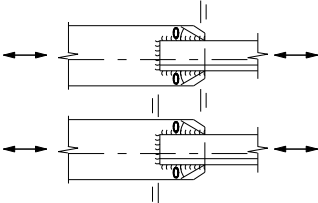
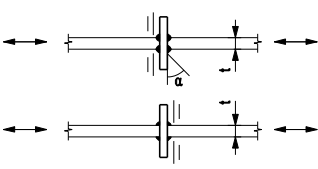
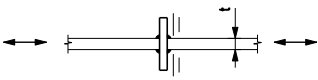
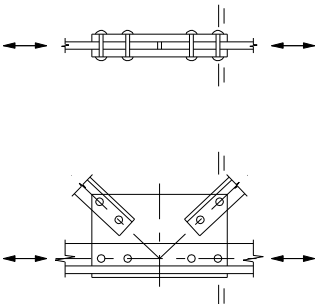
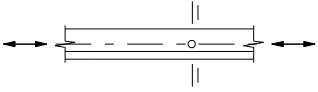
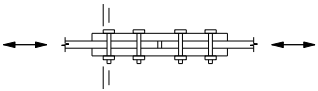
Classification of member and connection details for fatigue calculation

Table E2.4 shows detail categories for the more typical details of members and connections.

Table E2.4 - Member and connection detail categories

Reference Number	Illustration	Description	Detail Category Number
1		For continuous steel members: 1) Rolled Steel 2) Steel Panel a) Both sides are either rolled or planned side b) Both cutting sides are either automatic or semi-automatic (Cutting quality must correspond to GB 50205)	1 1 2
2		Transverse Butt Weld 1) Must be first grade welded seam that correspond to GB 50205 2) After additional finishing (especially polishing) of first grade welded seam	3 2
3		Polished transverse butt weld with different thickness (or width) should correspond to GB 50205.	2
4		Longitudinal butt weld - Welding must correspond to the second grade welding standard	2
5		Flange welded connection 1) Welded seam between flange plate and web plate a) Automatic welding, Second grade T - shaped butt and fillet grouped weld b) Automatic welding, Fillet weld, The appearance quality must correspond to the second grade c) Manual welding, Fillet weld, The appearance quality must correspond to the second grade	2 3 4

		<p>2) Welding connection between overlapping flange plate</p> <p>a) Automatic welding, Fillet weld, Appearance quality must correspond to the second grade</p> <p>b) Manual welding, Fillet weld, Appearance quality must correspond to the second grade</p>	<p>3</p> <p>4</p>
6		<p>End of transverse stiffener</p> <p>1) With continuous arc (use backward weld)</p> <p>2) With non-continuous arc</p>	<p>4</p> <p>5</p>
7		<p>Weld defects should not appear for ladder shaped joining plate, which uses butt weld to connect to flange beam, web plate, and truss member:</p>	5
8		<p>Members with rectangular shape joining plate welded to its flange or web with $l > 150\text{mm}$</p>	7
9		<p>Middle of flange plate (end of the outer plate welded)</p>	7
10		<p>Transitional (temporary) position of fillet weld used as tack weld</p>	6
11		<p>End of a two sides fillet weld member</p>	8
12		<p>End of a three sides fillet weld member</p>	7

13		Joining plate with a three sides or a two sides fillet weld (When calculating the width of joining plate, corresponding to the stress resulting the angle to increase, consider 0 - 30 degrees)	7
14		Members with T-Shaped butt weld (with K – shaped slope opening) and fillet group weld: two plate axes diverging less than 0.15t, second grade weld, weld end angle less than or equal to 45 degrees	5
15		Cross connected fillet weld, two plate axes diverging less than 0.15t	7
16	Fillet Weld	Use Shear Stress Amplitude Calculation on the most effective surface	8
17		Rivet connected members	3
18		Connection of slotted hole and bolt	3
19		Members with high strength friction grip bolt connections	2

Notes:

- 1) All butt weld connection shall be full penetration butt welds and comply with the requirements on workmanship, dimensions and details given in the Code.
- 2) All fillet welds shall comply with the requirements on workmanship, dimensions and details given in the Code.
- 3) The shear stress amplitude in reference no. 16, $\Delta\tau = \tau_{\max} - \tau_{\min}$, where the positive and negative sign of τ_{\min} is determined by the direction of τ_{\max} : when τ_{\min} and τ_{\max} are at the same direction, take the positive sign; when τ_{\min} and τ_{\max} are at the opposite direction, take the negative sign.
- 4) For calculating stresses in reference no.s 17 and 18: use the net sectional area, and for reference no.19 uses the gross sectional area.

E2.3.4 Structural integrity and robustness

General

A large amount of work has been carried out on structural robustness and the avoidance of disproportional collapse following the World Trade Center tragedy in the USA on 11th September 2001.

Two major studies have been completed, one by the US Federal Emergency Management Agency entitled World Trade Centre Building Performance Study and the other by the Institution of Structural Engineers entitled Safety in Tall Buildings and Other Buildings With Large Occupancy. In terms of recommendations affecting structural design, they essentially confirm the guidelines given in Eurocodes EC2 and EC3 and UK codes BS 5950 and BS8110. These earlier recommendations were originally formulated as the UK 5th Amendment to the UK Building Regulations following the 1968 progressive collapse of Ronan Point, a high-rise residential building of precast construction.

The principle structural issues to provide sufficient structural robustness given in these guidelines are:

- (a) Identifying any key elements in the structure whose failure would lead to a large part of the structure to collapse (for example a major column at ground floor of a high-rise building or a transfer plate). Then, considering various types of exceptional load (such as explosion, collision from aeroplane, lorry or train), which could conceivably arise and designing the element to resist that load.
- (b) Provide effective horizontal tension continuity ties around the building perimeter and internally at each principal floor (i.e. floors at 3.5 to 4.5m spacing, part mezzanine floors not necessarily included) connecting to vertical elements.
- (c) Provide vertical tension continuity ties at all principal columns and structural walls.
- (d) This 3 dimensional grid of tension continuity should be sufficiently strong enough such that the removal of a vertical element (except for a key element) will not result in collapse other than local failure to that element.
- (e) Design the structure to safely resist a minimum notional horizontal load (this may be the design wind load)
- (f) The UK codes suggest an explosion pressure of 34 kN/m². This value was derived from tests carried out in the UK following the Ronan Point collapse. For general design, this is still considered a reasonable value so is used by the Code; however higher values may be appropriate if more powerful explosives (e.g. from car bombs) or shaped demolition charges are considered as possible risks.

This clause gives recommendations on how to achieve structural integrity and robustness. These are based on current U.K. practice as codified in BS 5950 and BS 8110. The intention is to provide a structure that can tolerate damage without disproportionate collapse. Structural designers should develop an understanding of building systems as a whole, rather than as a set of discrete components, and conceive a dimensional structural system to safely carry the primary vertical and lateral loads to the ground.

There is a deem-to-satisfy approach by the provision of ties (in beams and columns). If ties are provided accordingly, the structure is robust.

In case ties cannot be provided to comply with the requirements as stipulated in the Code, structural elements may be removed one at a time to see if there is any disproportionate collapse. If this is too complex, the code also accepts the concept of key element. Key elements should be designed to perform satisfactorily for 34 kPa as given in clause 2.5.9 of the Code. If design of key element approach is chosen, there is no need to check for disproportionate collapse by the removal of structural elements one at a time.

Structural integrity should be provided by tying all elements together in both plan directions and vertically. This tension continuity allows:-

- (a) Edge columns to be restrained against buckling outward from the building.
- (b) Floors to span in catenary action if a support, say a single column below, is removed. As a minimum, the design ultimate value for the horizontal tie forces should be 75 kN per beam.
- (c) A portion of floor to hang from a column above if the column below is removed.

Particular elements of the structure that have a critical influence on its overall strength or stability should be identified as key elements. These elements should be designed to resist abnormal forces arising from extreme events.

The surrounding structure of non-key elements should be designed to survive the removal of that non-key element by establishing alternative load paths, i.e. bridging over the lost element. It is acceptable for large permanent deformations to occur in such accidental or extreme event loadings.

The systems providing lateral stability and resistance to horizontal forces, whether by bracing or frame action, should be robust and sufficiently distributed such that no substantial part of the building relies on a single lateral load resisting element.

Each part of a building between expansion joints should be treated as a separate structure i.e. should be robust in its own right.

Clause 2.3.4.2 of the Code gives recommendations on tension continuity tying of buildings and illustrates this in Figure 2.2.

Clause 2.3.4.3 of the Code gives recommendations on general tying, tying of edge columns, continuity of columns, resistance to horizontal forces and anchorage of heavy floors. The clause says that steel framed buildings designed as recommended in the Code may be assumed not to be susceptible to disproportionate collapse provided that the five conditions in the clause are met.

The clause defines that the size of the portion of the building at risk of collapse should not exceed 15% of the floor or roof area or 70 m² (whichever is less) at the relevant level and at one immediately adjoining floor or roof level, either above or below it. If it does, then the support element must be treated as a key element.

E2.3.5 Brittle fracture

Clause 2.3.5 of the Code is self-explanatory. Although brittle fracture is an ultimate limit state failure, it is a material issue and is discussed in detail in clause 3.2 of the Code and in clause E3.2 of this EM.

E2.4 SERVICEABILITY LIMIT STATES (SLS)

Clause 2.4 of the Code is generally self-explanatory.

E2.4.1 Serviceability loads

In the case of combined imposed load and wind load, only 80% of the full design values need to be considered when checking serviceability. In the case of combined horizontal crane loads and wind load, only the greater effect need to be considered when checking serviceability. A similar logic may be applied to other situations where the likelihood of a combination of serviceability loads acting together is lower than that of a single load type.

E2.5 LOADING

E2.5.1 General

Clause 2.5.1 of the Code is self-explanatory.

E2.5.2 Dead and imposed loading

Clause 2.5.2 of the Code is generally self-explanatory.

The clause says that for design in countries or regions other than Hong Kong, loads can be determined in accordance with local or national provisions. The Responsible Engineer

should however be careful when doing this since values of some imposed loads may vary from country to country (for example the UK value for car park design is 2.5 kN/m² compared with the Hong Kong value of 4 kN/m²). Load and material partial factors should not be taken from other codes and mixed.

E2.5.3 Wind loading

Clause 2.5.3 of the Code is generally self-explanatory.

This Code clause says that the minimum unfactored wind load should not be less than 1.0% of unfactored dead load in the appropriate load combinations 2 and 3 defined in clause 4.3 of the Code. This load shall be applied at each floor and calculated from the weight of that floor and associated vertical structure. This is unlikely to govern in Hong Kong but may in other regions where basic wind speeds are low. (for example, it can govern for some buildings in Singapore)

Internal structures such as temporary seating in a concert hall may be relatively light and not very stiff, thus a sensibly high value of lateral load must be applied to ensure a safe structure. The clause of the Code says that the design factored lateral load shall be the greater of 1% of factored dead plus imposed loads or that obtained from a factored lateral pressure of 1.0 kN/m², whichever is the greater. This pressure should be applied to the enclosing elevation of the structure, i.e. assuming it is clad whether it actually is or not. In effect, this is a hypothetical internal wind load.

E2.5.4 Loads from earth and water pressure

It should be noted that some recent geotechnical design codes derive worst credible earth and ground water loads rather than nominal. When worst credible earth and ground water loads are used, the value of the partial load factor may be taken as 1.2 instead of 1.4. Refer to BS5959-1: 2000 clause 2.2.4.

E2.5.5 Loading from differential settlement of foundations

Clause 2.5.5 of the Code is generally self-explanatory.

In some cases, it is reasonable to ignore foundation settlements in the design of superstructures. In other cases, the absolute and relative settlements may need to be taken into account when considering overall building movements from gravity and wind loads. The Responsible Engineer should use his or her judgement in establishing a reasonable analytical model including the flexibility of any piles and the founding strata.

E2.5.6 Load effects from temperature change

Clause 2.5.6 of the Code is generally self-explanatory. The clause draws attention to special structures such as pre-tensioned rod and cable structural systems where structural stability and designed pre-tension force very much depend on the assumed temperature change. The Responsible Engineer's attention is drawn to clause 13.3 of the Code which provides more detailed guidance on this.

E2.5.7 Loads from cranes

Clause 2.5.7 of the Code is self-explanatory. See also clause E13.7 in this EM.

E2.5.8 Notional horizontal forces

Notional horizontal forces and minimum lateral loads

Minimum lateral loads and notional horizontal forces are two separate issues, however there are some differences in wording in various different codes.

Minimum Lateral Loads (MLL)

MLL is to provide a minimum load to be considered for the structural design including foundations, global overturning etc. i.e. a minimum cut off for wind or seismic. Both clause 3.1.4.2 of BS 8110, clause 2.3.1.4 of the Code of Practice for the Structural Use of

Concrete and clause 2.4.2.3 of BS 5950 say it is to provide robustness. This load applies only to combinations 2 and 3.

BS8110 and the Code of Practice for the structural Use of Concrete define MLL as an ultimate load of 1.5% characteristic (unfactored) Dead Load (DL) i.e.:- $MLL = 0.015 \times DL$

BS5950 defines MLL as an ultimate load of 1.0% of factored DL ie:-

- for combination 2 $MLL = 0.01 \times 1.4 \times DL$

- for combination 3 $MLL = 0.01 \times 1.2 \times DL$

In each case, the load at each floor would be calculated from the weight of that floor (plus associated single storey of vertical structure).

The Code further defines a minimum internal wind pressure to be used of 0.5 kN/m^2 , this is consistent with the B(C)R.

Notional Horizontal Forces (NHF)

While NHF are not referred to in BS8110 or the Code of Practice for the structural Use of Concrete, they are a stability issue in BS5950 clause 2.4.2.4 to allow for imperfections etc and apply only to combination 1. They need not be considered on foundations nor be combined with other horizontal loads.

Their magnitude is 0.5% of factored DL + Live Load (LL) ie:

$NHF = 0.005 \times (1.4DL + 1.6LL)$ applied in combination 1 as an ultimate load.

The load at each floor would be calculated from the weight of that floor (plus associated single storey of vertical structure)

Clause 2.5.8 of the Code addresses a concern that for some very light structures, the NHF load may not be enough; so the Code additionally defines a Notional Lateral Pressure (NLP) of 0.5 kN/m^2 to be applied to the enclosing envelope of the structure and the greater of that or the NHF should be used. Again, by implication, this NLP would be applied in combination 1 as an ultimate load.

Furthermore, clause 2.5.8 of the Code requires these loads to be doubled for ultra sway sensitive structures.

The purpose of the notional stability loads is to take account of imperfections in structural geometry and to ensure that the lateral stiffness of a structure is sufficient to prevent overall buckling failure under the maximum vertical loads, i.e. to provide sufficient resistance to P- Δ effects. The purpose of placing a cut off to lateral load of a minimum of 1% of dead load is to ensure that the structure is not designed to an unsafely low lateral load. In cases where wind loads are low, such as in regions where the wind climate is benign or where the structural elevation will attract little wind load, the minimum value may govern.

Table E2.5 - Notional forces recommended by Eurocode 3: Part 1 (1993 & 2003)

Number of storey	1993			2003		
	Number of bay			Number of bay		
	1	2	4	1	2	4
1	0.50	0.46	0.42	0.43	0.41	0.39
2	0.42	0.38	0.35	0.43	0.41	0.39
3	0.37	0.33	0.31	0.35	0.33	0.32
5	0.32	0.29	0.26	0.29	0.27	0.26
10	0.27	0.25	0.23	0.29	0.27	0.26

Notes: Storey height equal to 2m.

$$N = 0.01 \times \phi \times \sum P$$

where $\phi = k_c k_s \phi_o$

$$\phi_o = 0.5$$

$$k_c = \sqrt{0.5 + 1/n_c}$$

$$k_s = \sqrt{0.2 + 1/n_s}$$

n_c is the number of column per frame

n_s is the number of storeys

$$N = 0.01 \times \phi \times \sum P$$

$$\phi = \alpha_h \alpha_m \phi_o$$

$$\phi_o = 0.5$$

$$\alpha_h = \frac{2}{\sqrt{h}} \quad \text{but } \frac{2}{3} \leq \alpha_h \leq 1.0$$

$$\alpha_m = \sqrt{0.5 (1 + 1/m)}$$

h is the height of structure

m is the number of columns

Given that geometrical imperfections exist, it would be logical to include their effects in the lateral combinations 2 and 3 since they would act additionally to wind load. However, for some types of building, this would greatly increase the overall lateral design load and would be a conservative, i.e. uneconomical, change in design. For tall buildings, it would imply that the imperfections all tend to cause the building to tilt one way whereas the statistical likelihood is that, for example, columns will be out of plumb in random directions. To some extent, the 1% dead load as minimum lateral load ensures that combinations 2 and 3 have a sufficiently high lateral load and the notional stability forces in combination with full dead and live loads is quite onerous. Also, the concept of partial load factor γ_3 takes account of structural variations. Thus, it is considered that a design would not be unsafe if the notional stability loads are only applied in combination 1.

Table 2.2 of the Code summarises the lateral forces to be considered in design for the three principal combinations of load.

E2.5.9 Exceptional loads and loads on key elements

Clause 2.5.9 of the Code is generally self-explanatory and the principles are repeated and amplified here for clarity.

Exceptional load cases can arise either from an exceptional load such as an impact from a vehicle (ship, lorry, aeroplane) or explosion, or from consideration of the remaining structure after removal of a key element.

In a building that is required to be designed to avoid disproportionate collapse, a member that is recommended in clause 2.3.4.3 of the Code to be designed as a key element should be designed to resist exceptional loading as specified in clause 2.5.9 of the Code. Any other steel member or other structural component that provides lateral restraint vital to the stability of a key element should itself also be designed as a key element for the same exceptional loading. The loading should be applied to the member from all horizontal and vertical directions, in one direction at a time, together with the reactions from other building components attached to the member that are subjected to the same loading, but limited to the maximum reactions that could reasonably be transmitted, considering the breaking resistances of such components and their connections.

The Code says that key elements and connections should be designed to resist an explosion pressure of 34 kN/m². This value is based on tests carried out in England following the partial collapse of the Ronan Point precast construction tower block in 1968.

Pressures from high explosives or gas or liquid fuels may be higher and in cases where the Responsible Engineer considers it necessary, he or she should seek specialist advice on a suitable explosion design pressure.

Similarly key elements and connections should be designed to resist the impact force from a vehicle where this could occur. Normal nominal design impact forces from vehicles shall be as specified in the current Code of Practice for Dead and Imposed Loads. It is noted that collision forces are calculated by converting the potential energy of the vehicle ($1/2 \times \text{Mass} \times \text{velocity}^2$) to work done on the structural element (Force \times distance to bring the vehicle to rest). Thus for heavy goods vehicles travelling at high initial speed and brought to a halt in a short distance, the calculated forces can become unmanageably large, see BS6779-1:1998 Highways parapets for bridges and other structures Annex A. In such a case, a better alternative may be to protect the key column with a crash barrier, which is designed to deform.

Table 4.3 of the Code contains the load factors and combinations with normal loads to be used in these situations and takes account of the reduced probability of other loads acting in combination with the exceptional event. It is noted that the extreme event load, for example the 34 kN/m^2 pressure, is considered to be an ultimate load; thus the partial load factor used is 1.0.

E2.5.10 Loads during construction

Clause 2.5.10 of the Code requires that loads on the permanent structure, which arise during construction, shall be considered in the design. This is a short and simple clause but overlooking it had lead to significant problems and failures in the past.

A particular case for designers to be aware of is when construction materials are stored on a partially complete structure which is not as strong as when completed, for example, if an area of slab is left uncast for a tower crane hole, then adjacent spans which are continuous in the permanent case will have no continuity at the edge of the hole in the temporary case. Another case is where unforeseen load paths may occur, perhaps from propping.

E2.5.11 Loading on temporary works in construction

Clause 2.5.11 of the Code is self-explanatory.

E3 MATERIALS

E3.1 STRUCTURAL STEEL

E3.1.1 General

Normal strength steel from international manufacturers

As stipulated in clause 3.1.1 of the Code, steel complying with one of the reference material standards in Annex A1.1 and basic requirements given in clause 3.1.2 and produced from a manufacturer with an acceptable Quality Assurance system shall be classified as Class 1 Steel.

The intention of the Code is to allow the use of steel and steel materials (for example bolts and nuts) from the major worldwide suppliers on a “level playing field” basis. The Code achieves this by using an approach based on a consistent set of acceptable reference standards from five major international regions which produce structural steel. These standards are listed in Annexes A1.1 and A1.7. These regions are:- Australia, China, the United States of America, Japan, and the European Union. [Note: this system allows the use of steel from another country, say from Korea, Malaysia or South Africa, (as a Class 1 steel as defined in the Code) as far as that steel complies with the steel material standard from one of the five regions.]

Normal strengths of steel are defined as having yield strengths ranging from 215 N/mm² (170 N/mm² for thick plates) to 460 N/mm². This range includes the lowest grade China steel Q235 up to the highest normally available structural steel strengths (the previously designated grade 55 steel) which are not specially heat treated.

Use of High and Ultra High Strength Steel

Various very high strength steel with yield stresses in the range 500 to 900 N/mm² are available from specialist manufacturers worldwide though 690 N/mm² is a more widely available upper yield strength value. Table D2 in Annex D of the Code lists some high strength steel and countries of supply. The steel is typically only available in plate form. In North America, an attempt was made to manufacture rolled I sections in high strength steel but they failed by cracks between flange and web.

Design issues for components made from high strength steel are buckling stability, reduced ductility and decreased weldability. These materials, which have higher strengths but the same stiffness as ordinary steel, may give advantages for certain ultimate limit states but with limited improvement against buckling. Their use does not improve the performance for fatigue and serviceability limit states. Correct welding procedures are essential and shall be specified. When high strength steel is used in compression, it shall be limited to compact sections where local buckling of outstands will not occur.

There have been some design and fabrication problems with its use in the past, these may have attributed to the relatively low ductility and weldability. Albeit high strength steel formed by the roller quenched and tempered (RQT) process method has the disadvantage of losing strength when heated during welding, the advances in welding technology has generally resolved these problems. Fire-protection or fire engineering becomes particularly critical for this steel.

Some supplier stated that they produce weldable steel plates up to 180 mm thick with yield strength of 690 MPa; and 30 mm thick with yield strength of 1100 MPa. Engineers should refer to supplier's documents for details and QA.

National building steel design codes generally do not yet provide design rules for high strength steel and its use worldwide has been limited. However, in plate form, it is used successfully in Australia and North America. and economics and environmental concerns require better and more efficient use of structural materials. Thus, as knowledge and experience of high strength steel use develops, it will become more widely used. Therefore, the Code allows the use of steel in the range above 460 N/mm² up to 690 N/mm² with restrictions. Plastic analysis and design is not permitted for steel with a yield strength greater than 460 N/mm².

Ultra high strength steel, defined as **Class UH**, with yield strengths greater than 690 N/mm^2 is not covered by the Code, but performance based design will allow their use. For such steel, the Responsible Engineer must justify each design on a case-by-case basis using parameters and formulae proposed by manufacturers and verified by himself. Because of the great difficulty in producing satisfactory welds in such steel, it is anticipated that they will mainly be used in bolted tension applications in the form of proprietary high strength tie rods or bars.

The Code covers hot rolled steel and cold formed structural hollow sections in clause 3.1 of the Code and cold formed steel open sections and profiled sheets in clause 3.8 of the Code.

The Code covers both elastic and plastic analysis and design. However, plastic analysis and design is not permitted for uncertified steel or for steel with a yield strength greater than 460 N/mm^2 .

Classes of normal strength steel

Clause 3.1.1 of the Code covers the design of structures fabricated from structural steel with a design strength not exceeding 460 N/mm^2 and defines three classes of steel. The clause is generally self-explanatory.

Table 3.1 of the Code summarises classes, strength grades and tests required.

E3.1.2 Design strength for normal strength steel

Clause 3.1.2 of the Code defines the design strength for steel and is generally self-explanatory. It also states the essentials of the basic requirements for this steel.

In practice, steel manufacturers typically quote “guaranteed minimum” strength values and 95% of tests show values above this. For example, for S275 steel, the mean strength of the steel is around 310 N/mm^2 and 275 is the mean less two standard deviations. This is part of the justification for using a material factor of 1.0 in the Code.

For convenience, the Code provides design yield strengths for the more commonly used grades and thicknesses of Class 1 steel supplied in accordance with European BS EN, Chinese GBJ, American ASTM, Australian AS and Japanese JIS standards for hot rolled steel. The design strengths p_y are given in Tables 3.2 to 3.6 of the Code.

A material factor of about 1.1 is already included in the design strengths for steel supplied in accordance with Chinese Standard GB 50017-2003 as given in Table 3.3 of the Code. It is recommended that this be retained for consistency with table 3.4 1-1 of the Chinese Standard. In this Code, a partial material factor γ_{m1} is then applied, with a value of 1.0.

The tables are not exhaustive and for rarer steel, the design strength p_y may be obtained from the formula given in clause 3.1.2 of the Code using values of minimum yield strength and minimum tensile strength from the product standard for that steel.

For commonly used grade 43C steel, the maximum contents for sulphur and phosphorous should not exceed 0.05% as stipulated in BS 4360: 1986. For equivalent grade S275J0 steel, the maximum contents for sulphur and phosphorous are reduced to 0.04% as stipulated in BS EN 10025: 1993. These maximum contents are further reduced to 0.03% as stipulated in BS EN 10025: 2004. Hence, the maximum contents for sulphur and phosphorous are set at 0.03% in clause 3.1.2 of the Code. While there is no intention to make the Code more stringent than the current reference standards, Class 1 steel products conforming to the materials reference standards from the five regions in Annex A1.1 are deemed to satisfy the chemical composition requirements. For Class 2 and Class 3 steel products, the chemical composition requirements as stipulated under “Weldability” in clause 3.1.2 of the Code should be strictly observed.

E3.1.3 Design strength for high strength steel

Subject to additional requirements and restrictions given in clause 3.1.3 of the Code, it defines an additional class of high strength steel with yield strengths greater than 460 N/mm² and not greater than 690 N/mm² and produced under an acceptable Quality Assurance system as **Class 1H** steel. The clause is self-explanatory.

For Class 1H steel products, the maximum contents for sulphur and phosphorous do not exceed 0.015% and 0.025% as stipulated in BS EN 10025-6: 2004. Hence, the maximum contents for sulphur and phosphorous are set at 0.025% in clause 3.1.3 of the Code. While there is no intention to make the Code more stringent than the current reference standards, Class 1H steel products conforming to the materials reference standards from the five regions in Annex A1.1 are deemed to satisfy the chemical composition requirements. Otherwise, the chemical composition requirements as stipulated in clause 3.1.3 of the Code should be strictly observed.

E3.1.4 Uncertified steel

The purpose of this clause is to allow steel with no mill certificate documentation to be used but with a conservatively low value of design strength and not in important situations. Australian code AS4100 defines this as unidentified steel. Use of unidentified steel is not discussed in BS5950. Generally, the use of such steel is discouraged. However, from time to time, contractors will wish to use it for economy. Thus, the Code does permit its use with restrictions. The span limit of 6 m follows from Buildings Department guidance that the Responsible Engineer is not required for such restricted spans.

For mechanical steel properties, the sample coupon test should typically pass the minimum tensile yield stress of 170 N/mm², ultimate breaking stress of 1.2 of yield stress, Charpy V-notch test and a minimum 15% elongation.

If welding is required, then chemical tests are required and the steel material should not have a carbon equivalent value (CEV) larger than those specified in BS EN 10025 for weldability requirement. It is noted that Eurocode 3 and the Chinese standard for use of low grade steel of grade 170 MPa or below allow such steel to be used as secondary members without chemical composition tests.

Clause 3.1.4 of the Code says that if class 3 uncertified steel is used, it shall be free from surface imperfections, it shall comply with all geometric tolerance specifications and shall be used only where the particular physical properties of the steel and its weldability will not affect the strength and serviceability of the structure. The design strength, p_y , shall be taken as not exceeding 170 N/mm² (while the tensile strength shall be taken as not exceeding 300 N/mm²).

E3.1.5 Through thickness properties

Clause 3.1.5 of the Code draws the attention of the Responsible Engineer to requirements for through thickness strength where steel plate is subjected to significant through thickness or "Z" stresses. For example, such situations can occur when plates are welded at right angles to thick plates. The essential requirement is an adequate strength and deformation capacity perpendicular to the surface to provide ductility and toughness against fracture. Particular weaknesses arise from laminations in the steel (lamellar tearing) or from a brittle central region of the plate (centreline segregation).

Lamellar tearing

This defect originates from inclusions in the steel which are distributed into planes of weakness as the steel is rolled. Subsequent tension across these laminations can cause failure. The welding procedures should be chosen so as to minimise tensile forces perpendicular to the plate. If necessary, material with high through thickness properties (e.g. HiZed steel) may be specified.

Centreline segregation

Centreline segregation is a material deficiency that may exist within the centre of continuously cast (concast) plate products and some sections. It arises from impurities on the surface of the molten steel being drawn down into the centre of the steel as it comes out of the vat (or furnace) into the roller chain. It can lead to local reductions in toughness and weldability that can cause cracking in tee butt and cruciform weld configurations.

The use of good welding practice and design details may be sufficient to avoid these problems, i.e. :

- Avoid tee, butt or cruciform welds in which the attachment plate is thicker than the through plate.
- Minimising through thickness tension especially at the edges of plates.
- Dressing any cut edges to remove any areas of increased hardness.
- Using smaller weld volumes.
- Developing weld details and processes that minimise the restraint to welds.

E3.1.6 Other properties

Clause 3.1.6 of the Code gives values for Young's modulus, Poisson's ratio and the coefficient of thermal expansion for steel and is self-explanatory. The clause gives a value of $14 \times 10^6 / ^\circ\text{C}$ for the coefficient of thermal expansion in order to be consistent with section 12 but for normal working temperatures of steel, i.e. less than 100°C , a value of $12 \times 10^6 / ^\circ\text{C}$ is appropriate.

In composite construction, normal weight concrete and reinforcement shall comply with the recommendations given in HKCC. However, the elastic modulus of reinforcement shall be taken as 205 kN/mm^2 , i.e. same as that of structural steel sections.

E3.2 PREVENTION OF BRITTLE FRACTURE

Brittle fracture can occur in welded steel structures subjected to tension at low temperatures. In certain situations, where fracture-sensitive connection details, inappropriate fabrication conditions or low toughness weld materials are used, brittle fracture can also occur at normal temperatures. The problem is tackled by specifying steel and welded joints with appropriate grades of notch toughness, usually implemented in practice by specifying grades of notch ductility in the Charpy test. Higher grades are required for thicker steel and joints.

Guidance on selection of appropriate Charpy grades of steel to provide sufficient notch toughness at the design temperature of the steel is given in clause 3.2 of the Code. In some contracts, the Responsible Engineer will provide requirements in the form of a performance specification and the steelwork fabricator will provide the correct Charpy grade to meet this specification.

Clause 3.2 of the Code gives descriptive guidance that brittle fracture should be avoided by ensuring parent steel plates and fabrication are free from significant defects and by using a steel quality with adequate notch toughness as quantified by the Charpy impact properties. The criteria to be considered are:- minimum service temperature, thickness, steel grade, type of detail, fabrication procedure, stress level.

The welding consumables and welding procedures should be chosen to give Charpy impact properties in the weld metal and heat affected zone of the joint that are equivalent to, or better than, that the minimum specified for the parent material.

In Hong Kong, the minimum service temperature T_{\min} in steel is normally taken as 0.1°C for external steelwork. For cold storage, locations subject to exceptionally low temperatures or structures to be constructed in other countries, T_{\min} should be taken as the minimum temperature expected to occur in the steel within the design working life.

The calculation procedure given in clause 3.2 of the Code is generally self-explanatory. The Code also contains in Table 3.7 tabulated values of maximum basic thickness for the normally available strengths of steel (in the range from 215 to 460 N/mm^2) and Charpy 27 Joule impact energies. These are given for a minimum design temperature of 0.1°C

appropriate for Hong Kong. They must be modified by the appropriate factor K given in Table 3.8 of the Code for type of detail and stress level present. For specified temperature at 20°C, the values of maximum basic thickness can be assessed using the formulae 3.2 to 3.4 of the Code.

Additionally, the maximum thickness of the component should not exceed the maximum thickness t at which the full Charpy impact value applies to the selected steel quality for that product type and steel grade. This will be given in the relevant acceptable standard for the particular steel product as listed in Annex A1.1 of the Code.

Tables 3.7 and 3.8 of the Code are derived from recent research and are based on an assumed surface flaw size (i.e. depth) of 0.15 of the plate thickness. An adequate “rule of thumb” has been found from the results of the latest fracture mechanics calculations, that for grade of steel **strength up to and including Grade 355**, the limiting surface flaw size for thicknesses twice that derived from Tables 3.7 and 3.8 should be half the surface flaw size for the basic limit case. In other words, the limiting flaw size for thicknesses twice that derived from Tables 3.7 & 3.8 should be 0.075 of the plate thickness. However, selection of grades of steel should normally comply with the requirements of the CoP and any deviations from this require formal approval by the Responsible Engineer and are likely to involve more stringent non destructive testing and acceptance standards. Any proposed deviation from the requirements of the Code should be supported by fracture mechanics analysis of the particular situation that must be submitted to the Responsible Engineer for his approval.

For detection of surface flaws in critical areas of steel plates, magnetic crack detection or dye penetrant testing should be carried out. To determine the depths of any surface flaw detected, ultrasonic testing in areas around the weld should be specified by the Responsible Engineer. The fracture mechanics calculations assumed a surface flaw aspect ratio (i.e. length to depth) of 10:1, and a practical aspect ratio of 3:1 (i.e. depth being one third of the length) would almost invariably over-predict the flaw depth and hence be conservative.

As an example of a possible non-compliance situation, a Grade 355 steel material is used to build up a truss for which the designer has found 100 mm thickness to be required from conventional stress analysis. The attachment of the braces to the tension chord member would be partial penetration butt-welded; and the K factor according to Table 3.8 of the Code would be 0.8. To use 100 mm thick material the Charpy impact properties would need to comply with 27 J at -50°C, whereas the maximum permitted thickness of the steel member for J0 material is calculated as $50\text{mm} \times 0.8 = 40\text{mm}$. In exceptional circumstances, the Responsible Engineer for a project might be prepared to accept lower Charpy impact properties with increased non destructive testing. If the limiting flaw size is reduced from 0.15 to 0.075 of the limiting plate thickness in the Code, (i.e. $0.075 \times 50\text{mm} = 3.75 \text{ mm}$ depth) the maximum thickness could be increased to $50\text{mm} \times 0.8 \times (0.15/0.075) = 80\text{mm}$. If, for special reasons, the Responsible Engineer is prepared to consider allowing J0 material to be used with a further increase in the maximum allowable thickness to 100 mm, this could only be accepted with a further reduction of the limiting defect size to 0.06 of the limiting basic thickness in the Code, i.e. $50\text{mm} \times 0.8 \times (0.15/0.06) = 100\text{mm}$. In this respect, it must be recognized that the likelihood of such defects occurring will increase with increasing thickness and the likelihood of ensuring that any/all such defects are detected and eliminated in a large structure will decrease. Besides checking the surface flaw sizes of steel plates, welding would also induce defects at the weld toes. One of the effective means to mitigate the detrimental effect, if the Responsible Engineer accepts reduced Charpy impact properties of this order, is to have the toes of the butt welds to be ground to a smooth radius of say 6 mm with full magnetic particle crack detection and no visible defects permitted. It is worth to emphasize again that such a solution can only be accepted with rigorous quality control and inspection to confirm that all susceptible regions have been treated satisfactorily and Responsible Engineers should only accept such proposals in extreme circumstances and with appropriate expert advice.

As a second example of a non compliant situation, Grade 355J0 steel material has been specified in a mega composite column construction, in which there is transient tension under wind load and the tensile stress exceeds $0.3 Y_{\text{nom}}$. The composite column is a built-up H section and the unstiffened outstand element is required to be butt-welded

while two splice cover plates, each of say 300mm long by 100mm thick, have been specified to be welded to both faces of the internal element. The K factor according to Table 3.8 of the Code for the above mentioned welded details would be 0.5. Hence, the maximum thickness of the steel member for J0 material is calculated as $50\text{mm} \times 0.5 = 25\text{mm}$. If the limiting flaw size is reduced from 0.15 to 0.075 of the basic limiting plate thickness, the Responsible Engineer might be prepared to accept that the maximum thickness could be increased to $50\text{mm} \times 0.5 \times (0.15/0.075) = 50\text{mm}$. In this respect, the likelihood of such defects occurring will increase with increasing thickness and the likelihood of ensuring that any/all such defects are detected and eliminated in a large structure will decrease. Besides checking the surface flaw sizes of steel plates, welding would also induce defects at the weld toes. One of the effective means to mitigate the detrimental effect is to have the toes of the butt welds to be ground to a smooth radius of say 6 mm with full magnetic particle crack detection and no visible defects permitted. It should also be noted that the situation has been aggravated by the presence of the cover plates, and a better solution is to adopt welding procedures that guarantee full penetration welds, confirmed as defect free by non destructive testing, and to omit the cover plates and the stress concentration effects they produce.

As noted above, any deviation from the CoP would require a fracture mechanics assessment based on drawings of the structure and details concerned, full information on the material properties from mill certificates, full information on the welding procedures and consumables and full information on the supervision, inspection and non destructive testing. The examples given above are provided to show possible solutions to difficult situations but should not be taken as automatically acceptable, since a general guidance may cause a serious risk of being misunderstood and misinterpreted. The above assumption in reducing initial flaw size would only apply to Grade 355 steel and below, and for higher grade of steel, justification by fracture mechanics calculations should be given if the maximum plate thickness calculation is to be deviated from the requirements as stipulated in this clause of the Code.

When steel materials of JR grade are used as external steelworks in Hong Kong, the maximum basic thickness as stipulated in Table 3.7 should be adjusted using equations 3.2 to 3.4. Occasionally, the mill certificates of such JR steel do not contain Charpy Test results, If so, additional Charpy Impact test at 20°C should be conducted to the requirements as stipulated in the relevant reference materials standards.

E3.3 BOLTS

Normal and high strength friction grip or preloaded bolts

Clauses 3.3.1 and 3.3.2 of the Code are self-explanatory. See also clause E14.4 of this EM. Bolts of grade 10.9 or above should not be galvanised.

E3.4 WELDING CONSUMABLES

Clause 3.4 of the Code is generally self-explanatory.

The general principle for steel with design strength not exceeding 460 N/mm^2 is that weld material should be at least as good as the parent metal in terms of strength and ductility.

As this may be difficult to achieve for high strength steel, the welding material, in such case, is allowed to be of a lower strength subject to being at least as ductile as the parent metal and the joint strength being based on the lower weld metal strength. However, weld materials of strength lower than that of the parent metal should not be used in an earthquake loaded situation.

E3.5 STEEL CASTINGS AND FORGINGS

Clause 3.5 of the Code is self-explanatory.

E3.6 MATERIALS FOR GROUTING OF BASEPLATES

Clause 3.6 of the Code is self-explanatory.

E3.7 MATERIALS FOR COMPOSITE CONSTRUCTION

Clause 3.7 of the Code is generally self-explanatory. It specifies the documents with which materials for composite construction other than structural steel must comply. These are:- concrete, reinforcement, shear studs and profiled sheeting used as permanent formwork and reinforcement for slabs.

Section 10 of the Code covers design for composite construction itself, noting that the Code covers the use of concrete and normal strength steel with limited strength. The Code does not forbid the use of higher strength steel or concrete and should the Responsible Engineer wish to use them, he or she would need to carry out a performance based justification in accordance with clause 2.1.6 of the Code.

E3.8 COLD-FORMED STEEL MATERIAL PROPERTIES

Clause 3.8 of the Code is self-explanatory.

E4 LOAD FACTORS AND MATERIAL FACTORS

E4.1 PARTIAL SAFETY FACTORS

Limit state philosophy, including discussion of the principles of limit state design, is covered in clause 1.2 in outline and in clause 2.2 of the Code. Individual load types are covered in clause 2.5 of the Code. Section 4 of the Code describes partial load and material factors and gives tables of load combinations to be used in various design cases.

Clause 4.1 of the Code is relatively short, thus a more detailed description is given here to clarify the underlying logic of the build up of the partial load and material factors. This should help in understanding how the factors can change in various design cases.

In limit state design, both cross section capacity and member resistance are checked against material yielding and structural instability respectively, and various load and material partial safety factors are incorporated for different modes of failure and limit states.

Ultimate design loads or factored loads Q_{ult} are obtained by multiplying characteristic loads Q_{char} by partial load factors γ_1, γ_2 :

$$Q_{ult} = \gamma_1 \gamma_2 Q_{char}$$

Design load effects S_{ult} , for example bending moments, are obtained from design loads by the appropriate design calculation and multiplying by a further partial load factor γ_3 :

$$S_{ult} = \gamma_3 (\text{effects of } Q_{ult})$$

The partial factor γ_1 allows for variation of loads from their characteristic (i.e. assumed working) values, γ_2 allows for the reduced probability that various loads acting together will reach their characteristic values and γ_3 allows for inaccuracies in calculation and variations in structural behaviour.

For simplicity, a single partial load factor γ_f is used in clause 4.1 of the Code.

Ultimate design resistance R_{ult} is calculated from dividing characteristic or specified material strengths by a materials partial factor γ_{m1} to allow for manufacturing tolerances and variations of material strengths from their characteristic values. In some codes, for example BS 5400 Part 3, the materials partial factor is explicitly split into one part to take account of reduction of strength below the characteristic value and another part to allow for manufacturing tolerances and other material defects.

In the Code, the resistance is the lesser of the yield strength Y_s divided by the partial material factor γ_{m1} or the ultimate tensile strength U_s divided by the partial material factor γ_{m2} , i.e. :-

$$R_{ult} = f(Y_s/\gamma_{m1} \text{ but } \leq U_s/\gamma_{m2})$$

where γ_{m1} allows for manufacturing tolerances and variations of material strengths from their characteristic values.

For satisfactory design of an element at ultimate limit states, the design resistance R_{ult} must be greater or equal to the design load effects S_{ult} :

$$R_{ult} \geq S_{ult}$$

For satisfactory design of an element at serviceability limit states, the same logic applies with changed values for the load factors, typically values of load factors for serviceability calculations are 1.0. The material factor on properties such as Young's modulus is 1.0.

In the Code, the partial load factors γ_1, γ_2 and γ_3 are multiplied together and given as a single value for a particular limit state. The material factor for strength calculations on structural steel is taken as 1.0, ie $f_y = p_y / 1.0$.

For strength design, the ultimate material design strength p_y is taken as the material yield stress. This is limited to a maximum value of the ultimate tensile strength divided by 1.2.

The most probable value of ultimate design strength is required for certain performance based calculations, for example in seismic design where one particular element must fail before another. This would require a partial materials factor of the order of 0.8. Guidance on this is given in section 4 of the Code.

E4.2 MATERIAL FACTORS

E4.2.1 Steel plates and sections

Clause 4.2 of the Code gives values of γ_{m1} and γ_{m2} for the various classes of steel plates and sections defined in the Code, generally for **Class 1** and 1H steels γ_{m1} is 1.0 and γ_{m2} is 1.2, i.e. the ultimate material design strength for steel: $p_y = Y_s / 1.0$.

Class 2 steel from a known source may be tested and if found to comply may also be used with material factors of $\gamma_{m1} = 1.1$ and $\gamma_{m2} = 1.3$. The rationale for using increased material factors rather than allowing the Class 2 steel to be reclassified as true class 1 is that the product specifications for Class 1 steel from the 5 regions give minimum requirements only. Typically, a good modern steel product from one of the 5 regions will be significantly better than these minima.

Steel plates, sections and weldable castings from an unknown source are defined as **Class 3**. The use of such steel is not recommended; but from time to time, it may be required to recycle previously used steel or steel where mill certificates have been lost. Such materials may only be used for minor structural elements where the consequences of failure are limited. Then, their design strength p_y is limited to 170 N/mm^2 . The Australian code AS4100 also limits the ultimate tensile strength of such steel to 300 N/mm^2 .

The most probable value of ultimate design strength is required for certain calculations, for example in seismic design where one particular element is designed to fail before another. This requires a partial materials factor γ_{m1} below 1.0 in order to reflect the higher actual ultimate tensile strength of the steel. In the absence of more detailed information, a value of 1/1.2 may be used. If records of mill certificates show that a different figure to 1/1.2 is appropriate to the difference between the characteristic yield strength and the average yield strength as rolled and supplied for fabrication, then that factor shall be used in place of 1/1.2.

E4.2.4 Grout for base plates and wall plates

Clause 4.2.4 of the Code is self-explanatory. It states that material factors for cement grout should be the same as for concrete of the same cube strength, implying that the ultimate design strengths in bearing, bond and shear are the same as for concrete of equivalent cube strength f_{cu} . It should, however, be noted that Young's modulus values for grout are significantly lower than that for concrete since grout entirely comprises cement paste. In the absence of more accurate information, a value of around 1/3 that of concrete of equivalent cube strength may be used.

E4.3 LOAD FACTORS AND COMBINATIONS

Clause 4.3 of the Code is generally self-explanatory and describes the three principal load combinations which must be considered for design.

The various types of load to which a structure may be subjected are given in clause 2.5 of the Code. Clause E2.5.8 of the EM discusses the rationale behind only requiring notional stability loads to be considered in load combination 1.

E4.3.1 Load combinations for normal ultimate limit state

Clause 4.3.1 of the Code is generally self-explanatory and the load factors and combinations given in Table 4.2 of the Code apply to strength and stability for normal design situations.

Where the action of earth or water loads can act beneficially, the Code provides that the partial load factor should not exceed 1.0. (The value of the partial load factor γ_f should be taken such that $\gamma_f \times$ the design earth or water pressure equals the actual earth or water pressure).

Clearly, the beneficial load factor for water pressure should be taken as 0.0, for example when checking an empty swimming pool for stability against uplift from the external water table, the water pressure inside the pool is zero and that for outside should be maximum.

The Code notes that collision loads are required to be considered as part of normal design, i.e. are not considered as an extreme event, they shall be treated as normal live loads with the appropriate safety factor.

As discussed in section 2 of the Code, differential settlements and temperature effects need only be considered when they are significant or when second order effects are important. Generally, they need not be considered at ultimate limit state provided that rotational capacity and ductility of the structural members and connections are sufficient. BS 5950 uses a load factor of 1.2 for temperature effects and says nothing about differential settlement. The Code of Practice for the Structural Use of Concrete proposes a load factor of 1.0 for temperature effects and 1.4 for differential settlement. The ACI code for concrete uses 1.4 for differential settlements in combination with dead loads and $0.75 \times 1.4 = 1.05$ in combination with dead and live loads. Steel structures are generally more ductile than concrete but possibly more susceptible to thermal load effects as they are less massive. Therefore, it might seem more logical to apply a load factor of 1.2 for both thermal and differential settlement effects since they are both caused by imposed deformations. However, the Code uses a value of 1.4 for differential settlements in load combinations 1 and 2, reducing to 1.2 in load combination 3.

E4.3.2 Load combinations for overhead traveling cranes

Clause 4.3.2 of the Code is self-explanatory.

E4.3.3 Load combinations for building assessment

Clause 4.3.3 of the Code is generally self-explanatory. The values of partial load factor given in Table 4.2 of the Code should normally be used. In assessing old structures, there may be some situations where engineering judgement has to be applied in justifying their structural capacity. This is discussed in section 17 of the Code and section E17 of this EM.

E4.3.4 Load combinations for temporary works in construction

The intention of the Code is to strike a balance between safety and economy for the structure. Engineering tradition in the past has sometimes been to reduce load factors, or, in the case of permissible stress codes, increase permissible stresses, particularly in the design of temporary works.

However, temporary works are prone to collapse for various reasons. Workmanship may be poor with non-concentric bracing connections or inadequate foundations. Old and damaged steel elements may be used. The lateral strength and stiffness of light temporary structures such as scaffolding or temporary support towers may be low because of low wind frontal area. Thus, the overall resistance to buckling of such structures may be poor.

Temporary works often fail because of inadequate support pads for scaffolding props, thus the Code states that temporary foundations shall be checked for the effect of differential settlements.

Therefore, the Code states that the load factors in Table 4.2 shall be used unless it is considered that the consequences of failure of a particular element is serious enough to warrant a higher load factor. It also states that in no circumstances should any adverse load factor be less than 1.2. This includes load factors for wind loads, in which the actual design wind speed given in a wind code may be lower for a temporary structure designed for a life shorter than a permanent structure.

E4.3.5 Load combinations for exceptional events

Various recent codes such as BS 8110, BS 5950 and the Code of Practice for the Structural Use of Concrete give guidance on load factors for exceptional events, taking account of the likelihood of their occurrences simultaneously. There is general consensus in recent codes, for example clause 2.3.1.5 of the Code of Practice for the Structural Use of Concrete and clause 2.4.5.3 of BS 5950, on an overall minimum partial load factor of 1.05 and likely live and wind loads of 1/3 of characteristic. Thus, a live or wind load factor of $1.05 \times 0.33 = 0.35$ is given in Table 4.3 of the Code for wind and live loads. The extreme event load may be the explosion pressure of 34 kN/m^2 or the impact force from a vehicle (e.g. ship, lorry, aeroplane). In the case where the remainder of the structure is being checked after an element such as a column has been removed, there is no extreme event load as such.

Exceptional load cases can arise either from an exceptional load such as a vehicle collision or explosion or from consideration of the remaining structure after removal of a key element. The magnitude of the load effect caused by the exceptional event is such that the load is considered to be an ultimate load, thus the partial load factor for the load effect is 1.0.

Table 4.3 of the Code contains the load factors to be used in these situations and take account of the low probability of other loads acting in combination with the exceptional even loads.

Fire resistant design is dealt with separately in clause 12.1.5 of the Code.

E4.3.6 Summary of partial load factors

Table 4.4 of the Code provides a useful summary of the various partial load factors used in the preceding sections and is self-explanatory.

E4.3.7 Load combinations for serviceability limit states

Clause 4.3.7 of the Code is self-explanatory. The Responsible Engineer should use engineering judgment and apply different serviceability load factors other than 1.0 if he or she considers it would provide a more realistic case. In particular, clause 2.4.1 of the Code allows a serviceability load factor of 0.8 when considering deflections from live and wind loads in combination. It is noted that this applies to situations such as the deflection of a roof beam.

E5 SERVICEABILITY LIMIT STATES

E5.1 GENERAL

Section 5 of the Code contains particular requirements and guidance for deflection control and building dynamics including advisory criteria for wind induced oscillation. It also covers durability and protection against corrosion.

E5.2 DEFLECTION

The deflection limits in Table 5.1 are only applicable to building structure in general and is not meant to cover all kinds of structures. For covered walkway and other similar structures, it is the onus of the designer to propose the deflection criteria with justification.

E5.3 WIND-INDUCED OSCILLATION

Clause 5.3 of the Code is generally self-explanatory.

E5.3.1 Wind sensitive buildings and structures

Clause 5.3.1 of the Code is self-explanatory.

E5.3.2 Serviceability limit state

Clause 5.3.2 of the Code is self-explanatory.

E5.3.3 Dynamic structural characteristics

Clause 5.3.2 of the Code is generally self-explanatory. Natural frequencies and structural damping should be measured or computed for a building or structure. Both parameters are sensitive to amplitude of motion; for composite structures, the concrete will crack at higher amplitudes reducing stiffness and frequency and increasing damping.

E5.3.4 Serviceability criteria for tall buildings

Engineering practice in various parts of the world regarding the design criteria for the comfort of occupants of tall buildings is provided in the form of non-mandatory guidance and thus most building design codes do not include specific requirements for comfort and serviceability requirements of such structures.

Clause 5.3.4 of the Code provides clear requirements for comfort design criteria. Hong Kong is in a region where typhoons occur regularly and where many tall buildings are constructed, and it was considered that clear requirements should be provided in the Code in order to better assist designers. In order to provide simple criteria, the clause was based on the following considerations:

1. Peak acceleration limit as a more appropriate criterion when compared to root-mean-square acceleration limit. This is based on the research results that human comfort is most related to the second or even third derivative of displacement. It is also based on the likelihood that a person will notice the largest (peak) rather than average acceleration in an event (storm). A root-mean-square value is obtained from squaring each of the 'n' numbers of acceleration values, summing them, dividing the sum by n and taking the square root of the result.
2. The 10-year return period has been used as this is consistent with the Chinese code JGJ3-2002 and National Building code of Canada 1995.
3. The 10-minute duration is used since it is a typical period of maximum response during an extreme wind event. This value has also been adopted in ISO 6897. The 10-minute mean wind speed can be obtained from the hourly mean wind speed but this conversion depends on the particular wind regime and advice should be sought from an experienced wind engineer or wind tunnel laboratory.

4. The value given in the table for occupied buildings is the same as that in the Chinese Code and has been used in many wind tunnel studies in Hong Kong.
5. The value for unoccupied buildings is the same as that in the Chinese code, and is based on experience when motion can start to interrupt normal walking patterns.
6. For simplicity, the adopted approach does not depend on the natural frequency of the structure.

Generally, the maximum wind load and deflection during the design level storm occurs along an axis in the along-wind direction. Cross-wind response may dominate in the case of certain tall and slender structures, (typically with an aspect ratio of 5:1 or greater), and in this case, occupant discomfort due to building motion may be an issue. Torsional effects and eccentricity between centres of building mass and stiffness can exacerbate building response to wind.

Excessive deflection may cause cracking of masonry, partitions and other interior finishes and building façade.

A useful “rule of thumb” for estimating the lowest natural frequency is to assume a value of $f_0=46/H$, where H is the height of the building in metres.

Clause 5.3.4 of the Code allows three approaches to address serviceability and comfort criteria:-

The first is to limit the top deflection to Height/500 and the inter storey drift to Storey Height/400.

Limiting deflection at the top most storey of a building to H/500 under the design wind load specified in the current Code of Practice for Wind Effects in Hong Kong will usually provide an acceptable environment for occupants in most typical buildings without the need for a dynamic analysis. However, the RSE should always consider each building on its merits.

The second is to carry out a dynamic computational serviceability analysis and design in order to justify compliance with the serviceability limits for tall buildings given in sub-clauses (a) and (b). In such a case, the design and detailing of cladding, curtain wall, partitions and finishes should also take into account the effects of deflection, inter storey drift and movement.

In addition to calculations, wind tunnel testing may or may not be carried out as recommended by the Responsible Engineer.

The third is to carry out a full performance based design. It was recognized during the Code drafting process that some designers might wish to make use of current best practice and recent research and carry out a performance based assessment of the acceptable movements and deflections of a tall building structure. Therefore, sub-clause (c) of clause 5.3.4 of the Code allows a performance based design to be carried out as an alternative to the requirements of sub-clauses (a) and (b) and the acceleration criteria given in the table under subclause (b).

Guidance for a performance based design approach

When adopting such a performance based approach, the comfort criteria would need to be agreed between the project client and designer. Occupant tolerance of motion is influenced by many factors including experience, expectation, frequency of building motion, frequency of exposure, and visual and audio cues. The designer might elect to use a motion simulator in order to better appreciate what different levels of acceleration actually feel like and to understand the frequency dependent nature of the perception of accelerations. Such a performance-based approach would normally include comprehensive wind tunnel testing.

E5.3.5 Serviceability criteria for communication and broadcasting towers

Clause 5.3.5 of the Code is self-explanatory.

E5.3.6 Reduction of wind-induced dynamic response

Clause 5.3.6 of the Code is self-explanatory.

E5.4 HUMAN INDUCED FLOOR VIBRATION

Clause 5.4 of the Code says that when the deflection limit for beams and floors are exceeded, it may be necessary to carry out a vibration assessment of the floor structure. Typically, this may be necessary for light weight and long span structures, for dance floors, rooms where gymnastics and aerobics occur, stadia especially at cantilevered terraces. It may also be necessary where sensitive production equipment is used, e.g. for chip making, and for operating theatres.

Reference should be made to relevant codes of practice and specialist literature, in particular the Canadian Code, The Steel Construction Institute Guide and the ASCE guide all provide up to date information on this topic.

When modelling floor systems for vibration analyses, the degree of fixity of floor beams should be realistically assessed. It is noted that end connections of steel beams, designed as simply supported i.e. "pins" for strength, will usually actually act as fixed ends for low loads and movements since the connection bolts will provide a weak frictional grip. Therefore such connections should be modelled more realistically as fixed in this case.

E5.5 DURABILITY

E5.5.1 General

Steelwork can be subjected to many different types of environmental exposure. Clause 5.5 of the Code provides general guidance for steelwork in buildings and some other structures subjected to more commonly occurring exposure conditions.

There is a perception that if steel is exposed to the atmosphere, then it will corrode. While this is hard to dispute, it should not be a reason for limiting the use of steel as a structural material. In many cases, it is found that the rate of corrosion is often tolerable, within conventional design limitations, such that no additional protection is required.

However, in many other instances, this may not be the case, particularly when considering buildings and the desires of clients and architects to have attractive looking structures. In these cases, it is necessary to provide additional protection to steel. The type of exposure environment will determine the rate at which corrosion occurs.

The guidance given in clause 5.5 of the Code is based on experience and good practice generally and the following references:-

- (1) CIRIA Report 174.
- (2) The manual for the design of steelwork building structures, published by the Institution of Structural Engineers.

The provisions of clause 5.5 of the Code assume that workmanship is carried out in accordance with clause 14.6 of the Code and that maintenance of paint systems is carried out in accordance with clause 13.8 of the Code. Proper specification, inspection and maintenance is required in order to avoid premature failure of an inappropriately chosen system.

The purpose of clause 5.5 of the Code is to provide general guidance on corrosion protection. It is not intended to be definitive or mandatory, nor does it attempt to prescribe particular solutions in detail. Detailed guidance on corrosion protection can be found in specialist literature.

The following factors should be taken into account in design of protective systems for steelwork in order to ensure the durability of the structure under conditions relevant both to its intended use and to its design working life.

- a) The environment of the structure, whether bimetallic corrosion is possible and the degree of exposure of the structure.
- b) Whether inspection and maintenance are easy, difficult or impossible. Access, safety and the shape of the members and structural detailing affect this.
- c) The relationship of the corrosion protection and fire protection systems.

More information concerning less common situations and more detailed guidance can be found in the CIRIA Report 174, BS EN 12944 and in proprietary literature from paint manufacturers. The Responsible Engineer should always evaluate data from proprietary literature with caution.

Typical exposure conditions

Table 5.2 of the Code classifies five types of exposure condition of increasing severity and provides commonly occurring examples of each. It is self-explanatory.

Maintenance regime

Clause 5.5.1.2 of the Code defines three classes of maintenance, depending on access, and notes that the degree of maintenance to be carried out to the protective system depends on the client's requirements for initial cost versus ongoing maintenance cost and also on the accessibility of the steelwork for carrying out the maintenance.

E5.5.2 Types of protection against corrosion

Clause 5.5.2 of the Code is generally self-explanatory. It describes various corrosion protection systems and gives guidance on their applicability for particular levels of exposure. The clause notes that all relevant information including the proposed maintenance regime should be considered before selecting an appropriate system.

Galvanizing

Clause 5.5.2.1 of the Code describes hot dip galvanizing and gives warnings on its use for high strength plates and bolts. In particular, high strength steel (in plate, rolled section or bar) of design strength greater than 460 N/mm² should not be galvanized in order to avoid the risk of hydrogen embrittlement cracking or annealing. Bolts of Grade 10.9 or higher grade or equivalent should not be galvanized for similar reasons.

Recent experience suggests that cracking of normal steel during galvanizing may occur and it is recommended that the Responsible Engineer should read the recent publication BCSEA/GA 40/05 written jointly by the British Steelwork Construction Association and the Galvanising Association. Generally, the galvanizing of components whose failure would be critical is not recommended.

Hollow sections should be vented in order to prevent pressure build up and possible explosion if they are to be galvanized.

It is noted that it is very difficult to adequately degrease and clean a galvanized surface such that paint will adhere to it. Proper application of an etch primer e.g. "British Rail T Wash" can work but in practice, painting over galvanizing is difficult to carry out successfully and requires correct choice of primer, thorough preparation and good workmanship.

Concrete casing

Clause 5.5.2.2 of the Code gives recommendations for concrete casing and is generally self-explanatory. Smooth mill scale has been known to reduce bond between steel and concrete in composite sections and thus if concrete is required to act compositely with steel to transfer significant shear stresses (over 0.1 N/mm²), then clause 5.5.2.2 of the Code states that the steel should be blast cleaned to remove mill scale before casing. The mesh arrangement and bar sizes of the alphanumeric reference for the standard wrapping fabric mesh D49 is defined in BS 8666: 2005.

Paint systems

Clause 5.5.2.3 of the Code advises that a suitable paint system should be selected using one of the references given or from manufacturer's guidance, however the Responsible Engineer should always evaluate data from proprietary literature with caution.

A possible paint system for severely exposed internal steel is:

Surface Preparation Blast clean to Sa2.5.

Primer coat: 50 microns Epoxy Zinc Phosphate applied in the shop.

Barrier coat: 125 microns Epoxy Micaceous Iron Oxide applied in the shop.

Finish coat: 50 microns Acrylic/Urethane applied on site after erection and touching up. (All paint thicknesses specified are dry film thicknesses)

A possible paint system for accessible moderate to severely exposed external steel is:

Surface Preparation Blast clean to Sa2.5.

Primer coat: 75 microns Zinc Rich Epoxy applied in the shop.

Barrier coat: 100 microns Epoxy Micaceous Iron Oxide applied in the shop.

Finish coat: 50 microns Acrylic/Urethane applied on site after erection and touching up.

Minimum thickness of steel

Clause E5.5.2.4 of the Code gives minimum thicknesses of steel to be used in various situations and is generally self-explanatory. The clause notes that the minimum thicknesses given may not apply to particular proprietary products and requires the Responsible Engineer to provide justification that the corrosion resistance of the product is suitable for its application in this case.

Sacrificial corrosion allowances for steel

Clause 5.5.2.5 of the Code allows use of additional thickness of steel as a sacrificial corrosion allowance where other systems are not practical but exposure conditions are severe. The clause gives general guidance on how to do this but notes that the sacrificial thickness shall be determined from the particular corrosion regime and required life of the structural element under consideration, thus a justification is required for each case and hard and fast general rules cannot be given.

E5.5.3 Corrosion from residual stresses

Clause 5.5.3 of the Code is generally self-explanatory.

E6 DESIGN METHODS AND ANALYSIS

E6.1 METHODS OF ANALYSIS

Practical structures contain frame (P- Δ) and member (P- δ) imperfections shown in Figure 6.1 of the Code. They need to be considered either in the analysis or in the design stage. Different levels of material yielding may be considered in a design as “no-plastic hinge”, “first plastic hinge” and “full plastic analysis”. To ensure strength and stability of a structure, material strength, P- Δ and P- δ effects and structural and member imperfections must be allowed for in either the analysis or the design stage, unless they can be verified to be insignificant by the value λ_{cr} in clause 6.3 of the Code.

Below is a table showing different levels of consideration for various types of analysis. One must be clear that P- Δ - δ effects and their imperfections must be considered in a design, either implicitly using charts or explicitly in an analysis. This is because the effects are present in all practical structures and their ignorance is on the unsafe side. On the other hand, for elastic or plastic analysis, one can consider and continue his analysis after formation of one or more plastic hinges, depending on whether he wants to achieve economical design or be conservative.

Non-linear effects	Simple design		First-order linear		Second-order P- Δ -only elastic		Second-order P- Δ - δ elastic		Advanced Analysis	
	Analysis	Design	Analysis	Design	Analysis	Design	Analysis	Design	Analysis	Design
P- Δ Effect with frame imperfection	X	✓	X	✓	✓	X	✓	X	✓	X
P- δ Effect with member imperfection	X	✓	X	✓	X	✓	✓	X	✓	X
Plastic hinges or Plastic zone	X	X	X	X	X	X	X	X	✓	X

Notes : X indicates the effect is not considered in analysis / design stage
 ✓ Indicates the effect is considered in analysis / design stage

The P- Δ and P- δ effects must be considered in either the design or the analysis stage, but not in both stages nor none, since they are unavoidable natural features of practical structures. For example, the first-order linear analysis ignores P- Δ moment and therefore the moment amplification method is used to consider the P- Δ moment. It also ignores the P- δ effect, so we need to use the Code to account for member buckling.

In many practical applications, the P- Δ and P- δ effects for sway frames can be considered using a design chart, which, however, cannot consider P- Δ moment in connection and restraining beam design. This is a limitation.

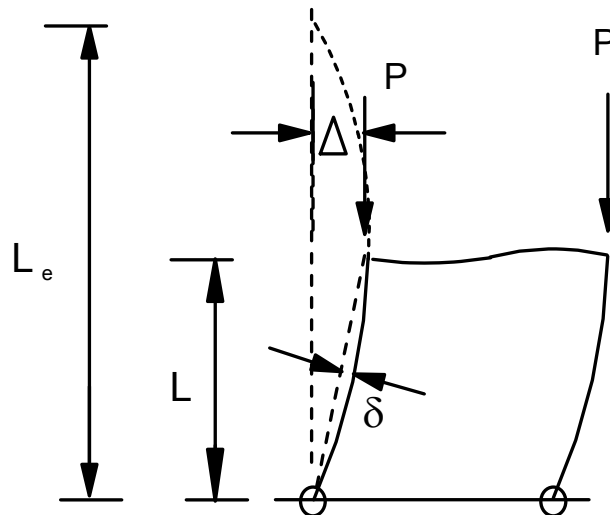


Figure 6.1 - Column effective length, P- Δ and P- δ moments

Clause 5.1.3 in BS 5950 (2000) allows one to use semi-rigid base stiffness as $0.2EI/L$ and EI/L for rigid connection. In Hong Kong, we sometimes just use infinity for rigid base connection. In the Code, we propose to follow the UK practice. This is more realistic since this nominally pinned connection (i.e. without purposely detailed pinned connection) will have some rotational stiffness. The ignorance of connection stiffness implies under-designing of connections and over-designing of columns because bending moment is not transferred from column to base connection. Although the consideration is more accurate, it makes the design and analysis more complicated.

Linear vs. non-linear analysis

Linear analysis refers to any analysis assuming a linear relationship between force, stress, strain and displacement. Non-linear analysis refers to any analysis not fulfilling this linearity requirement and in practice it includes P- Δ -only analysis, P- Δ - δ analysis with and without imperfections, advanced analysis allowing for plastic hinges and elasto-plastic buckling. These analysis and design methods are discussed in this section.

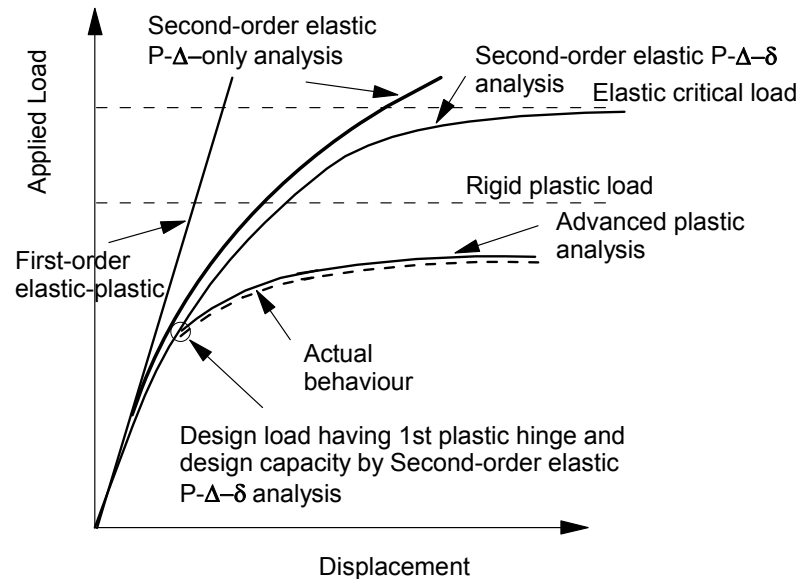
Non-linear analysis has been mistakenly considered by many as a tool to reduce structural weight. Non-linear analysis is only a tool of giving us a reliable prediction of design load which causes a structure to yield, to form the first plastic hinge or to attain its ultimate load capacity. As we all know, structural design is a probabilistic exercise giving us a confidence limit of a designed structure with failure probability in the order of 10^{-6} . Thus, using a less rational design method will give us inconsistent factor of safety for members in a structure that the structure can be heavier but less safe, because of inappropriate allocation of material to critical and redundant members. For example, if we assign effective length factor as 1 for all members whilst in reality, some redundant members have actual effective length factors less than 1 whereas key members have effective length factors larger than 1, the structure may collapse before reaching the design ultimate load due to buckling of key members.

One may ask why many structures do not collapse when using simplified and incorrect assumptions. In many incidents, a structure still stands even it is under-designed. The reason is that we may not encounter loads in excess of the ultimate loads which occur, say, once in every 50 years. It is interesting to note that temporary structures have a higher collapse rate than permanent structures since the actual loads of temporary structures is due to concrete weight etc. which are closer to the design loads. The philosophy for using non-linear analysis is that it is more reliable than the linear analysis to predict reliably the load capacity of a structure that it is less likely to collapse when the ultimate load really comes. Design is a probabilistic based exercise that we intend to have a consistent factor of all members (in some cases, key members should have a larger factor of safety), such that the probability of failure or collapse is under control and acceptable.

For the respectful and responsible profession of structural engineers, a coarse method of design should not be accepted for prestigious and important structures for the sake of safeguarding human lives.

The theory of P- Δ and P- δ analysis is well established but it is rather new in most national design codes. P- Δ effect is for frame sway buckling and P- δ effect is for member buckling. Using these terms will make the concept of frame and member buckling much clearer and makes the Code more advanced. The method of verification can be varied, such as keeping the structure elastic and ignoring any plastic action, making the elastic critical load factor larger than 10, so that the effects of P- Δ moment can be ignored.

Relationship between various methods of analysis can be seen in the Figure below.



General types of analysis

First-plastic hinge and first yield loads are the two most common methods for analysis and design. Other more advanced methods are expected to be used in future for more advanced and quality design.

Connection stiffness can be used in computer analysis to allow for semi-rigid support stiffness when using either the effective length method or P- Δ analysis.

E6.2 MODELS AND ASSUMPTIONS

Collapse of many steel structures is due to weak connection stiffness overlooking the P- Δ moment. The effective length method should not be used for the design of connections and restraining beams. Engineers should be aware of this limitation. Second paragraph on p.6-180 of Load Resistance Factor Design (LRFD) (1994) states “Connection shall also be designed to resist magnified end moments” (due to P- Δ effect).

Elastic critical load factor λ_{cr} is an indicator for sensitivity of a frame against sway buckling and it can be obtained by the eigenvalue analysis or by the deflection method below. λ_{cr} of a frame is defined as the ratio by which the factored loads would have to be increased to cause elastic instability. Member imperfection is not required for frame classification.

In the determination of elastic critical load factor by eigenvalue analysis, it is important to study the form of each buckling mode to see if it is a frame mode or a local column mode. The elastic critical load factor should be taken as the factor corresponding to the first sway buckling mode rather than the local buckling mode.

E6.3 FRAME CLASSIFICATION

λ_{cr} is a factor multiplied to the design load that will cause the structure to buckle elastically and it is used as an indicator for stability condition of a structure. Imperfection and material yielding are not considered and it is only an upper bound solution used for measuring the buckling stage of a structure. When λ_{cr} is larger than 10, the frame buckling or P- Δ effect can be ignored but member buckling is required for consideration. The structure under this condition is called non-sway frame. When λ_{cr} lies between 5 and 10, the P- Δ effect must be considered and the structure is classified as a sway frame. When λ_{cr} less than 5, the second-order effect must be considered via the full second-order analysis and hand method is no longer allowed since the error of assuming effective length etc. is too enormous.

Sway-sensitive and non-sway frames classification has been used in BS 5950 (2000). A slightly larger limit of 5 instead of 4 in BS 5950 (2000) for mandatory use of second-order analysis is adopted from AS 4100 (1998). As the pace of local construction is much faster than in UK, the value of "5" is used for the sake of safety. The value "15" for plastic design is adopted from Eurocode 3. Further, the multiples of 5, 10 and 15 for the three cases are numerically friendly. These numbers are similar to those in some codes such as AS 4100.

E6.3.2 Elastic critical load factor

A structure may need to be classified for sway instability. The elastic critical load factor λ_{cr} of a structure is determined and used to assess the stability condition of a sway structure. λ_{cr} is defined as the ratio of the buckling resistance of a perfect and elastic structure, P_{cr} divided by applied load. For example, a pin-pin column of length 3m and

second moment of area of 17,500 cm⁴, the elastic buckling load is $\frac{\pi^2 EI}{L^2} = 38382 \text{ kN}$. If

the factored design load is 10,000 kN, the elastic critical load factor λ_{cr} is then $38382/10,000 = 3.84$.

Figure 6.2 are used to show the sway and non-sway buckling modes.

Elastic critical load factor λ_{cr} can be obtained by computer analysis or by hand method.

Elastic critical load factor λ_{cr} is defined as the factor by which the design load must be multiplied in order to cause the structure to buckle elastically. λ_{cr} is used as an indicator for sway classification of frames and can be determined by following computer analysis or deflection methods.

The elastic critical load factor λ_{cr} may generally be determined from a standard computer procedure as,

$$\text{Det}(K_L + \lambda_{cr} K_G) = 0$$

where Det is the determinant of the sum of the linear (K_L) and geometric (K_G) stiffness matrices. λ_{cr} is the elastic critical load factor.

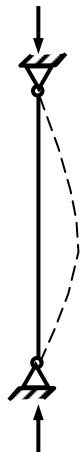
Deflection method

For the pin-pin column below, the end has no deflection and therefore the method cannot be used. As this column bends by itself with only P- δ effect but no P- Δ effect, this indicates the P- δ effect cannot be modeled by the sway displacement method.

For the column with both ends fixed but free to translate at one end, see below, the deflection method calculates the elastic critical load for the same column above as

$$\lambda_{cr} = \frac{F_N}{F_V} \frac{h}{\delta_N} = \frac{F_N L}{F_V \times \frac{F_N L^3}{12EI}} = 4.7, \text{ which is 22\% above the exact value.}$$

This case involves member end translation and the method can predict an approximate value.



Pinned-Pinned column



Fixed Free-to-translate column

Note that we cannot use directly the Euler's load as the design resistance of a column against the design load since material yielding, large deflection effect and initial imperfection effects have not been considered.

Equation 6.1 refers from Eurocode 3, which is the same as BS5950(2000) if we use notional force of 0.5% of the vertical force. Elastic critical load cannot be used directly as it ignores large deflection effect and material yielding, but it is an important yardstick to measure the stiffness of a structural system. The computational method can calculate both the sway and the non-sway cases, and the hand Deflection Method can be used only for the sway case. Member buckling due to $P-\delta$ effect cannot be determined by the deflection method in Equation 6.10.

Frame classification is for frame sway buckling check under clause 6.3.2. Member buckling check needs to be done by analysis allowing for $P-\delta$ effect.

E6.4 IMPERFECTIONS

E6.4.1 General

We need to consider the followings for completion of the requirements for a second-order $P-\Delta-\delta$ analysis. $P-\Delta$ is for global frame consideration and $P-\delta$ is for member curvature.

- P- Δ effect due to sway of a frame
- P- δ effect due to member bowing
- Frame imperfections for Δ effect
- Member imperfections for δ effect

These effects are existent in practical structures, analysis and design are required to simulate these effects, or else our structures will be under-designed.

E6.4.2 Frame imperfections

Practical structures contain random imperfections but we assume a conservative set of imperfections for safe design. The values of imperfections in this clause are consistent with the 0.5% notional force (i.e. $1/200=0.5\%$). The imposition of notional force is to account for the adverse effect of out-of-plumbness

E6.4.3 Member imperfections

BS 5950 (2000) gives only the Perry constant which can be indirectly used to calculate the member initial imperfections. The Robertson constants are used for the generation of

buckling tables in section 8. Eurocode 3 (2003) gives both the values of Perry constant and member initial imperfections, which are adopted above. Note: Euro Code 3 (2003) was the first draft version circulated in 2003, and the title of the document should be EN 1993-1-1:2005. From simple calculation, a Perry constant can be related to initial imperfection as follows.

$$\frac{\delta_0}{L} = 0.001 \times \frac{a}{y/r}$$

where δ_0 is the initial imperfection of the element,
 a is the Perry constant,
 L is the element length,
 y is the distance from the centroid to the extreme fibre of the section
 r is the radius of gyration

In most cases, the direction of imperfection shall be on the unfavourable side which is normally taken as the elastic critical mode. For a second order analysis taking account of lateral torsional buckling of a member in bending, the imperfections in both in-plane and out-plane should be considered.

Frames can be slightly inclined by the out-of-plumbness. This frame imperfection is simulated by notional force or geometrical imperfection in frame nodal coordinates.

Structural members are slightly curved and this is member imperfection. When using either the effective length or second-order analysis, imperfection is required for consideration since all practical structures contain imperfections which can be bow geometry and material residual stress. Because of this, several buckling curves are used in different codes. 1, 3, 4 and 5 number of buckling curves are used respectively in LRFD (1994), AS 4100 (1998), BS 5950 (2000) and Eurocode 3 (2003). We propose 5 buckling curves to be used in the Code with the new a_0 curve included for annealed sections. Curved element should be used for modeling of bow members in practice.

Both in-plane and out-of-plane initial bow imperfections should be considered in the 2nd order analysis, and these imperfections as given in Table 6.1 of the Code are applicable from a conservative design point of view. Computational softwares performing the second-order analyses have already accounted for such imperfections in both the principal axes.

Horizontal notional force

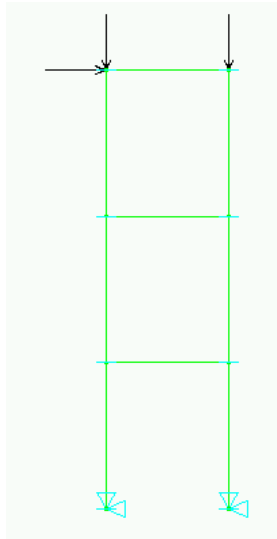
As can be seen in Figure 6.2 of the Code, notional force is aimed at producing a result similar to inclined structural geometry, which is unavoidable in practical structures. Out-of-plumbness is unavoidable in real structures.

E6.4.4 Elastic critical mode

As an alternative to horizontal notional force in clause 6.4.4 of the Code, the elastic critical mode can be used for global imperfections. The amplitude can be taken as building height/200.

The same imperfection amplitude as in Eurocode 3 (2003) of $h/200$, which is the same as 0.5% notional force, was adopted.

In some codes like BS 5950 (2000), notional forces are required to be applied simultaneously with wind force. Others consider that only one type of horizontal force is needed to be applied at a particular instance. As imperfect geometry can exist under any load cases, the more conservative consideration by BS 5950 (2000) is adopted here.



This frame under a pair of increasing point loads at top was studied using 0.5% notional force and out-of-plumbness. The most critical member ($A=1.37\text{cm}^2$, $I=175,000\text{cm}^4$, $Z=1,312\text{cm}^3$) is at lowest right hand column and the forces and moments computed in these two cases are the same as $N=2,573\text{kN}$ and $M=123\text{kN-m}$.

E6.5 SIMPLE DESIGN

Simple design is widely used in practice. Rigid structures include lift core, shear and core walls, braced frames, etc.

E6.6 FIRST-ORDER LINEAR ELASTIC ANALYSIS (FIRST-ORDER INDIRECT ANALYSIS)

A new term is introduced for the direct and indirect analyses. Direct analysis considers the $P-\delta$ and $P-\Delta$ effects with their imperfections, so the section capacity check is sufficient for safety. For indirect analysis, one or more of the non-linear effects is/are ignored (e.g. the use of straight element which ignores the $P-\delta$ effect) such that member design is still required. For instance, the $P-\Delta$ only analysis considers solely the sway moment effect, therefore moment amplification is not required; however, the member check is still required for structural adequacy. Furthermore, some codes like the LFRD (2010) requires the consideration of change of stiffness, the stiffness of which is reduced for members in compression, and this effect should also be allowed for in the Code.

E6.6.1 General

First-order elastic analysis ignores all second-order and non-linear effects and the equilibrium condition is based on the initial geometry. $P-\Delta$ and $P-\delta$ effects are not considered in the analysis but should be considered in the member design by the moment amplification or by the effective length method. This method can be used in cases where frames for which the elastic critical load factor λ_{cr} determined in section 6.7 of the Code is larger than 5.

The term “indirect analysis” is used to indicate that the method requires member design and assumption of effective length. When the axial force is very large, effective length method is more critical. When the frame sway is very large, moment amplification is more critical. Hence, both the effective length method and the moment amplification method should be used in member design. It is not safe to carry out checks only on elastic critical load factor by deflection method, which can only account for sway frame.

The second-order effect includes both global frame imperfection and member bow imperfection. Therefore, missing of them would disqualify the second-order direct analysis as the second-order indirect analysis.

E6.6.2 Moment amplification for sway frames

The moment due to horizontal load is amplified by the following factor.

The P- δ effect in the column should be considered by assuming the effective length as the column length between supports when using the effective length method, unless the effective length factor (L_e/L) can be determined by a more rigorous analysis to be less than unity. The bending moment due to horizontal loading should be multiplied by the factor in clause 6.6.2 of the Code.

To be consistent with the Code of Practice for the Structural Use of Concrete which ignores the P- Δ effect when it is less than 10%, the Code also ignores the effect when λ_{cr} is larger than 10, indicating the amplification factor is $\frac{\lambda_{cr}}{\lambda_{cr} - 1} = \frac{10}{9} = 1.11$ or the amplified factor of 11% \cong 10% used in the Code of Practice for the Structural Use of Concrete.

E6.6.3 Effective length for sway and non-sway frames

For simple structures where buckling is not critical, one may use the effective length method with careful selection of effective length factor (L_e/L) under this clause.

This clause is adopted from BS 5950 (2000). First-order linear elastic analysis is the conventional linear analysis ignoring all non-linear effects. A first-order linear analysis makes no difference to stress or load capacity computed from a long or a short column. To account for the buckling column effect under compression, design charts for column strength are used to reduce the load capacity of a long column with slenderness ratio larger than 15.

Two methods can be used for determination of effective length, which are as follows.

1. Moment amplification is a method to account for sway and change of geometry or the P- Δ effect.
2. Effective length method using charts from BS 5950 (2000). The Responsible Engineer should note that at $\lambda_{cr}=10$ and with both k_1 and k_2 values at 0.7, Figure 6.4a gives an effective length factor of 1.93, whilst Figure 6.4b gives an effective length factor of 0.78.

The method using directly the elastic critical load method in E.6 of Annex "E", p.187, BS5950 (2000) to find L_E ($L_E = \sqrt{\frac{\pi^2 EI}{\lambda_{cr} F_c}}$; where F_c is the axial force in the member under consideration) is not recommended for uses in the Code because of the following two drawbacks.

- The elastic critical load is only for the most critical column and other critical columns cannot be designed by the load factor.
- A small axial force F_c in the above equation leads to a very large effective length and thus large member size, which is not reasonable.

E6.7 SECOND-ORDER P- Δ -ONLY ELASTIC ANALYSIS (SECOND-ORDER INDIRECT ANALYSIS)

The term indirect analysis refers to the method that requires member design and section capacity check, which is not sufficient since some non-linear effects have been ignored.

E6.7.1 General

When a structure deflects laterally, deflections at various levels are generated which combine with the vertical forces to induce an additional P- Δ bending moment. This effect cannot be accommodated in the first-order elastic analysis and it is termed as the P- Δ effect.

An analysis method that does not consider all the factors required for a second-order direct analysis is termed as second-order indirect analysis. These factors include P- δ effect and local and global imperfections.

E6.7.2 Method of analysis

The second order P- Δ -only elastic analysis considers the P- Δ effect by referencing the equilibrium condition to the deformed structural geometry. This is normally carried out by a computational analysis based on an iterative procedure of adding deflections to original structural geometry. Frame imperfection in clause 6.4.3 should be allowed for in P- Δ -only analysis.

A frame may buckle in-plane or out-of-plane. The more critical one should be adopted for design.

E6.7.3 Applications and Limitations

Second-order P- Δ only analysis is a second-order analysis method to account for displacement or sway of joints only, but not for member curvature. Thus, it can only be used for calculating sway-induced moment, but not moment due to member bow, which is considered by the P- δ effect. So, P- Δ analysis is only a method to calculate the sway moment and cannot be used for section capacity check for structural adequacy. The effects of imperfection, change of stiffness under loads and P- δ effect have not been considered in this method.

E6.8 SECOND-ORDER P- Δ - δ ELASTIC ANALYSIS (SECOND-ORDER DIRECT ANALYSIS)

The term direct analysis referred herein considers all non-linear effects such that the section capacity check is sufficient for structural adequacy.

E6.8.1 General

Both the P- Δ and P- δ effects are accounted for in this method and the buckling strength of a structure should be checked by section capacity strength of its members.

When a structure deflects with its members deformed, both the P- Δ and the P- δ effect due to member bowing deflection under the influence of global and local imperfections are induced. Analysis allowing for these effects is termed the second-order direct P- Δ - δ analysis.

E6.8.2 Method of analysis

Second-order P- Δ - δ elastic analysis normally uses the large deflection and small strain structural analysis theory. The method is typically based on an incremental-iterative numerical procedure by applying the loads in increment and iterates for equilibrium in

each load increment. It should capture the behaviour of the structure in elastic state until the first plastic hinge is formed or until the load causing first yield is reached.

The second-order P-Δ-δ analysis can be based on a curved finite element capable of simulating the changing curvature of a member and calculating the most critical or highest stressed section along a member. Several straight elements are recently noted to be impractical to model a member for simulation of member curvature before and after being loaded because of uncertain direction of imperfections.

E6.8.3 Applications and Limitations

The method is limited to the capacity at first plastic hinge. Member local buckling, lateral-torsional and torsional buckling checks are carried out separately.

Second-order P-Δ-δ elastic analysis calculates the sway-induced P-Δ moment, as well as the P-δ moment due to member bowing. The method does not require the use of an effective length for the structural members and section capacity check is sufficient for member strength check.

The connections and connecting members should be designed using this refined bending moment allowing for first-order and second-order P-Δ-δ moments. The member buckling strength may be checked using the following formula.

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} \frac{F_c}{A_g p_y} + \frac{\overline{M}_x + F_c(\Delta_x + \delta_x)}{M_{cx}} + \frac{\overline{M}_y + F_c(\Delta_y + \delta_y)}{M_{cy}} \leq 1$$

$$\frac{F_c}{A_g p_y} + \frac{m_{LT}[\overline{M}_x + F_c(\Delta_x + \delta_x)]}{M_b} + \frac{m_y[\overline{M}_y + F_c(\Delta_y + \delta_y)]}{M_{cy}} \leq 1$$

where

F	is the axial force ;
A _g	is the gross sectional area ;
M _x and M _y	are the bending moments from the first-order linear analysis and about principal x- and y- axes ;
Z _x and Z _y	are elastic moduli (or plastic moduli for classes 1 and 2 sections, but not larger than 1.2 of their respective elastic values) ;
p _y	is the design strength ;
Δ	is the second-order moment due to change in structural geometry ;
δ	is the second-order moment due to change in member curvature.

Equation 6.13 is a more rigorous and economical checking equation and it is adopted from the BS5950(2000). The concept is to allow central core to take axial force and shear and the remaining area to take moment.

Equation 6.14 is an additional check for lateral-torsional buckling which is based on simple idealized beam boundary condition and not relying on stiffness or sway or non-sway buckling mode of a frame.

Second-order P-Δ-δ analysis accounts for both sway-induced P-Δ moment and member bow P-δ moment. Thus, it can be used for integrated analysis and design which represents analysis encompassing frame and member design against stability and buckling. To qualify for a P-Δ analysis, the software must be able to simulate buckling of a sway frame for P-Δ checking and buckling of a pin-pin column for P-δ check. The software must produce the correct result for simple structures including braced and unbraced portal frame, in which the former is controlled by P-δ effect and the latter by P-Δ effect. Benchmark examples for checking are available in literature. This method is similar to the Advanced Analysis in that both allow the section capacity check without assumption of effective length. They are different in that the Advanced Analysis allows the formation of more than one plastic hinges.

For beam with no buckling, there is no need to check against equation 6.13.

For beam subjected to buckling and $m_{LT} = 1$, check against equation 6.13 will suffice as M_b is always less than M_{cx} .

For Beam subjected to buckling and $m_{LT} < 1$, check against both equations 6.12 and 6.13 are required as it is not sure which equation would be more critical.

E6.9 ADVANCED ANALYSIS

This is a direct analysis that member design is not required. The method is a P- Δ - δ direct second-order plastic analysis allowing for imperfections and one or more than one plastic hinges and it is based on an incremental-iterative numerical procedure by applying the loads in increment and iterating for equilibrium in each load increment. It should capture the behaviour of a structure until the plastic buckling collapse load is reached.

E6.9.3 Applications and Limitations

The method directly locates the maximum limit load of a structure allowing for formation of plastic hinges and under different load cases.

Advanced analysis checks the equilibrium condition by consideration of the sway-induced P- Δ moment, the P- δ moment along a member due to member bowing and material yielding by the plastic hinge or plastic zone method. The effects arising from residual stresses, erections and structure strength, connection stiffness and ductility should be considered in the analysis and only the cross-sectional strength of every member needs to be checked for the formation of plastic hinge or yielding. The resistance is determined as the maximum load that the structure can withstand.

Local member buckling effects due to torsional, flexural-torsional, and local plate buckling (see section 7) should be checked separately in member design, unless their effects have been accounted for in the analysis.

“Advanced analysis” is first used in AS 4100 (1990) and then in AS 4100 (1998). Similar analysis types are included in various codes like the LRFD (2010). In essence, the method can be defined as any analysis method which captures the behaviour of a structure such that only the section capacity check is necessary and individual member design can be skipped. Account must be taken of the effects from relevant material properties, residual stresses, geometrical imperfections, second-order effects, erection procedure and interaction with the foundations. With these considerations, it will be sufficient for the method to check the satisfaction of section capacity requirement for each member and strength and rotation capacity for each connection in the frame.

The advantages and disadvantages of the method are as follows.

Advantages

1. Economical design since the designed structure will be lighter. It can be viewed as a material optimisation process by re-arranging the material correctly. The effective length is generally over-estimated about 80% of members in Hong Kong. Saving of weight is in the order of 20% against properly designed steel structures using the first-order linear analysis.
2. The method can consider the P- Δ moment in connection and in member design, which is mandatory in LRFD code. Effective length method does not provide information on this aspect.

When using the advanced plastic analysis, the design should be safer. Unlike the first-order linear analysis used in conjunction with the effective length method, some members will not be over-designed whilst others are under-designed. The under-designed members may lead to collapse. In some cases, the most critical member is not identified correctly because of the mistake made in effective length assumption.

3. Efficient design output, design is completed simultaneously with analysis. No need to waste time on checking the code for determination of elastic critical load factor λ_{cr} for frame classification.
4. Accurate in output since the determination of buckling effect is rigorous, but not by manual judgement which varies from one engineer to another.
5. Change of stiffness or stiffening and weakening effects of tension and compression members are considered.
6. Wider application, it accounts for complex cases, such as change of stiffness in the presence of axial force, sloping bracing members, snap-through instability, pre-tensioned structures etc.
7. More reliability, e.g. effect of adding bracing members can be seen directly.
8. Interactive behaviour can be considered. A system design instead of a member design approach is used.
9. Lesser chance of human error due to the use of design codes.
10. Capable of dealing with frames with different elastic critical load factors, even when it is less than 4 or 5.

Disadvantages

1. Super-position cannot be applied. Bending moment envelop cannot be used.
2. It is a new method which requires us to learn and be familiar with. However, with the changing technology and globalisation, it appears that engineers cannot avoid using better and new methods in order to remain competitive.

E6.10 BRACING MEMBERS

This clause makes reference from the Euro-code 3. When more bracing members are present, the imperfection can be reduced since the chance of having all bracings to have imperfections in the same direction is very small.

E6.11 CONNECTION CLASSIFICATION

Connection stiffness is doubted by many as an uncertainty and this problem exists in both first-order linear and second-order non-linear analyses. Tests on connections may be required and the information can then be input in a computer analysis. When pinned connection is assumed, we need to ensure sufficient ductility to allow for rotation. For a fixed connection, we need to ensure sufficient moment resistance in the connection. For semi-rigid connections, both these requirements of strength and ductility are required, but to a less stringent requirement.

The concept of a new concept of semi-rigid connection is well established but its applications are limited. Most engineers assume either rigid or pin joints, which are indeed the extreme cases for semi-rigid connections. The reason for this unpopular use is due to lack of experimental data in measuring connection stiffness. This clause allows one to design with allowance for semi-rigid connections.

E6.12 EXAMPLES

In the following four examples, it can be seen that the error for a linear analysis assuming an effective length equal to the member length could lead to a considerable error. The first example shows the importance of P- δ effect which is commonly ignored by structural analysis software and one should use this benchmark example to test the validity of a program for direct second-order analysis. Other examples show the presence of both P- δ and P- Δ effects.

E6.12.1 Simple benchmark example for testing of software

- A strut under axial force

The column of CHS 88.9x3.2, grade S275 steel and length 5m has a boundary condition as one end pin and one end fixed. Determine the axial load resistance and buckling load of the column by second order analysis. Do not assume effective length for the column as it is unknown for most compression members in real frames.

$$\text{Area} = 8.6200 \times 10^{-4} \text{ m}^2$$

$$\text{Second moment of area} = I = 7.9200 \times 10^{-7} \text{ m}^4$$

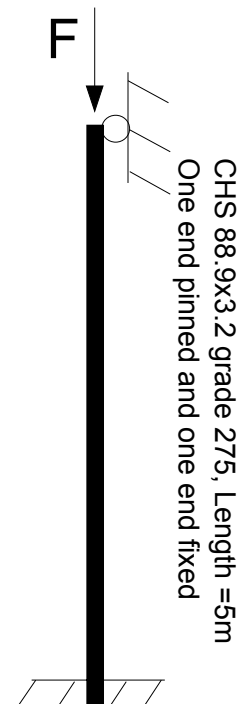
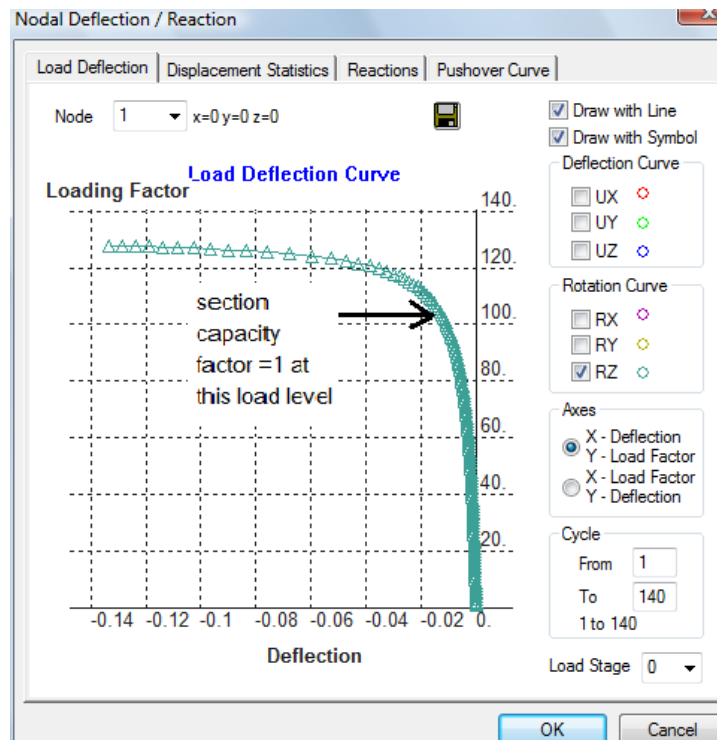
$$\text{Elastic modulus} = Z = 1.7800 \times 10^{-5} \text{ m}^3$$

$$\text{Plastic modulus} = S = 2.1360 \times 10^{-5} \text{ m}^3$$

$$\text{Design strength or yield stress} = 275 \text{ MPa}$$

Software unable to do the first example should never be used to do a second-order analysis.

If the software cannot tell the design resistance or buckling load when the load vs. deflection curve is flat for the above column (see below), it cannot tell the design and buckling resistance of a frame.



Dividing a member to two equal-length elements is still unable to place the imperfection to the location with maximum curvature which is not at the mid-length.

Input of imperfections is too inconvenient when we use 2 elements per member since we need to follow the buckling mode shape otherwise the imperfection does good instead of harm to the design resistance of a frame which is NOT what we want.

This results in the answer being over-estimated and the member is over-designed.

Method	Buckling resistance P_c (kN)	Error
1 st order linear with $L_e/L=0.7$	108.9	N.A.
2 nd order with imperfection $L/500$ to code	102.2	-6%
2 nd order with imperfection $L/1000$ (not to code)	113.4	+4%
No imperfection	$p_y A = 234$	+118%

Using the effective length factor of $L_e/L = 0.7$, the 1st order linear analysis from the Code is 108.9 kN

If we assume the imperfection as $L/500$ which is recommended in the Code, the computed resistance is 102.2 kN (-6%)

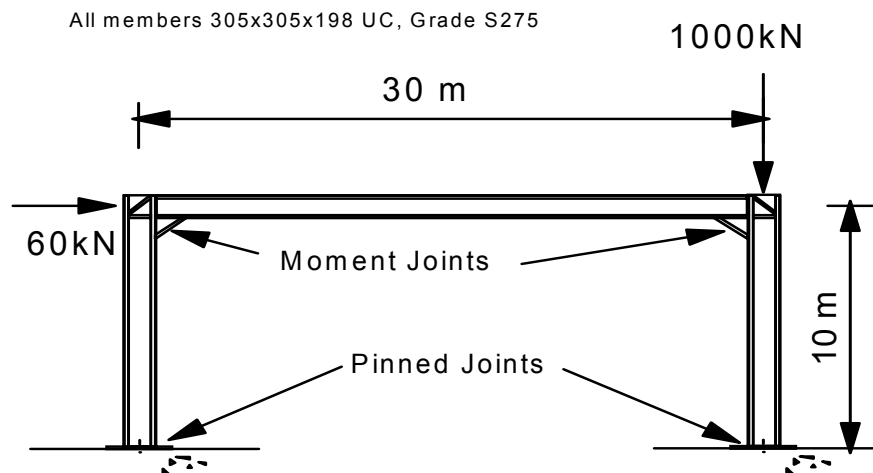
If we assume the imperfection as $L/1000$ (smaller than the Code), the computed resistance is 113.4 kN (+4%). This shows that imperfection is important in determining the resistance. If one ignores imperfection, the resistance becomes $p_y A = 237$ kN (+118% !) since no load versus deflection path could be plotted for a perfectly straight column which does not know where to deflect when under load.

E6.12.2 Design of a simple portal by amplification method

The portal frame shown in the figure below is analysed and compared with the Code used in association with the hand method of analysis. It is under a lateral load and a vertical force at the top of one of its column. The section used for both columns and beam is 356x368x153 H-section and grade S275 steel.

Properties of 356x368x153 H-section are as follows.

$$A = 195 \text{ cm}^2, I = 48,500 \text{ cm}^4, r = 15.8 \text{ cm}, Z = 2680 \text{ cm}^3$$



Hand Moment Amplification Method

Vertical Reaction on the left = $R_L = 60 \times 10,000 / 30,000 = 20 \text{ kN}$

Vertical Reaction on the right = $R_R = 1000 + 20 = 1,020 \text{ kN}$

Horizontal reaction of the left = $H_L = H_R = 60/2 = 30 \text{ kN}$

$M_A = M_D = 0$

$M_b = M_c = 30 \times 10 = 300 \text{ kN-m}$

Buckling analysis :-

$k_a = 1.0, k_B = (1/10)/(1/10+1.5 \times 1/30) = 0.67$

$(L_e/L)_{AB} = 2.9$ from HKSC

Similarly, $L_e/L_{CD} = 2.9$

$N_{of} = 2 \times \pi^2 EI / (2.9L)^2 = 2 \times \pi^2 \times 200,000 \times 485 \times 10^6 / (2.9 \times 10,000)^2 = 2277 \text{ kN}$

$\lambda = 2277 / (-20 + 1020) = 2.27$

Using Non-linear Analysis and Design Computer Software, λ_{cr} is 2.25

Amplified Moment = $M^* = M \lambda / (\lambda - 1) = 300 \times 2.35 / 1.35 = 522.2 \text{ kN-m}$

λ from sway index method is 2.5

For column of Euler buckling length of 1.0 $L = 10 \text{ m}$

Column slenderness = $10,000 / 158 = 63.3$

From Table 8 of the Code, $p_c = 214.4 \text{ N/mm}^2$

Axial Force = $P_c = A p_c = 19500 \times 214.4 = 4180.8 \text{ kN}$

Combined Load Check:

$F/P_c + M/M_r = 1,000 / 4,180.8 + 522.2 / 275 / 2680 / 10^{-3} = 0.948 < 1.0, \text{ O.K.}$

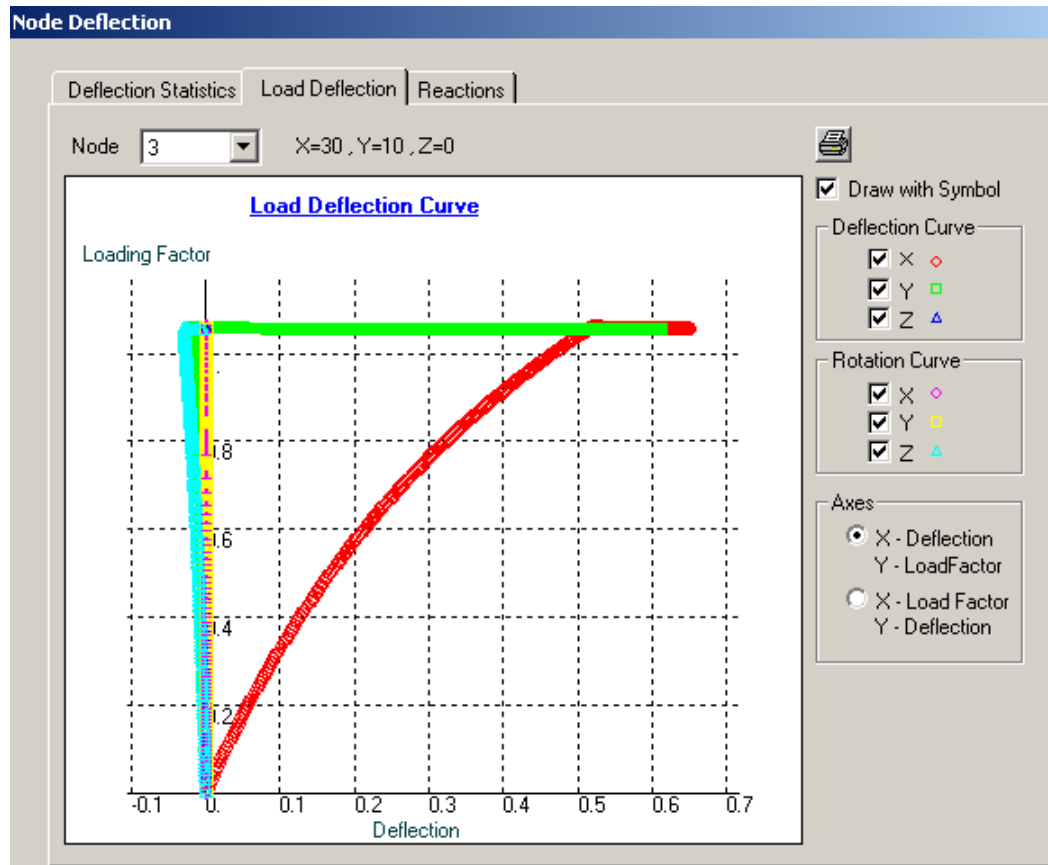
Second-order $P-\Delta-\delta$ analysis NOT allowing for moment re-distribution after yield

The section capacity factor at design load is $0.93 < 1.0, \text{ O.K.}$

Or the collapse load factor is 1.06. (i.e. combined load factor at design load = 0.94)

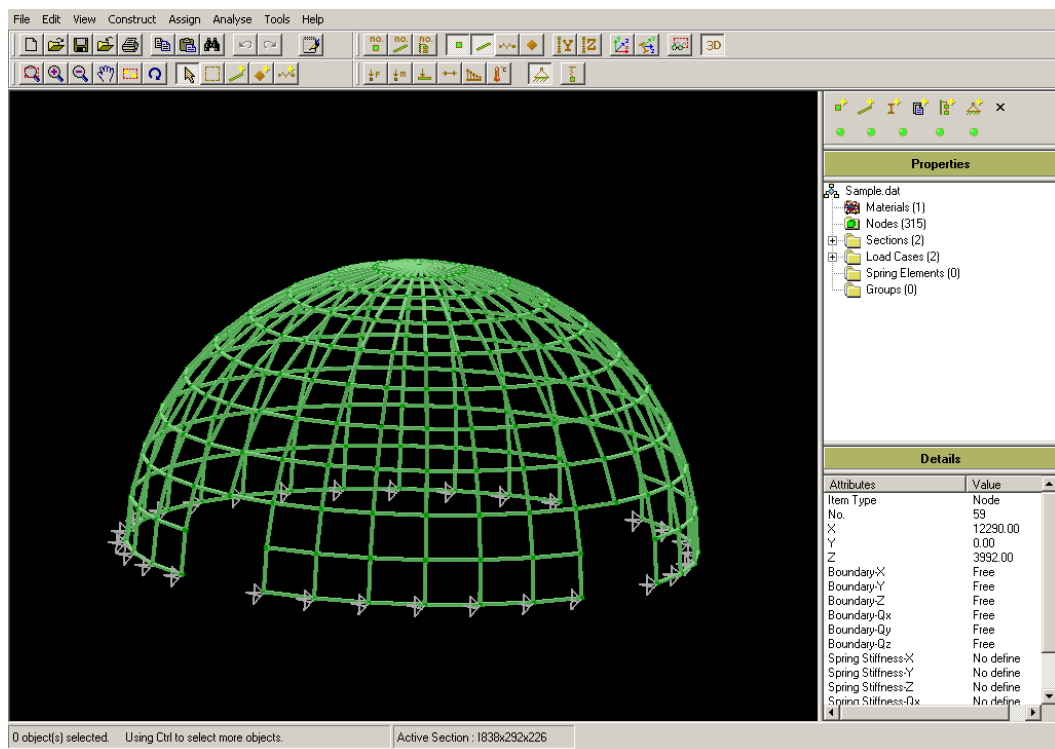
Elasto-plastic P- Δ - δ analysis

As seen below, the maximum load which can be sustained is 1.1. The small difference between the elastic and elasto-plastic analyses is small since the structure has little strength reserve.



E6.12.3 Design and analysis of a 24.6 m span dome

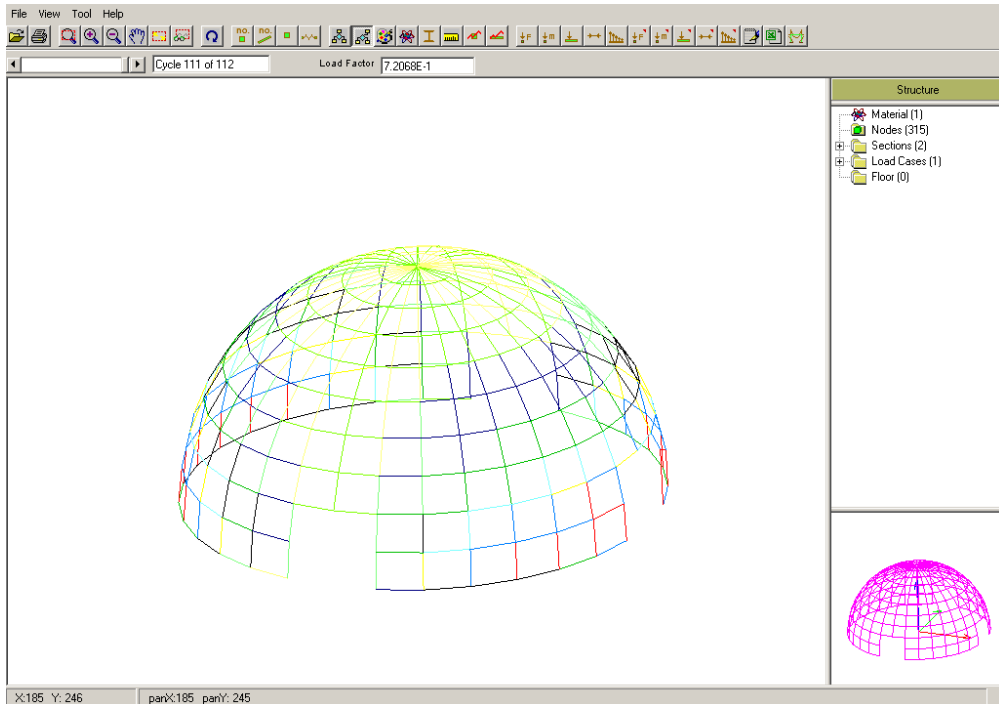
The dome of span 24.6m shown below is under member load due to wind perpendicularly to the member. All members are of size 219x16 CHS of steel grade S355.



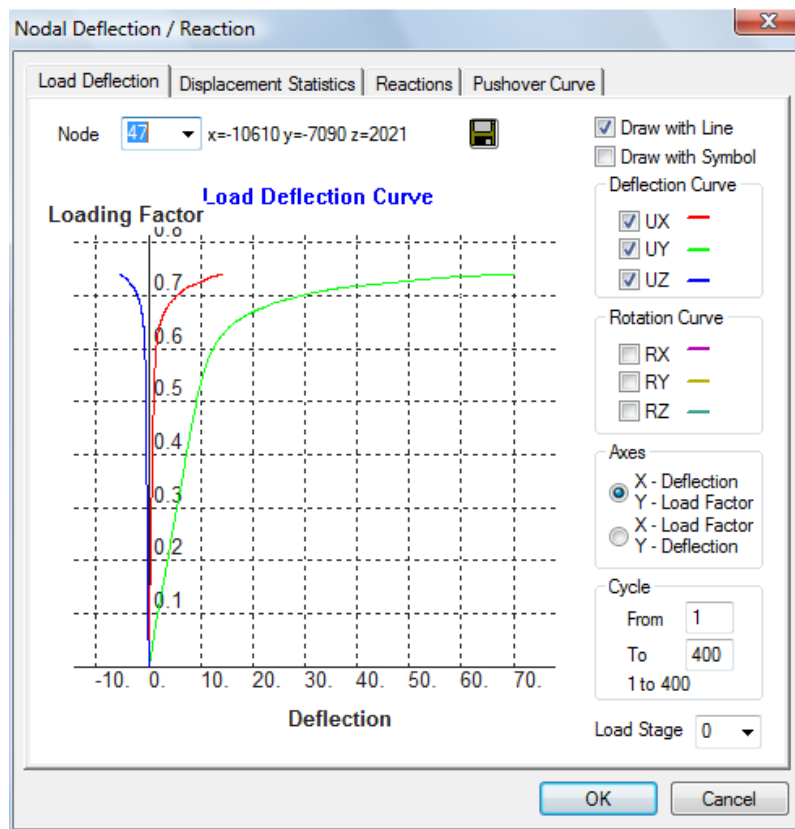
When using a linear analysis, the effective length cannot be determined by a rational and simple method. Assuming an effective length factor equal to 1 (i.e. assuming the effective length is the same as the member length) and using the elastic modulus, the resistance load factor is 0.52 of the design load when the first member fails in the check.

In the example, the load level satisfying the Equation for the formation of plastic moment allowing for axial force effect is 0.53 and the collapse load using the advanced analysis is 0.72. The similarity between the second-order direct analysis (first-plastic hinge design) and advanced analysis (plastic design) is significant because the assumption that the failure of a redundant member as the failure of the complete structure is very uneconomical and the elasto-plastic method determines the re-distribution of moment after the failure of the first few member. The figure below shows the collapse shape and load-deflection curve of the structure.

When using the elasto-plastic design method, ductility of steel material is required to be checked. This includes minimum elongation of 15% and plastic (type 1) sections. These checkings are not needed in second-order direct $P-\Delta-\delta$ analysis not allowing for moment re-distribution with design capacity being set equal to the load causing the first member to fail to section capacity check.



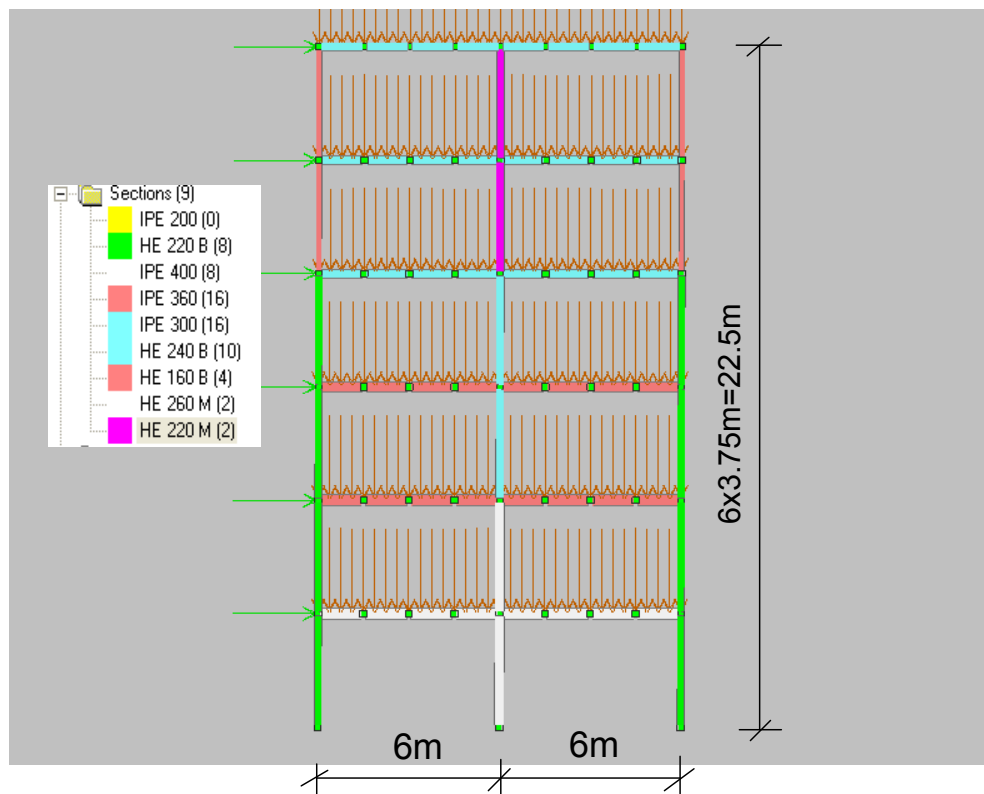
Members failed in section capacity check to equation 6.12 in the Code are indicated in red (Load factor making the first member failed in section capacity check is 0.62)



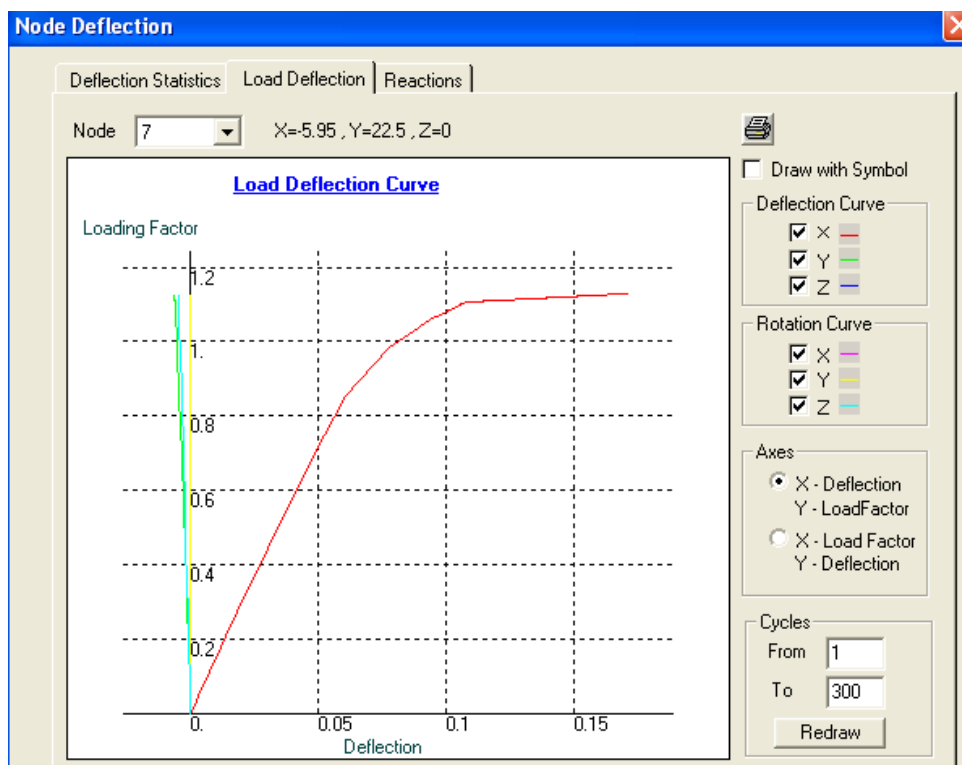
Load vs. deflection curve of the dome with ultimate load factor as 0.72 of design load

E6.12.4 Advanced analysis of 6-storey frame

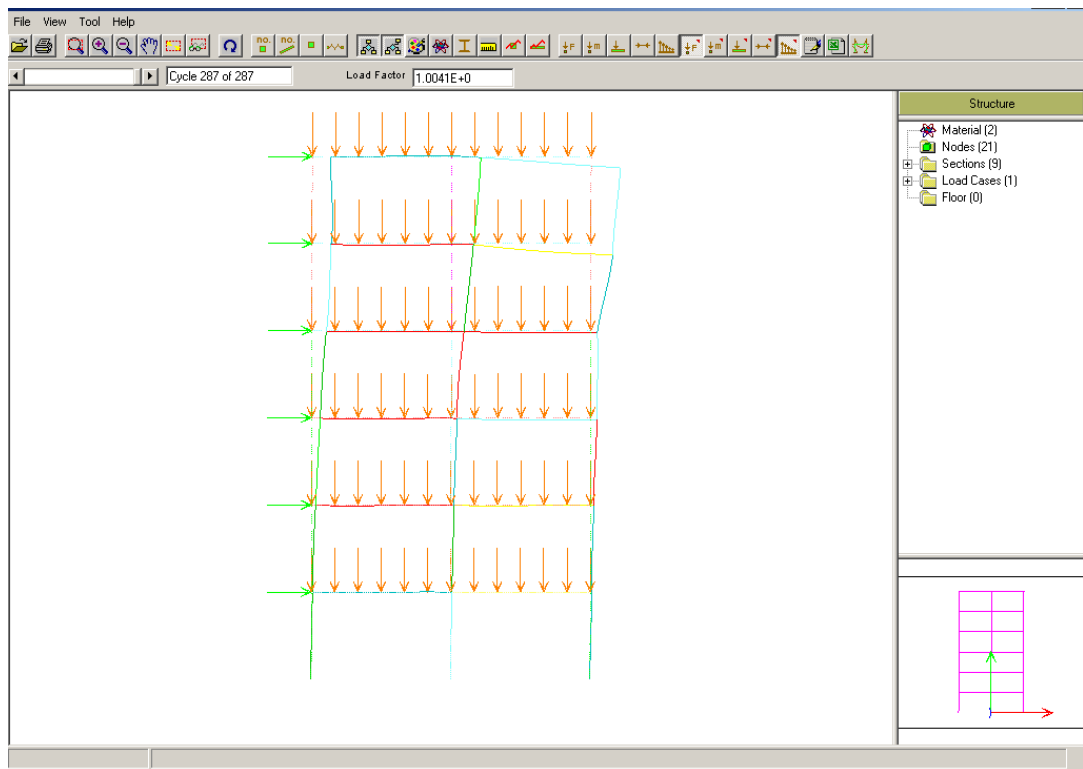
Vogel (1985) analysed a 6-storey steel frame using an elasto-plastic analysis. The structural layout and loading is indicated in the figures below.



The 6-storey Vogel frame



The load vs. deflection plot of the 6-storey frame



The results of analysis via the load vs. deflection plot are indicated in the figure above. The results should be compared with that by Vogel with close agreement. This example checks the validity of a non-linear analysis method used in a program for this type of analysis based on this code.

E6.12.5 Suggested checking method for Second-order analysis and design & Advanced Analysis

Background

It is essential to make sure one understands the concept underlying the second-order analysis and advanced analysis in Section 6 of the Code before he is allowed to use the method. In the Code, a software is required to consider the following points before it can be used.

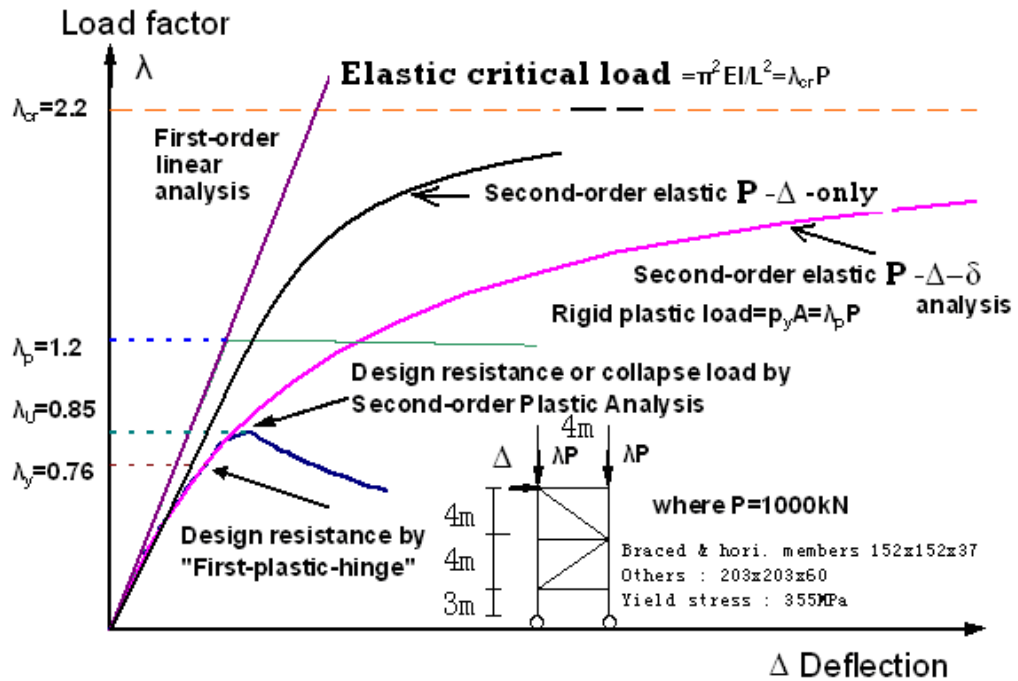
P- Δ effect due to sway of a frame

P- δ effect due to member bowing

Frame Imperfections attributing to big Δ effect

Member Imperfections attributing to small δ effect

These effects are existent in practical structures and analysis and design are required to simulate these effects, or else the structures will be under-designed.



Many computer programs can only carry out P- Δ -only type of second-order analysis, but not the P- Δ - δ analysis. Therefore, care should be taken as to define its application for finding the sway moment and the member design is still needed.

P- Δ -only analysis is used to find the amplified moment due to P- Δ effect or moment.

P- Δ - δ analysis can do the same as above plus checking of member resistance for P- δ moment, such that the checking member resistance to the Code is not required. Both methods must consider frame and member initial imperfections to code requirements.

Check of stage of sway stability by λ_{cr}

A structure needs to be classified for sway instability. The elastic critical load factor λ_{cr} of a structure is determined and used to assess the stability condition of a swaying structure. λ_{cr} is defined as the ratio of the buckling resistance of a perfect and elastic structure, P_{cr} divided by the applied load. For example, a pin-pin column of length 3 m and second moment of area of 17500 cm^4 , the elastic buckling load is $\frac{\pi^2 EI}{L^2} = 38382 \text{ kN}$.

If the factored design load is 10000 kN, the elastic critical load factor λ_{cr} is then $38382/10000 = 3.84$.

Use of Displacement method in Equation 6.1 in the Code

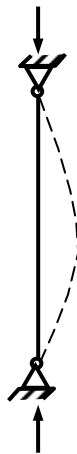
For the pin-pin column below, the end has no deflection and, therefore, the method cannot be used. As this column bends by itself with only P- δ effect but no P- Δ effect, this indicates that the P- δ effect cannot be modeled by the sway displacement method.

For the column with both ends fixed but free to translate at one end as shown below, the deflection method calculates the elastic critical load for the same column above as

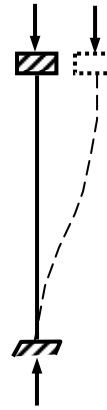
$$\lambda_{cr} = \frac{F_N}{F_V} \frac{h}{\delta_N} = \frac{F_N L}{F_V \times \frac{F_N L^3}{12EI}} = 4.7, \text{ which is 22\% above the exact value.}$$

(The reason for the error is due to the use of cubic element in the first-order linear analysis method for predicting the buckling load. In the same token, structural analysis programs for first-order linear analysis cannot be used to carry out a buckling analysis when an element is used to model a member)

This case involves member end translation and the method can predict an approximate value.



Pin Pin column



Fix / Free-to-translate column

We cannot use the Euler's load as the design resistance of a column against the design load since material yielding, large deflection effect and initial imperfection effects have not been considered. Similarly λ_{cr} cannot be used for direct design and it can only be used as a yardstick for instability condition.

Concept

There are 2 major non-linear effects and their associated imperfections are required to be considered. The term 'imperfection' is new to most engineers, but they are considered implicitly in buckling curves, such as the a, b, c and d curves for hot-rolled hollow sections, I-sections etc. In the Code, these imperfections need to be considered implicitly using the first-order linear analysis or explicitly using the second-order P- Δ - δ analysis.

(a) P- Δ effect

A frame sways under lateral load and the sway deflection makes the column and beam to be under an additional moment called the P- Δ moment.

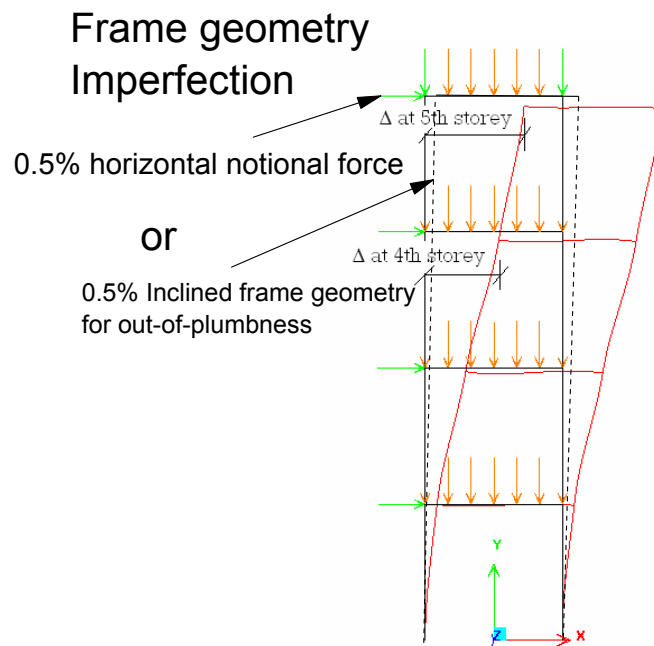


Figure 1 - P- Δ effect and its imperfections in a regular frame

(b) P- δ effect with imperfection

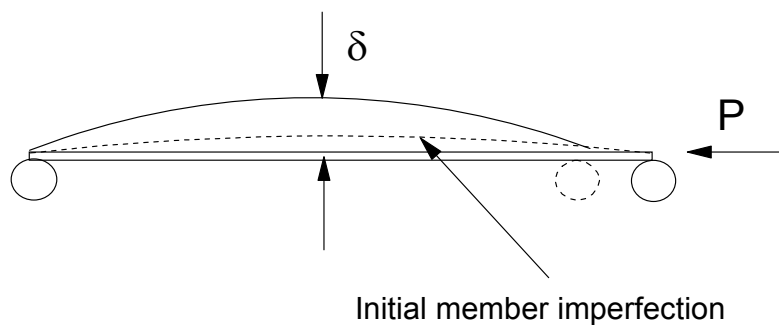


Figure 2 - P- δ effect with its imperfection in a member

Before one can use the second-order or the advanced analysis, he must demonstrate that the software should be able to allow for input of member imperfection. For frame imperfection of regular structures, one can apply 0.5% notional horizontal force, i.e. 0.5% of factored dead and live loads. For irregular structures, one has problem on how to apply the notional horizontal force and therefore the method of using the first elastic critical mode must be used (see section 6.4.4 of the Code). The shallow roof in figure 4 below is an example of a structure with problem in applying the notional force.

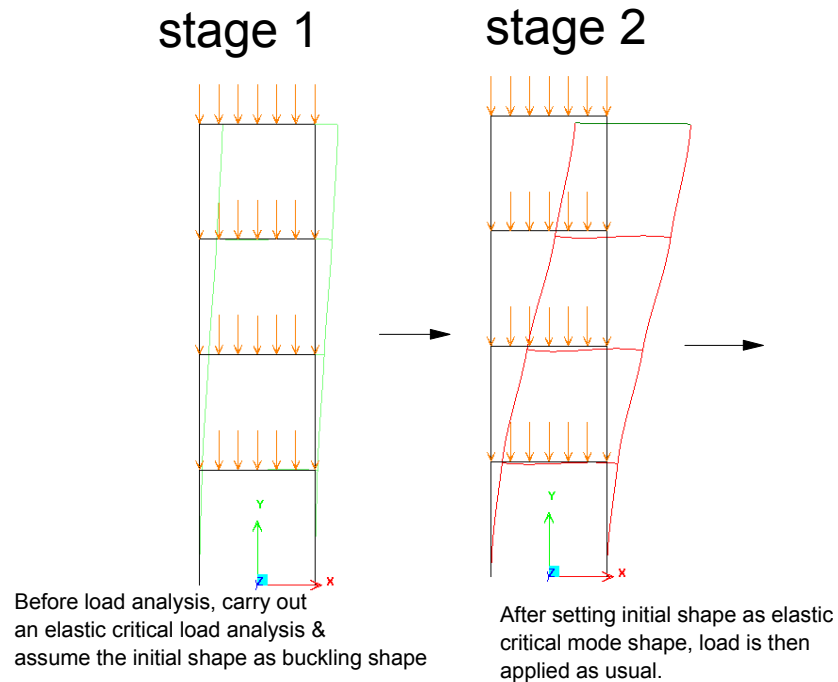
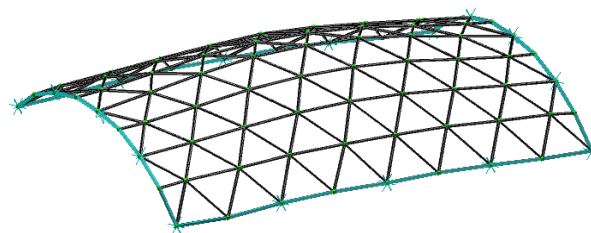


Figure 3 - Application of elastic critical buckling mode with amplitude set as out-of-plumbness as imperfection mode



Shallow roof dome requiring use of elastic critical mode as imperfection

Figure 4 - Shallow dome with problem in use of notional horizontal force for imperfection simulation

E6.12.6 An example on demonstration of the deflection method for finding the elastic critical load factor λ_{cr}

The 4-storey frame shown in figure 5 is designed. All members are 203x203x60 UC of steel grade S275 with the following properties.

Area = 76 cm², $I_x = 6103 \text{ cm}^4$, $I_y = 2047 \text{ cm}^4$, $Z_x = 582 \text{ cm}^3$, $Z_y = 199 \text{ cm}^3$,
 $r_x = 8.96 \text{ cm}$, $r_y = 5.19 \text{ cm}$
 $T = 14.2 \text{ mm} < 40 \text{ mm}$.

The structure is subjected to a pair of factored vertical point loads of 500 kN at top, with a notional horizontal force of 0.5% of factored dead and live loads applied at the same level (clause 2.5.8 of the Code). The design strength is 275 N/mm². The storey height and width are both 4m.

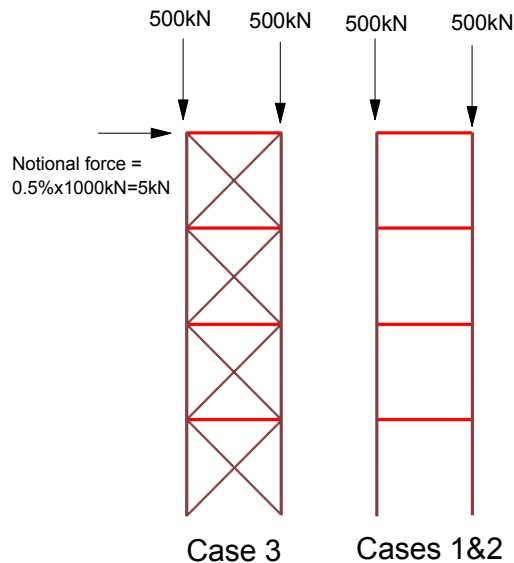


Figure 5 - The braced and unbraced frames studied

Using the method of sway index, the elastic buckling load factor, λ_{cr} , is calculated in case 1 as follows:

Table 1 - Deflections at various levels of the 4-storey frame

Storey	Deflections δ_i (mm) / sway indices ${}_i\phi$					
	Case 1 (Bent about minor axis, no bracing)		Case 2 (Bent about major axis, no bracing)		Case 3 (Bent about minor axis, fully braced)	
	δ_i	${}_i\phi$	δ_i	${}_i\phi$	δ_i	${}_i\phi$
1	6.3	0.00151	2.1	0.00053	0.1	0.00003
2	15.25	0.00230	5.3	0.00080	0.2	0.00003
3	24.63	0.00236	8.7	0.00085	0.4	0.00006
4	32.73	0.00202	11.6	0.00070	0.6	0.00006

Note: ${}_i\phi = \frac{\delta_i - \delta_{i-1}}{h}$

Local buckling check:

For rolled section:

Web $d/t = 17.3 = < 80\epsilon$ O.K.

Section Classification:

$B/T = 14.5$; $d/t = 17.3$ Section Class = 1 "Plastic"

Case1 Unbraced case

The maximum ϕ_s is 0.00236 and the λ_{cr} is $= 1/200/0.00236 = 2.12$

Using Non-linear Analysis and Design Computer Software, λ_{cr} is calculated as $2.136 < 5.0$, the manual method cannot be proceeded any further.

If we ignore this limitation, the effective length L_E :

$$L_E = \sqrt{\frac{\pi^2 EI}{\lambda_{cr} F_c}} = \sqrt{\frac{\pi^2 \times 205000 \times 2047 \times 10^4}{2.12 \times 500000}} = 6.25m$$

$$L_E/r_y = 6250/51.9 = 120.4$$

From Table 8.8c of the Code, permissible axial force $= 97 \times 7600 \times 10^{-3} = 737 \text{ kN}$

However, since λ_{cr} is less than 5.0 here, the above method can no longer be used in the Code. If we use advanced analysis (second-order plastic design), the collapse load factor is then 1.6.

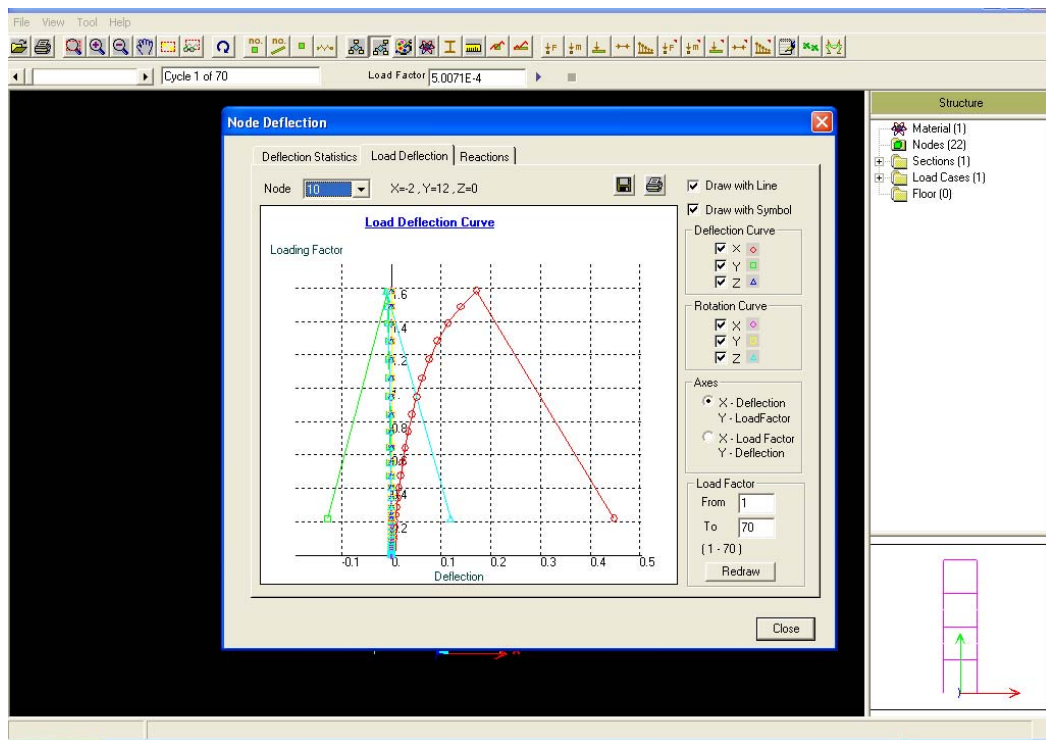


Figure 6 - Advanced analysis for case 1 with load factor 1.6

If we want to use manual method, there can be two solutions for this problem. The first is to rotate the major principal axis of members to resist loads, which is considered as case 2. The other option is to add bracing members, which is designated as case 3.

Case2 Unbraced case

Referring to Table 1, the selected ϕ_s is 0.00085 and the λ_{cr} is $= 1/200/0.00085 = 5.9 > 5.0$ and < 10 , sway sensitive frame.

Using Non-linear Analysis and Design Computer Software, λ_{cr} is 6.3 > 5.0 , the Code can be used.

Here we make use of λ_{cr} to find the effective length factor. The elastic critical load is $\lambda_{cr} F_c$.

Therefore, the effective length L_E

$$L_E = \sqrt{\frac{\pi^2 EI}{\lambda_{cr} F_c}} = \sqrt{\frac{\pi^2 \times 205000 \times 6103 \times 10^4}{5.9 \times 500000}} = 6.47 \text{ m}$$

$$L_E/r_x = 6470/89.6 = 72.2,$$

From Table 8.8b of the Code, permissible axial force $= 198 \times 7600 \times 10^{-3} = 1505 \text{ kN}$

Design load factor $= 1505/500 = 3.0$

Collapse Design Load Factor by Non-linear Analysis and Design Computer Software = 3.3

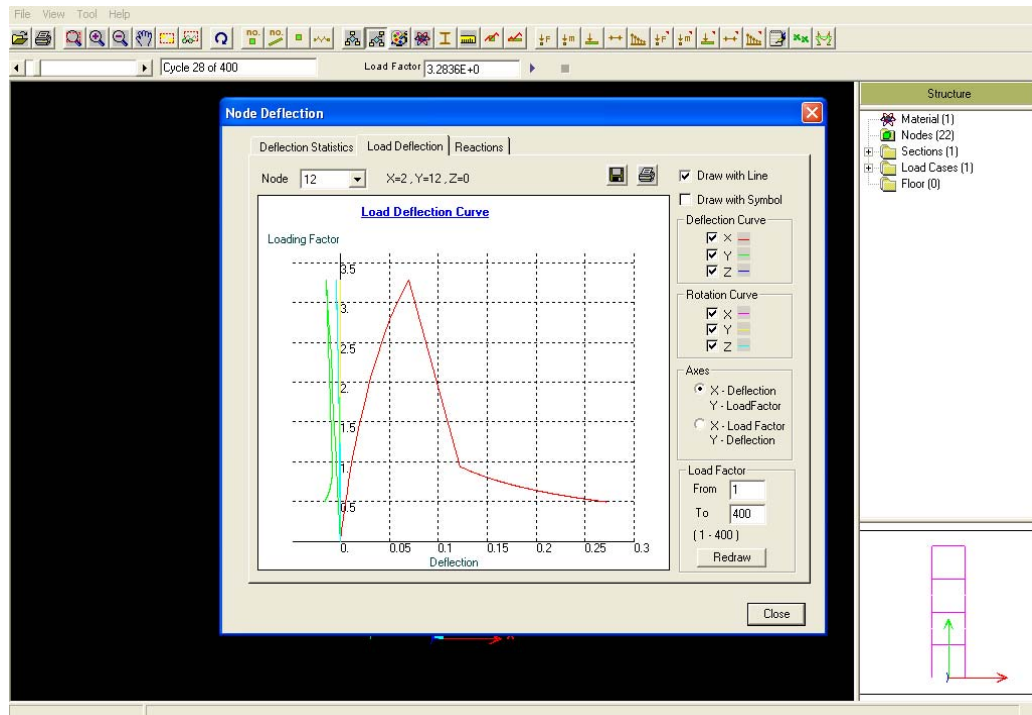


Figure 7 - Advanced Analysis for case 2 with collapse load factor as 3.3

Case 3 Fully braced case

Obviously the frame is non-sway and the beam is bent under single curvature. From Table E.3, consider column in the second level as the most critical.

From Table 6.2,

$$k_1 = \frac{\frac{I}{L} + \frac{I}{L}}{0.5 \frac{I}{L} + \frac{I}{L} + \frac{I}{L}} = 2/2.5 = 0.8 \quad \text{and} \quad k_2 = \frac{\frac{I}{L} + \frac{I}{L}}{0.5 \frac{I}{L} + \frac{I}{L} + \frac{I}{L}} = 2/2.5 = 0.8$$

From Figure 6.4b of the Code, for non-sway frame, $L_E/L = 0.85$,

Thus, effective length = $0.85 \times 4 = 3.4 \text{ m}$

$$L_E/r_y = 3400/51.2 = 66.4$$

From Table 8.8c of the Code, $p_c = 189 \text{ N/mm}^2$

$$P_c = 189 \times 7600 \times 10^{-3} = 1436 \text{ kN}$$

At design load, the axial force in column is 500 kN,

$$\text{Design load factor} = 1436/500 = 2.9$$

Second-order P-Δ-δ design load factor by Non-linear Analysis and Design Computer Software = 3.7 using imperfection parameter 1/500 or 0.5% notional horizontal force. The collapse load factor is 4.5.

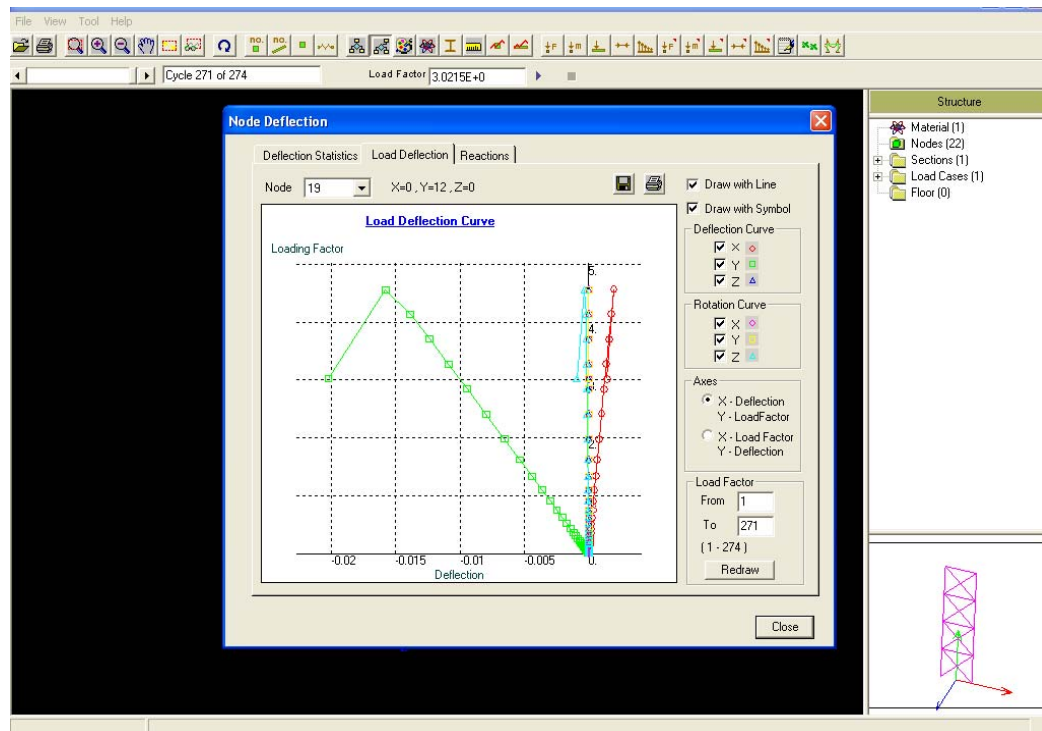


Figure 8 - Advanced Analysis for Case 3 with load factor 4.5

For the braced case, the use of notional horizontal force to find elastic critical load gives us a result of very large λ_{cr} . Thus, we can forget about the sway effect and use non-sway effective length for design. Conservatively, we can assume effective length equal to member length or use the chart for effective length.

E6.12.7 Some common terms used in non-linear analysis

Elastic Critical Load Factor λ_{cr} : a factor multiplied to the design load to cause the first element of a structure to buckle elastically without consideration of the large deflection and material yielding effects. The factor is an upper bound solution that cannot be used directly for design. λ_{cr} can be obtained in a computational analysis or from Equation 6.1 of the Code.

P-delta effects : refer to the second-order effects. There are two types, being P- Δ for sway in frame and P- δ in member bowing.

P- Δ effect : second-order effect due to nodal change of geometry or sway of a structure

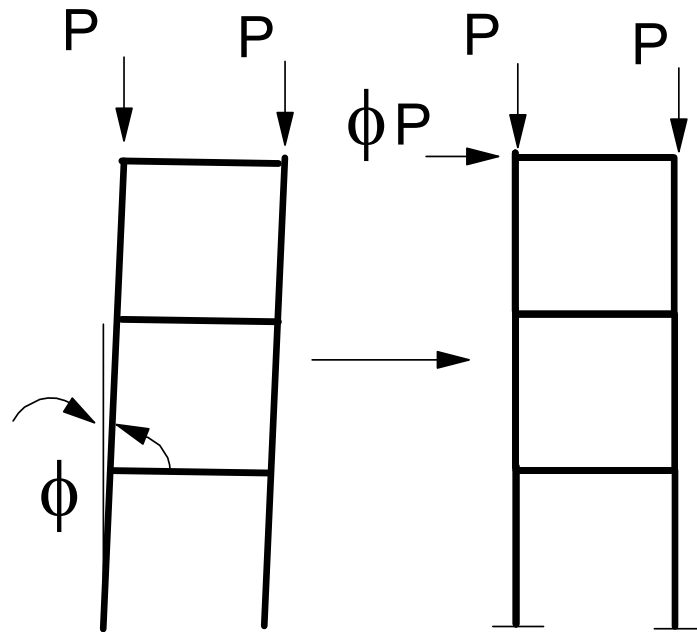
P- δ effect : second-order effect due to deflection along a member leading also to a change of member stiffness under load as well as an additional P- δ moment along a member. The effect of change in stiffness can be seen as “a member under tension is stiffer than under compression”.

First-order linear analysis : an analysis assuming the deflection and stress being proportional to load. It does not consider buckling nor material yielding, which are considered in member design using tables, figure and charts in various codes.

Imperfections : deformed member curvature and frame geometry to account for imperfect structural member and frame geometries.

Non-linear analysis : represent any analysis which does not assume a linear relationship between load, displacement, stress (σ) and E. This is a very broad term and in steel design, it includes second-order P- Δ -only analysis, P- Δ - δ analysis and advanced analysis.

Notional Horizontal Force : a small force applied horizontally to a structure to simulate lack of verticality and imperfection. It is also used to measure the lateral stiffness so that the elastic critical factor can be determined (see Equation 6.1 of the Code).



Lack of verticality is considered by notional horizontal forces to a truly vertical frame in an analysis model

Second-order P-Δ-ONLY analysis : an analysis used to plot the bending moment and force diagrams based on the deformed or sway geometry. It considers only the P-Δ effect but not the P-δ effect and checks the moment at member ends but not along members. Use of the Code for member design is needed. When using the Code, the effective length is set equal to member length or obtained from Figure 6.4b of the Code, since the sway effect at member ends has been considered in the P-Δ moment.

Second-order P-Δ-δ analysis : an analysis and design method allowing for P-Δ and P-δ effects, such that only checking of member cross sectional strength in clause 8.9.1 of the Code is required, because the effect due to effective length has been considered in the Δ and δ effects. The design load can be taken as the load causing the first plastic hinge or first yield. This condition is satisfied when the maximum design (action or applied) moment is \leq plastic moment ($=p_y S$) or elastic moment ($=p_y Z$). It considers both the P-Δ and P-δ effects, and therefore P-Δ-δ analysis is named. Effective length is not required to be assumed here.

Advanced analysis : an analysis and design method allowing for P-Δ and P-δ effects such that only sectional check of member resistance as for the above “Second-order P-Δ-δ analysis” is required. However, unlike the “Second-order P-Δ-δ analysis”, it does not stop at the first plastic or first yield moment. This method allows the formation of one or more plastic hinges by inserting a plastic hinge to the end of a yielded member when the design (applied) moment is equal to the plastic moment in a cross section until a plastic collapse mechanism is formed. This method has been used in seismic and static load design.

References

AISC, Load and resistance factor design specification for structural steel buildings (2010), American Institute of Steel Construction Chicago, USA.

AS 4100. Steel structures, (1998), Australian Standard

Vogel, U. “Calibrating frames”, Stahlbau, (1985), 54, 295-311

E7 SECTION CLASSIFICATION

E7.1 GENERAL

When a section is made of very thin plates, local buckling may occur before the section reaches its plastic moment or even starts to yield. As shown in the Figure below, the local plate buckling has an approximately half-sine curve with wave length about the plate width. There are two methods to check and reduce the sectional capacity, namely the effective width method and the effective stress method. The effective stress method determines the stress reduction for the whole section. It does not depend on the load case and therefore it is more convenient to use. The effective width method reduces the cross sectional dimensions but it depends on the load and requires more separated calculation for different load cases. The effective width method is, however, more economical.

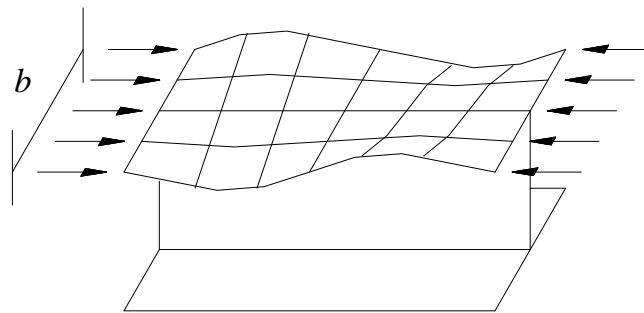


Figure E7.1 - Local plate buckling at top flange of an I-beam

The general concept

In elastic or plastic structural design, we assume cross sectional properties of members remain unchanged until the sections yield for elastic design or the sections form plastic hinges for plastic design. This requires that the thickness of the cross sections must be adequate not to have local plate buckling and therefore section classification is needed to ensure this. These requirements in plastic strength are summarized as follows:

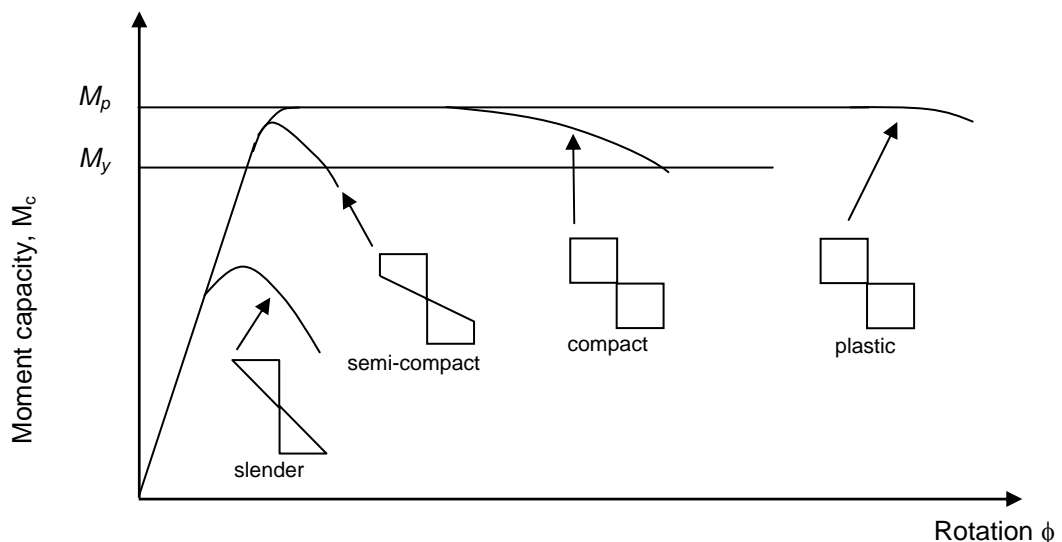


Figure E7.2 - Local buckling and plastic strength of cross-sections

Cross-sections subject to compression due to bending moment or axial load should be classified into Class 1 plastic, Class 2 compact, Class 3 semi-compact or Class 4 slender, depending on their local buckling strength. Cross-sections should be classified to

determine whether local buckling influences their capacity, without calculating their local buckling resistance.

The classification of each element of a cross-section subject to compression should be based on its width-to-thickness ratio. The dimensions of these compression elements should be taken as shown in Figure 7.1 of the Code. The elements of a cross-section are generally of constant thickness. For elements that taper in thickness, the thickness specified in the relevant standard should be used.

A distinction should be made between the following two types of element,

- (a) Outstand elements – attached to an adjacent element at one edge only, the other edge being free.
- (b) Internal elements – attached to other elements on both longitudinal edges and including:
 - *Webs* comprising internal elements perpendicular to the axis of bending
 - *Flanges* comprising internal elements parallel to the axis of bending

All compression elements should be classified in accordance with clause 7.2 of the Code. Generally, the complete cross-section should be classified according to the highest (least favorable) class of its compression elements. Alternatively, a cross-section may be classified with its compression flange and its web in different classes.

Circular hollow sections should be classified separately for axial compression and for bending.

For the design of compression elements with longitudinal stiffeners, reference should be made to BS 5400-3.

E7.2 CLASSIFICATION

Plate buckling is controlled and classified by the breadth to thickness ratio (b/t). Thicker plates or plates with smaller breadth are less likely to buckle than the thinner plates or plates with larger breadth. Plates with stiffeners will reduce the breadth by the distance between longitudinal stiffeners, thus increases the buckling resistance. Transversely placed stiffeners are not effective in reducing the local plate buckling resistance as they are unable to stiffen the long plate elements unless they are very closely spaced.

The limiting values for the 4 classes of sections are indicated in Table 7.1 in the Code. When a section is classified as slender, the effective stress and effective width method can be used. A more rigorous evaluation can be carried out by the non-linear finite element analysis. As slender hot rolled sections are uncommon, the clause refers to section 11 of the Code for the effective width method, which is limited to 8mm thick.

The limiting ratio including the use of ε is likely to be applicable to high strength steel, but no research evidence at the time of compilation of this EM is available to confirm the applicability to high strength steel. Non-linear finite element method can be applied for local buckling check in high strength steel plates.

For channel sections, the web is classified as slender section when the limiting value d/t is larger than 40ε .

E7.3 STRESS RATIOS FOR CLASSIFICATION

Clause 7.3 of the Code is self explanatory.

E7.4 FLANGES OF COMPOUND I- OR H-SECTIONS

Clause 7.4 of the Code is self explanatory.

E7.5 EFFECTIVE PLASTIC MODULUS

Clause 7.5 of the Code is self explanatory.

E7.6 EFFECTIVE WIDTH METHOD FOR SLENDER CROSS-SECTIONS

This is one of the two approaches to account for local buckling. In order to limit the moment and axial force to a level below the plastic moment or first-yield moment, this method reduces the sectional properties by decreasing the width of a plate making the cross section. For **the general methods in determining the effective section properties of class 4 slender cross-sections**, reference should be made to clause 3.6.2 – 3.6.6 of BS 5950-1.

E7.7 EFFECTIVE STRESS METHOD FOR SLENDER CROSS-SECTIONS

In this method, the design strength is reduced in order to limit the stress below the buckling stress of the cross sections. The same cross sectional properties can be used in design with this reduced design strength allowing for local buckling. Except for calculation of neutral axis for web d/t ratio, the method does not depend on load and therefore the results for reduction of p_y can be used for all load cases, which is much convenient.

E7.8 SHIFT OF THE CENTROID OF THE EFFECTIVE CROSS-SECTION

Clause 7.8 of the Code is self explanatory.

E7.9 WORKED EXAMPLES

Section classification of rolled universal I-beam

Determine classification of element of beam of 254×102×22 UB in S355 material as shown.

Solution

SECTION PROPERTIES:

$$D = 254\text{mm}, B = 101.6\text{mm}, t = 5.8\text{mm}, T = 6.8\text{mm}, d = 225.2\text{mm}$$

$$b = 101.6 / 2 = 50.8\text{mm}$$

SECTION CLASSIFICATION:

$$\text{Design yield stress, } p_y = 355\text{N/mm}^2, \therefore \varepsilon = \sqrt{\frac{275}{355}} = 0.88$$

For compression outstand flange of a rolled section, the limiting value β is 9ε .

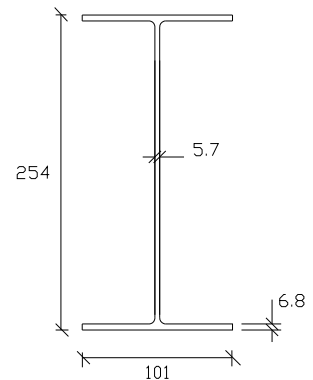
$$\text{For flange, } \frac{b}{T} = \frac{50.5}{6.8} = 7.5 < 9\varepsilon = 7.92 \quad \therefore \text{flange is compact section} \quad (\text{Table 7.1})$$

For bending the web of a rolled section, the limiting value β is 80ε .

$$\text{For web, } \frac{d}{t} = \frac{225.2}{5.8} = 39 < 80\varepsilon = 70.4 \quad \therefore \text{web is plastic section} \quad (\text{Table 7.1})$$

\therefore the beam element is class 2 compact section

Whole section is therefore class 2 compact section. It should be noted that when flange is class 2 compact section and web is class 1 plastic section, the whole section is regarded as compact section.



Effective width method for hot-finished RHS under uniform compression

The section 250x150x5 hot-finished RHS section Grade S355 is under compression of 1100kN and under a small moment causing negligible stress gradient. Determine the section properties for compression capacity of the section.

Solution

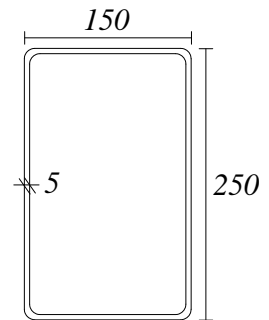
SECTION PROPERTIES:

$$D = 250\text{mm}, B = 150\text{mm}, t = 5\text{mm}, A_g = 3890\text{mm}^2$$

SECTION CLASSIFICATION:

Design strength, $p_y = 355\text{N/mm}^2$ (for $t = 5\text{mm} < 16\text{mm}$)

$$\therefore \varepsilon = \sqrt{\frac{275}{355}} = 0.88 \quad (\text{Table 3.2})$$



The width of RHS, $b = B - 3t = 150 - 3 \times 5 = 135\text{mm}$ (Table 7.2)

The depth of RHS, $d = D - 3t = 250 - 3 \times 5 = 235\text{mm}$ (Table 7.2)

For compression flange of a hot-finished section, the limiting value β is 40ε .

For flange, $\frac{b}{T} = \frac{135}{5} = 27 < 40\varepsilon = 35.2 \quad \therefore$ flange is non-slender section (Table 7.2)

For the web (general) of a hot-finished section, the limiting value β is $\frac{120\varepsilon}{1+2r_2}$.

$$\text{Stress ratio, } r_1 = \frac{F_c}{2dtp_{yw}} = \frac{1100 \times 10^3}{2 \times 235 \times 5 \times 355} = 1.32 \quad \therefore r_1 = 1 \quad (7.5)$$

$$r_2 = \frac{F_c}{A_g p_{yw}} = \frac{1100 \times 10^3}{3890 \times 355} = 0.8 \quad (7.6)$$

For web, $\frac{d}{t} = \frac{235}{5} = 47 > \frac{120 \times 0.88}{1 + 2 \times 0.8} = 40.62 \quad \therefore$ web is slender (Table 7.1)

\therefore the beam element is class 4 slender section

\therefore the RHS section of column member is class 4 slender section.

E8 DESIGN OF STRUCTURAL MEMBERS

E8.1 GENERAL

This section of the Code deals with the design of hot-rolled sections and it is modified from the BS 5950 (2000) with refinement of some equations for checking of beam and column buckling strength and inclusion of Chinese sections for capacity checks. Its features contain the use of 4 column curves against the single buckling curve in LFRD (1996) or the 5 buckling curves in Euro-code 3, consideration of $P-\delta$ effect due to loads, imperfection and load-induced deflection. All classes of steel can be designed in accordance with the sectional properties.

To determine the Perry constant, a curve-fitting process is required to find a constant, which gives a buckling curve close to the design or experimental buckling curve. Using the Perry constant, the buckling resistance of a strut can be determined.

E8.2 RESTRAINED BEAMS

Moment capacity of a restrained beam or in-plane beam, which refers to a beam with lateral restraint against lateral-torsional buckling, is detailed here. Its content is adopted from BS 5950 (2000).

Both elastic and plastic in-plane bending behaviour on the beam members should be considered in the design procedures of in-plane bending beam, since the section of beam may undergo elastic or plastic deformations in curvatures. The moment capacity of plastic and compact beam sections should refer to the plastic behaviour as zone ④ in Figure E8.1, and the plastic moment capacity for this section of the Code is developed. The moment capacity of semi-compact beam section should refer to the elastic moment as discussed in clause 8.1.1 of the Code, which bases on elastic behaviour as stress block ②. For slender sections, only effective elastic section or the effective stress method should be used, as the local buckling should be taken into account.

It should be reminded that the beam members with fully lateral restraints are only considered in this section of the Code so that lateral-torsional buckling does not occur before reaching the plastic moment capacity of beam member. Both the terms lateral-torsional and flexural-torsional buckling refer to beam buckling and they are the same. Torsional buckling refers to twisting mode of column buckling under axial force.

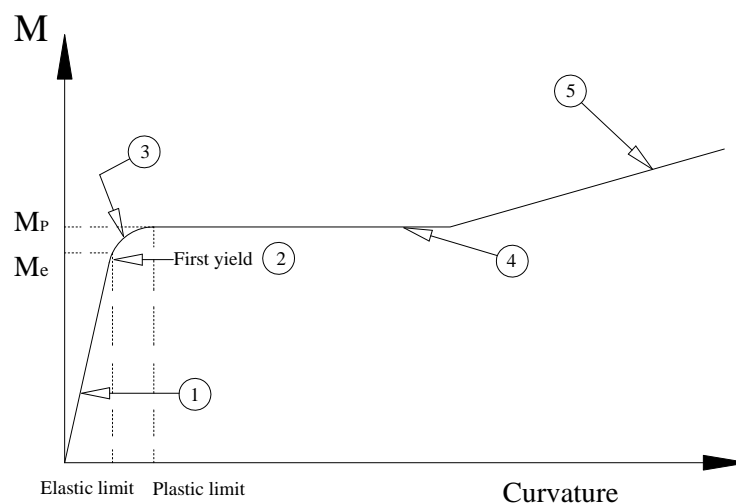


Figure E8.1 - Moment versus curvature of a laterally restrained beam

The use of 2.5% restraining force is a common practice in many codes. This value is larger than, but in the same order as, the restraining force for temporary structures of 1%. Design for top restraints at support and intermediate locations should refer to other national codes.

For low shear condition, the effective section properties and modulus can be obtained from Section 7 of the Code for section classification.

For high shear condition, the formulae adopted here is similar to BS 5950 (2000) version and different from the BS 5950 (1990) version.

E8.2.3 Beams with web openings

Two types of openings are common for steel beams, being the castellated beams for material saving and isolated openings for the passing of service ducts. The general requirements are given here for checking of bending and shear. A rigorous method is required for the design of specific members. In this respect, finite element method should be used

E8.3 LATERAL-TORSIONAL BUCKLING OF BEAMS

A beam not fully restrained along its length may buckle laterally by a combination of bending laterally and twisting action. The lateral-torsional buckling resistance moment can be substantially below the in-plane moment capacity of the same member. Design for lateral-torsional buckling is important for beams weak in torsional and minor axis stiffness and without fully lateral restraints.

For a simply supported beam under a pair of equal uniform end moments M_e , the beam is bent about its major axis. The beam is assumed as doubly symmetric, i.e. the shear centre is coincident with the centroid of beam section and the section is thin-walled and open. Only elastic material property is only assumed in this section of the Code. The beam buckles at a moment M_{cr} , with the deflected and twisted equilibrium position of the beam as shown in Figure E8.2.

Values of the parameters “u” and “x” used in design of unrestrained beams of I, H and channels with equal flanges can be taken respectively as 0.9 and D/T for hot-rolled sections and 1.0 and D/T for welded sections. Alternatively “u” and “x” for hot-rolled sections can be determined from Appendix 8.2 of the Code and these values for welded sections can be obtained from BS 5950 (2000).

Equivalent uniform moment factor, m_{LT} , is taken as the same for normal and destabilizing load, in which the adverse effect of the latter is considered by a larger effective length factor in Table 8.1. With this value of m_{LT} , it is appreciated that a beam under uniform moment is more likely to buckle than a beam under non-uniform moment. However, as cantilever beam is easier to buckle, m_{LT} is taken as 1.

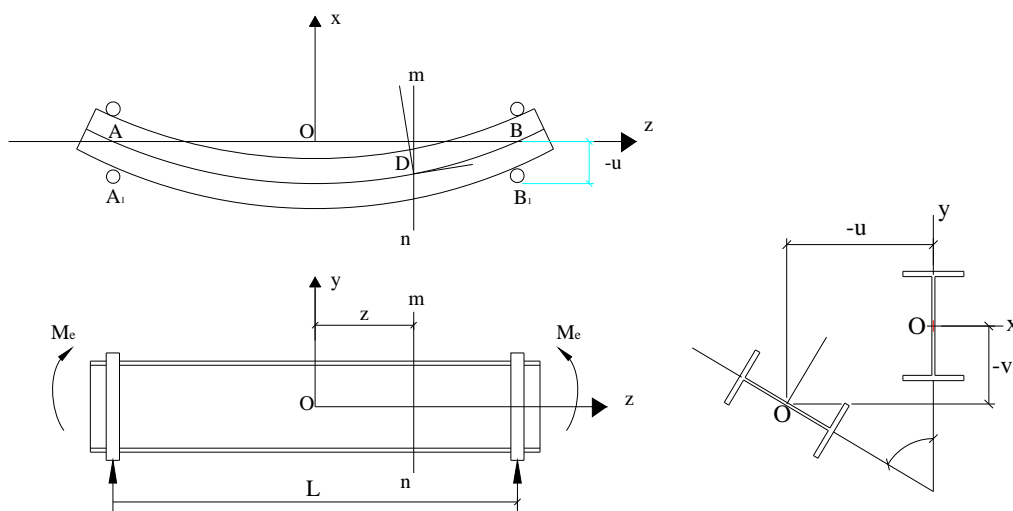


Figure E8.2 - Buckling of an unrestrained simply supported beam

E8.3.1 Intermediate and end lateral restraints

To reduce the effective length of a beam, lateral restraints are commonly added. The strength of the restraining members cannot be too small and must not be less than 2.5% of the applied load acting on the member being restrained and the restraints should be placed on the compression flange of the member being restrained.

The use of the parameter N_r is to consider the case when the restraining forces are not simultaneously required and therefore, a reduction factor can be imposed.

E8.3.3 Normal and destabilizing loads

When a load is applied on the top flange of a beam, it will create an additional disturbing moment when twisted, which reduces the load required to cause lateral-torsional buckling of a beam and it is called the destabilizing load. On the other hand, if the load is at shear centre or below, then it will not cause a disturbing twisting moment to reduce the buckling load of the beam and it is called the normal load.

E8.3.4 Effective length for lateral-torsional buckling

The effective length can generally be taken as the distance between lateral restraints accounting for detailing in boundary conditions. Various refined details and drawings in the Code should be referred to and the effect of these details is significant.

E8.3.5 Moment resistance to lateral-torsional buckling

In case where the sections are of odd shape or varied along the member length, the effective length can be directly obtained from a finite element buckling analysis via the elastic critical moment M_{cr} .

The equation here is to calculate the effective length, since most finite element package calculates the elastic critical moment and Equation 8.17 is then used to determine the effective length for use with the design table.

The equivalent moment factor is the same as BS 5950 (2000), which is based on sampling along a beam. The formula is different from BS 5950 (1990), which is empirical as follows.

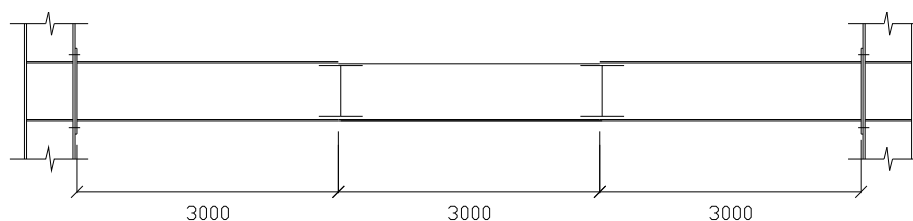
$$m_{LT} = 0.57 + 0.33\beta + 0.10\beta^2 \geq 0.43$$

in which β is the ratio of two end moments

The formulae are based on the 2000 version of BS 5950, which has some improvement over the 1990 version of BS 5950.

Example 8.1

An I-beam of 686×254×125 UB in S275 material of span 9m long with rigid connection restrained to rotate on plan supports two secondary beams as shown. The end supports can provide adequate lateral and torsional restraint, whereas intermediate lateral restraint prevents the compression flange of the main beam from lateral movement but free rotation is allowed. The unfactored dead load of 100kN and imposed load of 200kN are transferred from the secondary beams to the I-beam. The member capacities of the main steel beam shown in the figure below are checked.



Solution

DESIGN LOADING:

Considering the point loads from the secondary beams,

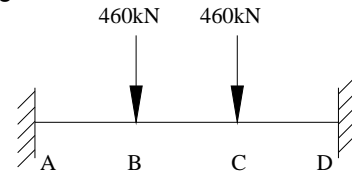
$$\omega = 1.4\omega_Q + 1.6\omega_G = 1.4 \times 100 + 1.6 \times 200 = 460 \text{ kN}$$

After considering the self-weight of the beam, the calculated design forces and moments are summarized as follows:

Maximum shear force, $V = 467.9 \text{ kN}$

Maximum hogging moment, $M_{\text{hog}} = 931.8 \text{ kNm}$

Maximum sagging moment, $M_{\text{sag}} = 465.9 \text{ kNm}$



SECTION PROPERTIES:

$$D = 677.9 \text{ mm}, B = 253 \text{ mm}, t = 11.7 \text{ mm}, T = 16.2 \text{ mm}, d = 615 \text{ mm}, x = 43.9,$$

$$I_x = 118000 \text{ cm}^4, S_x = 3994 \text{ cm}^3, Z_x = 3481 \text{ cm}^3, r_y = 5.24 \text{ cm}, A_g = 159 \text{ cm}^2, u = 0.862$$

SECTION CLASSIFICATION:

Design yield strength for flange, $p_{yf} = 265 \text{ N/mm}^2$ ($T > 16 \text{ mm}$) (Table 3.1)

Design yield strength for web, $p_{yw} = 275 \text{ N/mm}^2$

\therefore The design strength of the section, $p_y = 265 \text{ N/mm}^2$

For outstand compression flange of rolled section, limiting value β is 8ε .

For flange, $\frac{b}{T} = \frac{126.5}{16.2} = 7.81 < 8\varepsilon = 8\sqrt{\frac{275}{265}} = 8.15$ \therefore flange is plastic section (Table 7.1)

For web, $\frac{d}{t} = \frac{615}{11.7} = 52.6 < 80\varepsilon = 80\sqrt{\frac{275}{265}} = 81.5$ \therefore web is plastic section (Table 7.1)

\therefore the beam member is class 1 plastic section

SHEAR CAPACITY:

Shear area, $A_v = tD = 11.7 \times 677.9 = 7931.4 \text{ mm}^2$ (Clause 8.2.1)

Shear capacity, $V_c = p_y A_v / \sqrt{3} = 265 \times 7931.4 \times 10^{-3} / \sqrt{3}$ (8.1)
 $= 1213 \text{ kN} > V = 465.9 \text{ kN}$ (OK!)

MOMENT CAPACITY:

$V < 0.6 V_c$; so, it is low shear condition (Clause 8.2.2.1)

Moment capacity, $M_c = S_x p_y = 3994 \times 10^3 \times 265 \times 10^{-6}$ (8.2)

$= 1058 \text{ kNm} > M_{\text{sag}} = 465.9 \text{ kNm}$ (OK!)

or $> M_{\text{hog}} = 931.8 \text{ kNm}$ (OK!)

$= 1058 \text{ kNm} < 1.2 p_y Z_x = 1.2 \times 265 \times 3481 \times 10^3 \times 10^{-6} = 1107 \text{ kNm}$ (OK!)

LATERAL-TORSIONAL BUCKLING:

Load is applied to the top flange as is destabilising loads. (Clause 8.3.3)

In range of AB, $\beta = -931.8/465.9 = -2$ $\therefore m_{LT} = 0.44$

In range of BC, $\beta = 1$ $\therefore m_{LT} = 1$ (Table 8.4 (a))

Uniform moment, $\overline{M}_{AB} = m_{LT} M_x = 0.44 \times 931.8 = 410 \text{ kNm}$ (8.18)

$\overline{M}_{CD} = 460 \text{ kNm}$

Effective length of AB, $L_E = \frac{1}{2}(1.2 \times 0.8 L_{LT} + 1.2 L_{LT}) = 0.5 \times 2.16 \times 3 = 3.24 \text{ m}$ (8.14 & Clause 8.3.4.2)

(The mean value of one end support and an intermediate lateral restraint should be taken when the end conditions of AB segment are different.)

Effective length of BC, $L_E = 1.2 L_{LT} = 1.2 \times 3 = 3.6 \text{ m}$ (Clause 8.3.4.2)

Therefore, the effective length L_E of region BC is more critical. The resistance of lateral-torsional buckling should be based on the segment of BC.

Slenderness ratio, $\lambda = \frac{L_E}{r_y} = \frac{3600}{52.4} = 68.7$ (8.26)

$u = 0.862$ (for economical) and $\beta_w = 1$ (for plastic section) (Clause 8.3.5.3)

$v = \frac{1}{[1 + 0.05(\lambda/x)^2]^{0.25}} = \frac{1}{(1 + 0.05(68.7/43.9)^2)^{0.25}} = 0.972$ (8.27)

Equivalent slenderness, $\lambda_{LT} = uv\lambda\sqrt{\beta_w} = 0.862 \times 0.972 \times 68.7 = 57$ (8.25)

Bending buckling strength, $p_b = 213 \text{ N/mm}^2$ (Table 8.3 (a))

Buckling resistance moment, $M_b = p_b S_x = 213 \times 3994 \times 10^3 \times 10^{-6}$ (8.20)
 $= 851 \text{ kNm} > M_{AB} \text{ \& } M_{CD}$ (OK!)

E8.4 PLATE GIRDERS

Complex girders are not preferred in buildings because of the uncertain structural behaviour. Thus we concentrate more on simple girders with web stiffeners and users should refer to a bridge code such as BS 5400 for more types of stiffener design. The Code covers 3 common types of stiffeners “intermediate transverse stiffeners (for shear)”, “loading bearing stiffeners (for bearing)” and “load carrying stiffeners (for web buckling)”. Other complex stiffeners are not covered in BS 5950. For stiffener design, BS5400 is more comprehensive.

E.8.4.6 Shear buckling resistance

If the web depth-to-thickness ratio d/t ratio $> 62\varepsilon$ for welded section, shear buckling should be checked. No shear buckling check is required for hot-rolled section.

E8.4.8 End anchorage

The formulae in 2000 version of BS 5950 are based on tension field action, which is more economical. However, the tension field force can only develop when the members bounding the panel provide adequate anchorage. This is particularly important at the ends where no opposite bay is present to counter-act the tension force and therefore the end stiffeners are required for end anchorage.

E8.4.10 Web capacity and stiffeners design

Plate girder usually requires stiffeners while beams do not as they have smaller d/t ratios. There are always exceptional cases that beams may have larger d/t ratios. Stiffeners are required to strengthen members locally and one should check for the need of such provision.

E8.4.10.5.1

For the definition of the distance to the nearer end of the member from the end of the stiff bearing, b_e , please refer to BS 5950-1.

E8.5 BUCKLING RESISTANCE MOMENT FOR SINGLE ANGLE MEMBERS

Clause 8.5 of the Code is self-explanatory.

E8.6 TENSION MEMBERS

The computation of tension capacity in 2000 version of BS 5950 simplified and more easy to use than the formulae in 1990 version of BS 5950.

E8.7 COMPRESSION MEMBERS

When a column length increases, the geometric second-order effects, such as $P-\Delta$ and $P-\delta$ effects, reduce the ultimate compression resistance. The $P-\delta$ effect is referred to as the second-order effect due to the deflection along a member, whereas the $P-\Delta$ effect is caused by the load change of structural geometry, which creates an additional moment. The additional second-order moments are due to change of geometry and member curvature due to loads along members and at member ends and initial imperfections. Figure E8.3 shows the buckling behaviour considering and ignoring material yielding.

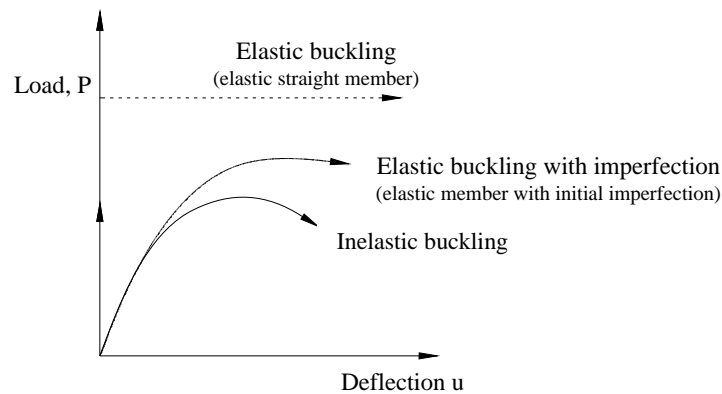


Figure E8.3 - Buckling of an elastic and inelastic column

E8.7.2 Effective length in general

The buckling resistance of a column depends not only on the column physical unrestrained length, but also the restraint conditions at column ends. Assessment of effective length affects greatly the buckling resistance, but it is unfortunately assessed by most engineers arbitrarily as 1. The Code provides some guidelines on the approximate assessment of effective length. For more accurate determination of buckling strength, the second-order elastic and advanced analysis should be used. Figure E8.4 shows the elastic, squash and buckling strength with imperfection of columns with various slenderness ratio. The Perry constants for classes of steel other than the Class 1 steel can be obtained from curve-fitting of experimental results or from reliable buckling loads of columns made of that particular class of steel.

As an error in effective length affects not linearly but in a quadratic relation to the buckling resistance of a member, uncertain assumption of effective length should not be made and a second-order analysis should be employed for checking of stability. Table 8.6 of the Code should only be used for simple structures insusceptible to buckling.

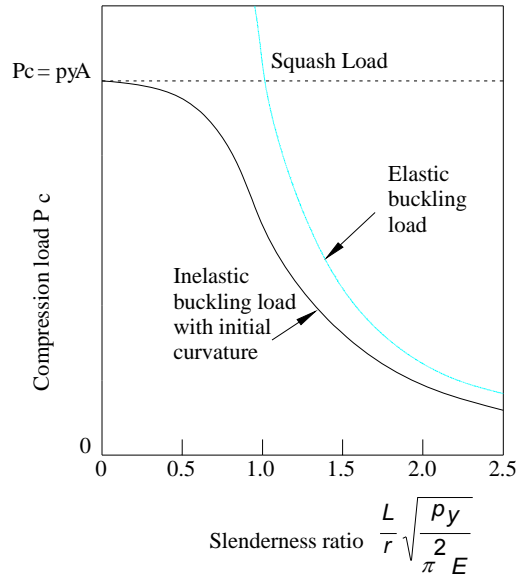


Figure E8.4 - Column buckling curves

Example 8.2

A 3 m high H-section steel column, which section is 254×254×73 UC in S275 steel material, supports only factored compression load 1000 kN. The column assumes to be fixed at the ground and top of column is held in position without rotational restrained as shown in the diagram. Check the compression capacity of the specified steel column.

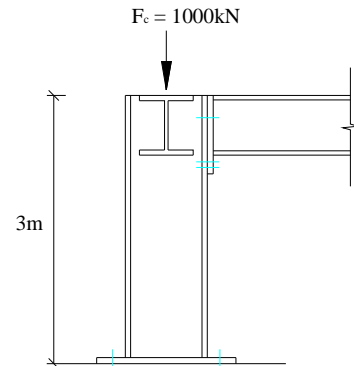
Solution

SECTION PROPERTIES:

$D = 254\text{mm}$, $B = 254\text{mm}$, $t = 8.6\text{mm}$, $T = 14.2\text{mm}$,
 $d = 200.2\text{mm}$, $r_x = 11.1\text{cm}$, $r_y = 6.46\text{cm}$, $A_g = 92.9\text{cm}^2$

$b = 254 / 2 = 127\text{mm}$

$T = 14.2\text{mm} < 16\text{mm} \therefore p_y = 275\text{N/mm}^2$



SECTION CLASSIFICATION:

(Table 7.1)

For flange, $\frac{b}{T} = \frac{127}{14.2} = 8.9 = 9\varepsilon = 9$

\therefore flange is non-slender section

For web, $\frac{d}{t} = \frac{200.2}{8.6} = 23.3 < \frac{80\varepsilon}{1+r_1} = \frac{80}{3.11} = 25.73$

\therefore web is plastic section

use $40\varepsilon = 40$

Stress ratio, $r_1 = \frac{F_c}{d t p_{yw}} = \frac{1000 \times 10^3}{200.2 \times 8.6 \times 275} = 2.11$ (7.1)

$r_2 = \frac{F_c}{A_g p_{yw}} = \frac{1000 \times 10^3}{9290 \times 275} = 0.391$ (7.2)

\therefore the H-section column member is class 2 compact section

COMPRESSION CAPACITY:

Buckling of the column about minor axis is more critical,
Effective length, $L_E = 0.85L = 2.55m$ (Table 8.6)

Slenderness ratio, $\lambda = \frac{L_E}{r_y} = \frac{2550}{64.6} = 39.5$ (Clause 8.7.4)

(Maximum thickness < 40mm, Rolled H-section & buckling critical about minor axis)
Compressive strength bases on buckling curve (c) (Table 8.7)

Compressive buckling strength, $p_c = 238 N/mm^2$ (Table 8.8 (c))

Compression capacity, $P_c = A_g p_c = 238 \times 9290 \times 10^{-3}$ (8.73)
 $= 2211 kN > F_c = 1000 kN$ (OK!)



E8.7.6 Compressive strength

Because of residual stress, some parts of the sections are “pre-stressed” to a value of residual cooling stress and this leads to a reduction of buckling resistance, but not on beam capacity as residual stress is self-equilibrium. For grades with design strength below 460 MPa, the reduction is 20 N/mm².

E8.8 TENSION MEMBERS UNDER COMBINED AXIAL FORCE AND MOMENTS

In some cases where a beam is under tension, axial force and moment, it may buckle laterally even though the axial tension is counteracting against lateral torsional buckling. The clause requires the designer to check for the case when the favourable tension force is absent.

E8.9.2 Member buckling resistance

An unrestrained beam-column should be checked for three scenarios for buckling, namely they are the beam buckling, buckling due to P-Δ moment from change of structural geometry and buckling due to P-δ moments from member loads. It has been found that when sway is large, the amplification method in Equation 8.80 is more critical. On the other hand, the effective length method in Equation 8.79 is more critical when sway is small. Thus three equations of Equations 8.79 to 8.81 are required for checking. Equation 8.79 is for checking against sway due to P-Δ moment where effective length due to member initial curvature is in control, Equation 8.80 is for checking against the case where sway moment due to lateral force is in control and Equation 8.81 is for combined axial force and lateral-torsional beam buckling check. Equation 8.79 is more critical than Equation 8.80 when the lateral force is small and vice versa.

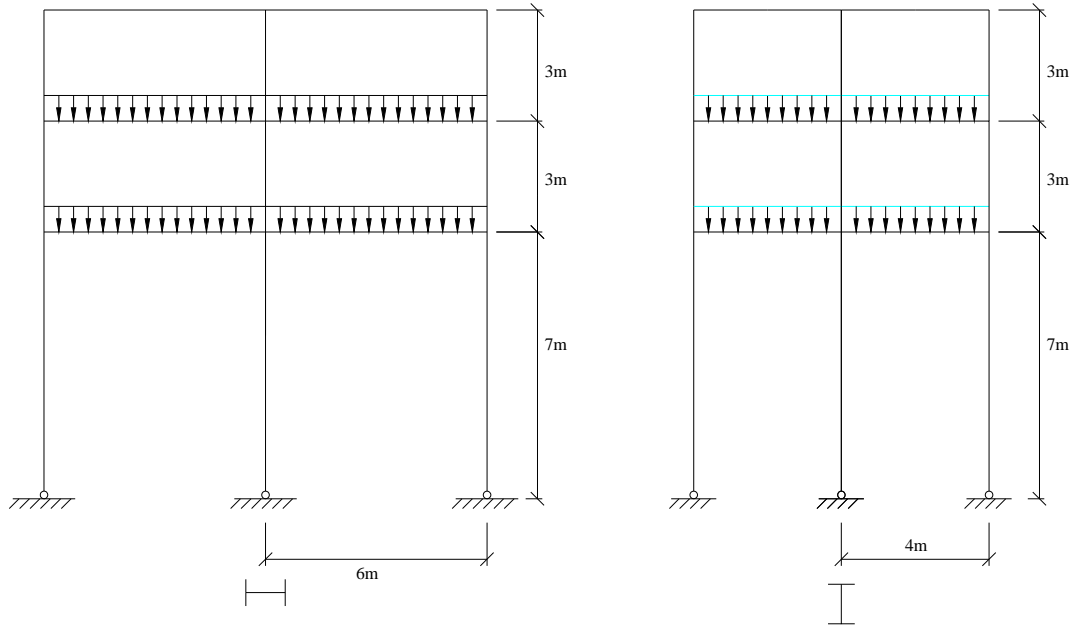
There has been some formulae putting the effect of axial force inside the moment resistance and they are more economical. This method assumes the centre core of material to take axial force with the remaining material to take moment (see Chan and Chui, 2000). However, these equations are only limited to plastic and compact sections and their use is complicated.

The interactive equations are the most widely ones with applicability covering different sectional types. However, for some sections like plastic and compact sections, interaction equations for economical design available in BS5950 (2000) or can be used.

Example 8.3

Compression member in unbraced multi-storey frame

The 3-storey frame of steel grade S275 and columns of 356x368x153 UC and beams of 356x127x33UB is a moment frame with rigid member connections and pinned bases. Check the compression resistance of the column. Dead load is 2kN/m² and Live load is 4.5kN/m².



Solution

$$\text{Design load} = 1.4 \times 2 + 1.6 \times 4.5 = 10 \text{ kN/m}^2$$

FRAME CLASSIFICATION:

The notional horizontal force due to factored total load is $10 \times 12 \times 8 \times 0.5\% / 2$ for centre node in each storey = 2.4kN in both directions. The notional force will be halved for end node in each storey. From linear analysis, the deflection about major and minor of the column under consideration are 3.26mm and 5.73mm respectively.

From frame classification,

$$\text{About major axis, } \lambda_{cr} = \frac{1}{200} \times \frac{h}{\delta_N} = \frac{7000}{200 \times 3.26} = 10.74 \quad \therefore \text{it is non-sway frame} \quad (6.1)$$

$$\text{About minor axis, } \lambda_{cr} = \frac{1}{200} \times \frac{h}{\delta_N} = \frac{7000}{200 \times 5.73} = 6.11 \quad \therefore \text{it is sway frame} \quad (6.1)$$

COMPRESSION RESISTANCE:

The boundary condition of the interior column should be assessed by considering it as a column in a sub-frame, while the stiffness of the connected beam contributing to the effective length of interior column is taken into account in the frame. Thus the determination of effective length of the interior column allows for the effect of restraining beam stiffness here.

The second moment of inertia of connecting beam (356x127x33 UB) is $I_x = 8192 \text{ cm}^4$

The second moment of inertia of column (356x368x153 UC) is $I_x = 48640 \text{ cm}^4$

About major axis, (non-sway frame)

Relative stiffness of beam in frame should be $1.0 \times \frac{I_x}{L}$ (Table 6.2)

$$\text{Stiffness of connected beam, } K_{11} = K_{12} = \frac{I_{bx}}{L} = \frac{81.92 \times 10^6}{6000} = 13653.3 \text{ mm}^3$$

Stiffness of designed column, $K_c = \frac{I_x}{L} = \frac{486.4 \times 10^6}{7000} = 69485.7 \text{ mm}^3$

Stiffness of upper column, $K_1 = \frac{I_x}{L} = \frac{486.4 \times 10^6}{3000} = 162133.3 \text{ mm}^3$

$$k_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}} = \frac{69485.7 + 162133.3}{69485.7 + 162133.3 + 2 \times 13653.3} = 0.895 \quad (\text{Figure 6.3})$$

$k_2 = 1$ (for pinned end)

Effective length, $L_E = 0.96L = 0.96 \times 7 = 6.72 \text{ m}$ (Figure 6.4 (b))

Slenderness ratio, $\lambda = \frac{L_E}{r_x} = \frac{6720}{158} = 42.5$ (Clause 8.7.4)

Maximum thickness < 40 mm

Compressive strength based on buckling curve (b) (Table 8.7)

For $T = 20.7 \text{ mm} > 16 \text{ mm}$, $p_y = 265 \text{ N/mm}^2$

Compressive buckling strength, $p_c = 239 \text{ N/mm}^2$ (Table 8.8 (b))

About minor axis, (sway frame)

Stiffness of connected beam, $K_{11} = K_{12} = \frac{I_{bx}}{L} = \frac{81.92 \times 10^6}{4000} = 20480 \text{ mm}^3$

Stiffness of designed column, $K_c = \frac{I_y}{L} = \frac{175.1 \times 10^6}{7000} = 25014.3 \text{ mm}^3$

Stiffness of upper column, $K_1 = \frac{I_y}{L} = \frac{175.1 \times 10^6}{3000} = 58366.7 \text{ mm}^3$

$$k_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}} = \frac{25014.3 + 58366.7}{25014.3 + 58366.7 + 2 \times 20480} = 0.67 \quad (\text{Figure 6.3})$$

$k_2 = 1$ (for pinned end)

Effective length, $L_E = 2.9L = 2.9 \times 7 = 20.3 \text{ m}$ (Figure 6.4 (a))

Slenderness ratio, $\lambda = \frac{L_E}{r_y} = \frac{20300}{94.6} = 214.5$ (Clause 8.7.4)

(Maximum thickness < 40 mm)

Compressive strength based on buckling curve (c) (Table 8.7)

Compressive buckling strength, $p_c = 36.5 \text{ N/mm}^2$ (Table 8.8 (c))

Therefore, the compression buckling of the column is critical about minor axis.

$$\begin{aligned} \text{Compression capacity, } P_c &= p_c A_g = 36.5 \times 19600 \times 10^{-3} && (8.73) \\ &= 715.4 \text{ kN} > F_c = 480 \text{ kN} && (\text{OK!}) \end{aligned}$$

The compression resistance of the column in the frame is greatly reduced when the frame is changed from the non-sway mode to the sway mode, because of the variation of the P-Δ effect.

E8.10 TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF COMPRESSION MEMBERS

For short, thin-walled and open sections, torsional buckling check is needed.

E8.11 PORTAL FRAME

Portal frames are simple single storey structures for sheltering and its design to BS5950 (2000), on which this part of the Code depends, is available in many standard texts such as “Design of single-span steel portal frames to BS5950-2000”, The Steel Construction Institute, 2004.

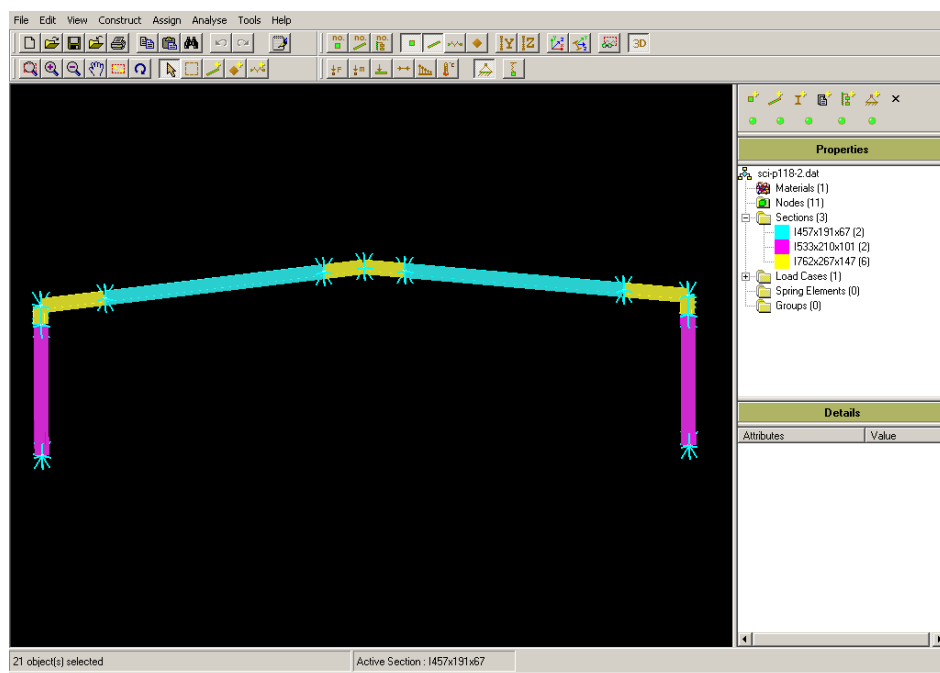
It is common to design a portal by plastic method and below is an example of plastic design by computer software.

Before the availability of fast-speed and low-cost personal computers, manual methods are used to design portals and special consideration should be paid to plastic hinges, column buckling and snap-through buckling of rafters. The use of advanced analysis has made the design simple and straight-forward to date. Below is an example of using the advanced analysis in designing a portal frame for which a design book requires extensive work for its analysis and design.

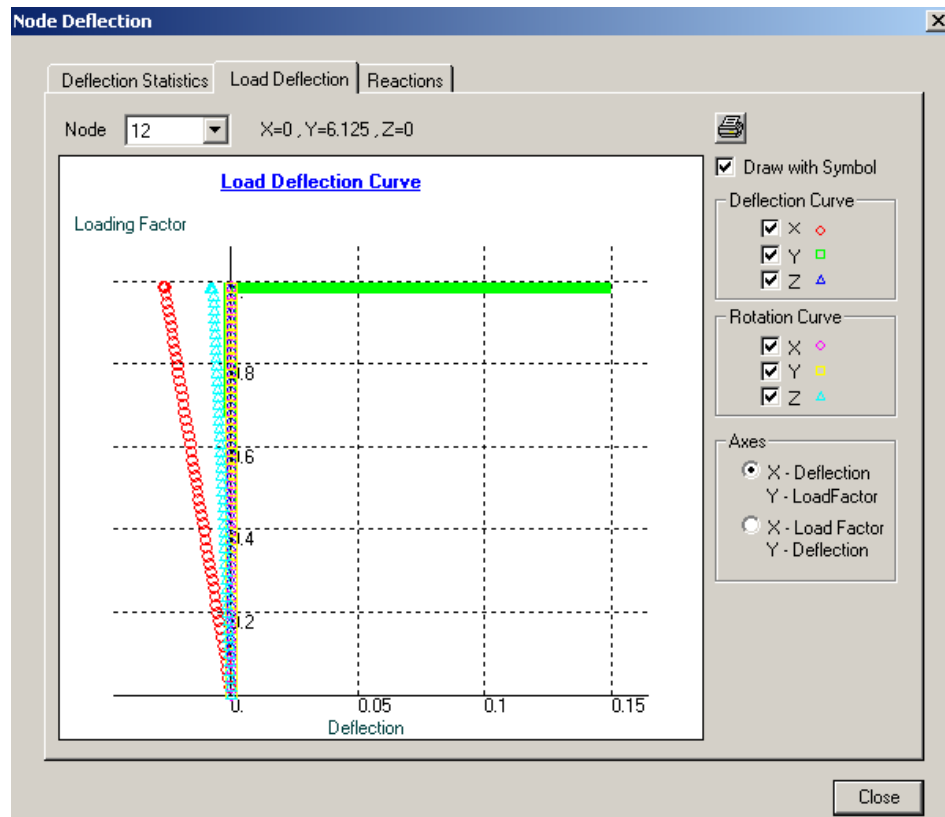
Example :

An advanced analysis below demonstrates the design of a portal using 1 element per rafter or column. The eave height of the portal is 7 m and the rise is 1.58m with its span 30 m. The rafter is 457x191x67 UB and the column is 533x210x101UB, both in S275 steel. The haunch is not expected to have yielding, and thus simulated by use of very large I-sections. The factored design load is 11.30 kN/m. The example is compared with the hand method in “Design of single-span steel portal frames to BS5950-2000”, The Steel Construction Institute, 2004, pp.116 based on the Code, which is more difficult and time consuming to apply.

The calculated design load is the same for both analyses. The advanced analysis indicates that the maximum design load factor is equal to 1 after which the load factor cannot be increased further.



The layout of the portal frame



The Load vs. deflection plot showing the maximum load is just equal to the design load

E8.12 LACED AND BATTENED STRUTS

Laced and batten struts are compound sections used to increase the second moment of area against buckling under compression. The arrangement of spacing and angles of lacings and battens should meet the requirement in order to prevent local buckling failure before the system buckles.

REFERENCES

Chan, S.L. and Chui, P.P.T., "Non-linear static and cyclic analysis of steel frames with semi-rigid connections", Elsevier, 2000, pp336.

E9 CONNECTIONS

E9.1 GENERAL

This section of the Code is based mainly on the BS5950 (2000). The commonly used welded and bolted connections are studied.

Typical connections used in steel buildings are shown below. They represent different degrees of strength, stiffness and rotation capacities.

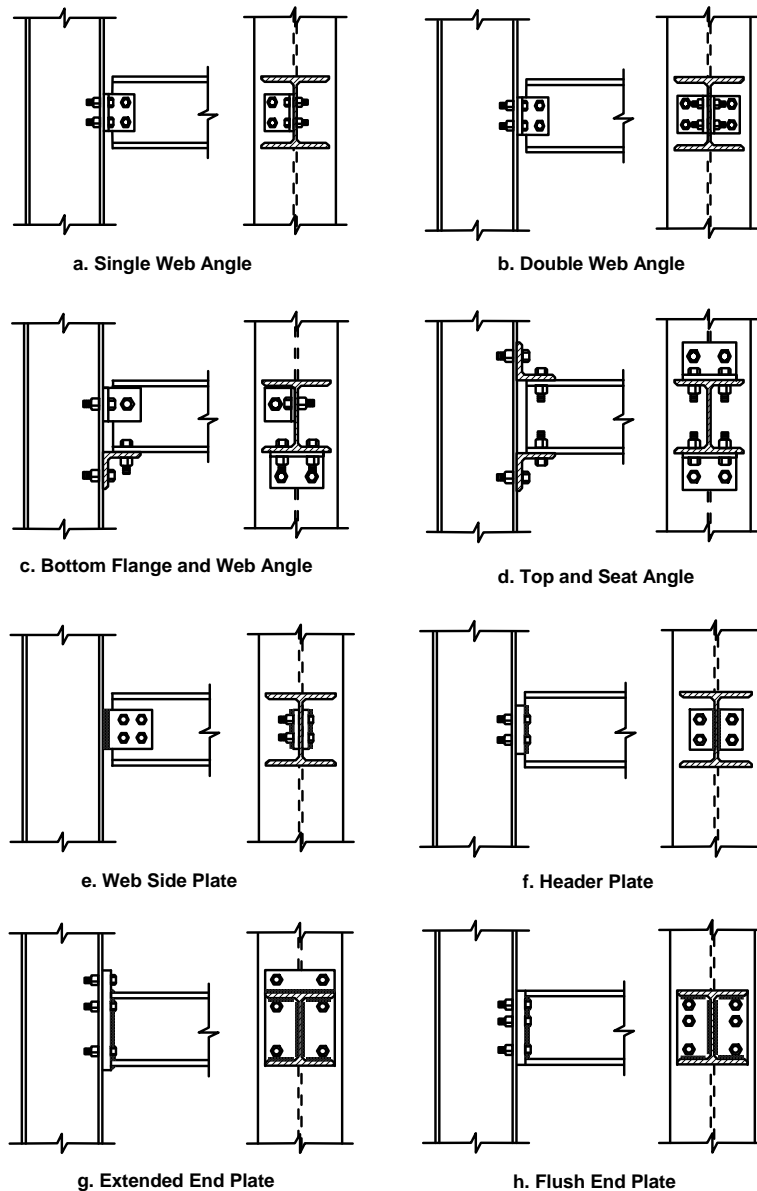


Figure E9.1 - Typical connections used in steel buildings

Most connections are assumed either pinned or rigid. Another form is the semi-rigid connection which, however, is more important for seismic design since it generally possesses a non-linear moment-rotation curve and absorbs energy. Practical connections will somehow transmit some but not all bending moment to other members and this may affect the load path in the structure. Thus, the effect must be allowed for in design and analysis. The effects of moment re-distribution affect the stress distribution state in a structure and must be allowed for in the design, or they should be proven to be insignificant. Nevertheless, semi-rigid connections are uncommon in Hong Kong.

In general, connections are required to transmit forces and moments and to accommodate rotations. In the Code, the rigid connection is designed to resist moment and force while the pinned connection is designed to resist force and accommodate rotations in simple construction.

Rotation of connections can be provided by using elements in connections which can deform appreciably. Detailing of connections should, as practical as possible, allow deformation to take place or moment to be transmitted. Rigid connections normally have connecting elements away from the centre of rotation in order to provide moment more effectively. On the other hand, connecting element such as cleats should be close to the centre of rotation for pinned connections in order to minimize resisting moment and rotation more easily.

Three connection types are available in the Code – they are the flexible connections, rigid connections and the semi-rigid connections. The stiffness should be detailed as close as possible to the assumption made in the analysis and design as it affects the moment and stress distributions in a framed structure. The strength of a connection should be capable of resisting the external load and the rotational capacity affects the maximum rotation that a connection can accommodate. The following figure shows the typical assumption for pinned, rigid and semi-rigid connections in terms of strength, stiffness and rotational capacities.

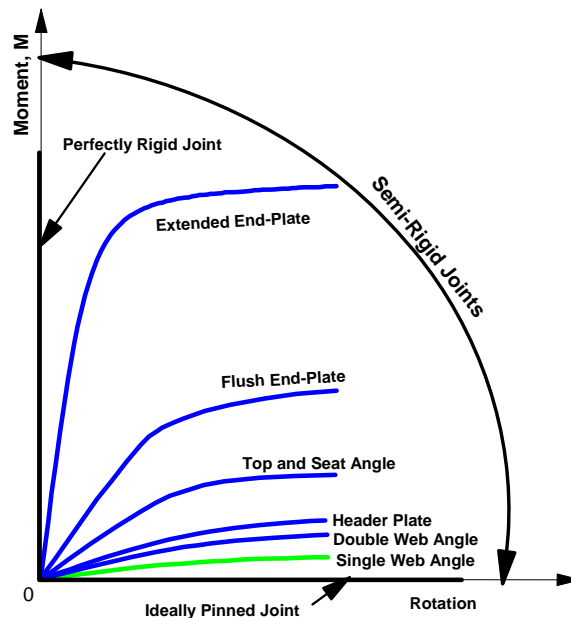


Figure E9.2 - Stiffness, Strength and Rotational Capacity of Pinned, Rigid and Semi-rigid connections

E9.2 WELDED CONNECTIONS

Butt weld and fillet welds are included in the section of welded connections. Butt welds are normally used for in-line jointing in plates or sections and fillet welds are used for tee or lap jointing.

Some precautions for avoiding problem in welding are given in this section of the Code. Explanation of the reason of action is given since following blindly an instruction without understanding the joint behaviour is dangerous.

E9.2.1 Through thickness tension

Lamellar tearing is a type of cracking perpendicular to the thickness dimension of a plate. Tension force on the surface of a thick plate can create lamellar tearing easily. Two methods are available to prevent this type of failure, the first method is the connection

detailing to avoid direct lamellar tearing of steel plate, and the second method is to use steel of sufficient strength in the transverse direction of the steel plate confirmed by the through-thickness tearing test when this transverse stress is greater than 90% of the design strength of the steel material.

Relevant standards of Australia, China, Japan and United States of America in mitigating the risk of lamellar tearing which will be updated in Annex A2.3.

Please also make reference to the latest articles on lamellar tearing of steel plates.

E9.2.2 Types of welds

The design strength of fillet welds should follow Table 9.2 of the Code for the lowest grade of material joined. In principle, the vector sum of applied stress should not be greater than the design strength of weld.

The strength of a butt weld should be taken as that of the parent metal. The throat thickness of partial penetration should be taken as depth of penetration -3mm for V (bevel) weld, and for J or U weld the same depth of penetration applies but it should not be less than $2\sqrt{t}$, where t is the connected plate thickness.

For weld illustration and symbolic representation, refer to BS EN 22553.

E9.2.3 Weldability and electrodes

Single sided fillet or partial penetration butt welds shall not be used to transmit a bending moment about the longitudinal axis of weld.

E9.2.4 Welded connections to unstiffened flanges

In a tee-joint of a plate to an unstiffened flange of an I, H or a box section, a reduced effective breadth shall be taken into account both for the parent material and for the welds. This consideration is to account for flexibility of connecting plates.

This criterion of effective breadth is valid for connections subjected to compression or tension. However, for connection under compression, web crippling should always be checked.

E9.2.5 Strength of weld

Fillet welds

Fillet weld is more commonly used because its cost is lower than full penetration weld. Fillet weld does not require end preparation of the element to be welded and the size or leg length is smaller. The amount of testing required for fillet weld is also smaller.

The Code provides information on maximum and minimum size of fillet welds.

BS 5950 (2000) and BS5400 allow deep penetration in fillet welds. They permitted a depth of $0.2a$ up to 2 mm in addition to the effective throat thickness 'a' only for submerged arc weld. AWS allows up to 3 mm as credit for penetration beyond the root. Here, this extra penetration is ignored in local practice and in consideration of the fact that the actual depth of penetration cannot be easily detected and verified by non-destructive tests. In practice, a minimum 3 mm weld electrode is used.

Effective length

Effective length is used to determine the length for calculation of strength of a fillet weld. The Code provides guidance on the determination of effective length.

End returns

The length of end return is specified in the Code. When electrode of grade different from that of the welded parent steel material is used, the lower strength should be used. Table 9.2a and 9.2b of the Code show the combined strength of welded parts in a welded connection.

Strength of fillet weld

Design strength of fillet weld is taken as 0.5 of ultimate strength of fillet weld metal satisfying BS EN standard and whereas 0.38 of ultimate strength of fillet weld metal is taken for satisfying GB standard. Weld should not be allowed to bend on its own axis.

Capacity of fillet welds

Two methods are available to calculate the strength of a weld group, namely (a) the simple method and (b) the directional method. The tensile strength of a weld is greater than its shear strength. Method (b) considers this fact and gives a more veracious behaviour on its characteristic. However, to define the angle θ requires prolonged calculation. The simplified method (a) is more conservative and simple to use.

Lap joints

This provision does not apply to full weld length of connected parts of uniform load distribution e.g. weld between web and flange of plate girder.

Penetration weld

Full penetration welds

A full penetration weld is a butt that has complete penetration and fusion of weld with parent metal throughout the thickness of the joint. In the use of matching welding consumables, which have all mechanical properties equivalent to, or better than, those specified for the parent metal, the design strength of full penetration butt weld can be taken as equal to the parent metal. If parent materials are of different grades, the lower grade steel should be assumed for all connected plates. However, the electrode for the highest grade should be used.

Two common types of butt welds are available as U and V butt joints. U butt joint has a depth of penetration equal to the depth of weld penetration and V butt joint has a depth 2 mm less than the depth of weld penetration.

Partial penetration weld

When connecting strength is not required to be as high as the strength of the capacity of the connection components, a partial penetration butt weld can be used.

E9.3 BOLTED CONNECTIONS

Two common bolt types are the non-preloaded bolts and the preloaded high strength friction grip (HSFG) bolts. Mixed use of these two types of bolts should be avoided to prevent confusion and error on site.

Strength grades available are 4.6, 8.8 and 10.9 whereas 12.9 bolts are not recommended in the Code because of lack of ductility.

Connections should be designed to realistic assumption in the analysis and member and frame design. The elements of bolts or connecting plates should have adequate strength to transmit moment. The connection should have enough ductility to rotation in pinned connections and to allow redistribution of extra stresses due to lack-of-fit tightening in fasteners and residual stress. When members do not meet at the same point, eccentric moment should be considered in design. When bolts are subjected to vibration in their service life, security by locking nuts or HSFG bolts should be used. For bolts under reversed loads where bolt slip is unacceptable, HSFG bolts should be used.

E9.3.1 Bolt spacings

The Code gives recommendations on the maximum and minimum spacing and end distance of bolts. All distances should be measured from the centre of holes. Typically, the hole size is 2 mm greater than the bolt diameter greater than 24 mm. For slot holes,

the measurement should be made from the centerline of the slot or the centre of end radius.

The minimum bolt spacing is to ensure bolts are fully effective and maximum spacing is to base on prevention of local buckling and sharing of bolts among the bolt group.

Limiting the maximum end and edge distances is intended to minimize the infiltration of moisture, thus preventing corrosion between the connected parts. More restrictive limitation may be imposed for severe conditions but up to the discretion of the designer.

E9.3.4 Sectional area of connected parts

In order to account for the beneficial effect of strain hardening at connections, the sectional area is allowed to increase by a ratio of 1.2 for grade S275 steel and 1.1 for grade S355 steel.

E9.3.5 Block shear

The block shear is under a combined action of shear and tension. It prevents the bolt group from both being too close to the edge and the plate thickness too small to facilitate the block mode of failure. Shear capacity check for individual bolt may need to be checked as appropriate.

E9.3.6 Design strength of bolts in shear

The Code gives clauses to check the strength of connections against shear, bearing and tension on bolts and the connected parts.

One more criterion is added to evaluate the bearing capacity of the connected parts. Recent research indicates that the material bearing strength is limited by bearing deformation of the hole. Its normal bearing capacity P_{bs} is equal to Cd_tU_s and C is 2.0, 2.4 or 3.0 depending upon the hole type and/or acceptability of hole ovalization at the ultimate load. BS 5950-1 (2000) is considered as conservative here.

The constant C is an empirical constant to account for various conditions.

Long grip length

This provision applies to ordinary bolts e.g. grade 4.6 only. Tests have demonstrated that the ultimate shear strength of high-strength bolts with a grip length of 8 or 9 diameters (very unusual in general practice) is no less than that of similar bolts having much shorter grips.

The use of grade 4.6 bolts is recommended because they have greater ductility. As this connection type assumed great efficiency in load sharing, ductile bolts are required.

Bolts through packing

This provision does not apply to preloaded bolted (friction-type) connections.

Shear capacity of preloaded bolts

Only high grade bolts of strength equal to or greater than 8.8 can be used for preloaded bolts. The shear in pre-loaded bolts is not the shear stress in the bolts as for non-preloaded bolts, but it refers to the frictional slip resistance of the bolts. The Code gives recommendations on the frictional resistance of preloaded bolts for various conditions of prying condition.

The use of K_s as 0.7 is an improvement from 1990 version of BS5950.

E9.3.7 Design strength of bolts in tension

The Code gives recommendations of tension strength of bolts for non-preloaded bolts in this clause. Care should be taken against loosening of bolts by sufficient tightening torque, which can be available in user manual by manufacture or other fabrication recommendation.

Tension capacity of bolts

The Code gives recommendation on tension strength of various bolt grades.

Prying force

Prying force exists when a bolt is under a lever action causing a force larger than the simple direct tension force in the bolt.

To visualize the prying action, the flange of a T-section in Figure 9.11 of the Code can be considered as a continuous beam with four point supports under a point load at its mid-span. The reactions in the supports are dependent on the beam stiffness.

It is noted that the checking of moment capacity of connected plate at the bolt line for prying force requirement as indicated on BS 5950 is not always adequate. The prying force is mostly determined by the stiffness and not the stress of the connected parts.

The prying force shall be calculated

$$Q = \left(\frac{b}{2n} F_t - \frac{\beta \gamma p_0 w t^4}{27 n b^2} \right) > \frac{1}{10} F_t \quad (\text{DN9.1})$$

where (see Figure 9.11)

F_t = half of the total applied tension

t = thickness of flange

w = effective width of flange, it may be taken as the distance between adjacent bolts or $3d$, here d is the diameter of the bolt.

b = distance between the center of bolt to toe of fillet weld or half of root radius

P_0 = minimum proof stress in bolt obtained from BS 3692

β = 1.0 for preloaded bolts
2.0 for non-preloaded bolts

γ = 1.5

$$n = 1.1 t \sqrt{\frac{\beta P_0}{p_y}} \quad (\text{DN9.2})$$

This formula for calculating the prying force is semi-empirical. Although it has theoretical basis, it is modified to agree with the results of laboratory tests. (see Structural Steelwork Connections by G.W. Owens.)

Further information can be found in the book Holmes M. and Martin L.H. "Analysis and design of structural connections" Ellis Horwood, 1983.

In formulae for pinned connections, the Code considers more detailed effect such as anchorage design than BS5950. This work is commonly used in construction.

E9.3.8 Combined shear and tension

Interaction equations are used for bolts under and free from prying force.

Theoretically, also proven by tests, the interaction of tension and shear forces can be closely defined by an ellipse. AISC and CISC consider this theory in their codes. However, BS5950 simply assumes this interaction as linear interaction. Here, the linear interaction is used conservatively.

Tension combined with preloaded bolts

In this type of connections, the gripping force will be weakened by tension force. But it is still assumed that there is no slip between the connected parts under shear force.

E9.3.9 Bolts combined with welds

Because of the problem of load sharing, only preloaded bolts designed to be non-slip may be designed to share load with welds and the preloaded bolts are finally tightened in place after welding. Ordinary bolts cannot be assumed to share loads with welds.

E9.3.10 Pin connections

When a pinned connection with negligible moment is required, pinned connection can be used. Device to prevent loosening of pin should be used to make the pin stay in position during the working life of the structure. Special precaution should be exercised for removable pins. The bending stress of pin by considering the pin as a beam should be evaluated and checked against the strength of the pin.

When high strength stainless steel rods are used, the same principle can be applied here.

E9.4 Baseplate and anchor construction

The actual mechanism of bearing pressure under a base plate is very complex and therefore a simple assumption of uniform stress under a width along the stanchion is adopted. Maximum design stress of 275 MPa is imposed on steel base plate to prevent fracture and therefore high grade steel has no advantage. 0.6 instead of 0.4 of concrete cube strength is used in the Code, which is in line with the change in 2000 version of BS5950.

Holding down bolt

Holding down bolts are widely used for connection between steel and concrete structures. The holding down bolts should be properly and effectively held in position.

For bonding and bearing of concrete, refer to BS 8110. Friction resistance is not allowed due to the fact that when erecting columns, the elevation and leveling / plumbing are usually adjusted either by leveling bolts or shim plates. They take up quite a large part of compression force. For shear force of great amount, shear key or other device should be used instead of impractically increasing the size of anchor bolts.

Column baseplate

The maximum stress induced in concrete foundation should not be larger than $0.6f_{cu}$, in which f_{cu} is the concrete 28 day cube strength to the Code of Practice for the Structural Use of Concrete or the strength of bedding material. The thickness of base plate should limit the stress in the plate to p_{yp} , but not larger than 275 N/mm^2 . Thickness should be designed to prevent from brittle fracture, if such steel base plate is exposed to external weather conditions. Axial force and moments should be considered for stress computation and holding down bolts may be used to resist tension in base plate. Lateral load should be included in the design.

BS 5950 (2000) uses $0.6 f_{cu}$ and effective area method, which are of great difference with the method in 1990 version. The more economical use of greater concrete strength should be adopted because there has been no reported problem since the use of 2000 version of BS5950.

Anchor bolt

Anchor bolt systems are widely used in Hong Kong for construction of steel canopy and curtain wall. The present approach is not available in BS 5950 and it is adopted from Canadian Code S16.1-94. Anchor bolts can be of two types, the first type being self-fabricated anchor bolt relying on the bond length and strength of embedded bars inside concrete, and the second type refers to factory made type anchors bolts. For the former type, sufficient bond length and strength of anchorage bars are required and the checking is given in the Code. For the latter type of factory made anchors, the recommendations by the manufacturer should be strictly followed.

Drill-in bolts constitute many collapse cases of canopy and scaffolding in Hong Kong since the on-site quality is difficult to control. Many installed drill-in bolts do not have sufficient embedded length when in use because of clashes with steel reinforcement.

Quality assurance test such as pull-out test of anchor bolts is required to ensure the safety of anchorage systems since many failures of anchor bolts are due to bad quality such as the embedded material of insufficient strength and inadequate embedment length.

E10 COMPOSITE CONSTRUCTION

E10.1 MATERIALS

E10.1.1 Structural steel

Refer to clause 3.1 of the Code for the material properties of the structural steel sections.

E10.1.2 Concrete

This section of the Code has adopted the essence of Eurocode 4, BS 5950: Part 3 and some other international codes, which are written in accordance with the latest researches and developments in steel construction, in formulating the design guides for composite elements.

During the literature review stage, it was noted that some codes of practice, e.g. BS 5400: Part 5, had not specified the design limitations such as concrete cube compressive strength and characteristic strength of steel. After verification with the latest researches and code developments, the limitations in strength of concrete and steel as stipulated in the Code are considered as appropriate.

Should the stipulated limitations be exceeded, adequate proof test(s) should be carried out to verify the design assumptions and demonstrate the compatibility of steel and concrete materials as well as the validity of the design formulae.

Information on the elastic modulus, creep coefficient, shrinkage coefficient and coefficient of thermal expansion for concrete may be obtained from the Code of Practice for the Structural Use of Concrete. The minimum grades are generally in line with the minimum grades recommended in the Code of Practice for the Structural Use of Concrete. They do not apply to existing structures.

E10.1.3 Reinforcement

Clause 10.1.3 of the Code is self-explanatory.

E10.1.4 Shear connectors

The shear strength of shear connectors is a function of the concrete strength, connector type and the weld, and is related to the diameter of the connector. The head of the stud is to prevent uplift. The common diameter of stud, which can be welded easily on site, is 19 mm and supplied in 75, 100 or 125 mm heights.

Although adequate deformation capacity is not defined in the Code, the shear connectors should be able to reach their maximum capacity at slips (relative displacement between the slab and beam) of 2 to 3 mm and retain this load fully to total slip of at least 6 mm. Additionally, the drop-off in load capacity at higher slips should not be excessive. A suggested criterion is that the load capacity at 8 mm slip is not less than 80% of the maximum value. For good serviceability performance, the slip at half of the maximum capacity should be less than 0.5 mm.

To ensure effectiveness of the shear connections in preventing separation between the concrete and steel section, the connectors should be designed to resist a nominal tensile force perpendicular to the plane of steel flange of at least 0.1 times the design shear resistance of the connectors.

E10.1.4.2 Other types of shear connectors

The requirement of deformation capacity other than the head shear stud given in Clause 10.3.2.2 of the Code should be determined by push out test as stated in the last sentence in Clause 10.3.2.1 of the Code.

General information for shear connector is given in Clause 6.6.1.1 of BS EN 1994-1-1. Details of push out test for shear stud are also given in Annex B of BS EN 1994-1-1 for reference.

E10.1.5 Profiled steel sheets

Exposed profiled steel sheeting acts as reinforcement and shall be adequately protected to resist particular atmospheric conditions. A zinc coating, if specified, should conform to the requirements of EN 10147 or with relevant standards in force. A zinc coating Z275 of total mass 275 g/m² (including both sides) is sufficient for internal floors in non-aggressive environment, but the specification may be varied depending on service conditions.

For details, refer to clause 3.8.1 and Section 11 of the Code.

E10.2 COMPOSITE BEAMS

E10.2.1 General

Sub-paragraph (2) of clause 10.2.1 of the Code specifies that steel section can be of yield strengths between 235 MPa and 460 MPa. However, similar clause in BS 5950: Part 3: Section 3.1: 1990 Cl. 3.1 limits steel section strength to 355 MPa with a note saying that the limitation is due to lack of test evidence using higher-strength steel. It also mentions in the first paragraph of the clause that if steel other than grades 43 and 50 (235 MPa and 355 MPa) is used, due allowance should be made for variations in properties including ductility. Please therefore note that if there are adequate tests, steel of 460 MPa can be comfortably used.

The sub-paragraph (2) only mentions Class 1 and 2 sections, that the design of which may be under the assumption that at the ultimate state the whole of the loading acts on the composite member. Class 3 and 4 are not included though similar clause is included in BS 5950: Part 3: Section 3.1: 1990 Cl. 5.1.2. (It is noted that at least Class 3 can be used as per 10.2.1(2) of the Code".)

The sub-paragraph (8) of clause 10.2.1 of the Code only mentions Class 1 and 2 sections, the design of which may be subject to the assumption that, at the ultimate state, the whole of the loading acts on the composite member. Class 3 and 4 are not included though similar clause in BS 5950: Part 3: Section 3.1: 1990 Cl. 5.1.2 is stipulated. (It is noted that at least Class 3 can be used as per 10.2.1(2) of the Code.)

E10.2.2 Analysis of internal forces and moments

Clause 10.2.2 of the Code is self-explanatory.

E10.2.3 Establishment of composite cross-sections

Effective breadth of slab

In a T beam, the construction of the concrete flange is limited by the influence of "shear lag" associated with in-plane strains within the slab. The effective breadth of the slab is not a precise figure, as it depends on the form of loading and the position along the beam. For compatibility between designs at the ultimate and serviceability limit states, the effective breadth is taken as span/4 for internal beams (divided equally between each side of the beam), but not exceeding the actual slab width considered to act with each beam.

When the slab (and hence decking) spans in the same direction as the beams under consideration, allowance is made for the combined flexural action of the composite slab and the composite beam by limiting the effective breadth to 80% of the actual breadth.

E10.2.5 Section capacities and properties of cross-sections

Resistances

Plastic moment capacity of composite beam

The plastic moment capacity is expressed in terms of the resistance of various elements of the beam as follows:

Resistance of concrete flange,	$R_c = 0.45 f_{cu} B_e (D_s - D_p)$	(E10.1)
Resistance of steel flange,	$R_f = B T p_y$	(E10.2)
Resistance of slender steel beam,	$R_n = R_s - R_v + R_o$	(E10.3)
Resistance of slender web,	$R_o = 38 \varepsilon t / 2 p_y$	(E10.4)
Resistance of shear connection,	$R_q = N Q$	(E10.5)
Resistance of reinforcement,	$R_r = 0.87 f_{sd} A_s$	(E10.6)
Resistance of steel beam,	$R_s = A p_y$	(E10.7)
Resistance of clear web depth,	$R_v = d t p_y$	(E10.8)
Resistance of overall web depth,	$R_w = R_s - 2 R_f$	(E10.9)

where

A	is the area of the steel beam;
A_s	is the area of the reinforcement in the effective cross section;
B	is the breadth of the steel flange;
B_e	is the effective breadth of the concrete flange;
D_p	is the depth of the profiled steel sheet;
D_s	is the overall depth of the concrete flange;
d	is the clear depth of the web;
f_{cu}	is the cube compressive strength of concrete;
f_{sd}	is the characteristic strength of the reinforcement;
N	is the actual number of shear connectors for positive or negative moments as relevant (minimum number, one side of the point of maximum moment);
p_y	is the design strength of structural steel (in N/mm^2);
Q	is the capacity of the shear connectors for positive or negative moments as relevant;
T	is the thickness of steel flange;
t	is the web thickness;
ε	is a constant $(275/p_y)^{1/2}$.

Positive moments, full shear connection

Full shear connection applies when R_q is greater than (or equal to) the lesser of R_c and R_s . In a composite section with full shear connection, where the steel beam has equal flanges, the plastic moment capacity M_c for positive moments is given by the following:

Case 1: $R_c < R_w$ (plastic neutral axis in web)

$$a) \quad \frac{d}{t} \leq 76\varepsilon \text{ or } \frac{d}{t} \leq \frac{76\varepsilon}{1 - R_c/R_v} \text{ (web compact)}$$

$$M_c = M_s + R_c \frac{(D + D_s + D_p)}{2} - \frac{R_c^2 d}{R_v 4} \quad (E10.10)$$

$$b) \quad \frac{d}{t} > \frac{76\varepsilon}{1 - R_c/R_v} \text{ (web not compact)}$$

$$M_c = M_s + R_c \frac{(D + D_s + D_p)}{2} - \frac{R_c^2 + (R_v - R_c)(R_v - R_c - 2R_o) d}{R_v 4} \quad (E10.11)$$

where

D	is the overall depth of the steel beam;
M_s	is the plastic moment capacity of the steel beam.

Case 2: $R_c \geq R_w$ (plastic neutral axis in flange)

a) $R_s > R_c$ (plastic neutral axis in steel flange)

$$M_c = R_s \frac{D}{2} + R_c \frac{(D_s + D_p)}{2} - \frac{(R_s - R_c)^2 T}{R_f 4} \quad (\text{E10.12})$$

Note: The last term in this expression is generally small.

b) $R_s \leq R_c$ (plastic neutral axis in concrete flange).

$$M_c = R_s \left\{ \frac{D}{2} + D_s - \frac{R_s (D_s - D_p)}{R_c} \right\} \quad (\text{E10.13})$$

Positive moments, partial shear connection

Partial shear connection applies when R_q is less than both R_c and R_s . In a composite section with partial shear connection, where the steel beam has equal flanges, the plastic moment capacity M_c for positive moments is given by the following:

Case 3: $R_q < R_w$ (plastic neutral axis in web)

a) $\frac{d}{t} \leq 76\epsilon$ or $\frac{d}{t} \leq \frac{76\epsilon}{1 - R_q/R_v}$ (web compact)

$$M_c = M_s + R_q \left\{ \frac{D}{2} + D_s - \frac{R_q (D_s - D_p)}{R_c} \right\} - \frac{R_q^2 d}{R_v 4} \quad (\text{E10.14})$$

b) $\frac{d}{t} > \frac{76\epsilon}{1 - R_q/R_v}$ (web not compact)

$$M_c = M_s + R_q \left\{ \frac{D}{2} + D_s - \frac{R_q (D_s - D_p)}{R_c} \right\} - \frac{R_q^2 + (R_v - R_q)(R_v - R_q - 2R_o) d}{R_v 4} \quad (\text{E10.15})$$

Case 4: $R_q \geq R_w$ (plastic neutral axis in flange)

$$M_c = R_s \frac{D}{2} + R_q \left\{ D_s - \frac{R_q (D_s - D_p)}{R_c} \right\} - \frac{(R_s - R_q)^2 T}{R_f 4} \quad (\text{E10.16})$$

Negative moments

In a composite section where the steel beam has equal flanges, the plastic moment capacity M_c for negative moments is given by the following:

Case 5: Plastic neutral axis in web

a) $\frac{d}{t} \leq 38\epsilon$ or $\frac{d}{t} \leq \frac{76\epsilon}{1 - R_r/R_v}$ (web compact)

i.e. $R_r < R_w$

$$M_c = M_s + R_r \left\{ \frac{D}{2} + D_r \right\} - \frac{R_r^2 d}{R_v 4} \quad (\text{E10.17})$$

b) $\frac{d}{t} > 38\epsilon$ and $\frac{d}{t} > \frac{76\epsilon}{1 + R_r/R_v}$ (web not compact)

i.e. $R_r < R_o$

$$M_c = M_s + R_r \left\{ \frac{D}{2} + D_r \right\} - \frac{R_r^2 + (R_v + R_r)(R_v + R_r - 2R_o) d}{R_v 4} \quad (\text{E10.18})$$

where

D_r is the distance from the top of the steel beam to the centroid of the reinforcement.

Case 6: Plastic neutral axis in flange

a) $\frac{d}{t} \leq 38\epsilon$ (web compact)

i.e. $R_r > R_w$

(i) $R_r < R_s$ (plastic neutral axis in steel flange)

$$M_c = R_s \frac{D}{2} + R_r D_r - \frac{(R_s - R_r)^2 T}{R_f} \frac{1}{4} \quad (\text{E10.19})$$

(ii) $R_r \geq R_s$ (plastic neutral axis outside steel beam)

$$M_c = R_s \left(\frac{D}{2} + D_r \right) \quad (\text{E10.20})$$

b) $\frac{d}{t} > 38\epsilon$ (web not compact)

i.e. $R_r \geq R_o$

(i) $R_r < R_n$ (plastic neutral axis in steel flange)

$$M_c = R_n \frac{D}{2} + R_r D_r - \frac{(R_n - R_r)^2 T}{R_f} \frac{1}{4} \quad (\text{E10.21})$$

(ii) $R_r \geq R_n$ (plastic neutral axis outside steel beam)

$$M_c = R_n \left(\frac{D}{2} + D_r \right) \quad (\text{E10.22})$$

Adequacy of composite section

Compare the moment capacity with the maximum ultimate moment. For beams subjected to uniform loading, the ultimate shear force at the ends of the span should not exceed V_c (where V_c is the lesser of the shear capacity and the shear buckling resistance, both determined from section 8 of the Code). For beams subjected to heavy point loads, it is necessary to carry out additional checks as clause 10.2.6.3 of the Code.

Second moment of area

For a composite section in which the steel beam has equal flanges, the second moments of area are given by:

$$I_g = I_x + \frac{B_e (D_s - D_p)^2}{12 \alpha_e} + \frac{A B_e (D_s - D_p) (D + D_s + D_p)^2}{4 \{A \alpha_e + B_e (D_s - D_p)\}} \quad (\text{E10.23})$$

$$I_p = I_x + \frac{B_e y_e^3}{3 \alpha_e} + A \left(\frac{D}{2} + D_s - y_e \right)^2 \quad (\text{E10.24})$$

where

α_e is the effective modular ratio defined in clause 10.2.3.4 of the Code;

y_e is the depth of the elastic neutral axis below the top of the concrete flange, which is given by:

$$y_e = \frac{(D + 2D_s)}{1 + \left\{ 1 + \frac{B_e}{A \alpha_e} (D + 2D_s) \right\}^{1/2}} \quad (\text{E10.25})$$

$$I_n = I_x + \frac{A A_s (D + 2D_r)^2}{4(A + A_s)} \quad (\text{E10.26})$$

For a composite section in which the steel beam has equal flanges, the elastic section moduli are given by:

Case 1: Elastic neutral axis in concrete flange for composite cross-section under positive moment.

The elastic section modulus for the concrete flange, Z_p , and the elastic section modulus for the bottom flange of the steel member, Z_s , are given by:

$$Z_p = I_p \alpha_e / y_e \quad (\text{E10.27})$$

$$Z_s = I_p / (D + D_s - y_e) \quad (\text{E10.28})$$

This case applies when:

$$A < \frac{(D_s - D_p)^2 B_e}{(D + 2D_p) \alpha_e} \quad (\text{E10.29})$$

where concrete on the tension side of the elastic neutral axis is taken as cracked, and thus, the properties of the cracked section are used.

Case 2: Elastic neutral axis in steel member for composite cross-section under positive moment.

The elastic section modulus for the concrete flange, Z_p , and the bottom flange of the steel member, Z_s , are given by:

$$Z_g = I_g \alpha_e / y_g \quad (\text{E10.30})$$

$$Z_s = I_g / (D + D_s - y_g) \quad (\text{E10.31})$$

This case applies when:

$$A \geq \frac{(D_s - D_p)^2 B_e}{(D + 2D_p) \alpha_e} \quad (\text{E10.32})$$

where the concrete is uncracked, and thus, the properties of the gross section are used.

The depth y_g of the elastic neutral axis below the top of the concrete flange is given by the expression:

$$y_g = \frac{A \alpha_e (D + 2D_s) + B_e (D_s - D_p)^2}{2\{A \alpha_e + B_e (D_s - D_p)\}} \quad (\text{E10.33})$$

Case 3: Elastic neutral axis in steel member for composite cross-section under negative moment.

The elastic section modulus for the stress in the reinforcement, Z_r , and the bottom flange of the steel member, Z_s , are given by:

$$Z_r = I_n / y_r \quad (\text{E10.34})$$

$$Z_s = I_n / (D + D_r - y_r) \quad (\text{E10.35})$$

The depth y_r of the elastic neutral axis below the centroid of the reinforcement is given by the expression:

$$y_r = \frac{A (D + 2D_r)}{2(A + A_s)} \quad (\text{E10.36})$$

E10.2.5 (7) (e)

Please refer to Clause E10.3.3.

E10.3 SHEAR CONNECTION

E10.3.2 Design resistance of shear connectors

Clause 10.3.2 of the Code is self-explanatory.

E10.3.3 Provision of shear connectors

Partial shear connection

The degree of shear connection, N_a / N_p , is defined as the number of shear connectors provided (N_a) divided by the number required for full shear connection (N_p). Limits are placed on the degree of shear connection as in clause 10.3.3 of the Code.

For spans up to 10 meters, the degree of shear connection should not be less than 0.4. For spans between 10 and 16 meters, the degree of shear connection should exceed $(L - 6) / 10$ where L is the beam span in meters.

The moment capacity M_c of a composite section with partial shear connection is obtained from E10.2.5. This depends on the location of the plastic neutral axis (PNA) as indicated in Figure E10.4.

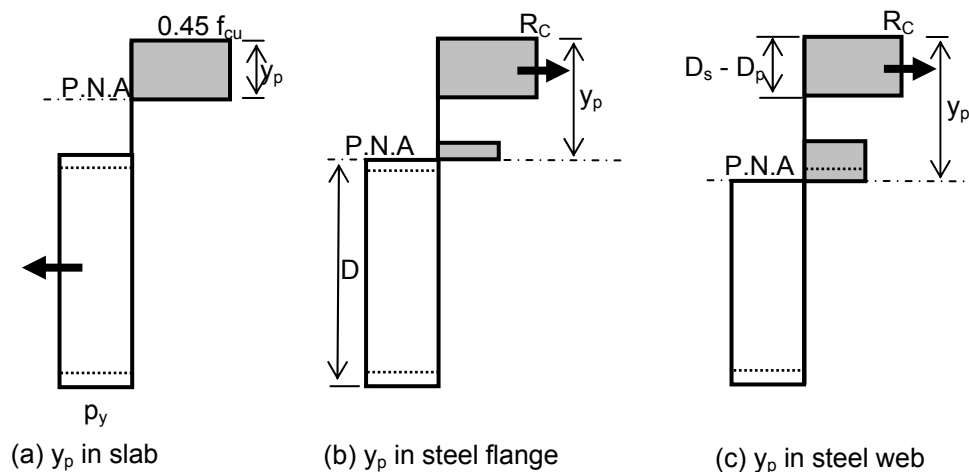


Figure E10.4 - Stress-block analysis of composite section for partial shear connection

E10.4 COMPOSITE SLABS WITH PROFILED STEEL SHEETS

Clause 10.4 of the Code is self-explanatory. Composite slabs with profiled steel sheets have been very popular in building construction in many parts of the world for more than 30 years, in particular, in high-rise steel-concrete composite framed buildings. In recent years, their use is becoming more and more popular in Hong Kong, not only in commercial high-rise buildings but also in low to medium-rise offices, long span footbridges and building envelopes. The structural behaviour of composite slabs with profiled steel sheets is covered in various parts of the British steel code BS5950. One of the major advantages of profiled steel sheets in the construction of floor slabs is that neither timber formwork nor temporary support is required as in conventional construction of reinforced concrete slabs. Moreover, profiled steel sheets also provide safe working platforms and effective protection to workers.

Construction loads are explained in BS5950: Part 4 as occurring during the concreting operation and are due to heaped concrete, the weight of operatives and equipment in general working areas. The construction load considered in the design of the decking is taken as 1.5 kN/m^2 . The loads carried by the profiled steel sheets take account of good construction practice. Care should be taken to ensure that the self-weight of profiled steel sheets, wet concrete and reinforcements and the construction loads are not exceeded by excessive heaping of concrete, or storage of materials or equipment on the profile steel sheets prior to concreting.

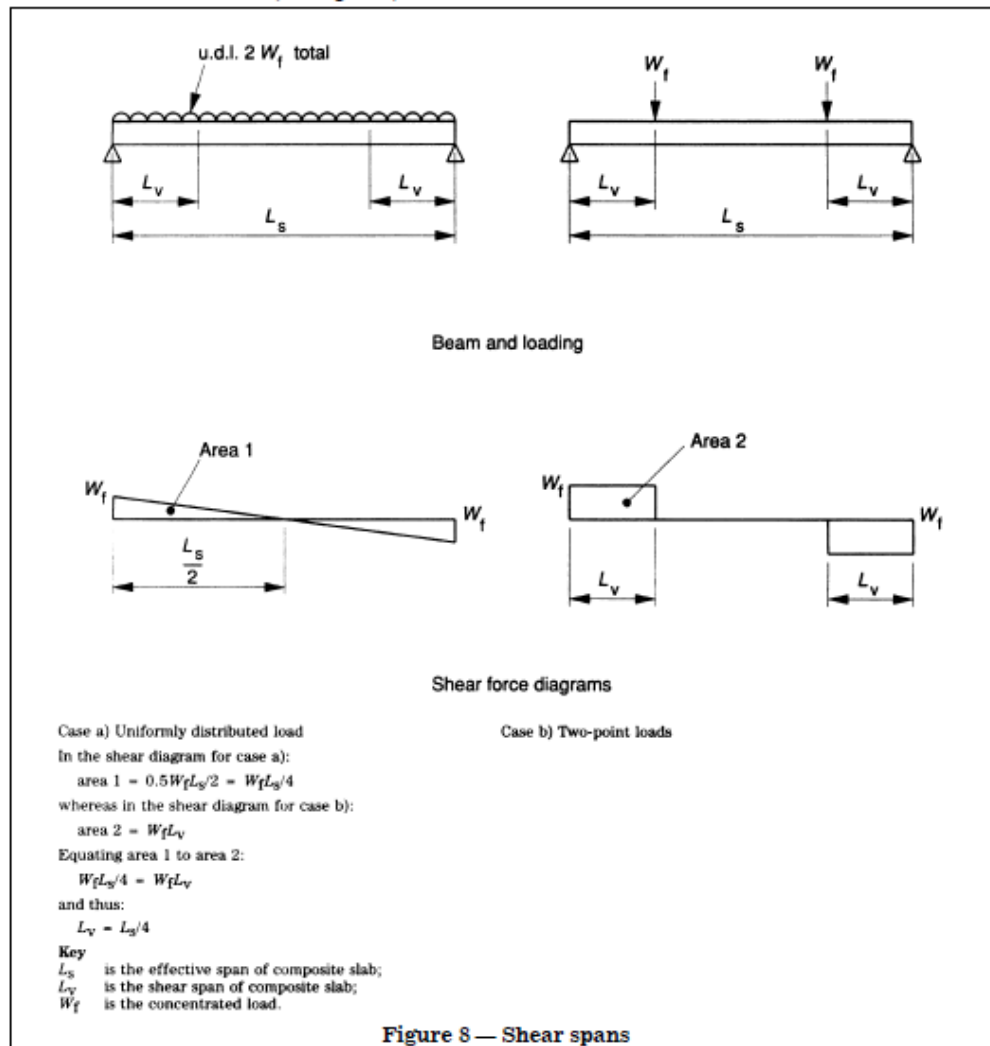
E10.4.3.5 Design of composite slabs in composite stage

The minimum concrete slab thickness does not include non-combustible screeds.

The loading arrangement, whether u.d.l. or point load, is not mentioned. Figure 8 of BS 5950: Part 4:1994 is inserted for guidance of determination.”

6.4.2 Shear span L_v

The shear span L_v should be taken as the distance from the support to the point within the span where at shear-bond failure a transverse crack in the concrete is deemed to occur (see Figure 8).



E10.4.7 Construction details

The rationale of specifying the minimum bearing length is referenced from Clause 4.6 of BS 5950-4.

E10.5 COMPOSITE COLUMNS

The first code of practice on composite construction in the UK is *CP117: Part 1: 1965 Composite construction in structural steel and concrete: Part 1: Simply supported beams in buildings*. In 1979, the design recommendations for steel-concrete composite columns using the limit state format became available in both Britain and the United States. In 1994, *Eurocode 4: Part 1.1* presents the latest recommendations for composite column construction using the methods based on the European buckling curves given in *Eurocode 3*. In China, the current edition of *GBJ17-88 Code of design of steel structures* also presents the general requirements in strength and stiffness for steel-concrete composite structures. Clause 10.5 of the Code aims to provide practical guidance to local practicing engineers in designing composite columns for local construction in accordance with British and European design recommendations fully compatible with local practice such as load factors and material factors.

Provision on ductility of composite columns should refer to the Code of Practice for the Structural Use of Concrete.

E10.5.3 Simplified method of design

In general, a composite column must be designed for the ultimate limit state. For structural adequacy, the internal forces and moments resulting from the most unfavourable load combination should not exceed the design resistances of the composite cross-sections. While local buckling of the steel sections may be eliminated, the reduction in the compression resistance of the composite columns due to overall buckling should be allowed for together with the effects of residual stresses and initial imperfections. Moreover, the second order effects in slender columns as well as the effect of creep and shrinkage of concrete under long term loading must be considered if they are significant. The reduction of flexural stiffness due to cracking of the concrete in tension should also be considered.

Clause 10.5.3 of the Code presents a simplified design method for composite columns in braced or non-sway frames with doubly symmetrical and uniform cross-section over the column length using the European buckling curves. This method shall comply with the recommendations given in clause 6.7 of Eurocode 4: Part 1.1: 2004. It should be noted that equation 10.69 of the Code presents the definition of the characteristic value of compressive capacity. However, η_a and η_c in equation 10.69c should be taken as 1.0 and 0.0 for infilled circular hollow sections respectively. This method is formulated in such a way that only hand calculation is required in practical design and the calculations of various design parameters for composite columns.

Design formulae for composite column

The moment capacity of a doubly symmetric composite cross-section may be evaluated as follows:

$$M_{cp} = \rho_y (S_p - S_{pn}) + 0.5 \alpha_c f_{cu} (S_{pc} - S_{pcn}) + f_{sd} (S_{ps} - S_{psn}) \quad (E10.37)$$

where

$\alpha_c = 0.53$ for all infill hollow sections

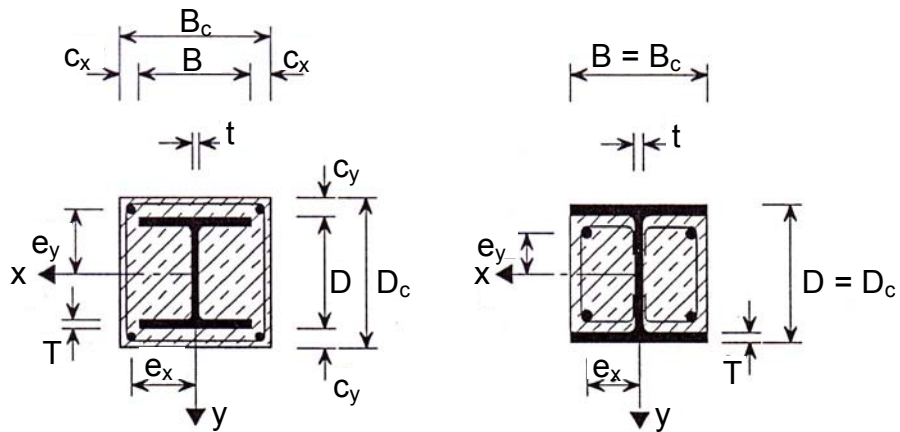
$= 0.45$ for fully or partially encased H sections

S_p, S_{ps}, S_{pc} are the plastic section moduli for the steel section, the reinforcement and the concrete of the composite cross-section respectively (for the calculation of S_{pc} , the concrete is assumed to be uncracked).

S_{pn}, S_{psn}, S_{pcn} are the plastic section moduli of the corresponding components within the region of $2 d_n$ from the middle line of the composite cross-section.

d_n is the depth of the neutral axis from the middle line of the cross-section.

Concrete encased steel sections



S_p are given in section property tables for steel sections

$$S_{ps} = \sum_i^n (A_{si} |e_i|) \quad (E10.38)$$

where

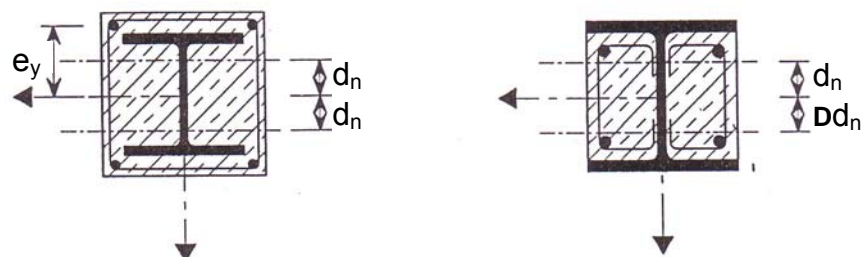
e_i are the distances of the reinforcements of area A_{si} to the relevant middle line

$$S_{psn} = \sum_i^n (A_{sni} |e_{ni}|) \quad (E10.39)$$

where

A_{sni} are the area of reinforcements within the region of $2 d_n$ from the middle line, and e_{ni} are the distances of the reinforcements from the middle line.

Major axis bending



$$S_{pc} = \frac{B_c D_c^2}{4} - S_p - S_{ps} \quad (\text{E10.40})$$

$$S_{pcn} = B_c d_n^2 - S_{pn} - S_{psn} \quad (\text{E10.41})$$

where

S_{pn} is defined as below.

The equations for the position of the neutral axis, d_n , are given for selected positions in the cross-sections. The resulting value of d_n should lie within the limits of the assumed region. A_{sn} is the sum of the reinforcement area within the region of $2d_n$.

Case a. Neutral axis in the web: $d_n \leq \left[\frac{D}{2} - T \right]$

$$d_n = \frac{A_c \alpha_c f_{cu} - A_{sn} (2f_{sd} - \alpha_c f_{cu})}{2B_c \alpha_c f_{cu} + 2t(2p_y - \alpha_c f_{cu})} \quad (\text{E10.42})$$

$$S_{pn} = t d_n^2 \quad (\text{E10.43})$$

Case b. Neutral axis in the flange: $\left[\frac{D}{2} - T \right] \leq d_n \leq \frac{D}{2}$

$$d_n = \frac{A_c \alpha_c f_{cu} - A_{sn} (2f_{sd} - \alpha_c f_{cu}) + (B-t)(D-2T)(2p_y - \alpha_c f_{cu})}{2B_c \alpha_c f_{cu} + 2B(2p_y - \alpha_c f_{cu})} \quad (\text{E10.44})$$

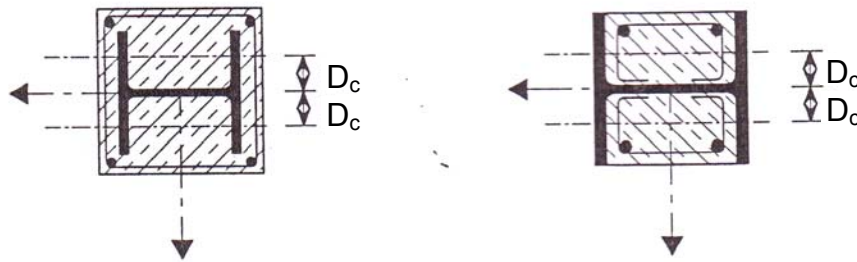
$$S_{pn} = B d_n^2 - \frac{(B-t)(D-2T)^2}{4} \quad (\text{E10.45})$$

Case c. Neutral axis outside the steel section: $\frac{D}{2} \leq d_n \leq \frac{D_c}{2}$

$$d_n = \frac{A_c \alpha_c f_{cu} - A_{sn} (2f_{sd} - \alpha_c f_{cu}) + A(2p_y - \alpha_c f_{cu})}{2B_c \alpha_c f_{cu}} \quad (\text{E10.46})$$

$$S_{pan} = S_{pa} \quad (\text{E10.47})$$

Minor axis bending



$$S_{pc} = \frac{D_c B_c^2}{4} - S_p - S_{ps} \quad (\text{E10.48})$$

$$S_{pcn} = D_c d_n^2 - S_{pn} - S_{psn} \quad (\text{E10.49})$$

where S_{pn} is defined as below.

The equations for the position of the neutral axis, d_n , are given for selected positions in the cross-sections. The resulting value of d_n should lie within the limits of the assumed region.

Case a. Neutral axis in the web: $d_n \leq \frac{t}{2}$

$$d_n = \frac{A_c \alpha_c f_{cu} - A_{sn}(2f_{sd} - \alpha_c f_{cu})}{2D_c \alpha_c f_{cu} + 2D(2p_y - \alpha_c f_{cu})} \quad (\text{E10.50})$$

$$S_{pn} = D d_n^2 \quad (\text{E10.51})$$

Case b. Neutral axis in the flange: $\frac{t}{2} \leq d_n \leq \frac{B}{2}$

$$d_n = \frac{A_c \alpha_c f_{cu} - A_{sn}(2f_{sd} - \alpha_c f_{cu}) + t(2T - h)(2p_y - \alpha_c f_{cu})}{2D_c \alpha_c f_{cu} + 4T(2p_y - \alpha_c f_{cu})} \quad (\text{E10.52})$$

$$S_{pn} = 2T d_n^2 - \frac{t^2(D - 2T)}{4} \quad (\text{E10.53})$$

Case c. Neutral axis outside the steel section: $\frac{B}{2} \leq d_n \leq \frac{B_c}{2}$

$$d_n = \frac{A_c \alpha_c f_{cu} - A_{sn}(2f_{sd} - \alpha_c f_{cu}) + A(2p_y - \alpha_c f_{cu})}{2D_c \alpha_c f_{cu}} \quad (\text{E10.54})$$

$$S_{pn} = S_p \quad (\text{E10.55})$$

For the calculation of the resistance P^E and M^E at the additional point E, the neutral axis should be located so that P^E is close to the average of P_{pm} and P_{cp} .

For a neutral axis in the flanges: $\frac{t}{2} \leq d_E \leq \frac{B}{2}$

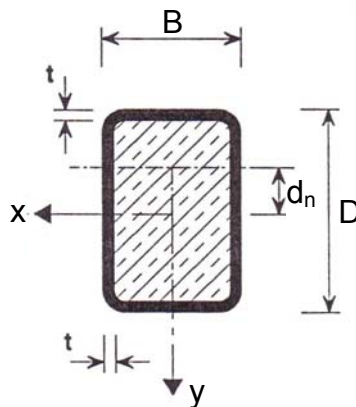
$$P^E = D_c(d_E - d_n)\alpha_c f_{cu} + 2T(d_E - d_n)(2p_y - \alpha_c f_{cu}) + A_{sE}(2f_{sd} - \alpha_c f_{cu}) + P_{pm} \quad (\text{E10.56})$$

Provided also that $\frac{t}{2} < d_n < \frac{B}{2}$ and A_{sE} is the sum of the areas of reinforcement lying in the additional compression region between d_E and d_n . The plastic section moduli are calculated by substituting d_n with d_E in the above relevant equations for S_{pn} and S_{pcn} .

Concrete filled hollow sections

Major axis bending

a) Rectangular hollow sections



$$S_{pc} = \frac{(B-2t)(D-2t)^2}{4} - \frac{2}{3}r^3 - r^2(4-\pi)\left[\frac{D}{2} - t - r\right] - S_{ps} \quad (\text{E10.57})$$

$$d_n = \frac{A_c \alpha_c f_{cu} - A_{sn}(2f_{sd} - \alpha_c f_{cu})}{2B \alpha_c f_{cu} + 4t(2p_y - \alpha_c f_{cu})} \quad (\text{E10.58})$$

$$S_{pcn} = (B-2t)d_n^2 - S_{psn} \quad (\text{E10.59})$$

$$S_{pn} = B d_n^2 - S_{pcn} - S_{psn} \quad (\text{E10.60})$$

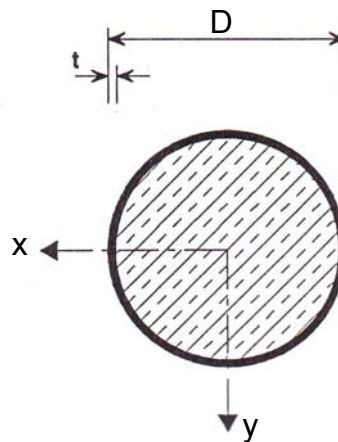
For the calculations of the resistances at the additional point E, P^E and M^E , the neutral axis is located half-way between d_n and the border of the section, so that $d_E = 0.5 d_n + 0.25 D$.

$$P^E = B(d_E - d_n)\alpha_c f_{cu} + 2t(d_E - d_n)(2p_y - \alpha_c f_{cu}) + A_{sE}(2f_{sd} - \alpha_c f_{cu}) + P_{pm} \quad (\text{E10.61})$$

where A_{sE} is the sum of the areas of reinforcement lying in the additional compression region between d_E and d_n .

The plastic section moduli are calculated by using the above equations for S_{pcn} and S_{pn} substituting d_n by d_E .

b) Circular hollow sections



The same equations given for the rectangular hollow sections may be used as a good approximation for the circular hollow sections by substituting $B = D$ and $r = D/2 - t$.

Minor axis bending

The same equations as for major axis may be used by exchanging the dimensions D and B as well as the subscripts y and z .

E10.5.4 Shear connection and load introduction

Clause 10.5.4 of the Code is self-explanatory.

E10.5.5 Detailing provisions

Clause 10.5.5 of the Code is self-explanatory.

E10.6 WORKED EXAMPLES

E10.6.1 Composite beam

Consider a simply supported composite beam and slab with steel decking of 12 m span and 2.8 m width as shown in Figure E10.16 under uniformly distributed load as follows:

Design Data

Dead load

125 mm slab (concrete density = 24 kN/m^3)	3.0 kPa
steel decking	0.2 kPa

Superimposed dead load

50 mm screeding (material density = 24 kN/m^3)	1.2 kPa
services	0.75 kPa
partition	1.0 kPa

Imposed load

Office area	3.0 kPa
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Steel beam

S355 steel section

Concrete

Grade 30 normal weight concrete

Shear studs

Shear stud 19 mm diameter with heads and 100 mm as nominal height

Decking

Breadth of trough, b_r	=	150 mm
Depth of decking, D_p	=	50 mm
Trough spacing	=	300 mm

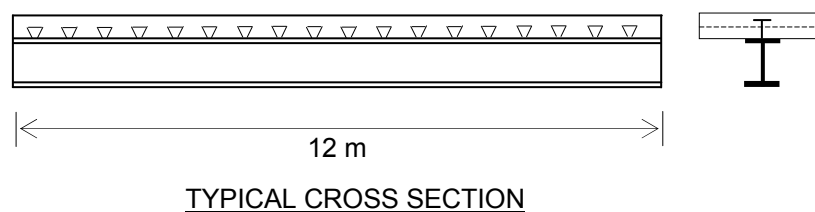
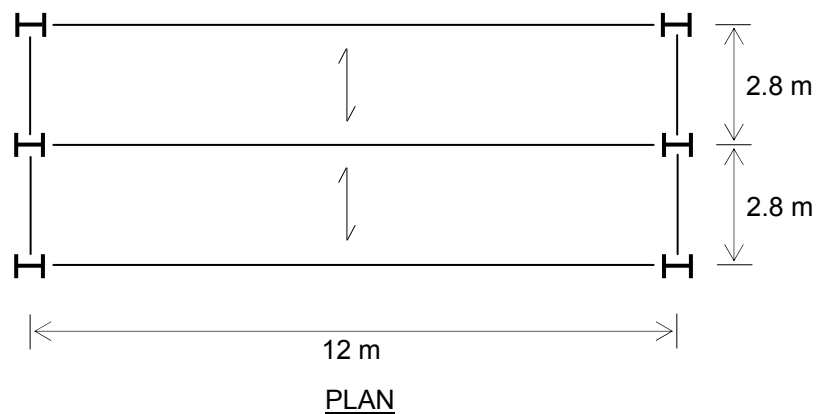


Figure E10.16 - Simply supported composite beam

Design loads for ultimate limit state

<u>Construction stage</u>	kPa
125 mm concrete slab	3.0
steel decking	0.2
steel beam (assume)	<u>0.2</u>
	3.4

$$\begin{aligned} \text{construction load} &= 4.5/2.8 \\ &= 1.6 \quad \geq 1.5 \end{aligned}$$

Clause 10.4.4.2(2)

$$\begin{aligned} W &= (1.4 \times 3.4 + 1.6 \times 1.6) \times 2.8 = 7.32 \times 2.8 = 20.5 \text{ kN/m} \\ M &= 20.5 \times 12^2 / 8 \\ &= \mathbf{369 \text{ kNm}} \end{aligned}$$

<u>Composite stage</u>	kPa
125 mm concrete slab	3.0
steel decking	0.2
steel beam (assume)	0.2
screeding	1.2
services	0.75
partition	<u>1.0</u>
	6.35
Imposed load (office)	3.0

$$\begin{aligned} W &= (1.4 \times 6.35 + 1.6 \times 3.0) \times 2.8 = 13.69 \times 2.8 = 38.33 \text{ kN/m} \\ M &= 38.33 \times 12^2 / 8 \\ &= \mathbf{690.0 \text{ kNm}} \end{aligned}$$

Construction stage design

Try UB457x152x52 Grade50

Section properties and dimensions

$$\begin{aligned} D &= 449.8 \text{ mm} & A &= 66.6 \text{ cm}^2 \\ B &= 152.4 \text{ mm} & I_x &= 21370 \text{ cm}^4 \\ t &= 7.6 \text{ mm} & Z_x &= 950 \text{ cm}^3 \\ T &= 10.9 \text{ mm} & S_x &= 1096 \text{ cm}^3 \\ d &= 407.6 \text{ mm} \end{aligned}$$

Design strength, $p_y = 355 \text{ N/mm}^2$ ($T=10.9 < 16\text{mm}$)

$$\begin{aligned} M_s &= 355 \times 1096 \times 10^3 \times 10^{-6} & \text{or} &= 355 \times 1.2 \times 950 \times 10^3 \times 10^{-6} \\ &= \mathbf{389.1 \text{ kNm}} & \text{or} &= 404.7 \text{ kNm (not critical)} \\ &> M = \mathbf{369 \text{ kNm}} & \text{OK} & \end{aligned}$$

Beam satisfactory for positive moment capacity in the construction stage.

The steel beam is assumed to be fully restrained against lateral buckling with intermediate bracing members as well as properly stud-welded profiled steel decking.

Composite stage design

Effective width of compression flange

$$B_e = 12000 / 8 \times 2 = 3000 \text{ mm} > b = 2800 \text{ mm}$$

$\therefore B_e = 2800 \text{ mm}$

clause 10.2.3.3
(eq. 10.5)

Resistance of concrete flange

$$R_c = 0.45 f_{cu} B_e (D_s - D_p)$$

$$= 0.45 \times 30 \times 2800 \times (125 - 50)$$

$$= 2835 \text{ kN}$$

clause 10.2.5(7)
(eq. 10.10)

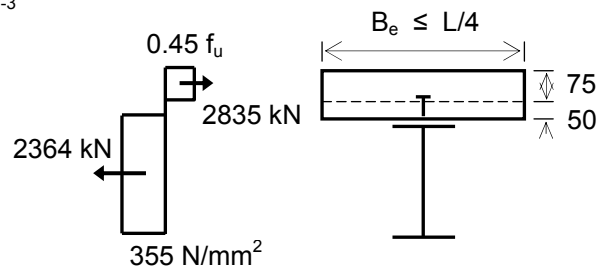
Resistance of steel section

$$R_s = A p_y$$

$$= 66.6 \times 10^2 \times 355 \times 10^{-3}$$

$$= 2364 \text{ kN}$$

clause 10.2.5(7)
(eq. 10.11)



$R_s \leq R_c$ (plastic neutral axis in concrete flange); assume full shear connection

$$M_c = R_s \left\{ \frac{D}{2} + D_s - \frac{R_s (D_s - D_p)}{R_c} \right\}$$

$$= 2364 \left\{ \frac{449.8}{2} + 125 - \frac{2364 (125 - 50)}{2835} \right\} \times 10^{-3}$$

$$= 753.2 \text{ kNm}$$

(eq. E10.13)

Shear connector capacity

$$P_k = 76.3 \text{ kN}$$

Clause 10.3.2.2 –
Table 10.7

$$P_p = 0.8 P_k$$

$$= 0.8 \times 76.3$$

$$= 61.0 \text{ kN}$$

Clause 10.3.2.1
(eq. 10.19a)

For 2 studs per trough

$$k = 0.5 \times \frac{b_r}{D_p} \times \left(\frac{h}{D_p} - 1 \right)$$

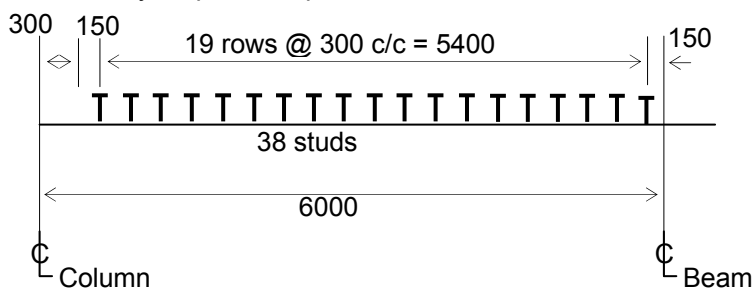
$$= 0.5 \times \frac{150}{50} \times \left(\frac{100}{50} - 1 \right) = 1.5 > 0.8 \quad \text{No reduction}$$

clause 10.3.2.3
(eq. 10.22)

$$\therefore P_p = 0.8 \times 61.0 \text{ kN}$$

$$= 48.8 \text{ kN}$$

Shear connector layout per half span



Resistance of shear connectors

$$\begin{aligned} R_q &= N P_p \\ &= 38 \times 48.8 \\ &= 1854 \text{ kN} \end{aligned} \quad (\text{eq. 10.12})$$

Minimum degree of shear connection

$$\begin{aligned} k_{sc} &= 1 - (0.75 - 0.03 L_E) \left(\frac{355}{p_y} \right) \\ &= 1 - (0.75 - 0.03 \times 12) \left(\frac{355}{355} \right) \\ &= 0.61 \end{aligned} \quad (\text{eq. 10.15a})$$

Degree of shear connection

$$k_{sc} = \frac{R_q}{R_s} = \frac{1854}{2364} = 0.78 > 0.61(\text{min}) \quad (\text{eq. 10.13a})$$

$k_{sc} < 1.0$ (Partial shear connection)

Plastic moment capacity of the composite cross-section with partial shear connection, M_{co} ,

Linear interpolation method

$$\begin{aligned} M_{co} &= M_s + k_{sc} (M_c - M_s) \\ &= 389.1 + 0.78 \times (753.2 - 389.1) \\ &= 673.1 \text{ kNm} \\ &< 690.0 \text{ kNm} \quad \quad \quad \mathbf{NOT OK!} \end{aligned} \quad (\text{eq. 10.14})$$

Plastic stress block method

$$\begin{aligned} R_f &= B T p_y \\ &= 152.4 \times 10.9 \times 355 \times 10^{-3} \\ &= 589.7 \text{ kN} \end{aligned} \quad (\text{eq. E10.2})$$

$$\begin{aligned} R_w &= R_s - 2 \times R_f \\ &= 2364 - 2 \times 589.7 \\ &= 1184.6 \text{ kN} \end{aligned} \quad (\text{eq. E10.9})$$

$$R_q = 1854 \text{ kN}$$

$R_q \geq R_w$ (plastic neutral axis in steel flange)

$$\begin{aligned} M_{co} &= R_s \frac{D}{2} + R_q \left\{ D_s - \frac{R_q (D_s - D_p)}{R_c} \right\} - \frac{(R_s - R_q)^2 T}{R_f} \frac{1}{4} \\ &= \left\{ 2364 \times \frac{449.8}{2} + 1854 \times \left[125 - \frac{1854 (125 - 50)}{2835} \right] \right. \\ &\quad \left. - \frac{(2364 - 1854)^2}{589.7} \times \frac{10.9}{4} \right\} \times 10^{-3} \\ &= 531.67 + 186.28 - 1.2 \\ &= 716.8 \text{ kNm} \\ &> 690.0 \text{ kNm} \quad \quad \quad \mathbf{OK!} \end{aligned} \quad (\text{eq. E10.16})$$

Summary of moment capacities

Linear interpolation method	673.1kNm	< 690kNm	NOT OK!
Plastic stress block method	716.8kNm	> 690kNm	OK!

E10.6.2 Composite encased column

Consider a pinned composite encased column under axial compression only. No reinforcement bars are used.

Design Data

Actions (factored values)

Applied axial force	P	=	2000 kN
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Buckling length

System length	L	=	4.0 m
Buckling coefficient	k_x	=	1.0
	k_y	=	1.0

Steel section

Universal column	UC 203x203x46 S355		
Modulus of elasticity	E	=	205 kN/mm ²

Concrete

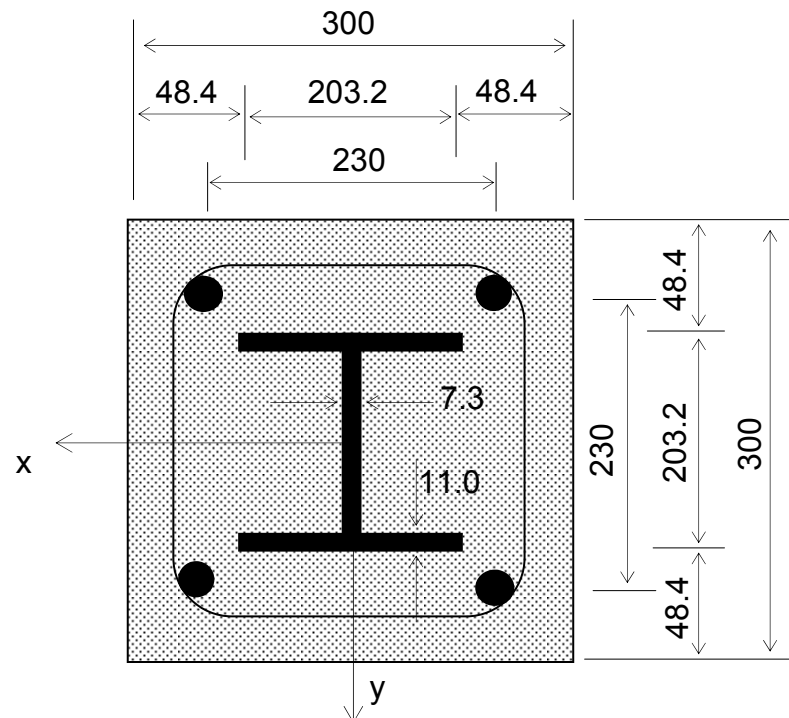
C45 normal weight concrete			
Characteristic strength	f_{cu}	=	45 N/mm ²
Secant modulus of elasticity	E_{cm}	=	26.4 kN/mm ²
Correction factor	K_e	=	0.6

Reinforcement

Characteristic strength	f_y	=	460 N/mm ²
Modulus of elasticity	E_s	=	205 kN/mm ²
Using 4 bars of 12mm diameter	A_s	=	452 mm ²

Material partial safety factors

For strength calculation	γ_c	=	1.5
	γ_s	=	1.15



Steel

For $T = 11.0 \text{ mm} < 16 \text{ mm}$,

$$\begin{aligned} p_y &= 355 \text{ N/mm}^2 \\ \varepsilon &= \sqrt{\frac{275}{355}} = 0.88 \\ E &= 205 \text{ kN/mm}^2 \\ A &= 5880 \text{ mm}^2 \\ I_x &= 45.65 \times 10^6 \text{ mm}^4 \\ I_y &= 15.39 \times 10^6 \text{ mm}^4 \end{aligned}$$

Reinforcement

$$\begin{aligned} f_y &= 460 \text{ N/mm}^2 \\ \gamma_s &= 1.15 \\ f_{sd} &= \frac{460}{1.15} = 400 \text{ N/mm}^2 \\ A_s &= 452 \text{ mm}^2 \\ I_{sx} &= 452 \times (230/2)^2 = 5.983 \times 10^6 \text{ mm}^4 \\ I_{sy} &= 452 \times (230/2)^2 = 5.983 \times 10^6 \text{ mm}^4 \end{aligned}$$

Concrete

$$\begin{aligned} f_{cu} &= 45 \text{ N/mm}^2 \\ \gamma_c &= 1.5 \\ f_{cd} &= \frac{45}{1.5} = 30 \text{ N/mm}^2 \\ E_{cm} &= 26.4 \text{ kN/mm}^2 \\ A_c &= 300 \times 300 - 5880 - 452 \\ &= 83668 \text{ mm}^2 \\ I_{cx} &= 300 \times 300^3 / 12 - 45.65 \times 10^6 - 5.983 \times 10^6 \\ &= 623.4 \times 10^6 \text{ mm}^4 \\ I_{cy} &= 300 \times 300^3 / 12 - 15.39 \times 10^6 - 5.983 \times 10^6 \\ &= 653.6 \times 10^6 \text{ mm}^4 \end{aligned}$$

Compression capacity, P_{cp}

Clause 10.5.3.2(1)
(eq. 10.58a)

$$\begin{aligned} P_{cp} &= A p_y + 0.45 A_c f_{cu} + A_s f_{sd} \\ &= (5880 \times 355 + 0.45 \times 83668 \times 45 + 452 \times 400) \times 10^{-3} \\ &= 2087.4 + 1694.3 + 180.8 \\ &= 3963 \text{ kN} \end{aligned}$$

Steel contribution ratio, δ

clause 10.5.3.2(2)
(eq. 10.59)

$$\begin{aligned} \delta &= \frac{A p_y}{P_{cp}} \\ &= 0.53 \\ &> 0.2 \quad \text{and} < 0.9 \quad \therefore \text{OK} \end{aligned}$$

Clause 10.5.3.1(2)
(eq. 10.52)

Check against column buckling

$$(EI)_{e,2} = EI + K_e E_{cm} I_c + E_s I_s$$

Clause 10.5.3.3(5)
(eq. 10.71)

$$\begin{aligned} (EI)_{e,2,x} &= 205 \times 45.65 + 0.6 \times 26.4 \times 623.4 + 205 \times 5.983 \\ &= 9358 + 9874 + 1226 \\ &= 20459 \text{ kNm}^2 \end{aligned}$$

$$\begin{aligned}
 (EI)_{e,2,y} &= 205 \times 15.39 + 0.6 \times 26.4 \times 653.6 + 205 \times 5.983 \\
 &= 3155 + 10353 + 1226 \\
 &= 14734 \text{ kNm}^2
 \end{aligned}$$

$$\begin{aligned}
 L_{E,x} &= 1.0 \times 4.0 \text{ m} &= 4.0 \text{ m} \\
 L_{E,y} &= 1.0 \times 4.0 \text{ m} &= 4.0 \text{ m}
 \end{aligned}$$

$$\begin{aligned}
 P_{cp,k} &= A p_y + 0.68 A_c f_{cu} + A_s f_y && \text{Clause 10.5.3.3(4)} \\
 &= (5880 \times 355 + 0.68 \times 83668 \times 45 + 452 \times 460) \times 10^{-3} && \text{(eq. 10.69a)} \\
 &= 2087.4 + 2560.2 + 208 \\
 &= 4856 \text{ kN}
 \end{aligned}$$

$$P_{cp,cr} = \frac{\pi^2 (EI)_{e,2}}{L_E^2} \quad \text{Clause 10.5.3.3(4)} \quad \text{(eq. 10.70)}$$

$$P_{cp,cr,x} = \frac{\pi^2 \times 20459}{4.0^2} = 12620 \text{ kN}$$

$$P_{cp,cr,y} = \frac{\pi^2 \times 14734}{4.0^2} = 9089 \text{ kN}$$

$$\bar{\lambda} = \sqrt{\frac{P_{cp,k}}{P_{cp,cr}}} \quad \text{Clause 10.5.3.3(4)} \quad \text{(eq. 10.68)}$$

$$\bar{\lambda}_x = \sqrt{\frac{4856}{12620}} = 0.620$$

$$\bar{\lambda}_y = \sqrt{\frac{4856}{9089}} = 0.731$$

$\therefore \bar{\lambda}_y$ is more critical

Reduction factor χ for column buckling Clause 10.5.3.3(3)

Use buckling curve "c", and $\alpha = 0.49$.

(eq. 10.67)

$$\begin{aligned}
 \phi &= \frac{1}{2} \left[1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \\
 &= \frac{1}{2} \left[1 + 0.49 (0.731 - 0.2) + 0.731^2 \right] \\
 &= 0.897
 \end{aligned}$$

The reduction factor χ for column buckling,

(eq. 10.66)

$$\begin{aligned}
 \chi &= \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \\
 &= \frac{1}{0.897 + \sqrt{0.897^2 - 0.731^2}} \\
 &= 0.705
 \end{aligned}$$

$$\therefore \chi P_{cp} = 0.705 \times 3963 \text{ kN} \quad \text{(eq. 10.65)}$$

$$\begin{aligned}
 &= 2794 \text{ kN} \\
 &> P = 2000 \text{ kN} \quad \text{OK}
 \end{aligned}$$

E10.6.2 Composite in-filled column

Consider a composite column with in-filled rectangular hollow section. No reinforcement bars are used.

Design Data

Actions (factored values)

Applied axial force	P	=	11000 kN
Applied moment	M_x	=	215 kNm
	M_y	=	0 kNm
End moment ratio	r	=	-0.5

Buckling length

System length	L	=	4.8 m
Buckling coefficient	k_x	=	1.0
	k_y	=	0.85

Steel section

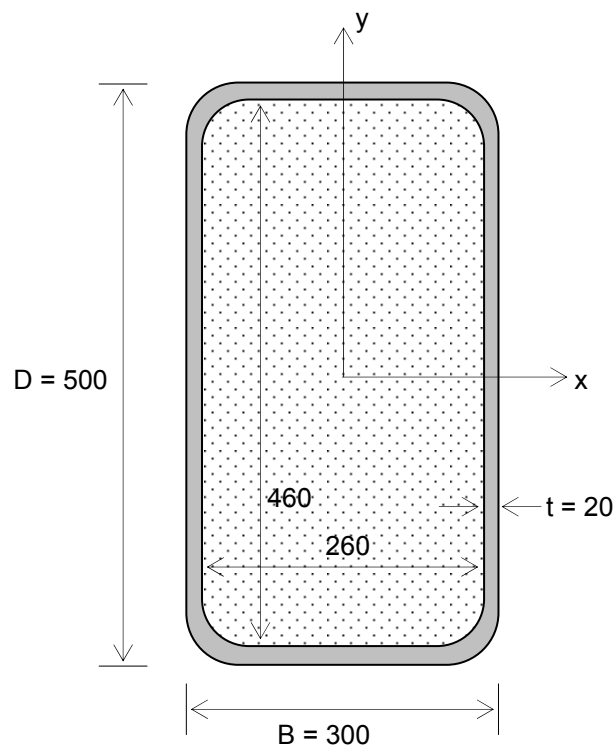
Rectangular hollow section 500x300x20 S355			
Modulus of elasticity	E	=	205 kN/mm ²

Concrete

C45 normal weight concrete			
Characteristic strength	f_{cu}	=	45 N/mm ²
Secant modulus of elasticity	E_{cm}	=	26.4 kN/mm ²
Correction factor	K_e	=	0.6

Material partial safety factors

For strength calculation	γ_c	=	1.5
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Radius of the corner of the structural hollow section may be neglected

Concrete

$$\begin{aligned}f_{cu} &= 45 \text{ N/mm}^2 \\ \gamma_c &= 1.5 \\ f_{cd} &= \frac{45}{1.5} = 30 \text{ N/mm}^2 \\ E_{cm} &= 26.4 \text{ kN/mm}^2 \\ A_c &= 460 \times 260 = 119600 \text{ mm}^2\end{aligned}$$

Steel

$$\begin{aligned}\text{For } t &= 20 \text{ mm} > 16 \text{ mm}, \\ p_y &= 345 \text{ N/mm}^2 \\ \varepsilon &= \sqrt{\frac{275}{345}} = 0.893 \\ E &= 205 \text{ kN/mm}^2 \\ A &= 500 \times 300 - A_c = 30400 \text{ mm}^2\end{aligned}$$

$$\begin{aligned}\frac{D}{t} &= \frac{500}{20} = 25 \\ &< 48 \varepsilon = 43 \quad \text{OK}\end{aligned}$$

Table 10.11

Compression capacity, P_{cp}

$$\begin{aligned}P_{cp} &= A p_y + 0.53 A_c f_{cu} + A_s f_{sd} \\ &= (30400 \times 345 + 0.53 \times 119600 \times 45 + 0) \times 10^{-3} \\ &= 10488 + 2852 \\ &= 13340 \text{ kN}\end{aligned}$$

Clause 10.5.3.2(1)
(eq. 10.58b)

Steel contribution ratio, δ

$$\begin{aligned}\delta &= \frac{A p_y}{P_{cp}} \\ &= \frac{10488}{13340} \\ &= 0.79 \\ &> 0.2 \quad \text{and} < 0.9 \quad \therefore \text{OK}\end{aligned}$$

clause 10.5.3.2(2)
(eq. 10.59)

Clause 10.5.3.1(2)
(eq. 10.52)

Check against column buckling

Clause 10.5.3.3

Concrete

$$\begin{aligned}I_{cx} &= \frac{460^3 \times 260}{12} = 2109 \times 10^6 \text{ mm}^4 \\ I_{cy} &= \frac{460 \times 260^3}{12} = 674 \times 10^6 \text{ mm}^4\end{aligned}$$

Steel

$$\begin{aligned}I_x &= \frac{500^3 \times 300}{12} - I_{cx} = 1016 \times 10^6 \text{ mm}^4 \\ I_y &= \frac{500 \times 300^3}{12} - I_{cy} = 451 \times 10^6 \text{ mm}^4\end{aligned}$$

$$(EI)_{e,2} = EI + K_e E_{cm} I_c + E_s I_s \quad \text{Clause 10.5.3.3(5)} \\ \text{(eq. 10.71)}$$

$$(EI)_{e,2,x} = (205 \times 1016 \times 10^6 + 0.6 \times 26.4 \times 2109 \times 10^6 + 0) \times 10^{-6} \\ = 208.28 \times 10^3 + 33.41 \times 10^3 \\ = 241.7 \times 10^3 \text{ kNm}^2$$

$$(EI)_{e,2,y} = (205 \times 451 \times 10^6 + 0.6 \times 26.4 \times 674 \times 10^6 + 0) \times 10^{-6} \\ = 92.46 \times 10^3 + 10.68 \times 10^3 \\ = 103.1 \times 10^3 \text{ kNm}^2$$

$$L_{E,x} = 1.0 \times 4.8 \text{ m} = 4.8 \text{ m} \\ L_{E,y} = 0.85 \times 4.8 \text{ m} = 4.08 \text{ m}$$

$$P_{cp,k} = A p_y + 0.8 A_c f_{cu} + A_s f_y \quad \text{Clause 10.5.3.3(4)} \\ = 10488 + (0.8/0.53) \times 2852 + 0 \quad \text{(eq. 10.69b)} \\ = 10488 + 4305 \\ = 14793 \text{ kN}$$

$$P_{cp,cr} = \frac{\pi^2 (EI)_{e,2}}{L_E^2} \quad \text{Clause 10.5.3.3(4)} \\ \text{(eq. 10.70)}$$

$$P_{cp,cr,x} = \frac{\pi^2 \times 241.7 \times 10^3}{4.8^2} = 103.5 \times 10^3 \text{ kN}$$

$$P_{cp,cr,y} = \frac{\pi^2 \times 103.1 \times 10^3}{4.08^2} = 61.1 \times 10^3 \text{ kN}$$

$$\bar{\lambda} = \sqrt{\frac{P_{cp,k}}{P_{cp,cr}}} \quad \text{Clause 10.5.3.3(4)} \\ \text{(eq. 10.68)}$$

$$\bar{\lambda}_x = \sqrt{\frac{14793}{103.5 \times 10^3}} = 0.378$$

$$\bar{\lambda}_y = \sqrt{\frac{14793}{61.1 \times 10^3}} = 0.492$$

$\therefore \bar{\lambda}_y$ is more critical

Reduction factor χ for column buckling

No reinforcement bars are used, i.e. $\rho_s = 0\%$
Use buckling curve "a", and $\alpha = 0.21$.

Clause 10.5.3.3(3)
Table 10.13

$$\phi = \frac{1}{2} \left[1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \quad \text{(eq. 10.67)} \\ = \frac{1}{2} \left[1 + 0.21 (0.492 - 0.2) + 0.492^2 \right] \\ = 0.652$$

The reduction factor χ for column buckling,

$$\begin{aligned}\chi &= \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} && \text{(eq. 10.66)} \\ &= \frac{1}{0.652 + \sqrt{0.652^2 - 0.492^2}} \\ &= 0.926\end{aligned}$$

$$\begin{aligned}\therefore \chi P_{cp} &= 0.926 \times 13340 \text{ kN} && \text{(eq 10.65)} \\ &= 12353 \text{ kN} \\ &> P = 11000 \text{ kN} \quad \text{OK}\end{aligned}$$

Moment resistance along x-axis

$$M_{cp} = \rho_y (S_p - S_{pn}) + 0.5 \alpha_c f_{cu} (S_{pc} - S_{pcn}) + f_{sd} (S_{ps} - S_{psn}) \quad \text{Clause 10.5.3.4 (eq. 10.73)}$$

$$\alpha_c = 0.53 \text{ for in-filled hollow section}$$

$$S_{pc} = \frac{460^2 \times 260}{4} = 13750 \times 10^3 \text{ mm}^3$$

$$S_p = \frac{500^2 \times 300}{4} - S_{pc} = 5000 \times 10^3 \text{ mm}^3$$

$$d_n = \frac{A_c \alpha_c f_{cu}}{2 B \alpha_c f_{cu} + 4 t (2 \rho_y - \alpha_c f_{cu})} \quad \text{(eq. E10.58) (Fig. 10.18)}$$

$$= \frac{460 \times 260 \times 0.53 \times 30}{2 \times 300 \times 0.53 \times 30 + 4 \times 20 \times (2 \times 345 - 0.53 \times 30)}$$

$$= 30.0 \text{ mm}$$

$$\Rightarrow 2d_n = 60.0 \text{ mm}$$

Concrete

$$S_{pcn} = \frac{60.0^2 \times 260}{4} = 233.4 \times 10^3 \text{ mm}^3$$

Steel

$$S_{pn} = 2 \times \frac{60.0^2 \times 20}{4} = 36.0 \times 10^3 \text{ mm}^3$$

$$\begin{aligned}M_{cp} &= [345 \times (5000 - 36) \times 10^3 + && \text{(eq 10.73)} \\ &0.5 \times 0.53 \times 45 \times (13750 - 233.4) \times 10^3 + 0] \times 10^{-6} \\ &= 1712.6 + 161.2 + 0 \\ &= 1873.8 \text{ kNm}\end{aligned}$$

Combined compression and uni-axial bending

Clause 10.5.3.5

Axial resistance ratio due to the concrete, $\chi_{pm} = \frac{P_{pm}}{P_{cp}}$ Clause 10.5.3.5(7)

$$\begin{aligned}P_{pm} &= \alpha_c A_c f_{cu} && \text{(eq. 10.76a)} \\ &= 0.53 \times 460 \times 260 \times 45 \times 10^{-3} \\ &= 2852 \text{ kN}\end{aligned}$$

$$\chi_{pm} = \frac{2852}{13340} = 0.214$$

Design axial resistance ratio, $\chi_d = \frac{P}{P_{cp}}$ Clause 10.5.3.5(7)

$$\chi_d = \frac{11000}{13340} = 0.825 > \chi_{pm}$$

$$\chi_n = \frac{(1-r)\chi}{4} = \frac{1-(-0.5)}{4} \times 0.926 = 0.347 \quad (\text{eq. 10.83a})$$

The moment resistance ratio, μ

$$\mu = \frac{(\chi - \chi_d)(1 - \chi_n)}{(1 - \chi_{pm})(\chi - \chi_n)} \quad \text{for } \chi_d \geq \chi_{pm} \quad \begin{array}{l} \text{Clause 10.5.3.5(7)} \\ \text{(eq. 10.82a)} \end{array}$$

$$\begin{aligned} &= \frac{(0.926 - 0.825)(1 - 0.347)}{(1 - 0.214)(0.926 - 0.347)} \\ &= 0.145 \end{aligned}$$

$$\begin{aligned} 0.9 \mu M_{cp} &= 0.9 \times 0.145 \times 1873.8 && \text{Clause 10.5.3.5(6)} \\ &= 244.5 \text{ kNm} && \text{(eq. 10.81)} \\ &> M = 215 \text{ kNm} && \text{OK} \end{aligned}$$

E11 DESIGN OF COLD-FORMED STEEL OPEN SECTIONS, SHEET PROFILES, HOLLOW SECTIONS AND SHEET PILE SECTIONS

Section 11 of the Code was previously written specifically for thin gauge sheet profiles with thickness up to 4 mm and for thin open sections with thickness up to 8mm only.

Strenuous efforts were paid to revamp Section 11 of the Code by clarifying the following :

- 1) The use of thin gauge open sections and sheet profiles with nominal thickness up to 4 mm only;
- 2) For the use of hollow section with nominal thickness up to 22 mm. Tight corners of cold-formed sections are controlled by external corner profiles while the welding in cold-formed regions are generally controlled by internal radius/thickness ratio as stipulated in clause 11.7.5 of the Code;
- 3) For the use of cold-formed sheet pile sections with nominal thickness up to 16mm, the tight corners are controlled by inside radius while the welding in cold-formed regions are generally controlled by internal radius/thickness ratio as stipulated in clause 11.7.5 of the Code; and
- 4) For cold-formed open sections with nominal thickness of 4mm and above clause 11.8 of the Code is essentially applicable.

Although this section permits the use of hand calculation to determine the loading capacity of cold-formed thin gauge open sections and sheet profiles with nominal thickness up to 4 mm, it is highly recommended to substantiate the calculation with loading test.

For the use of thin gauge open sections and sheet profiles, the following must be submitted to the Building Authority:

- a) Provision of detailed design calculation to demonstrate the structural adequacy of the proprietary product;
- b) Submission of the mill certificate of the steel sheeting material;
- c) Provision of HOKLAS or equivalent test reports of the above proprietary product; and
- d) Submission of Quality Assurance documents of the manufacturer demonstrating that the quality assurance of such proprietary product has been carried out by a factory with ISO 9001 quality assurance certification.

E11.2 MATERIAL PROPERTIES

BS 2989: 1992 provides guidance on the tolerances for cold-formed sections and was formerly the standard for the steel grades, before being replaced by BS EN 10147.

E11.3 SECTION PROPERTIES

The design thickness of the steel should be taken as the nominal base metal thickness exclusive of coatings. The main difference between the behaviour of cold-formed steel sections and hot rolled steel sections is that thin plate elements tend to buckle locally under compression. Cold-formed cross-sections are therefore usually classified as "slender" because they cannot generally reach their full compression resistance based on the amount of material in the cross-section. Therefore, effective section properties should be used in structural calculations.

E11.3.1 Gross section properties

For steel up to 3.2 mm thickness, it is sufficient to use mid-line idealized dimensions, that is assuming the material is concentrated at the mid-line of the section with the actual round corners replaced by intersections of the flat elements. Account should be taken of the thickness of galvanizing, which reduces the actual thickness by about 0.04 mm.

The approximation is recommended in both BS 5950 and Eurocode 3. A correction formula may be used to evaluate the exact properties considering the difference in the

round and the sharp corners.

E11.3.2 Effective section properties under tension

Clause 11.3.2 of the Code is self-explanatory.

E11.3.3 Effective section properties under compression and bending

Flat thin elements will buckle under compression due to their slenderness while the corners remain fully effective. The effective width of each flat element depends on the buckling coefficient, which is a function of element type, section geometry and stress distribution. It is important to note that the mid-line idealized dimensions may be used to evaluate all the local buckling coefficients.

Generally, the effective width ratio for the elements with both longitudinal edges supported (stiffened elements) is much higher than that of the elements with only one edge supported (unstiffened elements). Edge stiffeners should be of a minimum size so that flange elements are considered as stiffened.

Effective section properties may be evaluated in accordance with other established design procedures given in the following codes of practice:

- AISI
- AS/NZ 4600
- Eurocode 3: Part 1.3

For simplicity, detailed calculation methods for sections and sheet profiles with intermediate stiffeners are not provided. Refer to Eurocode 3 for details.

E11.3.4 Local buckling

(1) Elastic buckling

The full compression resistance of a perfectly flat plate supported on two longitudinal edges can be developed for a width-to-thickness ratio of about 40. At greater widths, buckles form elastically causing a loss in the overall compressive resistance of the plate. This is due to the inability of the more flexible central portion to resist as much compression as the outer portions, which are partly stabilized by the edge supports.

$$\begin{aligned} p_{cr} &= \frac{K \pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2 & (E11.1) \\ &\approx 0.904 E K \left(\frac{t}{b}\right)^2 \text{ N/mm}^2 \end{aligned}$$

where

b is the plate width, and
 t is the steel thickness.

The term K , referred to as the buckling coefficient, represents the influence of the boundary conditions and the stress pattern on plate buckling. Normally, plates are considered to be infinitely long but have various support conditions along their longitudinal edges. The two common cases are, firstly, simple supports along both edges, and, secondly, one simple support and the other free edge. In the first case K is 4, whereas in the second, K reduces dramatically to 0.425. This indicates that plates with free edges do not perform well under local buckling. These cases are illustrated in Figure E11.1.

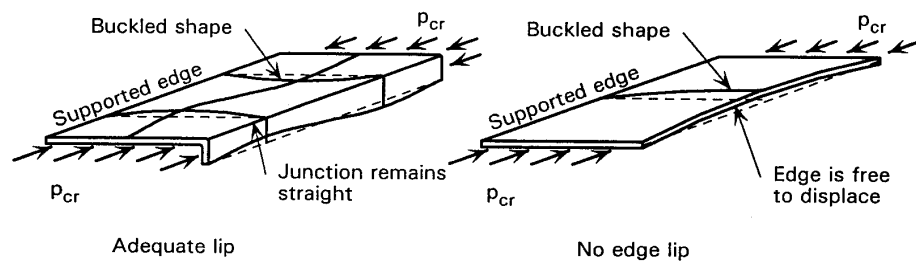


Figure E11.1 - Local buckling of plates with different boundary conditions

The value of K may be enhanced considerably when the rotational stiffness provided by the adjacent plates is included, or, alternatively, when the loading conditions do not result in uniform compression. Different cases for sections in bending and pure compression are given in Appendix B of BS 5950: Part 5.

(2) Post-critical behaviour

Plate elements are not perfectly flat, and therefore begin to deform out-of-plane gradually with increasing load, rather than buckle instantaneously at the critical buckling stress. This means that the non-uniform stress state exists throughout the loading regime, and tends to cause the plate element to fail at loads less than the critical buckling value. This is a dominant effect in the b/t range from 30 to 60 (for plates simply supported on both edges).

However, there are opposing effects for plate elements with higher b/t ratios. Firstly, "membrane" or in-plane tensions are generated, which resist further buckling, and secondly, the zone of compression yielding extends from the longitudinal supports to encompass a greater width of the plate elements. These post-critical effects cause an increase in the load-carrying capacity of wide plate elements ($b/t > 60$) relative to that given by Equation (E11.1).

The parameter which is used to express the behaviour of plate elements in compression is the "effective width". This is the notional width, which is assumed to act at the yield strength of the steel. The remaining portion of the plate element is assumed not to contribute to the compression resistance, as illustrated in Figure E11.2.

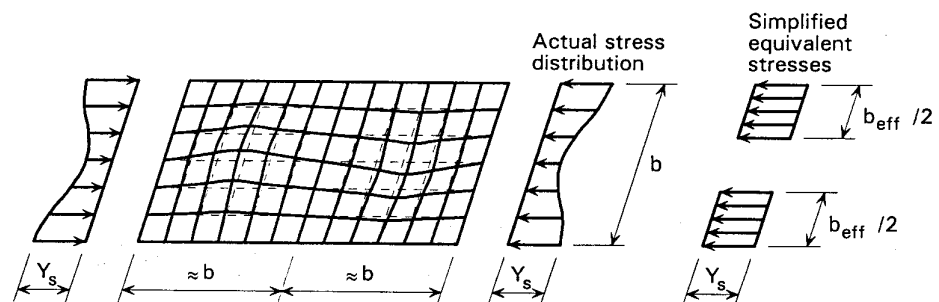


Figure E11.2 - Illustration of effective width of compression plate

The effective width concept can be modified to take the above factors into account. A semi-empirical formula for the effective width, b_e , of a plate element under compression is presented in clause 11.3.4.4.1 of the Code. The full compression resistance of a real (slightly non-flat) plate element supported on two longitudinal edges can be developed at a b/t ratio of less than approximately 30, and this therefore represents the most efficient spacing between stiffeners or folds in a cross-section.

E11.3.5 Flange curling

Clause 11.3.5 of the Code is self-explanatory.

E11.4 MEMBERS UNDER LATERAL LOADS

E11.4.2 Moment capacity

Clause 11.4.2 of the Code is self-explanatory.

E11.4.3 Shear capacity

Clause 11.4.3 of the Code is self-explanatory.

E11.4.4 Combined bending and shear

Clause 11.4.4 of the Code is self-explanatory.

E11.4.5 Web crushing capacity

Local failure at supports, or at locations of point loads, can occur as shown in Figure E11.3. This reduces the load-carrying resistance of the member. It is taken into account by an empirical formula representing the web crushing load.

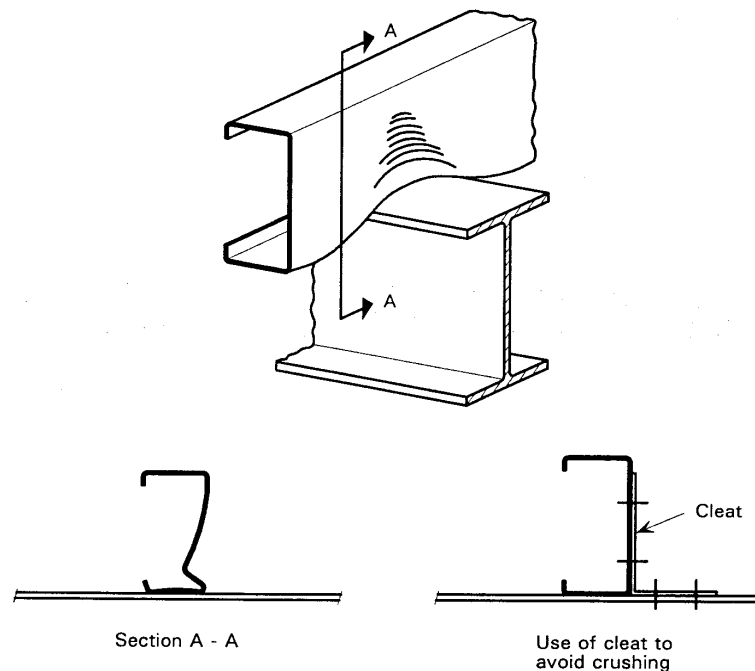


Figure E11.3 - Web crushing at a support

This effect is largely a function of the width of the support, the thickness of the steel, and the height/thickness ratio of the section. The web crushing capacity of flat section webs, P_w (in kN) under concentrated forces, either loads or reactions shall be evaluated using the equations given in the Code.

The reaction (or point load) is applied close to the end of the member, in which the web is free to move laterally. The support reaction or point load should not exceed the web crushing resistance. This can be best achieved by increasing the width of the supports or the thickness of the steel section. Enhanced capacities are given for double C sections with back to back webs, or webs with both flanges held in position (see Table 8 of BS 5950: Part 5).

E11.4.6 Combined bending and web crushing

Clause 11.4.6 of the Code is self-explanatory.

E11.4.7 Lateral buckling

Clause 11.4.7 of the Code is self-explanatory.

E11.4.8 Calculation of deflection

Clause 11.4.8 of the Code is self-explanatory. Recommended deflection limits are given in clause 5.2 of the Code.

E11.4.9 Effects of torsion

Clause 11.4.9 of the Code is self-explanatory.

E11.5 MEMBERS UNDER AXIAL LOADS

Clause 11.5 of the Code is self-explanatory.

E11.6 CONNECTIONS

Clause 11.6 of the Code is self-explanatory.

E11.7 DESIGN FOR HOLLOW SECTIONS

Clause 11.7 of the Code is self-explanatory. The clause 11.7.5 of the Code on welding at cold-formed zones is applicable to all cold-formed sections.

E11.8 DESIGN FOR COLD-FORMED SHEET PILE SECTIONS

Clause 11.8 of the Code is self-explanatory. Different kinds of steel grades are introduced when tighter corners are required.

This clause is also applicable for cold-formed open sections with nominal thickness of 4mm and above.

E11.9 WORKED EXAMPLES

E11.9.1 – Profiled steel sheet

Consider a double span profiled steel sheet as shown in Figure E11.4 under uniformly distributed load as follows:

Design Data

Dead load

Self-weight of profiled steel sheet 0.2 kN/m^2

Imposed load

Construction load 3.0 kN/m^2

Steel sheet

Thickness, t	=	1 mm
Depth of decking, d	=	50 mm
Unit width	=	110 mm
Sloping distance, d_w	=	51.91 mm
Internal radius of corner, r	=	4 mm
Design yield strength, p_y	=	275 N/mm^2
Elastic modulus, E	=	205 kN/mm^2

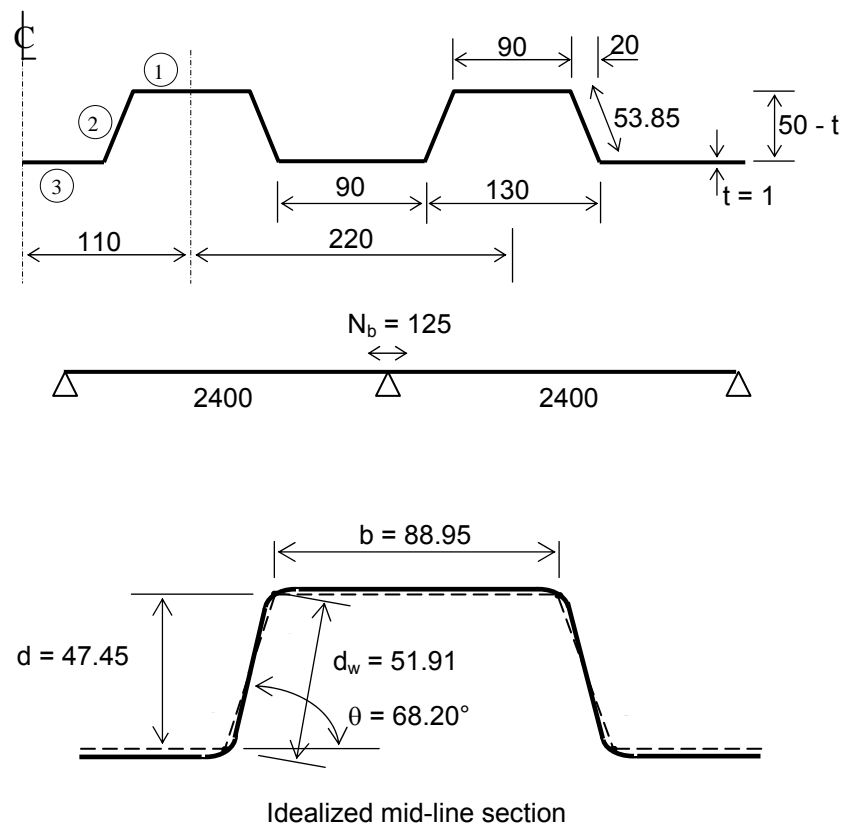
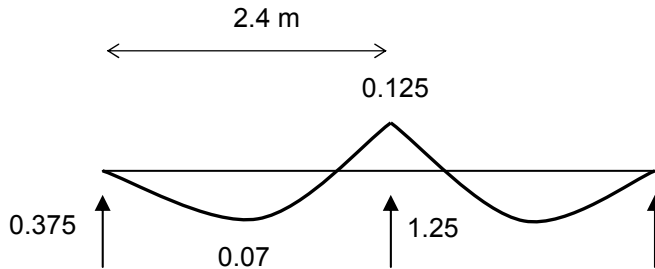


Figure E11.4 - Profile steel sheets

Loading (ULS)

$$\begin{aligned} \text{Design load} &= 1.4 \times 0.2 + 1.6 \times 3 \\ &= 5.08 \text{ kN/m per meter} \end{aligned}$$



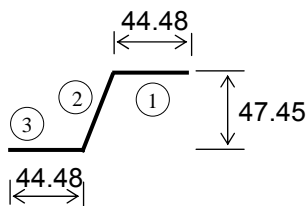
$$\begin{aligned} \text{Design moment} \\ M_{\text{hog}} &= 0.125 \times 5.08 \text{ kN/m} \times 2.4^2 \\ &= 3.66 \text{ kNm} \\ M_{\text{sag}} &= 0.07 \times 5.08 \text{ kN/m} \times 2.4^2 \\ &= 2.05 \text{ kNm} \end{aligned}$$

$$\begin{aligned} \text{Design shear force} \\ \text{(at end support)} \\ V &= 0.375 \times 5.08 \text{ kN/m} \times 2.4 \\ &= 4.57 \text{ kN per meter} \end{aligned}$$

$$\begin{aligned} \text{(at internal support)} \\ F_w &= 1.25 \times 5.08 \text{ kN/m} \times 2.4 \\ &= 15.24 \text{ kN per meter} \end{aligned}$$

Gross section properties

$$t = 1.0\text{mm} - 0.04\text{mm} = 0.96\text{mm} \quad (\text{E11.3.1})$$



Element	A	y	Ay	I_c	Ay^2
1	42.70	47.45	2026	$44.48 \times 0.96^3 / 12$ =3.280	24025
2	49.83	23.73	1182	$49.83 \times 47.45^2 / 12$ =9349	0
3	42.70	0	0	$44.48 \times 0.96^3 / 12$ =3.280	24025
Total	135.2	--	3208	9356	48050

$$\bar{y} = 3208 / 135.2 = 23.73 \text{ mm}$$

$$\begin{aligned}
I_x &= 9356 + 48050 = 5.741 \times 10^4 \text{ mm}^4 && \text{per 110 mm} \\
&= 52.19 \times 10^4 \text{ mm}^4 / \text{m} \\
Z_x &= 52.19 \times 10^4 / 23.73 \\
&= 2.199 \times 10^4 \text{ mm}^3 / \text{m} \\
M_x &= 275 \times 2.199 \times 10^4 \times 10^{-6} && \text{(eq. 11.26a)} \\
&= 6.05 \text{ kNm} / \text{m} \\
&> M_{\text{hog}} = 3.66 \text{ kNm} && \text{OK.} \\
&> M_{\text{sag}} = 2.05 \text{ kNm} && \text{OK.}
\end{aligned}$$

Effective section properties (cl 11.3.4.4)

Effective width of a flat stiffened flange: (cl. 11.3.4.4.1 and 11.3.4.4.3)

$$p_{cr} = 0.904 E K \left(\frac{t}{b} \right)^2 \quad \text{(eq. 11.11)}$$

$$K = 7 - \frac{1.8h}{0.15 + h} - 0.091 h^3 \quad \text{(eq. 11.12 b)}$$

$$h = \frac{d_w}{b} = \frac{51.91}{88.95} = 0.584$$

$$K = 7 - 1.432 - 0.018 = 5.55$$

$$\begin{aligned}
p_{cr} &= 0.904 \times 205000 \times 5.55 \left(\frac{0.96}{88.95} \right)^2 \\
&= 119.8 \text{ N/mm}^2
\end{aligned}$$

Effective width

$$b_e = \beta b \quad \text{(eq. 11.8)}$$

$$\beta = \left\{ 1 + 14 (\sqrt{\rho} - 0.35)^4 \right\}^{-0.2} \quad \text{when } \rho > 0.123 \quad \text{(eq. 11.9 b)}$$

$$\rho = \frac{f_c}{p_{cr}} \quad \text{(eq. 11.10)}$$

$$= \frac{275}{119.8} = 2.295 > 0.123$$

$$\beta = \left\{ 1 + 14 (\sqrt{2.295} - 0.35)^4 \right\}^{-0.2} = 0.518$$

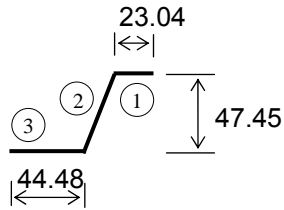
$$\begin{aligned}
b_e &= 0.518 \times 88.95 \\
&= 46.08 \text{ mm}
\end{aligned}$$

Effective width of a flat web:

$$d_w / t = 51.91 / 0.96 = 54.07 \quad \text{(cl.11.3.4.4.5i)}$$

$$< 70\varepsilon = 70 \times \sqrt{\frac{275}{275}} = 70$$

∴ The web is fully effective



Element	A	y	Ay	I_c	$A\bar{y}^2$
1	22.12	47.45	1050	$23.04 \times 0.96^3 / 12$ =1.843	17330
2	49.83	23.73	1182	$49.83 \times 47.45^2 / 12$ =9349	908.5
3	47.70	0	0	$44.48 \times 0.96^3 / 12$ =3.279	18064
Total	114.7	--	2232	9354	36303

$$\bar{y} = 2232 / 114.7 = 19.46 \text{ mm}$$

$$I_x = 9354 + 36303 = 4.566 \times 10^4 \text{ mm}^4 \quad \text{per 110 mm}$$

$$= 41.51 \times 10^4 \text{ mm}^4 / \text{m}$$

$$Z_x = 41.51 \times 10^4 / (47.45 - 19.46)$$

$$= 1.483 \times 10^4 \text{ mm}^3 / \text{m}$$

$$M_x = 275 \times 1.483 \times 10^4 \times 10^{-6} \text{ kNm / m} \quad (\text{eq. 11.26a})$$

$$= 4.08 \text{ kNm / m}$$

$$> M_{\text{hog}} = 3.66 \text{ kNm} \quad \text{OK.}$$

$$> M_{\text{sag}} = 2.05 \text{ kNm} \quad \text{OK.}$$

Web crushing capacity

(cl. 11.4.5.2)

$$P_w = 0.15 c_o t^2 \sqrt{E p_y} \left(1 - 0.1 \sqrt{\frac{r}{t}} \right) \left(0.5 + \sqrt{\frac{N_b}{50 t}} \right) \left\{ 2.4 + \left(\frac{\theta}{90} \right)^2 \right\} \quad (\text{eq. 11.39})$$

$$= 0.15 \times 1 \times 0.96^2 \sqrt{205000 \times 275} \left(1 - 0.1 \sqrt{\frac{4}{0.96}} \right) \left(0.5 + \sqrt{\frac{125}{50 \times 0.96}} \right) \left(2.4 + \left(\frac{68.2}{90} \right)^2 \right)$$

$$= 1038 \times 0.796 \times 2.114 \times 2.974$$

$$= 5195 \text{ N / web}$$

$$= 5.195 \text{ kN / web}$$

$$= 5.195 \text{ kN / web} \times 9.09 \text{ webs / m}$$

$$(\text{No. of webs / m} = 1000 / 110 = 9.09 \text{ webs / m})$$

$$= 47.22 \text{ kN / m}$$

$$> F_w = 15.24 \text{ kN / m} \quad \text{OK}$$

Combined bending and web crushing

(cl. 11.4.6)

$$\frac{F_w}{P_w} = \frac{15.24}{47.22} = 0.323 < 1.0$$

(eq. 11.40)

$$\frac{M}{M_c} = \frac{3.66}{4.08} = 0.897 < 1.0$$

(eq. 11.41)

$$\frac{F_w}{P_w} + \frac{M}{M_c} = 0.323 + 0.897$$

(eq. 11.42 c)

$$= 1.22 < 1.25 \quad \text{OK}$$

Web shear capacity

(cl. 11.4.3.2)

For a web without longitudinal stiffeners,

$$\begin{aligned} \lambda_w &= \frac{d_w}{t} \sqrt{p_y / E} \\ &= \left(\frac{51.91}{0.96} \right) \sqrt{\frac{275}{205000}} \\ &= 1.980 \end{aligned}$$

(eq. 11.31)

$$\begin{aligned} p_v &= 0.6 p_y \quad \text{for} \quad \lambda_w \leq 2.33 \\ &= 0.6 \times 275 \\ &= 165 \text{ N/mm}^2 \end{aligned}$$

(eq. 11.30 a)

$$\begin{aligned} V_c &= p_v d t \\ &= 165 \times 47.45 \times 0.96 \times 10^{-3} \\ &= 7.52 \text{ kN / web} \\ &= 7.52 \text{ kN / web} \times 9.09 \text{ webs / m} \\ &= 68.36 \text{ kN / m} \end{aligned}$$

(eq. 11.32)

Combined bending and shear

(cl. 11.4.4)

$$\frac{V}{V_c} = \frac{15.24 / 2}{68.36} = 0.111 < 1.0$$

(eq. 11.33)

$$\frac{M}{M_c} = \frac{3.66}{4.08} = 0.897 < 1.0$$

(eq. 11.34)

$$\left(\frac{V}{V_c} \right)^2 + \left(\frac{M}{M_c} \right)^2 = 0.012 + 0.805$$

(eq. 11.35)

$$= 0.817 < 1.0 \quad \text{OK}$$

Deflection check

(cl. 11.4.8)

Effective width of a flat stiffened flange: (cl. 11.3.4.4.1 and 11.3.4.4.3)

$$p_{cr} = 0.904 E K \left(\frac{t}{b} \right)^2 \quad (\text{eq. 11.11})$$

$$K = 7 - \frac{1.8h}{0.15 + h} - 0.091 h^3 \quad (\text{eq. 11.12 b})$$

$$h = \frac{d_w}{b} = \frac{51.91}{88.95} = 0.584$$

$$K = 7 - 1.432 - 0.018 = 5.55$$

$$p_{cr} = 0.904 \times 205000 \times 5.55 \left(\frac{0.96}{88.95} \right)^2 = 119.8 \text{ N/mm}^2$$

Effective width

$$f_{ser} = p_y / 1.4 = 275 / 1.4 = 196.4 \text{ N/mm}^2$$

$$b_e = \beta b$$

$$\beta = \left\{ 1 + 14 (\sqrt{\rho} - 0.35)^4 \right\}^{-0.2} \quad \text{when } \rho > 0.123 \quad (\text{eq. 11.8})$$

$$\rho = \frac{f_{ser}}{p_{cr}} \quad (\text{eq. 11.9 b})$$

$$= \frac{196.4}{119.8} = 1.639 > 0.123 \quad (\text{eq. 11.10})$$

$$\beta = \left\{ 1 + 14 (\sqrt{1.639} - 0.35)^4 \right\}^{-0.2} = 0.614$$

$$b_e = 0.614 \times 88.95 = 54.62 \text{ mm}$$

In case of using load factor 1.6,

$$f_{ser} = p_y / 1.6 = 275 / 1.6 = 172 \text{ N/mm}^2$$

$$\rho = 1.435$$

$$\beta = 0.656$$

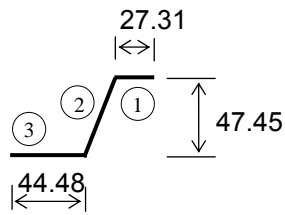
$$b_e = 0.656 \times 88.95 = 58.34 \text{ mm}$$

∴ $b_e = 54.62 \text{ mm}$ is more critical*Effective width of a flat web:*

$$d_w / t = 51.91 / 0.96 = 54.07 \quad (\text{cl. 11.3.4.4.5i})$$

$$< 70\varepsilon = 70 \times \sqrt{\frac{275}{196.4}} = 82.83$$

∴ The web is fully effective



Element	A	y	Ay	I _c	A \bar{y}^2
1	26.22	47.45	1244	27.31×0.96 ³ /12 =2.014	19157
2	49.83	23.73	1182	49.83×47.45 ² /12 =9349	545.9
3	42.70	0	0	44.48×0.96 ³ /12 =3.279	17805
Total	118.8	--	2426	9354	37508

$$\bar{y} = 2426 / 118.8 = 20.42 \text{ mm}$$

$$\begin{aligned}
 I_{x,r,s} &= I_{x,r,h} \\
 &= 9354 + 37508 = 4.686 \times 10^4 \text{ mm}^4 \quad \text{per 110 mm} \\
 &= 42.60 \times 10^4 \text{ mm}^4 / \text{m}
 \end{aligned}$$

$$\begin{aligned}
 I_{ser} &= \frac{2I_{xg} + I_{x,r,s} + I_{x,r,h}}{4} \\
 &= \frac{2 \times 52.19 \times 10^4 + 42.60 \times 10^4 + 42.60 \times 10^4}{4} \\
 &= 47.40 \text{ mm}
 \end{aligned} \tag{eq. 10.38}$$

$$\begin{aligned}
 \delta &= \frac{1}{185} \frac{wL^4}{E I_{ser}} \\
 &= \frac{1}{185} \frac{3 \times 2400^4}{205000 \times 47.40 \times 10^4} \\
 &= 5.54 \text{ mm}
 \end{aligned} \tag{eq. 11.51}$$

$$\frac{L}{180} = \frac{2400}{180} = 13.33 \text{ mm}$$

$$\delta < \frac{L}{180} \quad \text{OK} \quad (\text{No ponding effect is considered})$$

E11.9.2 Cold-formed steel purlin

Consider a double span purlin as shown in Figure E11.5. The span, L , is equal to 4.5 m with a purlin spacing of 1.2 m. A lipped C section C200x65x2.4 is adopted as the purlin member with the following design data:

Area, A	=	8.07 cm ²
Radius of gyration, r_y	=	2.26 cm
Section modulus, Z_{xx}	=	47.59 cm ³
Effective section modulus, Z_{xr}	=	45.20 cm ³
Second moment of area, I_{xx}	=	475.8 cm ⁴
Effective second moment of area, I_{xr}	=	451.9 cm ⁴
Design yield strength, p_y	=	350 N/mm ²
Elastic modulus, E	=	205 kN/mm ²
Factored wind load (uplift)	=	5.0 kPa

Design coefficients are as follows:

Bending moment	=	$-0.125 wL^2$	(internal support)
	=	$0.0703 wL^2$	(near mid-support of end span)
Reaction	=	$1.25 wL$	(internal support)
	=	$0.375 wL$	(end support)

Equivalent uniform moment factor for lateral torsional buckling is conservatively taken as unity.

The point of contra-flexure may be assumed to occur at $0.75L$ from the end support.

Consider the critical span of purlin under hogging moment region, check for the moment resistance of the purlin member under uplift. In case of severe lateral torsional buckling identified, add bracing(s) as necessary at suitable location(s), and re-check the purlin member for structural adequacy.

Also, check the section under combined bending and shear over the internal support.

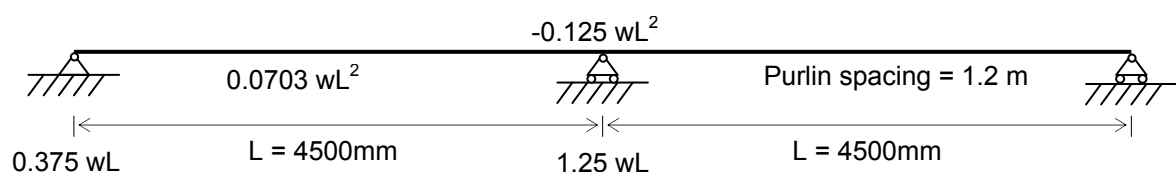


Figure E11.5 - Double span purlin

Factored wind load (uplift) = 5.0 kPa

$$\begin{aligned} W_{design} &= 5.0 \times 1.2 \text{ m} = 6.0 \text{ kN/m} \\ M_{hog} &= 0.0703 W_{design} L^2 \\ &= 0.0703 \times 6.0 \times 4.5^2 \\ &= 8.54 \text{ kNm} \end{aligned}$$

$$\begin{aligned} M_{sag} &= 0.125 W_{design} L^2 \\ &= 0.125 \times 6.0 \times 4.5^2 \\ &= 15.19 \text{ kNm} \end{aligned}$$

Reaction at end support

$$\begin{aligned} R &= 0.375 W_{design} L \\ &= 0.375 \times 6.0 \times 4.5 \\ &= 10.13 \text{ kN} \end{aligned}$$

Shear force at internal support

$$\begin{aligned} V &= 1.25 W_{design} L / 2 \\ &= 1.25 \times 6.0 \times 4.5 / 2 \\ &= 16.88 \text{ kN} \end{aligned}$$

$$\begin{aligned} M_y &= Z_{xx} \times p_y \\ &= 47.59 \times 350 \times 10^{-3} \\ &= 16.66 \text{ kNm} \end{aligned}$$

$$\begin{aligned} M_c &= Z_{xr} \times p_y \\ &= 45.20 \times 350 \times 10^{-3} \\ &= 15.82 \text{ kNm} \\ &> M_{hog} = 15.19 \text{ kNm} \quad \text{OK} \end{aligned}$$

Elastic lateral buckling moment resistance, M_E

(cl. 11.4.7.4.2)

$$M_E = \frac{\pi^2 A E d}{2(L_E / r_y)^2} C_{tw} \quad \text{(eq.11.48a)}$$

Assume the roof cladding provide full restraint in sagging moment region

$$\begin{aligned} L_E &= 0.75 \times 4500 \text{ mm} \\ &= 3375 \text{ mm} \end{aligned}$$

$$\begin{aligned} L_E / r_y &= 3375 / 22.6 \\ &= 149.3 \end{aligned}$$

$$C_{tw} = \sqrt{\left\{ 1 + \frac{1}{20} \left(\frac{L_E t}{r_y d} \right)^2 \right\}} \quad \text{(eq.11.49a)}$$

$$\begin{aligned} &= \sqrt{\left\{ 1 + \frac{1}{20} \left(149.3 \times \frac{2.36}{200} \right)^2 \right\}} \\ &= 1.075 \end{aligned}$$

$$\begin{aligned} M_E &= \frac{\pi^2 \times 807 \times 205000 \times 200}{2 \times (149.3)^2} \times 1.075 \times 10^{-6} \\ &= 7.87 \text{ kNm} \end{aligned} \quad \text{(eq.11.48a)}$$

The buckling moment resistance, M_b

(cl.11.4.7.4.1)

$$M_b = \frac{M_E M_Y}{\phi_B + \sqrt{\phi_B^2 - M_E M_Y}} \leq M_c$$

(eq. 11.45)

$$\eta = 0.002 (L_E / r_y - 40) = 0.219$$

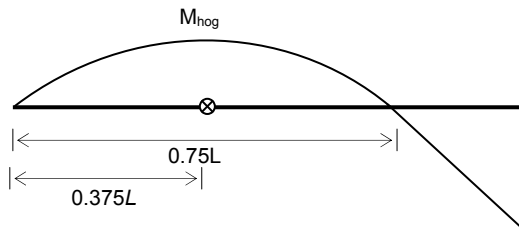
(eq. 11.47b)

$$\begin{aligned} \phi_B &= \frac{M_Y + (1 + \eta)M_E}{2} \\ &= \frac{16.66 + (1 + 0.219) \times 7.87}{2} \\ &= 13.13 \text{ kNm} \end{aligned}$$

(eq. 11.46)

$$\begin{aligned} M_b &= \frac{7.87 \times 16.66}{13.13 + \sqrt{13.13^2 - 7.87 \times 16.66}} \\ &= 6.70 \text{ kNm} \\ &< M_{hog} = 8.54 \text{ kNm} \quad \text{Not OK} \end{aligned}$$

(eq. 11.45)



Add bracing at $0.75L / 2 = 0.375L$

$$\begin{aligned} L_E / r_y &= 0.375 \times 4500 / 22.6 \\ &= 74.67 \end{aligned}$$

$$\begin{aligned} C_{tw} &= \sqrt{\left\{ 1 + \frac{1}{20} \left(74.67 \times \frac{2.36}{200} \right)^2 \right\}} \\ &= 1.019 \end{aligned}$$

(eq.11.49a)

$$\begin{aligned} M_E &= \frac{\pi^2 \times 807 \times 205000 \times 200}{2 \times (74.67)^2} \times 1.019 \times 10^{-6} \\ &= 29.84 \text{ kNm} \end{aligned}$$

(eq.11.48a)

$$\eta = 0.002 (L_E / r_y - 40) = 0.069$$

(eq. 11.74)

$$\begin{aligned} \phi_B &= \frac{M_Y + (1 + \eta)M_E}{2} \\ &= \frac{16.66 + (1 + 0.069) \times 29.84}{2} \\ &= 24.28 \text{ kNm} \end{aligned}$$

(eq. 11.72)

$$\begin{aligned} M_b &= \frac{29.84 \times 16.66}{24.28 + \sqrt{24.28^2 - 29.84 \times 16.66}} \\ &= 14.67 \text{ kNm} \\ &> M_{hog} = 8.54 \text{ kNm} \quad \text{OK} \end{aligned}$$

(eq. 11.45)

Shear capacity, V_c

(cl. 11.4.3.1)

$$\begin{aligned} p_{v,y} &= 0.6 p_y \\ &= 210 \text{ N/mm}^2 \end{aligned}$$

(eq.11.27)

$$\begin{aligned} p_{v,cr} &= \left(\frac{1000 t}{d_w} \right)^2 \text{ N/mm}^2 \\ &= \left(\frac{1000 \times 2.36}{200} \right)^2 \\ &= 139.2 \text{ N/mm}^2 \end{aligned}$$

(eq. 11.28)

$$\begin{aligned} V_c &= p_{v,y} t d \quad \text{but} \quad < p_{v,cr} t d \\ &= 139.2 \times 2.36 \times (200 - 2.36) \times 10^{-3} \\ &= 64.93 \text{ kN} \end{aligned}$$

(eq. 11.29)

Combined bending and shear at internal support

(cl 11.4.4)

$$\frac{V}{V_c} = \frac{16.88}{64.93} = 0.26 < 1.0$$

$$\frac{M}{M_c} = \frac{15.19}{15.82} = 0.96 < 1.0$$

$$\left(\frac{V}{V_c} \right)^2 + \left(\frac{M}{M_c} \right)^2 = 0.068 + 0.922$$

$$= 0.99 < 1.0 \quad \text{OK}$$

E12 FIRE RESISTANT DESIGN

E12.1 DESIGN PRINCIPLES

Section 12 of the Code aims to provide guidance on fire resistant design in steel and steel-concrete composite structures, which deals primarily with minimizing the risk of structural collapse and restricting the spread of fire through the structure.

Passive fire protection systems include spray-applied protection, board protection, intumescent coating as well as concrete encasement. Further details may be found in Chapter 34 of Fire protection and fire engineering of the Steel Designers' Manual, 6th edition, published by the Steel Construction Institute in 2003.

E12.1.1 Basic requirements for fire resistance for a structure

Clause 12.1.1 of the Code is self-explanatory.

E12.1.2 Fire exposure

The standard time - temperature curve is defined in accordance with ISO 834 as well as BS476: Part 20.

The standard fire exposure is usually adopted in compartmentation fires according to prescriptive design approach, while the natural fire exposure is often adopted in fires in large enclosure according to performance-based design approach.

E12.1.3 Fire limit states

Current assessment methods include (a) standard fire tests, (b) limiting temperature methods, (c) performance-based design methods, and (d) simplified calculation methods. Methods (a), (b), and (d) are based on BS5950: Part 8 while method (c) is established from Advanced Calculation Model proposed in Eurocode 3: Part 1.2 and Eurocode 4: Part 1.2.

For prescriptive design approach, the fire limit states of structural members are usually established or verified through standard fire tests. Reference should be made to clause 12.2.2 of the Code for the failure criteria on load-bearing capacity, integrity and insulation for standard fire tests.

E12.1.4 Physical and mechanical properties at elevated temperatures

The physical properties of steel and concrete at elevated temperatures given in Table 12.1 of the Code are compiled according to Eurocodes 3 and 4 as well as BS5950: Part 8. It should be noted that these data are 'engineered' codified values, which should only be used in simplified thermal calculations. Reference should be made to established recommendations for the complete definitions on various temperature-dependent physical properties.

For various temperature-dependent mechanical properties, Table 12.2a of the Code is compiled according to Eurocode 3 and BS5950: Part 8 while Table 12.2b of the Code is compiled according to Eurocode 3. Data given in Tables 12.2c and 12.2d of the Code are sourced from Eurocode 4. Reduction factors for bolts and welds in Table 12.3 of the Code are sourced from Eurocode 3.

For hot rolled reinforcing steel, the strength reduction factors at elevated temperatures should be taken from Table 12.2a of the Code corresponding to 2% strain.

In the absence of other information, the following strain levels in hot rolled steel at elevated temperatures are recommended:

- 2.0% for composite members in bending
- 1.5% for steel or non-composite members in bending
- 0.5% for all other members

In practice, the choice of the strain levels depends on the deformation characteristics of the structural members as well as the permissible strain levels of fire protection materials, if any. It should be noted that the recommended strain levels above are generally considered to be conservative, and it will be beneficial whenever the actual deformation of the structural members is considered in design.

Reduction factors of elastic modulus of hot rolled steel and cold worked reinforcing bars at elevated temperatures are given as follows (sourced from EN1994: Part 1.2: 2005):

Temperature	Reduction factors for elastic modulus	
	Hot rolled steel	Cold worked reinforcing bar
20 °C	1.000	1.00
100 °C	1.000	1.00
200 °C	0.900	0.87
300 °C	0.800	0.72
400 °C	0.700	0.56
500 °C	0.600	0.40
600 °C	0.310	0.24
700 °C	0.130	0.08
800 °C	0.090	0.06
900 °C	0.0675	0.05
1000 °C	0.0450	0.03
1100 °C	0.0225	0.02
1200 °C	0.0000	0.00

E12.2 FIRE RESISTANCE DERIVED FROM STANDARD FIRE TESTS

This section of the Code is consistent with the prevailing Code of Practice for Fire Safety in Buildings issued by the Buildings Department. Moreover, additional information about standard fire tests is provided for completeness.

E12.2.1 Fire resistance of structural members

Clause 12.2.1 of the Code is self-explanatory.

E12.2.2 Failure criteria for standard fire tests

The fire resistance of a structural member should be determined with respect to load-bearing capacity, integrity and insulation.

Most countries including European countries, Japan, Singapore and China do not have a specific code on fire resistance, and most of them refer to BS476: Parts 20 and 21. Moreover, ISO 834 also adopts the insulation requirements as stipulated in BS476, and so does the Code of Practice for Fire Safety in Buildings issued by the Buildings Department. The values of these two maximum temperature rises, namely, 140 and 180 °C, should be adopted for consistency with current established practice.

E12.3 FIRE RESISTANCE DERIVED FROM LIMITING TEMPERATURE METHOD

The limiting temperature method is established in late 1980's to assess whether or not fire protection to steel beams and columns is needed. The design rules are semi-empirical, which have been calibrated against fire tests of beam and column members with practical dimensions. Reference should be made to BS5950: Part 8 or other established recommendations for details on the limiting temperatures and the load ratios of beam and column members. In general, fire protection is often needed for steel beams and columns under practical loading and supporting conditions.

E12.4 FIRE RESISTANCE DERIVED FROM PERFORMANCE-BASED DESIGN METHODS

E12.4.1 Basis of analysis

This section of the Code is based on Section 4 “Advanced calculation models” of Eurocode 4: Part 1.2. Further information on performance-based design methods may be found in various parts of PD 7974: Application of fire safety engineering principles to the design of buildings (2001) published by British Standards Institution.

E12.4.2 Thermal response

Clause 12.4.2 of the Code is self-explanatory.

E12.4.3 Mechanical response

Clause 12.4.3 of the Code is self-explanatory.

E12.4.4 Validation of performance-based design methods

The performance-based design methods given in Section 12 of the Code are generally for element design of steel and steel-concrete composite structures, which are not well covered in recognized codes or design manuals. In general, validation of performance-based design by tests is not required if the design is well justified with recognized codes, design manuals or relevant test data whenever available.

E12.4.5 Simplified calculation methods

Clause 12.4.5 of the Code refers to the moment capacity method given in BS5950: Part 8. The method is established in the late 1980s to provide simple design rules to evaluate the load carrying capacities of both steel and steel-concrete composite beams at elevated temperatures after calibration against fire test data. The method is readily extended to composite slabs with profiled steel deckings. In addition, parts of the re-entrant type profiled steel decking, which are embedded in concrete, may be utilized to resist the bending moments at fire limit state, depending on the temperature distribution of the surrounding concrete.

E12.5 Performance requirements

This clause is written to promulgate the use of structural fire engineering approach to analyse the structural response under fire exposure, taking into account of predicted degradation of steel design strength and Young's modulus. The aim is to justify the non-protection of the steel structure when exposed to fire. If fire protection is required, the protection scale should be kept at the minimum. On the other hand, every endeavour should be made to explore new fire resistant materials if fire protection is definitely required.

E13 PERFORMANCE-BASED DESIGN GUIDANCE FOR PARTICULAR TYPES OF STRUCTURES, INCLUDING GUIDANCE ON GENERAL MAINTENANCE OF STEEL STRUCTURES

E13.1 HIGH-RISE BUILDINGS

Section 13 of the Code provides guidance on the design of various types of steel or steel – concrete composite structures. It is intended to act as an aide-memoire to designers rather than a textbook on the design of particular types of structure.

E13.1.1 Structural systems for high-rise buildings

Tall buildings are common in Hong Kong and East Asia and particular systems and combinations of materials are evolving. The most popular structural system for super high rise structures is demonstrated by buildings such as No.2 IFC and the Cheung Kong Centre. It comprises a concrete core with a limited number of very large perimeter columns of composite steel and concrete (mega columns). In addition to carrying vertical loads, these columns act with very strong beams (outriggers), between one and four levels, which are placed as regularly as possible up in the building to assist the core in contributing to the overall stiffness and strength of the structure. The core wall will typically carry all the lateral shear and a proportion of the bending caused by lateral loads. The main floors are typically of concrete cast on permanent metal formwork acting compositely with steel beams.

The most efficient structural system for super high rise buildings is the large space frame, or external mega truss as used by the John Hancock Centre, Chicago and the Bank of China, because it maximises bending stiffness by placing material at the perimeter and tying it together.

The Hong Kong and Shanghai Bank and the Commerzbank headquarters building in Frankfurt use giant portal frame systems (mega frames).

Perimeter tube structures also provide highly efficient structural systems but the relatively closely spaced perimeter columns are structurally dense in contrast to the open areas provided by mega column – outrigger systems. Central Plaza in Hong Kong is a good example of such a system executed in reinforced concrete. A more recent and unusual example is the Swiss Reinsurance building in London, UK, which has an external steel diagrid tube.

A summary of the principal structural systems used for high-rise steel and steel composite buildings are:

- (a) Steel perimeter columns, floor beams acting compositely with concrete floor and a concrete core providing lateral stability.
- (b) Steel perimeter moment frame providing lateral stability, steel and concrete composite floor and concrete core.
- (c) Tube in tube systems which are a development of perimeter tube systems.
- (d) Outrigger systems are structural systems which couple perimeter mega columns by strong floor members usually made up of steel trusses called the “outriggers”. Usually, an outrigger system may comprise a concrete core with a limited number of large perimeter mega columns of composite steel and concrete construction. In addition to carrying vertical loads, these columns act with outrigger beams, between one and four levels, which are placed as regularly as possible up in the building to assist the core in contributing to the overall stiffness and strength of the structure. The core wall will typically carry all the shear and a proportion of the bending caused by lateral loads. The main floors are of concrete cast on permanent metal formwork acting compositely with steel beams.
- (e) External mega truss or space frame provides the most efficient structural system for super high rise buildings. This system maximises overall building bending stiffness by placing material at the perimeter and tying it together.
- (f) Giant portal frame systems (mega frames) providing lateral stability. Floors are typically of concrete acting compositely with steel beams. Cores may be of steel

or concrete and mainly carry vertical loads. The Hong Kong and Shanghai Bank is an example of such a system.

E13.1.2 Stability issues for high-rise buildings

Overall rigid body stability

Lateral loads on tall buildings in a typhoon wind climate are large. The stability system of such buildings should be checked for stability against overturning as a rigid body under design load combination 2 in **Section 4** of the Code. (1.0 x DL - 1.4 x WL). Currently in Hong Kong, the overall stability must also comply with the Building (Construction) Regulations, see clause 2.3.2.2 of the Code. This requires overturning stability to comply with 1.0 x DL - 1.5 x WL. If uplift tensions occur between superstructure elements, e.g. columns or core, and pile caps or between pile caps and piles, they must be reinforced for.

Second-order effects

Second-order P- Δ and P- δ effects may be significant for high-rise buildings and shall be evaluated and allowed for. They can be considered directly by a second-order analysis using the P- Δ - δ analysis in section 6 of the Code.

Alternatively, the P- Δ effect shall be allowed for by amplifying the moment using equation 6.9 in **Section 6** of the Code. The P- δ effect should be considered by the effective length method using the non-sway column buckling length or conservatively the column length as the effective length. If the column is free to sway, the P- Δ effect should be evaluated by moment amplification and the column effective length should be determined from effective length factor for sway frame using figure 6.4a of the Code.

The Canadian code uses the factor $U_2 = \frac{1}{1 - \left[\frac{\sum C_f \Delta_f}{\sum V_f h} \right]}$ to amplify the moment due to

lateral load (see the code for symbol meanings). This is similar to the suggestion given in the Code.

The American code gives two amplification factors, B_1 and B_2 . B_1 accounts for second order effects caused by displacements of the members i.e. between braced points. These are the P- δ effects. B_2 accounts for second order effects caused by overall displacement of the frame i.e. displacement of braced points. These are the P- Δ effects.

E13.1.3 Considerations for particular details

Clause 13.1.3 of the Code is generally self-explanatory.

Consideration should be given to construction issues such as the number of floors of bare steel column that can be erected prior to encasing or filling with concrete and how concrete construction joints will be made.

Both elastic and long-term differential shortening caused by creep and shrinkage shall be considered.

E13.1.4 Considerations for design against extreme events

Clause 13.1.4 of the Code is generally self-explanatory, and clause 2.3.4 of the Code provides guidance on structural integrity, design against progressive collapse and design of key elements.

Tying of very large columns

The lateral stability and tying in of very large (mega) columns require special consideration as the restraint forces can be large. In accordance with clause 2.3.4.3 (b) of the Code, the restraint force should be 1% of the maximum factored dead and imposed load in the column, and this should be applied to steel columns. For composite steel and concrete columns, reference to concrete design codes suggests that a higher restraint force of 2½ % or 3% should be used. For very large columns with loads of hundreds of Mega Newtons, this would lead to very high restraint forces. As an alternative, an appropriate non-linear buckling analysis may be carried out to evaluate the restraint

forces required. This analysis may justify a higher or a lower restraint force than the 1% value, and is likely to justify a lower force than the 3% value. The horizontal restraint forces may actually be provided by strong tie beams at key floors spaced in accordance with the column buckled form or may be distributed to beams at every floor.

In some situations, as judged by the Responsible Engineer, mega columns may need to be considered as key elements.

E13.1.5 Wind engineering for high-rise buildings

Clause 13.1.5 of the Code is generally self-explanatory. For guidance and requirements on lateral deflections and accelerations of high-rise buildings, refer to clause 5.3 of the Code.

The Code requires that the horizontal deflection at the top of a building should be limited to Height/500 unless performance based calculations are carried out to justify a larger value. Such calculations may include consideration of the effect of inter storey drifts on cladding, vertical service risers and lifts. They may also include considerations of dynamic effects under wind load, such as vortex shedding and buffeting, and an assessment of whether or not building accelerations will be sufficiently low so as not to cause unacceptable discomfort to occupants. Generally, cladding and partitions will contribute to overall building stiffness to a greater or lesser extent and thus reduce overall deflections and increase damping.

General

Effects of wind on buildings should be considered in structural, foundation and cladding design and comfort of occupants and pedestrians. Control of deflection and acceleration should follow clause 5.3.4 of the Code. Vortex shedding and wind response in cross-wind direction should be considered, especially in design of slender structures. The damping effect may be considered for evaluation of actual structural response.

Wind has several effects on the design and satisfactory performance of tall buildings.

- (1) Wind loads are an important part of the loads for design of the foundations and vertical structure.
- (2) Wind causes the building to deflect and affects cladding design.
- (3) Wind causes accelerations which may not be acceptable to users of the building.
- (4) Windiness around the base can make conditions difficult for pedestrians depending on the activities to be carried out.

It is important to understand the wind loads in the advanced stage of the design. The design of tall buildings presents a few special wind risks that are not covered well by any of the current design codes.

- (1) The natural frequency of tall buildings is lower than that for shorter buildings, and closer to the natural frequencies of wind energy.
- (2) The height of the building increases the exposure to stronger winds and reduces the amount of turbulence in the wind.
- (3) Slender buildings generate turbulence through a process called vortex shedding. This can lead to cross-wind oscillations which increase as the wind speed increases and the turbulence reduces.
- (4) The inherent structural damping of buildings reduces with increasing slenderness of the building.
- (5) These factors can lead to a wind response in the cross-wind direction, which are larger than the wind response in the direction of the wind and which also cause higher building accelerations.

Figures E13.1.1 and E13.1.2 illustrate the global design forces and their effects on a tall building.

The principal structural systems used for tall steel and steel composite buildings used in Hong Kong and the region are described in clause E13.1.1. Figure E13.1.3 shows a

composite perimeter frame structure with enlarged corner columns to increase overall bending efficiency and figure E13.1.4 show a typical outrigger structure.

Figure E13.1.5 shows the principles of details for typical floor beams and Figure E13.1.6 shows the principles of composite mega column plan bracing to resist buckling and for robustness tying.

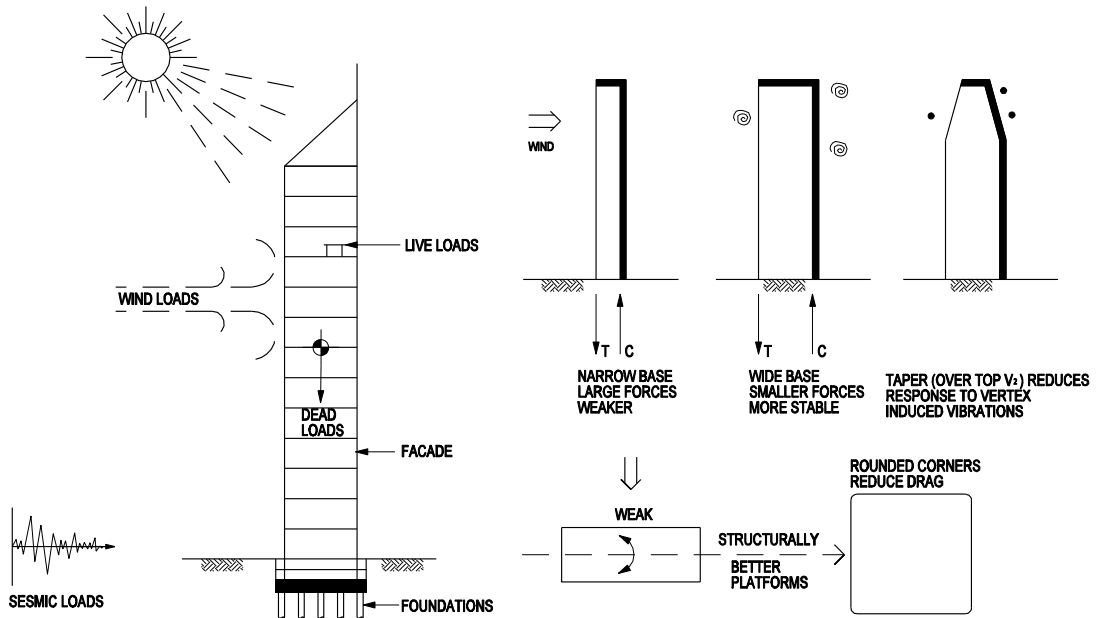


Figure E13.1.1 - Forces acting on tall buildings

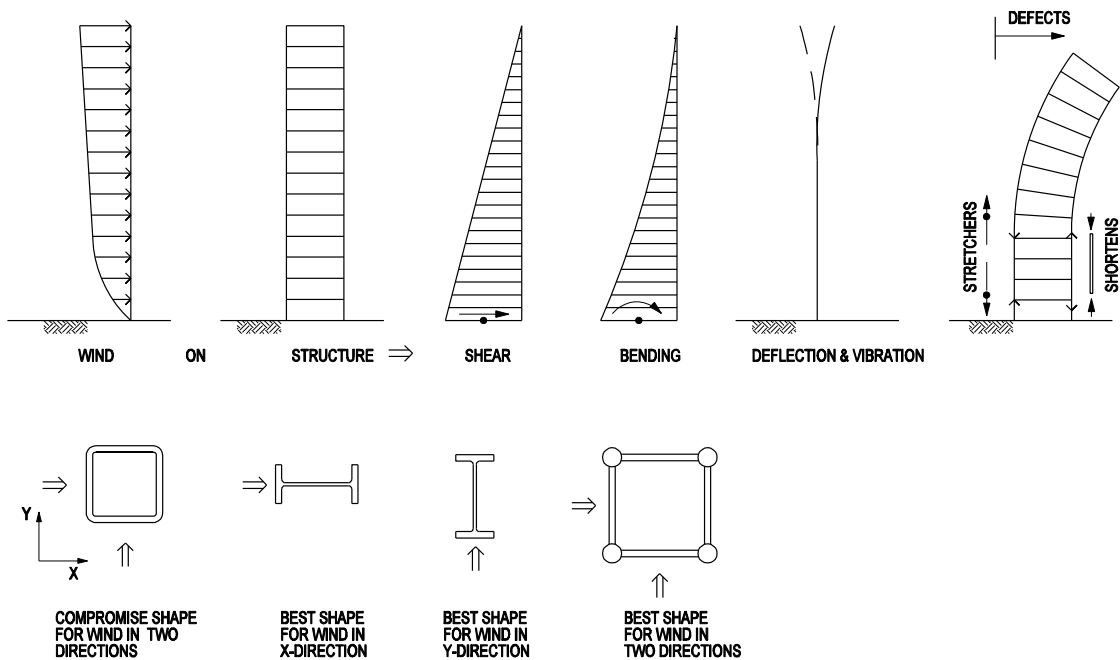


Figure E13.1.2 - Effects of forces on tall buildings

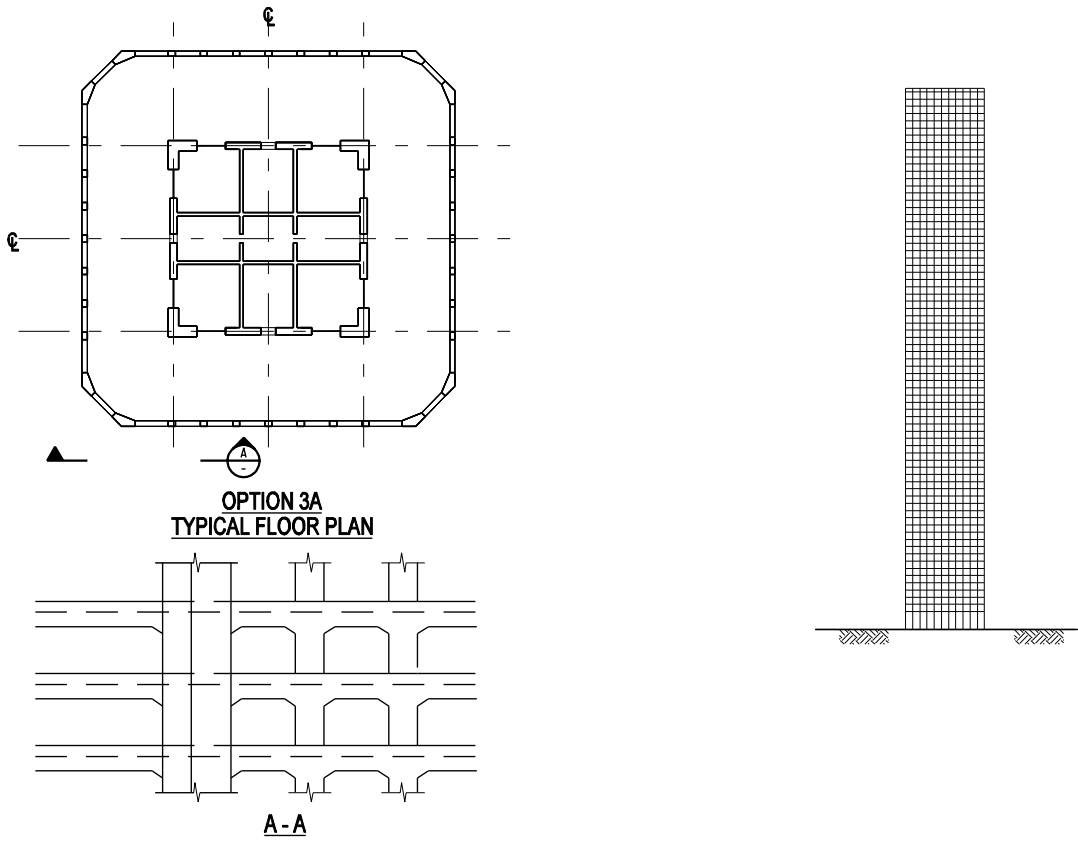


Figure E13.1.3 - Example of perimeter moment frame system

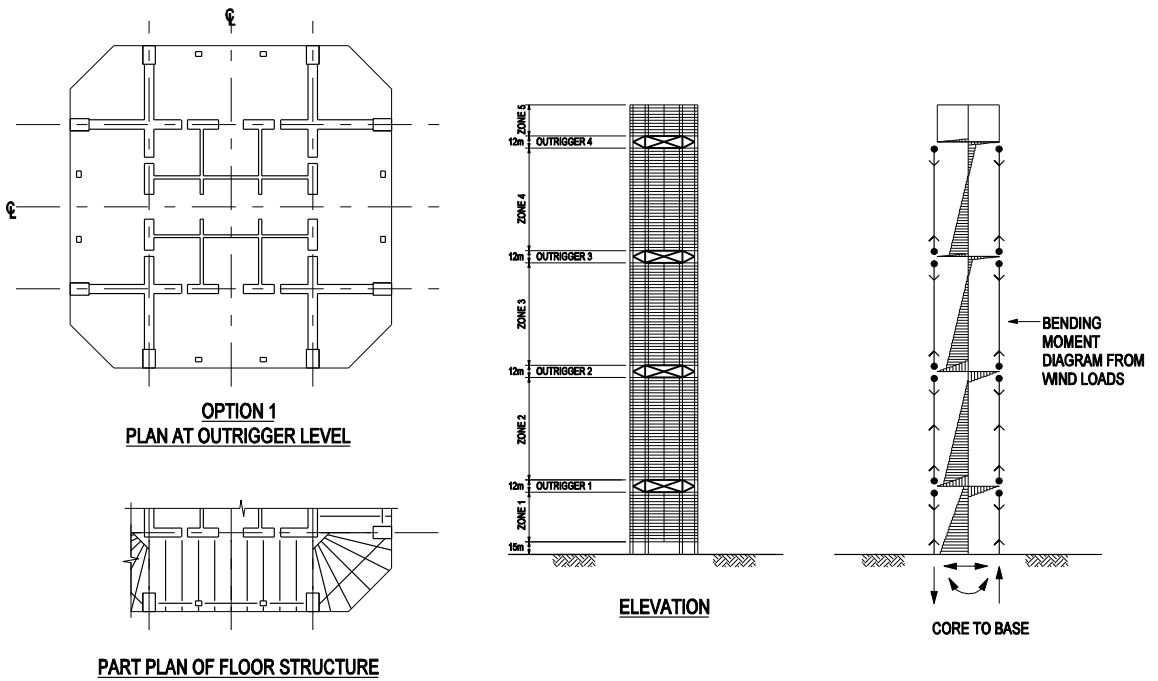


Figure E13.1.4 - Example of outrigger system

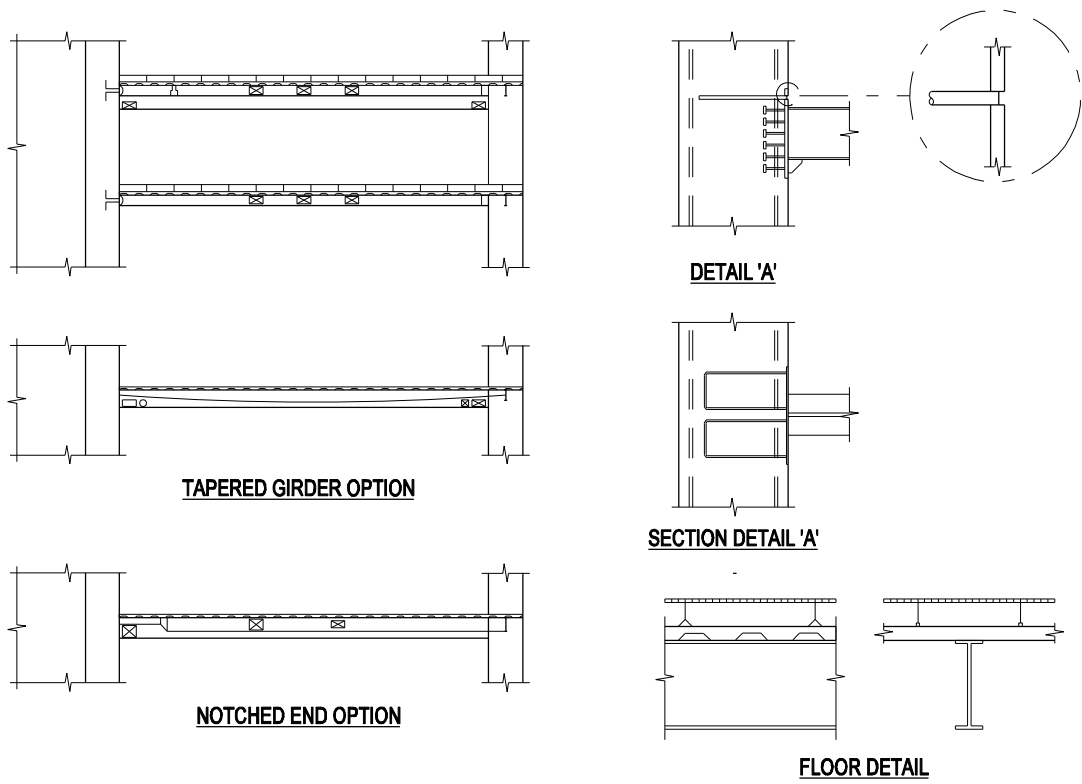


Figure E13.1.5 - Typical floor details including beam to core connection

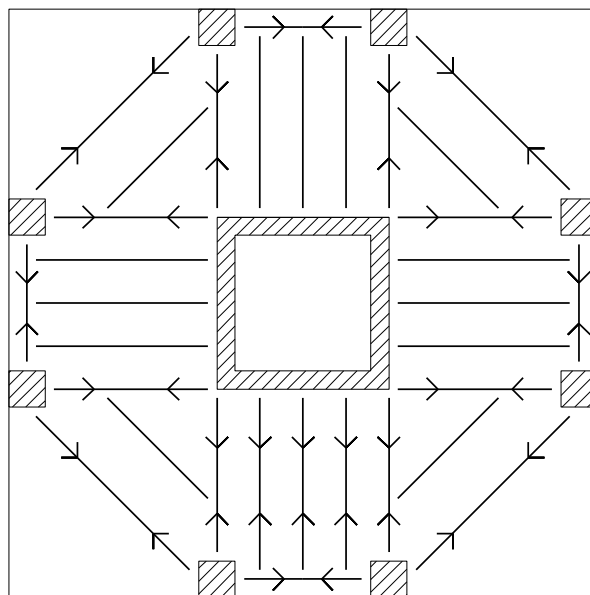


Figure E13.1.6 - Robustness ties for mega columns

E13.2 GUIDANCE ON DESIGN OF TRANSMISSION TOWERS, MASTS AND CHIMNEYS

E13.2.1 Structural systems for transmission towers, masts and chimneys

Clause 13.2.1 of the Code gives guidance on general issues for the design of steel transmission towers, masts and chimneys and is generally descriptive and self-explanatory. The Responsible Engineer should also refer to appropriate specialist literature and specific codes of practice for the design of steel chimneys; for example, the Practice Note for Authorized Persons, Registered Structural Engineers and Registered Geotechnical Engineers APP-8 (PNAP APP-8) or BS 4076: 1989 and transmission towers, for example BS 8100: Parts 1 to 4.

E13.2.2 Overall stability of towers, masts and chimneys

Clause 13.2.2 of the Code is self-explanatory.

E13.2.3 Particular details

Clause 13.2.3 of the Code is self-explanatory.

E13.2.4 Considerations for design against extreme events

Clause 13.2.4 of the Code is self-explanatory.

E13.2.5 Serviceability issues

Clause 13.2.5 of the Code is self-explanatory.

E13.2.6 Design issues for steel chimneys

Clause 13.2.6 of the Code is generally self-explanatory.

Buckling of thin walled chimneys requires consideration. Clause 13.2.6 (c) of the Code is updated from PNAP APP-8 permissible stress philosophy to a limit state approach. The relevant PNAP clauses are:-

(ii) To control buckling, the allowable compressive stress in the chimney structure should be limited to 115 MN/m^2 in the case of a chimney with effective height to diameter ratio of less than 21 and diameter to thickness ratio of less than 130. It should be reduced further for higher aspect ratios.

A 25% increase in allowable stresses due to wind load should not be applied to the design of the chimney.

(iii) Where the temperature is higher than 315°C , a reduction coefficient should be applied to the allowable stresses in the steel.

The limiting compressive stress of 140 N/mm^2 given in the Code is obtained by applying a partial load factor of 1.2 to the allowable value of 115 N/mm^2 . For simplicity and general dominance of wind loads and self weight, the design of a single value is used for all three principal load combinations.

E13.2.7 Construction and corrosion protection of steel chimneys

Clause 13.2.7 of the Code is generally self-explanatory. General guidance on corrosion protection is given in clause 5.5 of the Code. It should be noted that chimney gases are often highly corrosive.

E13.2.8 Wind-excited oscillations of circular chimneys

Clause 13.2.8 of the Code is self-explanatory. The method given may also be applied to the assessment of steel masts such as architectural features on buildings.

E13.3 GLASS AND FAÇADE SUPPORTING STRUCTURES

Clause 13.3 of the Code is generally self-explanatory.

E13.4 TEMPORARY WORKS IN CONSTRUCTION

In Hong Kong and many other places, collapse of temporary structures such as scaffolds is common. It is due to two main reasons, namely as relatively inferior quality for this type of structures and errors in design. The common use of a larger factor of safety of 2 indicates the profession has less confidence in its design. Permanent structures usually have smaller problem since buckling is prevented by wall and slab elements. For bare steel frames used as temporary structures, assumption of effective length is commonly erroneous and their buckling is common. This is essential to watch out for excessively deformed or damaged temporary structures on site. Also, temporary structures under heavy loads from permanent structures should receive special attention of sway and non-sway stability determined from the elastic critical load factor λ_{cr} .

E13.4.1 Design philosophy

Clause 13.4.1 of the Code is self-explanatory.

As shorter temporary structures have less severe consequence when collapse, the requirement for a proper design is looser.

Temporary structures including falseworks and scaffolding were noted to have a higher collapse rate than permanent structures. This section is aimed at providing a guide for the engineers on the design of this type of dangerous structures.

For temporary structures of height greater than 5m, the designer shall check the value of λ_{cr} using second order analysis. For temporary structures shorter than 5m, the designer should exercise professional judgment or to conduct second order analysis.

E13.4.2 Second-order effects

Clause 13.4.2 of the Code is self-explanatory.

Both the frame imperfection, the $P-\Delta_0$ imperfection and the member imperfection in the form of curvature or the $P-\delta_0$ imperfection are required to be considered in a design.

E13.4.3 Out-of-plumbness

Clause 13.4.3 of the Code is self-explanatory.

E13.4.4 Fitness tolerance

Clause 13.4.4 of the Code is self-explanatory.

E13.4.5 Member imperfections

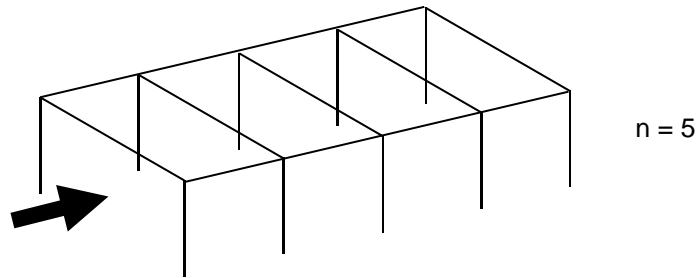
Member imperfection in columns of temporary structures should be taken as,

$$\delta = \frac{L}{500} \quad (13.7)$$

and this value may be reduced when columns are placed in parallel as,

$$\delta = \frac{L}{500} \cdot \frac{1}{\sqrt{n}} \quad (13.8)$$

where n is the number of structural elements arranged parallel to each other and similarly supported and propped, with their deformations of the same magnitude due to systematic influences can be excluded.



E13.4.6 Support settlements and flexible supports

Clause 13.4.6 of the Code is self-explanatory.

E13.4.7 Over-turning

Clause 13.4.7 of the Code is self-explanatory.

E13.4.8 Tolerance and clearance

Clause 13.4.8 of the Code is self-explanatory.

E13.4.9 New and used systems

Reduction of buckling resistance of a used system is required in BS code. Whether a unit is old or new is not judged by whether or not it has been used before, but by the condition as whether the members are bent and whether or not extensive rusting is founded. The judgment should be given by an experienced engineer or the Responsible Engineer. The classification system is not only useful for classifying a scaffolding unit, but also to encourage scaffolders to take a good care of the units and to keep them in good conditions, such as painting everything after use.

Provided that the members are not damaged noticeably and the units are in good conditions, the design strength of “used” materials of proprietary or non-proprietary systems in temporary structure should be taken as 85% of the design values for “as new” materials. This recommendation is in accordance with section 6 of BS5973: 1993 “Code of practice for Access and working scaffolds and special scaffold structures in steel”.

E13.4.10 Module Testing

Except when the second-order $P-\Delta-\delta$ analysis is used, load resistance of scaffolds can hardly be calculated. In most cases, a full-scale test is called upon to determine the nominal reference load resistance of a scaffold.

Modular scaffold should be designed and used in accordance with the manufacturer’s recommendations. Full justifications including buckling design check by second-order analysis and further tests may be required for critical scaffold modules not covered by the manufacturer’s recommendations.

Module testing for the proprietary scaffolding and temporary structural unit should be carried out for height not previously tested or substantiated in manufacturer’s manual in order to confirm the accuracy of computed design resistance.

E13.5 LONG SPAN STRUCTURES

Long span structures are necessary for stadia and roofs over exhibition halls, airports, railway stations, aircraft maintenance hangers and similar buildings in order to provide large column free spaces.

Particularly with such structures, the engineering can influence the form of the building. There is a paradox that it has only more recently been possible for clever structural tricks to allow designers to free building forms from strict functional logic. Even if hidden from view or covered by cladding, designers should strive for good proportions and simplicity of main concepts, but pay attention to key details at an early stage in the design process.

Long span structures can use concrete in the form of arches and shells or, more rarely, timber or even masonry. However, the predominant material used today is structural steel. The steelwork is often exposed and its appearance is often important. Structural weight and buckling stability are key structural design issues whilst important non structural design issues include fire, crowd management, lighting, comfort, orientation of occupants within the building and appearance.

Long span structures using apparently simple and graceful forms such as arches, three dimensional grid shells, catenary cables or properly proportioned trusses are often elegant.

Concepts for structural systems and shapes can be inspired by Nature, by studying both ancient and more recent engineering and architectural history, by observation of other technologies and from brainstorming with other design team members.

Structural design is only a part of the effort required to build a building, analysis is only a part of design, computing only a tool for analysis. When used correctly, the computer can remove the need for complex and tedious calculations, allow us to test a number of schemes and obtain a 'feel' for their relative merits. Increasingly, we will rely on sophisticated computer analysis, particularly when non-linear effects are important or where seismic loads must be considered. But when computers are used unwisely, garbage may emerge; thus the skills to carry out reality checks on complex designs must be developed and preserved.

E13.5.1 Systems for long span structures

Suitable structural systems for large spans may be classified as direct force systems, bending/trussed systems or air supported systems.

Direct force systems

Direct force systems, that is where forces are carried in the plane of the member, i.e. arches or catenaries, are generally more efficient than bending structures for large spans. Compression systems of arches or shells and tension systems of cable stayed or cable net forms are possible. Direct force systems need external space for abutments to carry the large in-plane compressions or anchorages to take tension reactions, they also need significant height for structural efficiency and to reduce the external reactions.

Truss or bending systems

Bending systems may be portal frames or one-way trusses or two-way space frames depending on the plan aspect ratio of the space to be enclosed. If this ratio is more than 1 to 1.5 or so, there will probably be little advantage to be gained from a two-way space frame system. Frames can be portalised to improve efficiency, giving rise to horizontal reactions which could be tied through the base slab. Truss span to depth ratios in the range 15 to 18 are reasonable and precambering can mitigate self weight deflections.

Air supported systems

Large spaces can be enclosed by air supported fabric structures, for example B.C. Place stadium, Vancouver, or the scheme by Arup for the Bridge of Don in Scotland. To cater for live, wind down force, snow and self weight, an internal overpressure of around 3 to 5kPa is required. Normal atmospheric pressure is about 100kPa, thus the difference is not noticeable to occupants. However, fans must run continually to maintain the pressure in a leaky system. For large spans, tie down cable are required and the roof must be able

to span as a catenary above the heads of occupants should there be a catastrophic loss of pressure.

The steelwork for long span building structures is often exposed and thus its appearance is important. As designers, we are often too stressed and not enough geometry/stability oriented, structural weight and buckling stability are key design issues for long span structures.

For the particular case of stadia, a typical roof system comprises girders or trusses spanning between the ends of the stadium and along the front edge i.e. above the pitch to seating boundary. Typically, secondary trusses span between the main girder, truss or arch to the stadium rear perimeter. Tertiary trusses or beams may be used. Stadia seating frames may be steel, precast concrete or insitu concrete. Bleacher seating is typically of precast concrete.

To summarise, primary structural systems for long span roofs may be:

- (a) Concrete shells or steel grid shells.
- (b) Concrete or steel arches.
- (c) Space frames.
- (d) Portal frames.
- (e) Long span trusses or prestressed concrete beams.
- (f) Cable stayed or cable suspension systems with masts.
- (g) Hybrid trusses with steel tie rods and masts – “tree” systems.
- (h) Air supported systems.

E13.5.2 Overall stability of long span structures

Clause 13.5.2 of the Code says that various critical stability checks shall be made for long span steel structural elements.

The avoidance of geometrical instability is of critical importance for long span structural elements and checks should be made on overall system buckling, member buckling and snap through instability. System imperfections and lack of fit should be considered.

Lightweight long span roofs may be subject to stress reversals under wind uplift with the possibility of truss bottom chords going into compression. Failures of shell structures have been caused by unsymmetric loading, for example from drifting snow combined with wind.

The method and sequence of construction will influence the design and should be properly taken into consideration, in particular the stability of partially completed structure shall be ensured during construction.

The erection of long span structures can be a major exercise and possible methods should be considered at the design stage. Often, it will be necessary to assemble large items such as steel arches on site, possibly on the ground and then lift up, or by assembly in final position on temporary falsework. Sometimes, a complete roof may be assembled at ground level and lifted into place by strand jacks working from temporary or permanent towers. Temporary or permanent support bearings may be necessary and form part of an erection strategy.

A properly sequenced fast track design and construction programme is often required and it is necessary for the design team and contractor to work closely together as particular construction methods must be considered at design stage.

E13.5.3 Particular details

Clause 13.5.3 of the Code draws attention to particular key design details which should be considered properly at an early stage in the design process. Some particular details requiring special consideration are:

- (a) Springing details for arches or shells.
- (b) Steel masts and their bases.

- (c) Cable fixings.
- (d) Connections of main truss elements.
- (e) Connections of secondary to main trusses to provide restraint against buckling.
- (f) Need for access to roof for maintenance, lighting, television cameras.

The dimensional tolerance of interconnected components forming a large span should be carefully thought through.

Increasingly, today's stadia are required to act as multi-functional entertainment centres and require to be fully covered at times, thus an opening roof becomes necessary. The design of opening roofs requires coordination between mechanical and structural systems designers.

E13.5.4 Considerations for design against extreme events

Crowd barriers must be designed to resist large crowd loads without collapse. There have been a number of serious accidents at stadia caused by crowds surging forward and people being crushed. (refer to IStructE guidance)

Long span roof trusses and main masts should be designed as key elements, i.e. the designer should consider possible extreme event loadings which may occur and, if practical, design key elements to carry forces arising from them. For example, main columns should generally be treated as key elements and the collapse of a single span of a multi span structure should not cause the collapse of adjacent spans.

Fire engineering may be used to justify that fire protection may not be required for roof steelwork. Intumescent paint or water cooling may be considered where it is required to expose steelwork which needs protection.

E13.5.5 Serviceability issues

Clause 13.5.5 of the Code requires that the following serviceability issues need to be addressed:

- (a) Deflections. Absolute values of deflection may be quite high and can be mitigated with precamber so long as ponding is avoided and stability is not in doubt. Deflection limits for long span trusses under live and wind loads depend on circumstances. A value of span/360 may be used for preliminary design in the absence of other requirements. Significantly, smaller deflection limits will be required for applications such as: aircraft hanger doors and stadia opening roofs.
- (b) Vibration from crowds, especially at concerts and events where rhythmic movements may occur.
- (c) Wind induced oscillations of roof elements and cables. Wind tunnel testing may be advantageous to assist understanding of loads and dynamic effects more precisely.
- (d) Access for maintenance of roof steelwork can be very difficult, therefore a high quality protective system should be specified for the steelwork. Corrosion protection may be provided by a high quality paint system or by galvanising or flame sprayed zinc, with additional layers of paint over. In such a case, the metallic protection must be carefully prepared to take painting.
- (e) Movements caused by wind and temperature change need to be considered. Interface details between roof and façade walls may need to accommodate large relative movements.

Figures 13.5.1 to 13.5.5 illustrate some systems for long span structures.

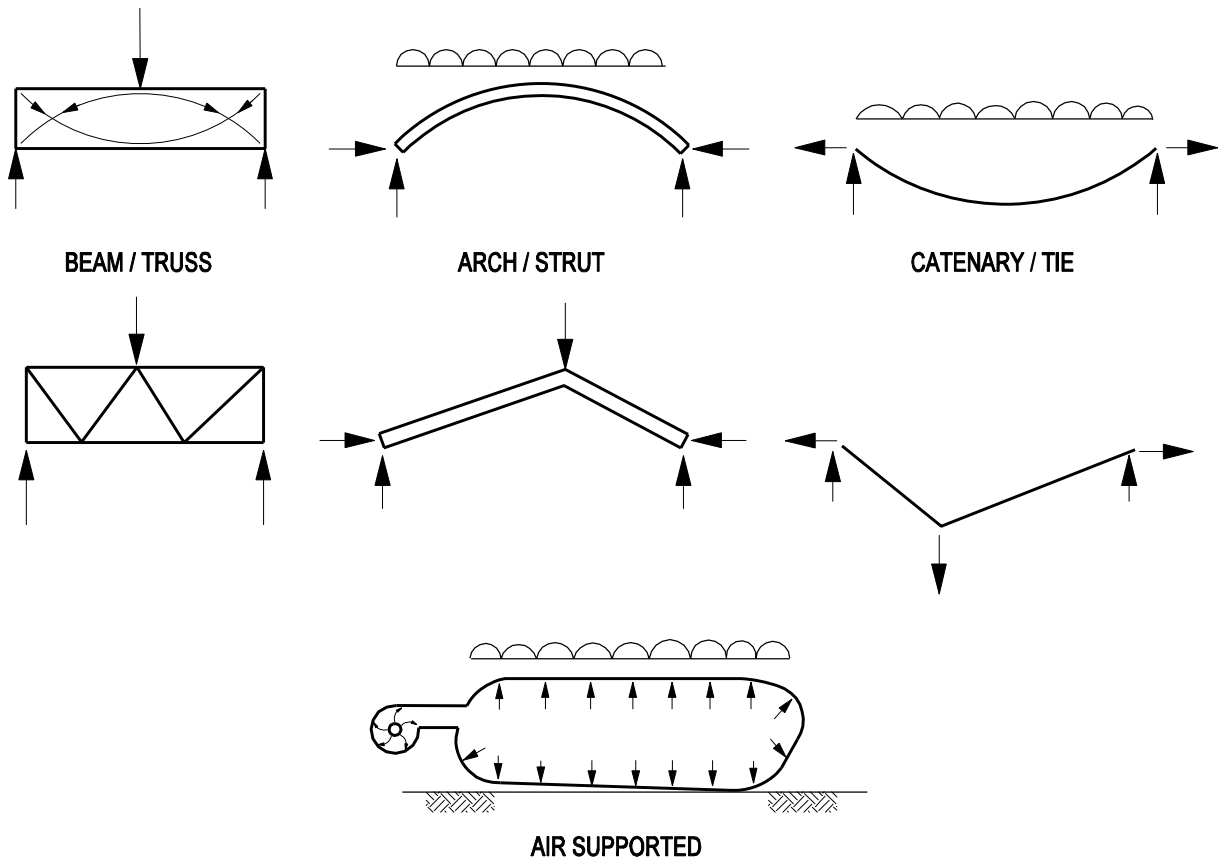


Figure E13.5.1 - Systems for Long Spans

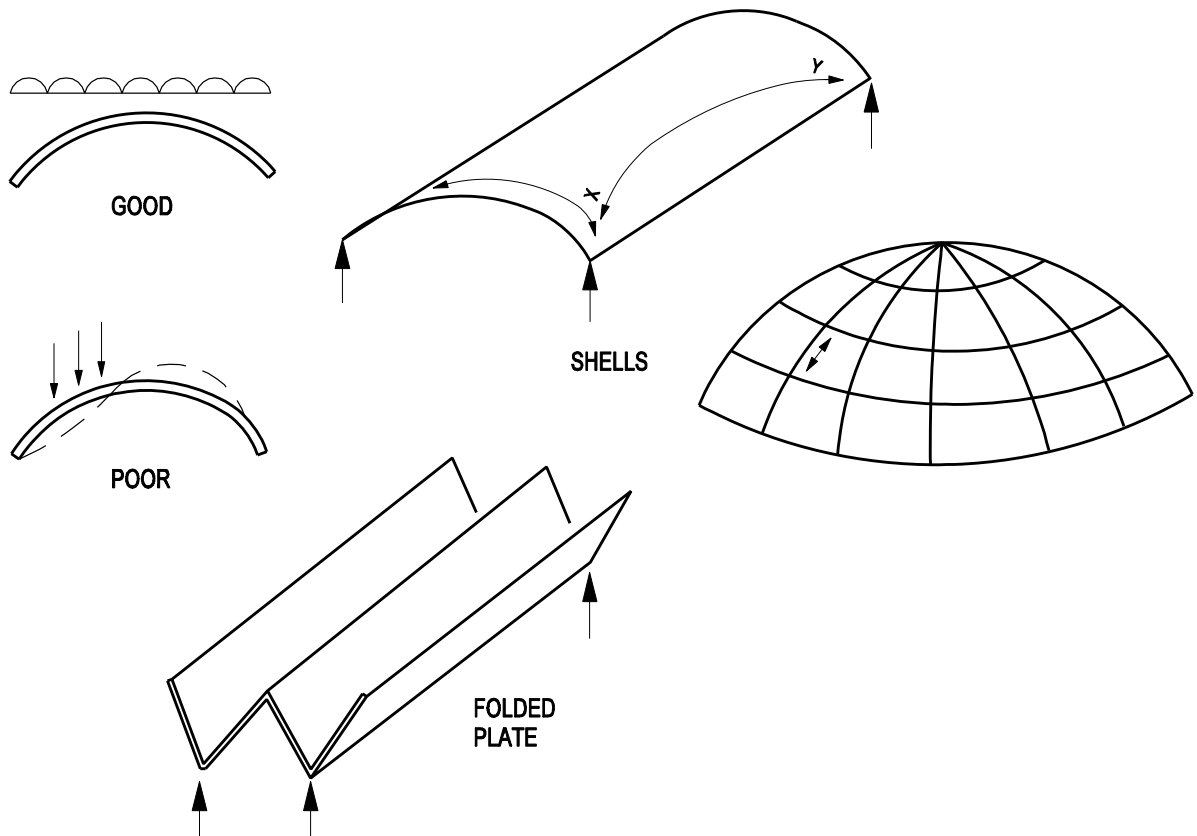


Figure E13.5.2 - Compression Systems

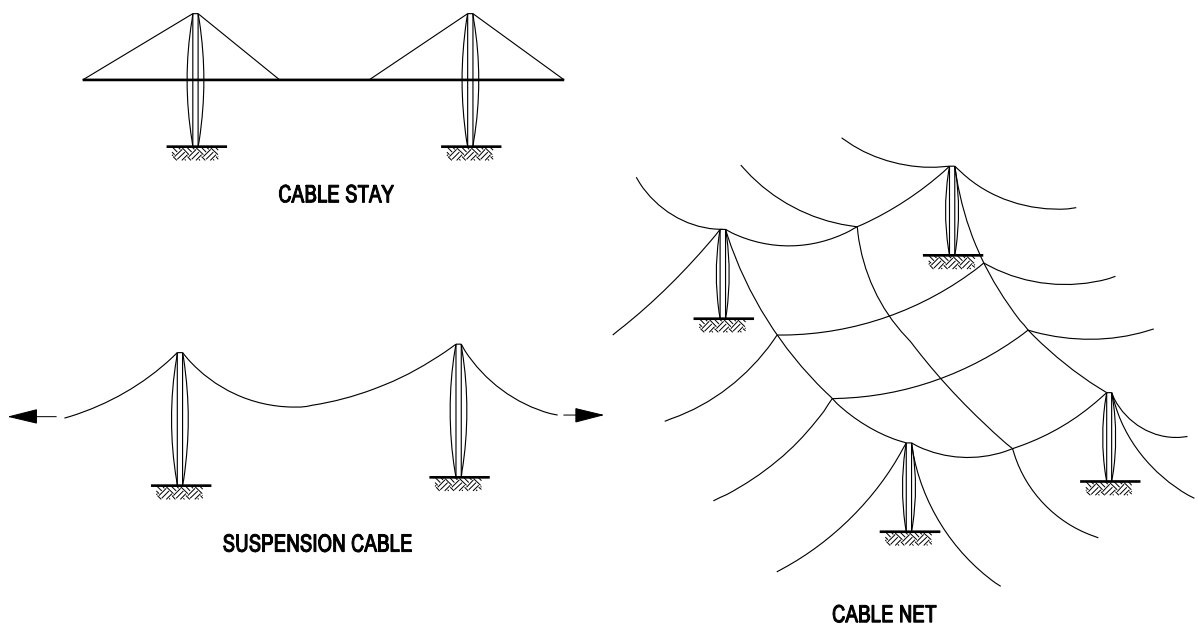


Figure E13.5.3 - Cable Systems

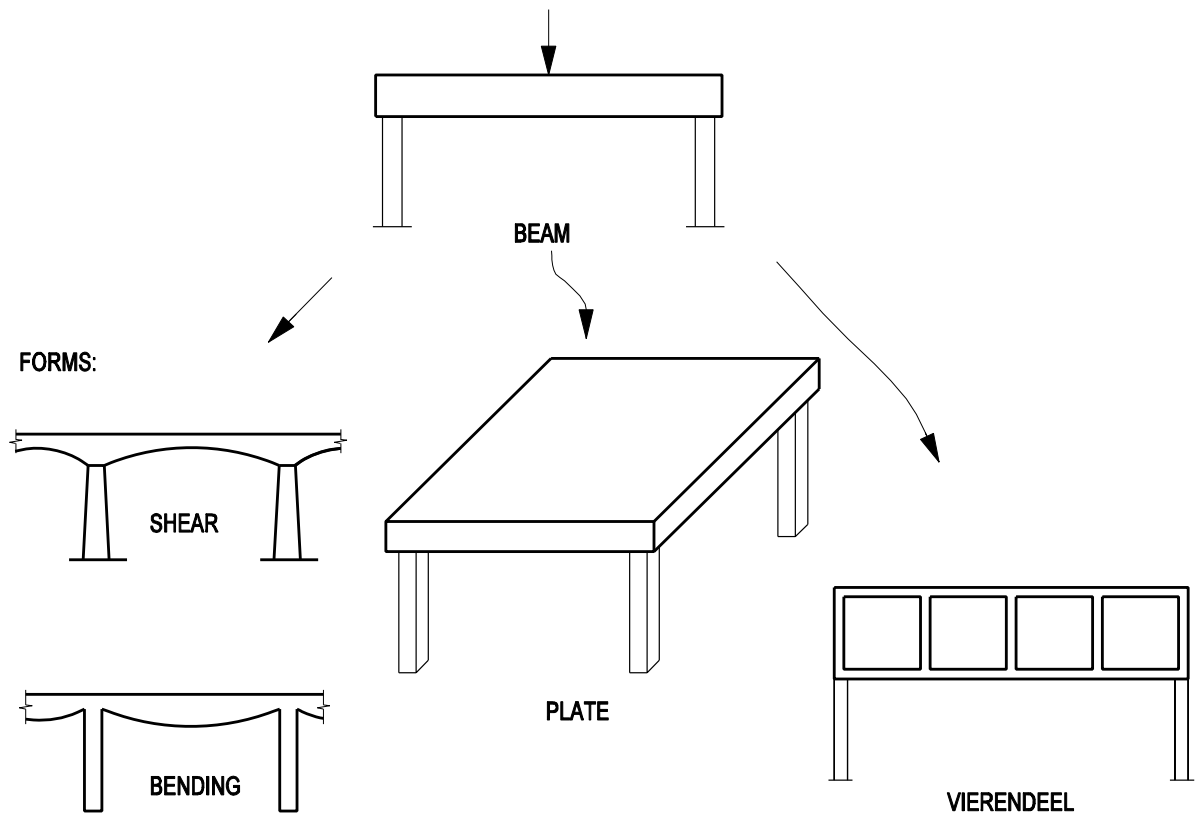


Figure E13.5.4 - Bending Systems

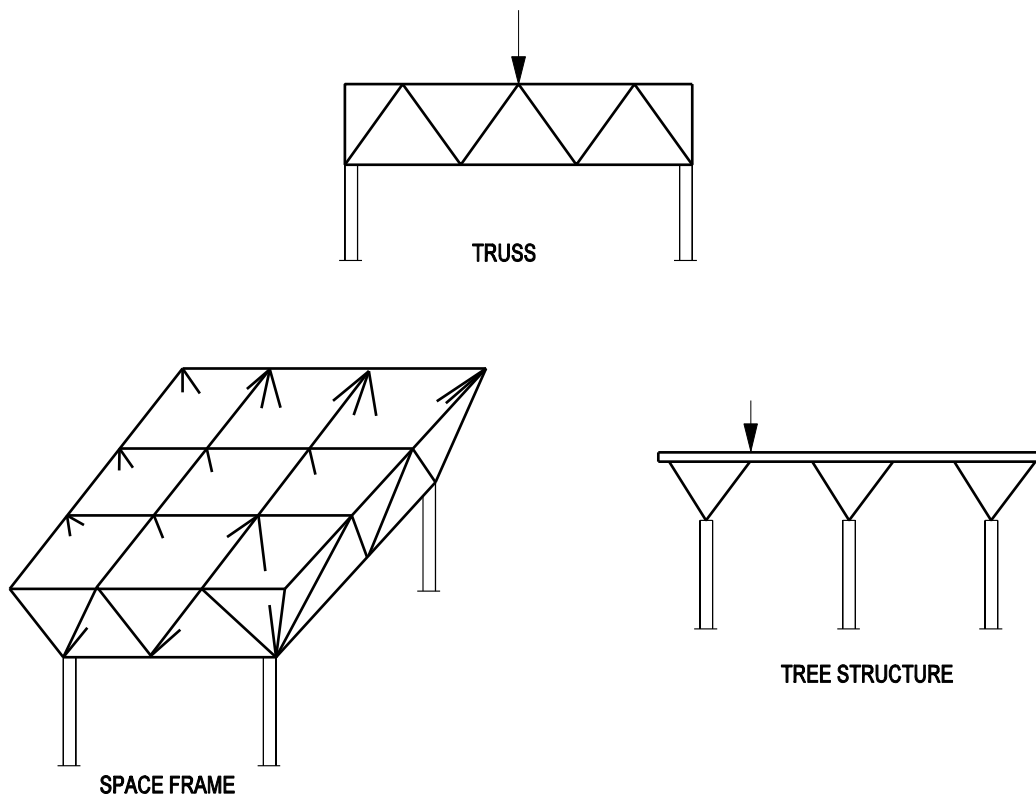


Figure E13.5.5 - Truss Systems

E13.6 FOOTBRIDGES

Clause 13.6 of the Code is generally self-explanatory.

E13.6.3.3.1 Fatigue – General

- (b) Cold-formed steelwork should not be used for footbridges in which fatigue predominates, unless adequate data for the fatigue assessment are available for non-fatigue sensitive elements such as parapet, sign gantry and bridge accessories.

E13.6.4 Vibration and oscillation

The human-induced vibration frequency caused by walking and jogging are generally 2 Hz and 2.5 Hz; hence, the design natural frequency of footbridge can reasonably be set at 3 Hz, in which occurrence of dynamic resonance is highly suppressed.

E13.7 DESIGN LOADS FROM OVERHEAD RUNWAY CRANES, TOWER AND DERRICK CRANES AND MOBILE CRANES

Clause 13.7 of the Code gives informative guidance on types of cranes and recommendations on the loads which static and runway cranes apply to building structures. The design of cranes themselves is a specialized activity which is not covered by the Code. BS 2573 may be referred to for the design of cranes themselves and this standard gives requirements for dynamic loads and fatigue.

If more reliable information is not available, it is prudent to design using the well tried requirements given in old codes such as BS 449 for dynamic loads. These are given in clause 13.7.3.1 of the Code and are to use an increase of 25% on static vertical loads (i.e. crane self weight plus hook load). A horizontal load of 10% of vertical wheel loads should be taken transverse to the rails and 5% along the rails should be taken.

The Responsible Engineer should apply sound engineering judgment when considering design loads and load combinations from cranes. This requires an understanding of the various types of crane and how they operate.

Cranes will impose large and fluctuating loads on structures. If the crane is heavily used, then fatigue may need to be considered in the design of the supporting structural elements.

E13.7.1 Types and classifications of static cranes

Clause 13.7.1 of the Code is self explanatory and describes various types of static crane, however the descriptions are repeated and expanded here for ease of reference and clarity. The clause also describes mobile cranes which may be used on a site and can apply construction loads to parts of the permanent structure.

Overhead runway cranes

Overhead runway cranes comprise a main girder supported on rails at each end. The load is hoisted and carried by a trolley which traverses along the main girder. The main girder can traverse along the end rails and thus the load can be moved in two planar dimensions over the building area. The rails are supported on a gantry girder which requires to be designed as part of the building structure.

Tower and derrick (or luffing) cranes

Tower cranes consist of a horizontal girder attached to a vertical mast supported from a temporary base attached to the permanent building structure or on a separate foundation at a suitable location, e.g. a lift shaft or light well.

Derrick cranes are often used for steel erection in Hong Kong and are typically attached to a mast which is extended from the building in a similar way to that for tower cranes.

From time to time, the Responsible Engineer for a building structure will need to consider the temporary loads imposed on the permanent structure from a tower or derrick crane. He or she may also need to check its design adequacy, since the cranes may be fabricated from countries with different and lower wind loads than can arise during typhoons. Clause 13.7.3.4 of the Code requires the Responsible Engineer to obtain all relevant crane loading data from the main contractor and tower crane supplier and carry out checks, or have checks carried out by the crane supplier, to justify the crane and permanent structure for the crane loads.

Mobile cranes

Mobile cranes may be lorry mounted, typically with a telescopic box section boom and supported on jacks attached to the crane by outriggers when in use.

Alternatively, cranes may be mounted on a wide tracked base platform, typically with a trussed boom.

Parts of the permanent structure may be required to support mobile cranes during construction and in this case, the Responsible Engineer is required in clause 13.7.3.5 of

the Code to obtain loading data from the contractor or crane supplier. This data should include loads arising from an envelope of boom positions in plan slew and azimuth angles.

E13.7.2 Design issues for crane support structures

Clause 13.7.2 of the Code is generally self explanatory.

The deflection limits for crane support girders may be more stringent than for normal beams. In the absence of manufacturers data, vertical deflections should be limited to span/600 and horizontal deflections to span/500.

Local compressive stresses at the top of the web of a crane girder will be high and will fluctuate as the wheels pass over a particular point. Both local web bearing and weld capacity require consideration.

E13.7.3 Loading from cranes

Clause 13.7.3 of the Code is generally self explanatory.

Loading from overhead traveling cranes

For overhead travelling cranes, the vertical and horizontal dynamic loads and impact effects should either be established in consultation with the crane manufacturer or by using engineering judgment and conservative values for dynamic and horizontal effects.

Loads arise from the dead load of the crane, the hook load being lifted, horizontal loads from braking and skewing (crabbing) and buffer collision loads.

Partial load factors are given in Table 13.3 of the Code for vertical and horizontal loads from overhead travelling cranes. These factors should be applied to the dynamic vertical wheel loads, i.e. the static vertical wheel loads increased by the appropriate allowance for dynamic effects.

Where a structure or member is subject to loads from two or more cranes, the Responsible Engineer should use engineering judgment to consider those load combinations which could occur.

E13.8 GUIDANCE ON MAINTENANCE OF STEEL STRUCTURES

Steel structures require relatively little maintenance provided there is no change to the environment for which any protection was originally specified. Degradation through ongoing corrosion is the most common issue.

Maintenance may be an aesthetic requirement, to restore and preserve the appearance of a structure, rather than a structural requirement.

The extent and frequency of such maintenance as is required can be controlled by specification of appropriate protection initially in the case of new construction. For existing construction, an assessment should be made of the nature and condition of the steelwork and any initial repair/maintenance work undertaken as necessary in conjunction with the establishment of an appropriate regime for future work.

Access needs to be considered when planning maintenance. Where this is not possible - either physically or in practical terms, e.g. where an external stanchion is built into the external wall – this needs to be considered when specifying the original protection.

BS 5493: 1977 Code of practice for Protective coating of iron and steel structures against Corrosion (declared obsolescent but remains current and is cited in B(C)R) contains useful information.

BS EN ISO 12944-8: 1998 Code of practice for Protective coating of iron and steel structures against corrosion, Part 8: Development of specifications for new work and maintenance contains a flow chart for planning maintenance work.

E13.8.1 General

Clause 13.8.1 of the Code is self explanatory.

E13.8.2 Consideration of maintenance in the original design

Clause 13.8.2 of the Code is self explanatory.

E13.8.3 Maintenance of existing construction

Clause 13.8.3 of the Code is self explanatory.

While the focus is very much on the avoidance of corrosion, there are structures where fatigue is a design issue, or vibration is anticipated. In such cases, holding-down bolts should be checked for tightness and any welds checked for cracking. Bearings will also require inspection.

E13.8.4 Health and safety issues

Clause 13.8.4 of the Code is generally self-explanatory. The key health and safety issues are: access, correct use of materials and environmental considerations. It is advisable to prepare a safety plan as part of the maintenance working procedure to ensure all safety issues are addressed. Further advice is given in clause E17.2.2.

E14 FABRICATION AND ERECTION

Section 14 of the Code gives detailed guidance and requirements on fabrication and erection of steel structures. Much of its content is in the form of specifications for workmanship and is self-explanatory. It is sourced from Hong Kong steelwork practice, Arup specifications, UK NSSS and BS5950: Part 2.

E14.1 DIMENSIONS AND TOLERANCES OF SECTIONS

Clause 14.1 of the Code states that dimensions and tolerances of hot rolled sections and cold-formed sections shall comply with reference standards as given in Annex A1.8 of the Code. Requirements for straightness are particularly important for members carrying axial loads or bending where buckling can occur.

Clause 3.1.2 of the Code requires that steel product manufacturers roll sections to their stated nominal sizes within their specified +/- tolerances such that average section sizes and properties are at least the nominal values. The Responsible Engineer should ensure that any steel used complies with this or he/she shall take account of any adverse variation in his design.

E14.2 WORKMANSHIP – GENERAL

E14.2.1 Identification

The intention of clause 14.2.1 of the Code is to ensure that it is possible to identify the steel material throughout the production and fabrication process from casting of the steel to fabrication into a component to its final destination in the completed structure.

E14.2.3 Cutting

Clause 14.2.3 of the Code gives requirements and limitations on cutting. Flame cutting processes may leave hard edge surfaces which require removal by grinding. Hand flame cutting is generally not permitted but in practice will be carried out, thus it is important that such cut edges are dressed.

Hardness tests should test the plate centreline to check for high hardness value, which could indicate if centreline segregation has occurred.

The shearing process can work harden the sheared edge, hence the limitations on thickness.

Re-entrant corners must be notch free and formed with a minimum radius of 5mm to reduce the possibility of cracking from stress concentration.

The clause in the Code says that columns and compression members intended to be in direct bearing shall be fabricated to the accuracy given in section 15 of the Code. Cold sawing by accurate modern circular saws can leave a clean square edge which will normally produce a surface acceptable for bearing without further end milling.

E14.2.4 Shaping and forming

Where hot forming is used, details of the process including temperature, application time and cooling time should be agreed between the Responsible Engineer and the fabricator. This is to ensure that the proposed temperature and method of heating and cooling will not adversely affect the properties of the steel.

Hot forming of cold formed thin gauge members and sheeting or of Quenched and Tempered high strength steel is not permitted by the Code because these steels derive strength from the forming or tempering process, which will be lost when heated above 600°C to 650°C.

Care should be taken to avoid local wall buckling of rolled hollow sections.

See also clause E14.2.7.

E14.2.5 Holing

Clause 14.2.5 of the Code is generally self explanatory.

Table 14.1 of the Code gives tolerances on drilled and punched hole diameters. The tolerances for fitted bolts are tighter. All tolerances are over the exact size. It is noted that bolt holes for normal shear connections and for high strength friction grip connections are generally 2 mm over the specified bolt size for 16 mm to 24 mm bolts and 3 mm for 27 mm to 36mm bolts.

The punching process is similar to the shearing process and can result in locally severely work hardened edges, thus clause 14.2.5.5 of the Code places limitations on punching. To reduce these limitations, undersize punching followed by reaming may be used. Punching should not be permitted in situations where fatigue loading may occur or where high ductility is required for plastic design or in seismically critical connections. Punching may also cause dishing of the punched plate which, if severe, would reduce the strength of a friction grip connection.

Clause 14.2.5.7 of the Code gives requirements for fabrication of holes for fitted bolts and pins.

E14.2.7 Curving and straightening

At temperature above 850-900°C steel, e.g. quenched and tempered steel typically with yield strengths of 460 N/mm² and above, will lose part of their strength. Thus, hot bending along the entire length of the element should not be carried out above these temperatures on such steel to avoid affecting their steel yield and tensile strengths and ductility. The Code ensures this by disallowing hot methods of bending for such steel.

Spot straightening of normal steel can be carried out by locally heating part of the section to 650°C, that part of steel then expands, deforms plastically and on cooling shrinks and straightens the section.

Induction bending will heat the whole of a cross section to 885 to 900°C (cherry red colour) and may be used for steel with yield strengths up to 460 N/mm² other than the quenched and tempered steel. Other means of heating to this temperature may be employed subject to the restrictions of clause 14.2.7(i) of the Code. See also clause E14.2.4 on shaping and forming.

E14.3 WORKMANSHIP – WELDING

E14.3.1 General

Clause 14.3.1 of the Code is generally self-explanatory.

For consistency, the clause requires that all welding operations for a particular project shall be carried out in accordance with one set of standards, either to the American Welding Standards or to the British versions of the Eurocodes, as contained in Annex A1.4.1.

The requirements for welding given in clause 14.3 of the Code are generally applicable to welding of higher strength steel but additional and more stringent procedures will often be necessary for such materials.

E14.3.3 Welding procedure specifications

Clause 14.3.3 of the Code is generally self-explanatory.

Avoidance of lamellar tearing- requirement for through thickness properties

The welding procedures should be chosen so as to minimise the risk of lamellar tearing. If necessary, material with through thickness properties shall be used.

E14.3.4 Assembly

Clause 14.3.4 of the Code is generally self-explanatory.

Unless they are properly made, tack welds can act as local stress raisers in fatigue situations; thus they should be removed unless inspected and found to be satisfactory.

Similarly, attachments required for fabrication or erection purposes can act as stress raisers, thus should be made in accordance with the requirements for a permanent weld; and when removal is necessary, it should be properly carried out to avoid damage to the surface of the parent material.

The sequence of welding a joint or a sequence of joints to control distortion requires experience from the designer and fabricator.

E14.3.6 Non-destructive testing of welds

Clause 14.3.6 of the Code is prescriptive and generally self-explanatory. The various types of inspection and testing that are normally carried out on welds are:

- a) Visual inspection for surface defects and measurement of weld size.
- b) Use of a penetrating dye which will concentrate in and thus assist in showing surface cracks.
- c) Use of magnetic particles. This involves spraying the weld surface with iron or steel filings in a liquid suspension and then applying a magnetic field by use of a powerful magnet with adjustable shoes. The magnetic field will be interrupted by surface defects which will therefore be highlighted by the particles.
- d) Ultrasonic testing where an ultrasonic transmitter sends waves through the body of the weld and steel plate and a receiver picks them up. The waves are distorted by internal discontinuities such as cracks and inclusions. Skilled interpretation of the waveform pictures is required.

Visual inspection of welds

Visual inspection should always be made over the full length of welds before carrying out any further non destructive testing which is specified. Obviously, any welds which will be rendered inaccessible by subsequent work shall be examined prior to the loss of access.

Visual inspection shall be carried out at all welds by a qualified welding inspector.

Regarding clause 14.3.6.3 of the Code, it cites that a suitably qualified person for visual inspection of welds may be a welding inspector who can provide evidence of having trained and assessed for competence in visual inspection of relevant types of welds.

In this context, the “suitably qualified person” may be construed as either a certified welding inspector with requisite qualifications (i.e. ASNT Level II, PCN Level 2, or CSWIP 3.1 or equivalent) or a competent person who can provide evidence of training and working experience in visual inspection of welds. Besides, the visual inspection of welds can also be carried out by HOKLAS laboratories accredited for this activity.

In the meantime, professional structural engineer of TCP – T5 level topped up with visual inspection course should have sufficient knowledge in carrying out visual inspection on welds to ensure weld quality. However, it is a good practice that a suitably qualified person such as a qualified welding inspector as mentioned above could be appointed to ensure weld quality.

Hold times before final Non Destructive Testing

Owing to the risk of delayed cracking, a period of at least 16 hours should generally be allowed before the final inspection is made of as-welded fabrications. Table 14.2b of the Code provides illustrative hold times for steel of various thicknesses and carbon equivalent values.

Surface flaw detection

Where a closer examination of a weld surface is required, magnetic particle inspection (MPI) shall be used. If magnetic particle inspection equipment is impractical, dye penetrant inspection (DPI) may be used.

Final surface flaw detection of a welded joint shall be carried out after completion of the weld and the hold times given in Table 14.2b of the Code.

Table 14.3a of the Code provides recommendations on the scope and frequency of inspection of various types of welds.

Ultrasonic examination

Where ultrasonic examination (U/S) is required, it shall be made in accordance with the requirement as contained in Annex A1.4.4 of the Code. Ultrasonic examination of the welded joint shall be carried out after completion of the weld in accordance with the correct hold times.

Ultrasonic examination can be used to examine the through-thickness properties of the parent material for weld geometries susceptible to lamellar tearing.

Because of the skilled nature of the process, operators carrying out final ultrasonic examination of the weld shall hold a valid certificate of competence from a nationally recognised authority.

Acceptance criteria and corrective action

Unless otherwise specified, the acceptance criteria for weld imperfections shall meet the minimum requirement as contained in Table 14.3b of the Code. Welds that do not comply with the requirement shall be repaired in accordance with approved welding repair procedures.

Any corrected welds shall be inspected again with increased hold times and shall meet the requirements of the original welds.

In cases where fatigue can occur, more stringent acceptance standards than these minima will need to be specified by the Responsible Engineer.

Table 14.3c of the Code defines zones and weld orientation and table 14.3d of the Code illustrates characteristic defects of welds in steel structures.

E14.3.7 Shear stud welding

Clause 14.3.7 of the Code is generally self-explanatory. Studs are usually welded using a proprietary stud welding gun which holds the stud, starts an arc and maintains it for a preset time, then plunges the stud into the weld pool.

The process is often carried out through the galvanised metal decking of typical 1 mm thickness. This usually does not cause problems but top surfaces of beams should not be painted (a prefabrication primer may not cause problems). Problems can arise from moisture from condensation overnight.

Studs subjected to the bend test shall not be straightened but are assumed to be effective in the permanent works.

E14.3.8 Further information on underwater welding

Underwater welding may be required in some situations during building works, such as for buildings on steel piles over the sea or special structures such as underwater observation platforms. The normal welding practices and procedures which apply to surface operations are required but important differences exist and this note discusses these. Underwater welding can be divided into three main types:-

Coffer dam welding

This is essentially a dry operation in a normal air environment. It is achieved by constructing a rigid structure to exclude water and provide a safe dry environment for welders to work at ambient pressure. Most commonly used for civil engineering works, harbour works and ship repairs.

Wet underwater welding

This is conducted by a welder/diver in the water using breathing apparatus and the arc is directly exposed to the wet environment. The most common welding process is shielded metal arc welding (SMAW) using electrodes that have been waterproofed. Flux-cored arc welding is also used when greater deposition is required. A hybrid approach to wet welding is semi-dry using a small chamber over the repair area which is pressurised to exclude water. The welder/diver only places his hands in the chamber and views the welding through a window in the side of the chamber. Close surface support is required for communications, breathing gas and welding control. The main advantage of wet

welding is that it is the most versatile and economic method. Disadvantage is that the weld quality is not as good as that conducted in the dry but research has produced much improved quality.

Hyperbaric welding (dry)

In this operation, a chamber is used to seal the area to be welded (such as a pipeline) and the chamber is pressurised. The pressure forces out the water and maintains it at a pre-determined level. The welder/diver works in his diving suit/helmet using the surface supplied breathing gas. The chamber can also be completely sealed from the water and have either ambient pressure or even at 1 atmosphere if required. This will require air locks and bell transfer for the welder/diver.

Welding processes used are SMAW, FCAW and gas tungsten arc welding (GTAW) and give high integrity welds. These are mainly used for rig repairs, pipelines tie-ins and risers. Research has produced automatic welding systems that give excellent weld quality down to 500 m. It is expected that further research will give even greater depths. As with wet welding, surface support is crucial for communications, environment control, breathing gas and welding operations.

Risks

When humans are involved, the main risks are:-

- Electric shock, which can be reduced by proper controls,
- Explosion, from the build up of explosive gases, and ,
- Occupational health hazards.

Occupational Health Hazards

Divers are exposed not only to the possibility of drowning but also to a variety of occupational safety and health hazards such as respiratory and circulatory risks, hypothermia, low visibility, and physical injury. Some health hazards are as follows:

- (a) Pain and barotrauma from expanding or contracting gases while transiting between pressure zones are the most direct effects predictable from Boyle's Law. The most common sites of pain from trapped gases are the digestive tract, sinuses, teeth, middle ear, and lungs (the latter particularly during ascent).
- (b) Decompression sickness (DCS) is the most commonly known of the many dysbarisms. It is sometimes referred to as evolved gas dysbarism, compressed air sickness, or caisson worker's syndrome. DCS is completely different from the preceding direct effects. DCS is caused indirectly by the formation of inert gas bubbles (nitrogen or helium if used) at one or more locations within the body, corresponding to the symptoms. The gas is absorbed slowly at depth from the blood to the tissues. The amount of gas absorbed depends upon depth and time. During ascent, divers can easily decompress to lower pressures at rates much faster than the stored gases can be reabsorbed back into the blood and exhaled out of the body. If the pressure ratio is too large, bubbles form and symptoms of DCS occur, ranging from mildly irritating to severe and may result in death if not treated quickly.
- (c) Dysbaric Osteonecrosis (sometimes also called Aseptic Bone Necrosis) manifests itself as regions of bone and marrow necrosis. The condition is diagnosed through high-quality radiography and by excluding other causes.

Specifications

It can be concluded that underwater welding is a highly specialised task that requires careful control and supervision. It can only be attempted by the use of a highly skilled and trained workforce and depend strongly on teamwork and respect for safety.

A review of various approaches to the specification for underwater welding has shown that the most applicable document to be AWS D3.6M:1999, Specification for Underwater Welding. This specification contains sections on classification, qualification, workmanship, procedure and welder approval and inspection. In addition, which is most

helpful to engineers, is the classification system for weld quality which will assist in avoiding onerous specification of the weld quality.

E14.4 WORKMANSHIP – BOLTING

E14.4.1 General

Clause 14.4.1 of the Code is generally self-explanatory. Clause 14.4 of the Code covers bolting in the shop and on site. The most widely used bolt is currently ISO grade 8.8 with a design tension strength of 560 N/mm². Bolts with an ultimate tensile strength exceeding 1000 N/mm² should generally not be used for structural steelwork because of low ductility.

E14.4.2 Ordinary bolted assemblies

Much of clause 14.4.2 of the Code is self-explanatory.

It is a good practice to minimise the different sizes of bolts used on a site, say to M16 and M20 grade 8.8 or M20 and M24 grade 8.8. Different grades of the same diameter of bolt should not be used, if at all possible, to avoid the risk of a weaker bolt being used than required for a particular connection.

It should be noted that Chinese standards use the same bolt strength designations “8.8” and “10.9” as do other acceptable standards for the 5 regions but the actual design values are lower, as given in Tables 9.5, 9.6 and 9.8 of the Code.

The combinations of ordinary bolts and nuts, which may be used, are given in Table 14.4 of the Code. A footnote to the table says that any bolt assemblies, which seize when being tightened, shall be replaced; this is because seizure may lead to torsional shear plus tension failure. Other footnotes say that nuts for galvanized or sheradized 8.8 bolts shall be class/grade 10 and those for sheradized 10.9 bolts shall be class/grade 12. This is because when bolts and nuts are galvanized, it is usual that the manufacturer will tap the nut threads oversize in order to fit the galvanized bolt threads. Therefore, the nut is required to be stronger than for the case when it is not galvanized in order that in a matched assembly of nut and bolt, the nut must be sufficiently strong so that the bolt shank fails in tension prior to the nut or bolt threads stripping. Typically, the manufacturer should supply the higher grade of nut automatically, but this may not always happen.

A suitable plate, or heavy duty washer, shall be used under the head and nut when bolts are used to assemble components with oversize or slotted holes. Such washers may be required in situations where holes are enlarged on site to fit misplaced holding down bolts. It may then be necessary to weld the thick washer plate to the baseplate to permit transfer of shear force.

When the bolt head or nut is in contact with a surface which is inclined at more than 3° from a plane at right angles to the bolt axis, a taper washer shall be placed to achieve satisfactory bearing.

Tightening of assemblies with non-preloaded bolts

Clause 14.4.2.8 of the Code is self-explanatory.

Fitted bolts

Precision bolts may be used as fitted bolts when holes are drilled or reamed after assembly, so that the clearance in the hole is not more than 0.3 mm. Fitted, or close tolerance, bolts may be used instead of friction grip bolts in situations where slip of bolts in normal tolerance holes would lead to unacceptable joint rotations. An example of this would be a flange splice to carry bending moment. Close tolerance holes can be achieved by match drilling i.e. by first clamping the plates then drilling through all plies together.

E14.4.3 Pre-loaded bolt assemblies

Much of clause 14.4.3 of the Code is self-explanatory. The terms slip critical bolts, high strength friction grip bolts or pre-loaded bolts generally have the same meaning, that is the bolt is tightened to a high proportion of its tensile strength in order to clamp the plies of the joint together.

Tightening of pre-loaded bolt assemblies

The use of friction grip bolts shall comply with the specification as contained in Annex A1.3 of the Code.

Connected parts intended to transfer force in friction shall be firmly drawn together with all bolts partially tightened in a similar manner to assemblies with non-preloaded bolts. The hardened washer should be placed under the nut or head being turned. The joint shall then be examined and if there is any remaining gap which may affect the integrity of the joint, it shall be taken apart and a pack inserted before recommencing the tightening procedure. Tightening procedures shall be carried out progressively from the middle of each bolt group to the free edges. Additional cycles of tightening shall be carried out, if necessary, to achieve uniform preloading.

Friction grip bolts may be tightened using the torque control method, part-turn method, or direct tension to BS 7644 or other equivalent and the manufacturer's recommendations.

Torque spanners and other devices shall be re-calibrated in accordance with BS 4604 or other equivalent document. Torque spanners should be checked and, if necessary, re-calibrated at least once per shift.

Direct tension indicators such as "Coronet" washers have raised pips, which deform plastically to a preset gap when the correct bolt tension, is achieved. The gap is measured with "go / no go" feeler gauges.

If, after complete tightening, a bolt or nut has to be slackened off, the whole bolt assembly should be scrapped since the bolt may have yielded and there is a risk of failure on retightening.

E14.5 WORKMANSHIP – ERECTION

E14.5.1 Erection method statement

Clause 14.5.1 of the Code requires an erection method statement to be prepared and checked in accordance with the design rules, in particular against resistance of the partly erected structure to erection and other temporary loading. The statement should describe procedures to be used to safely erect the steelwork.

Although the contractor is responsible for preparing the method statement and for carrying out the erection safely, it is good practice for the Responsible Engineer to conceive and describe at least one possible method of construction for his design. This is particularly important for large or unusual structures, for example where there are long spans, or the need to erect heavy outrigger steelwork at height for a tall building.

E14.5.4 Column base plates and slabs

Clause 14.5.4 of the Code is generally self-explanatory. Base packs may be left permanently in place, but they should not prevent proper grouting and should have sufficient cover such that they will not lead to unsightly rust stains.

E14.5.5 Grouting

Clause 14.5.5 of the Code is generally self-explanatory. The steelwork erector will usually erect sub assemblies of the structure "loose" and adjust alignments, levels, verticality, etc with guys before tightening bracing and fixing the column bases by grouting.

E14.5.6 Stability

Clause 14.5.6 of the Code is self-explanatory but repeated here because of its importance. The intention is to avoid collapse of partly assembled structure by ensuring lateral stability and structural capacity is always provided against possible loading during assembly.

E14.5.7 Alignment of part of the structure

Clause 14.5.7 of the Code is self-explanatory.

E14.6 PROTECTIVE TREATMENT

E14.6.1 General

Clause 14.6.1 of the Code is prescriptive and self-explanatory.

E14.6.2 Materials

Clause 14.6.2 of the Code is generally self-explanatory. The Code requires that all reinstatement of damaged coatings shall be made good to the standard of the original work using the same methods and materials. This can be difficult to achieve in practice since site conditions are usually not as ideal as factory conditions for paint application. The Responsible Engineer and site supervisory staff should try to encourage the contractor to achieve a reasonably durable repair.

E14.6.5 Hot-dip galvanizing

Clause 14.6.5 of the Code is generally self-explanatory. Vent holes are required in hollow members which will be galvanised to prevent air or moisture in the closed section expanding from the heat of galvanising and bursting the member. See also clause E5.5.2 of the EM and clause 5.5.2.1 of the Code, which describes hot dip galvanizing and gives warnings on its use for high strength plates and bolts.

Recent experience suggests that cracking of normal steel during galvanizing may occur and it is recommended that the Responsible Engineer read the recent publication BCSA/GA 40/05 written jointly by the British Steelwork Construction Association and the Galvanising Association. Generally, the galvanizing of components whose failure would be critical is not recommended.

E15 ACCURACY OF FABRICATION AND ERECTION

E15.1 GENERAL

Section 15 of the Code gives guidance on the permitted deviations in dimensions of the steelwork during fabrication and after erection and is generally self-explanatory. Clause 15.1 of the Code also gives self-explanatory guidance on combining tolerances. The intention is to accept that individual components can have reasonable tolerances of manufacture and fabrication but that these cannot be simply added together such that the geometrical setting out of the overall structure exceeds reasonable overall tolerance limits.

Clauses 15.3 to 15.8 of the Code contain diagrams showing the permitted deviations for various types of fabricated components. Clause 15.9 of the Code gives permitted deviations for cold formed sections, Clause 15.10 of the Code for foundations, walls and holding down bolts and clauses 15.11 and 15.12 of the Code give permitted deviations for the completed structure after erection.

As stated in the Code, accumulation in permitted deviations in the pieces supplied and in fabrication shall not cause the structure to be erected outside the permitted deviations for erection.

The following table summarises previous local practice used by major public sector bodies in Hong Kong:

User	Reference	Fabrication	Erection
Highway Department	Structural Design Manual: 1997	BS 5400 Part 6	Not specified. In general refer to BS 5400
Kowloon Canton Railway Corporation	Standard Specification: 2000	BS 5950 Part 2 for building or BS 5400 Part 6 for bridge	8 items incorporated
HKSAR Government	General Specification for Civil Engineering Works: 1992	BS 5950 Part 2 or BS 5400 Part 6 as appropriate	8 items incorporated
Architectural Services Department	General Specification	BS 5950	Not specified. In general refer to BS 5950
Housing Authority	Technical Specification: 2000	BS 5950 Part 2 or BS 5400 Part 6 as appropriate	8 items incorporated

E15.2 PERMITTED DEVIATIONS IN THE CROSS SECTION OF ROLLED COMPONENTS

Clause 15.2 of the Code states that permitted deviations in the cross section of rolled components are given in the product standards in Annex A1.8 of the Code.

PERMITTED DEVIATIONS IN VARIOUS COMPONENTS AFTER FABRICATION

Clauses 15.3 to 15.8 of the Code give diagrams of permitted deviations of various elements and components after fabrication and are self-explanatory.

E15.9 PERMITTED DEVIATIONS OF COLD FORMED SECTIONS

Permitted deviations of cold formed sections are given in section 15.9 of the Code and are generally self-explanatory. Clause 15.9.1 of the Code requires measurements other than thickness to be taken at least 200 mm from ends of members because distortions can occur during cutting and cropping processes.

Clause 15.9.2 of the Code notes that thickness tolerances are given in the relevant product standard for thin sheet steel. Clause 15.9.3 of the Code provides tables giving permitted deviations for internal and external widths of formed sections.

Clauses 15.9.4, 15.9.5, and 15.9.6 of the Code give tolerances for length, angle between adjacent plates and straightness respectively. In the case of complex cross sections, clause 15.9.6 of the Code allows that permitted deviations may be agreed between the designer and the manufacturer.

Clauses 15.9.7, 15.9.8, and 15.9.9 of the Code give tolerances on angle of twist, compound members and flatness and are self-explanatory.

E15.10 PERMITTED DEVIATIONS FOR FOUNDATIONS, WALLS AND HOLDING DOWN BOLTS

The diagrams of permitted deviations given in clause 15.10 of the Code are generally self-explanatory. It should be noted that problems of setting out and tolerance often occur at the interface between steel and concrete components, i.e. at wall plates where steel beams are connected to reinforced concrete walls and particularly at holding down bolts.

Often, the physical interface coincides with an interface between two contracts, for example the main contractor and the steelwork erection subcontractor. The designer should recognise this and, although the tolerances given in the Code are reasonable, may consider providing a design for the interface connection which allows for modification on site. For example, the use of extra large holes in the column baseplates and the use of large dimension thick washer plates should be specified.

Other practical measures are to use a template to install the holding down bolts or survey the as-built bolt positions before drilling the baseplate to suit.

If holding down bolts are cast loose in tubes or cones, it is essential that their freedom to move should be checked before the concrete hardens.

E15.11 APPLICATION OF PERMITTED DEVIATION FOR ERECTED COMPONENTS

Clause 15.11 of the Code gives guidance on measurement and assessment of tolerances of erected structures and is self-explanatory.

E15.12 of the Code gives Permitted Deviations of Erected Components and Structures and clause 15.12 of the Code gives permitted deviations of erected structures and components of structure and are generally self-explanatory.

The clause 15.12.4 of the Code on multi-storey column plumb allows columns in an individual storey to deviate by the greater of 5mm or 1/600 of storey height, but restricts the overall deviation of a column to 50mm from its theoretical setting out. This overall tolerance is clearly of particular importance for tall buildings.

E15.12 PERMITTED DEVIATIONS OF ERECTED COMPONENTS AND STRUCTURES

Clause 15.12 of the Code is self-explanatory. Clauses 15.10, 15.11 and 15.12 of the Code are effectively a construction tolerance specification and, as such, the Responsible Engineer may wish to revise them slightly for a particular project. However, this should be done with care and understanding since increase of tolerance may affect design assumptions, for example allowing an increase of out of verticality will increase gravitational overturning effects.

E16 LOADING TESTS

E16.1 GENERAL

E16.1.1 Scope

Section 16 of the Code gives recommendations for carrying out tests on structures or components of structures. Clauses 16.1 to 16.3 of the Code cover general testing and clause 16.4 of the Code covers testing of composite slabs. The testing procedures described in section 16 of the Code do not cover scale model testing or fatigue testing.

Testing may be considered to form a part of a performance based justification of a design by establishing the capacity of an existing structure or component or to verify design or construction that is not entirely in accordance with the rest of the Code.

Testing must not be used to reduce levels of safety below those generally assumed in the Code.

The load tests described in section 16 of the Code are to verify or establish the design strength of a structure or component. They are proof, strength and failure tests. Tests may also be undertaken to establish design data to be used in a calculation model.

Strength tests may be carried out on one or more prototypes, which are intended to develop a number of similar structures.

Proof tests must be applied to every structure or component of the class being proposed in order to prove the acceptability of each.

E16.1.2 Requirement for testing

Testing is required when a structure or part of a structure has not been justified by calculations, where there is argument or doubt about whether the structure or part of it complies with the Code or when materials or design justification is not covered by the Code or other acceptable design method.

E16.1.3 Recommendations for conduct of tests

Clause 16.1.3 of the Code gives recommendations for carrying out tests and is generally self explanatory.

Particular care should be taken when testing an existing structure, see also clause 17.4 of the Code. It is essential to establish a clear purpose for the test and how to interpret the results, which may not provide a precise "yes or no" answer. A careful assessment of structural conditions before execution is a fundamental requirement. A support framework in proximity to the structure should be considered to avoid less than expected performance leading to failure.

E16.2 PROOF, STRENGTH AND FAILURE TESTS

E16.2.1 Proof and strength tests

General

Proof and strength tests of a structure or component are tested to particular levels of load. A proof test is to confirm that the structure performs adequately; a strength test confirms that it can sustain a particular design load and can be used to accept similar items. A structure to be strength tested should first undergo a proof test and it is recommended that a failure test should follow the strength test.

A proof test is a non-destructive test, although there may be permanent local distortions and the effect of these on future use of the structure should be considered before testing. Any departure from linear behaviour during the proof test should be noted and reasons for such behaviour should be found. A strength test is likely to lead to significant residual deflection.

Serviceability behaviour can be obtained from proof and strength tests.

Test loads and test criteria

Clauses 16.2.1.2 and 16.1.3 of the Code describe test loads and test criteria for proof and strength tests and are self-explanatory.

E16.2.2 Failure test

Failure criterion

Failure of a test specimen should be considered to have occurred if there is collapse or fracture, if a crack begins to spread in a vital part of the specimen or if the displacement becomes grossly excessive.

Examples of types of test

A proof test might be carried out where the adequacy of design of a floor bay was in doubt after construction. The complete bay would be loaded, say with water bags, to a proof load value as defined in clause 16.2.1.2 of the Code.

A strength test might be carried out to justify the design of a new type of component, say a space frame connection node.

A failure test would be to continue the strength test up to failure.

E16.3 TEST CONDITIONS, METHODS AND PROCEDURES

E16.3.1 Test conditions

Clause 16.3.1 of the Code describes the required test conditions, i.e. how the test rig and component to be tested should be set up.

The clause also notes the importance of considerations for safety in the layout and design of the test and that any expected or unexpected failure of the test specimen must not result in hazard.

E16.3.2 Loading and unloading

Clause 16.3.2 of the Code gives requirements on application of the test loads and is self-explanatory. It allows a bedding down load to be applied and removed prior to the test. There may be situations where the Responsible Engineer has reasons not to apply a bedding down load.

E16.3.5 Relative strength coefficient

Clause 16.3.5 of the Code is self-explanatory.

E16.3.6 Quality control of load testing

Clause 16.3.6 of the Code is self-explanatory.

E16.3.7 Contents of test report

Clause 16.3.7 of the Code gives guidance on the format and content of the test report and is self-explanatory.

E16.4 TESTING OF COMPOSITE SLABS

Clause 16.4 of the Code gives guidance on the testing of composite slabs and is self-explanatory.

E17 GUIDANCE FOR EVALUATION AND MODIFICATION OF EXISTING STRUCTURES

E17.1 GENERAL APPROACH

Section 17 of the Code provides guidance for repair, alteration and addition (A and A) works of steel structures and of other structures where steel is proposed for the new part.

It is intended to act as an aide-memoire to designers rather than a textbook on refurbishment and A and A works.

E17.2 STRUCTURAL ASSESSMENT SURVEY

Clause 17.2 of the Code is self explanatory.

E17.2.1 Original materials

Clause 17.2.1 of the Code is self explanatory.

E17.2.2 Appraisal report

Structural assessment survey

The assessing engineer should gather and examine all possible evidence about the structure from various sources as described below.

Existing drawings and documents

The best source of good information is record drawings and specifications of the original design. For buildings and structures in Hong Kong, the following organisations may have records:

- (a) Buildings Department for all private buildings, except exempted houses in the New Territories.
- (b) Housing Authority for public housing.
- (c) Architectural Services Department for government buildings.
- (d) Antiquities and Monuments Office.
- (e) Public utilities such as MTR, CLP, WSD, CEDD.
- (f) The building owner or management committee may have records for newer buildings.

Historical studies and verbal information

Descriptions of older buildings of historical interest may be found in guidebooks, newspaper archives or historical studies. Sometimes, these may contain useful photographs. In Hong Kong, the archives of the Antiquities and Monuments Office or Universities may be consulted.

Useful information may be obtained from discussion with local people, for example, village heads or archaeologists.

Structural survey

Having obtained information from initial desk studies, a site visit and structural survey should be carried out. Detailed guidance on how to do this is given in the references.

- (a) Permission to carry out the survey must be sought.
- (b) Suitable safety precautions must be taken.
- (c) Notes, sketches and photographs should be made. It is helpful to walk around the structure in a logical way, for example, move clockwise outside the building, then inside floor by floor. Use of a portable dictation machine can speed up note taking. (Take adequate size of memory card and spare batteries for the digital camera. Binoculars are also useful.)

- (d) Examine the building from outside to note any signs of differential settlement and defects. It can be helpful to sight along a line which should be level, for example, a line of window sill. Use of a plumb bob and standard builder's level can also provide useful information ahead of any more detailed level survey that might be required.
- (e) Areas for detailed examination and possible opening up of walls and floors should be identified. Locations of trial pits for examination of foundations should be planned.
- (f) Possible locations for taking material samples for tests should be identified.
- (g) It is very likely that further visits will be necessary to confirm dimensions and missing items, which become obvious during the initial assessment report drafting stage.

Detailed site investigations

Detailed site investigations, opening up of the superstructure and trial pits to examine foundations should be specified as necessary following the initial inspection.

Original materials

The materials used in the existing structure should be identified, initially from information on original drawings and inspection. If material properties cannot be established to a reasonable level of accuracy, then it may be necessary to remove samples for destructive testing. The locations for taking samples must be carefully chosen to minimise damage, to avoid significant weakening and to provide sufficient and reliable data, particularly on chemical composition and weldability of iron and steel. A better range of properties may be obtained from thicker sections wherever practical. Suitable methods of making good shall be specified.

If corrosion of steel sections has occurred, then an approximately quantified estimate of the amount, i.e. percentage loss of section, should be made. A suitable method of treatment and a means of verification of its correct application should be devised.

The references in this EM and in Annex A.2 of the Code give information on identifying old types of steel and wrought and cast iron.

Safety during the survey

Safety on site is vital. There are particular safety issues during structural surveys and these are given in this EM.

The following safety points should be noted when carrying out surveys and inspections. They are not exhaustive:

- (a) Wear comfortable old clothes covering arms and legs and giving some protection from dirt, insect bites, rodent bites.
- (b) Be aware of wasp or bees nests in uninhabited buildings.
- (c) Wear a safety helmet.
- (d) Wear protective footwear with good grip.
- (e) Take hand cleaning wipes and tissues.
- (f) Take a good torch and spare batteries.
- (g) Do not inspect alone, in case of accidents. If you must inspect alone, leave details of location, expected time of return and take a working mobile telephone with you.
- (h) Ensure ladders are securely held by another person and tied if possible. Do not use scaffolding or other access equipment unless it has been inspected by a competent person.
- (i) Use safety harness with adequate attachments/anchorage when inspecting at height and note the risks of fatal accident if you fall and dangle from a harness.

- (k) Do not descend unventilated manholes or basements. Be aware of the risk from poisonous or inflammable gases and from carbon dioxide or lack of oxygen.
- (l) Take care when inspecting old floors, look out for rot and damage and holes/openings that may be inadequately protected.
- (m) Beware when working near an unfenced hole or edge.

Appraisal report

The appraisal report shall describe the findings of the document studies, site surveys, material tests and any analysis and design check calculations carried out. A suggested contents list is as follows:-

- 0. Executive Summary
- 1. Introduction, (Brief, Terms of reference, Location of building, Requirements for refurbishment and new structure.)
- 2. Documents examined and other historical information obtained.
- 3. Description of surveys and inspections carried out.
- 4. Description of building and foundations. (clarify structural systems, type of foundations, state of materials, outline of calculation checks carried out, defects)
- 5. Conclusions and recommendations
 - Appendix A. containing photographs and figures
 - Appendix B. containing calculations
 - Appendix C. containing results from material tests, logs of trial pits etc.

E17.3 DESIGN AND ANALYSIS ISSUES

E17.3.1 Structural appraisal analysis and design check

Hand, and if necessary simple computer, analyses of this system should be carried out. It is seldom necessary to set up complex computer analyses, it is much more important to establish a realistic structural system.

The analysis may be carried out in several stages and with increasing rigour as required.

It is often helpful to carry out a comparative study of the existing structure and the structure including the planned additions. Results from the principal load combinations of Dead + Live, Dead +/- Wind and Dead + Live + Wind should be compared and the differences in overall and critical element forces noted.

As a rule of thumb, and subject to the existing structure being in good condition, where such differences normally exceed marginally, say within 5% or less, it is likely that the proposed change of use can be justified without additional strengthening of the existing structure.

As a starting point, the load factors and combinations used for the appraisal should be taken from Table 4.2 of the Code.

In certain situations, there may be evidence to justify lower load factors.

As discussed in clause 4.1 of the Code, the concept of partial load and material factors allows for uncertainties in variation of loads, in probability of loads acting together, in inaccuracies of calculation and structural behaviour, and in variation of material strengths. If some of these uncertainties can be reduced or removed, then it is reasonable to reduce the appropriate partial safety factor.

Two points should be noted. One, if a building is standing and in reasonable condition, then its overall safety factor must be more [at the very least] than 1.0. Two, reducing safety factors in any assessment calculations does not reduce the actual factor of safety of an existing building.

In no case should the partial load factor be less than 1.2 under normal load combinations.

Load factors and combinations used for the appraisal should be taken from clause 4.3.3 of the Code. In certain situations, there may be evidence to justify lower load factors.

Load factors and combinations for design of any new additions shall be taken from clause 4.3.1 of the Code.

The possibility of fatigue loading on original elements should be considered.

E17.3.2 Overall stability of existing and new structure

Clause 17.3.2 of the Code is self-explanatory.

E17.3.3 Details for connection of new to old structure

Suitable connection details shall be designed. Some examples of particular details for connection of new steelwork to existing structures are shown in Figures E17.1 to E17.4. They are:-

Figure E17.1 – Baseplate detail for connection of new steelwork to existing concrete structure.

Figure E17.2 – Detail for connecting new steel beams to existing concrete or masonry walls.

Figure E17.3 – Detail for connecting new composite slab to existing concrete or masonry walls and allowing differential vertical movement.

Figure E17.4 – Detail for new steel beam to strengthen existing slab, showing springing procedure to load new beam.

Some principles for such connections are:-

- To provide tolerance, for example in fitting a new steel beam between existing concrete beams.
- To allow for relative movements.
- To avoid damage, e.g. the cutting of existing main reinforcement.
- To ensure that load will be transferred to any new structure, for example by use of flat jacks, sand jacks and well planned construction sequences.

E17.3.4 Upgrading of original structure

Clause 17.3.4 of the Code is self-explanatory.

E17.3.5 Considerations for design against extreme events (fire, accident, terrorism)

It may be difficult or even economically unfeasible to apply some current standards on robustness or fire protection to existing buildings designed in the past. However, a suitable level of safety must be provided to the building users. Therefore, each structure should be assessed on its merits and a specific risk analysis and performance based design may be required in order to justify an acceptable level of safety against fire and extreme events.

As a guideline, if new works (for example, installation of a new steel interconnecting stair) do not alter the principal vertical and lateral load carrying elements of structural system, which has stood without obvious signs of distress for many years, then a full analysis and design of the building and assessment against modern code robustness requirements may not be necessary.

If, on the other hand, the new works are major and will affect the principal structural systems, or a structural appraisal reveals particular vulnerabilities, then an assessment as described above should be made.

E17.3.6 Serviceability issues

Clause 17.3.6 of the Code is self-explanatory.

E17.4 LOAD TESTS

Clause 17.4 of the Code is self-explanatory.

There is a risk of damage occurring during load tests of existing buildings, and while such tests will be expensive, the relative merits of strengthening should be considered.

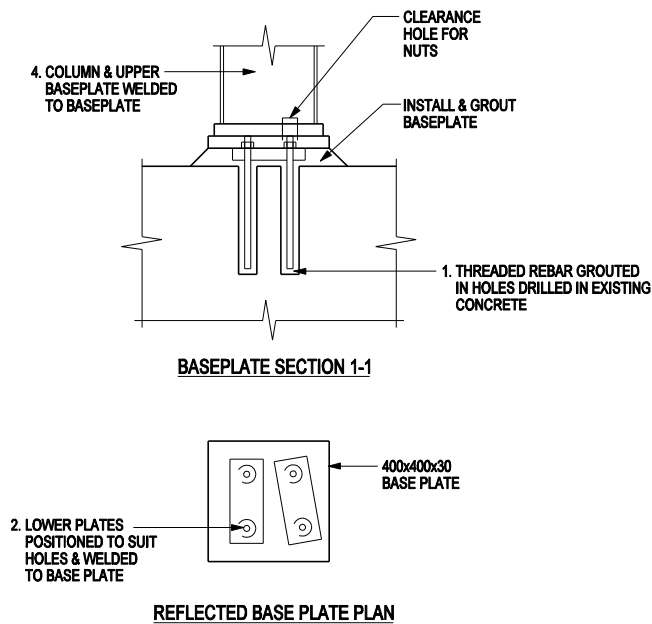


Figure E17.1 - Baseplate detail for connecting new steelwork to existing concrete structure

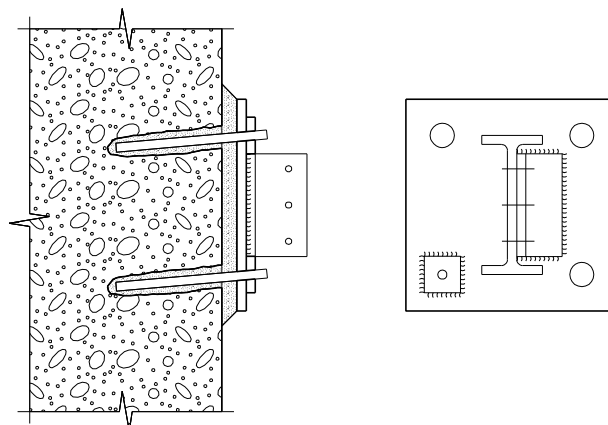


Figure E17.2 - Detail for connecting new steel beam to existing concrete or masonry wall

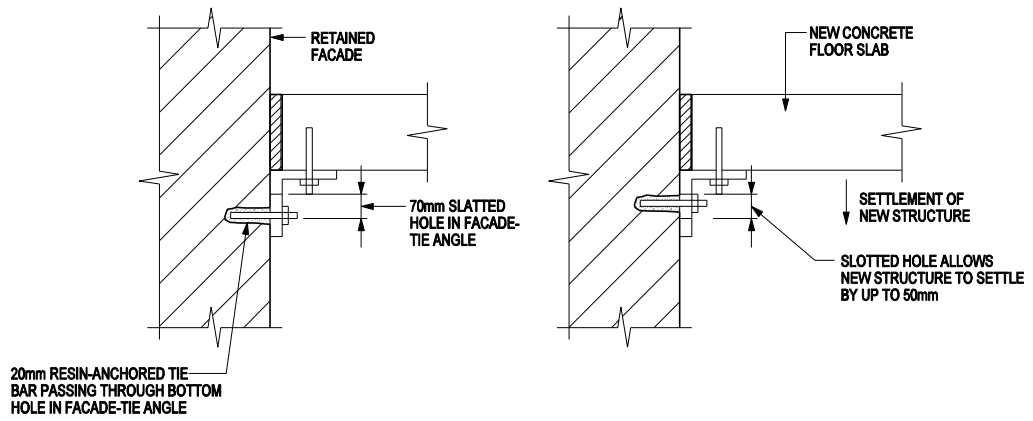


Figure E17.3 - Detail for abutting a new composite slab to existing concrete or masonry walls and allowing differential vertical movement

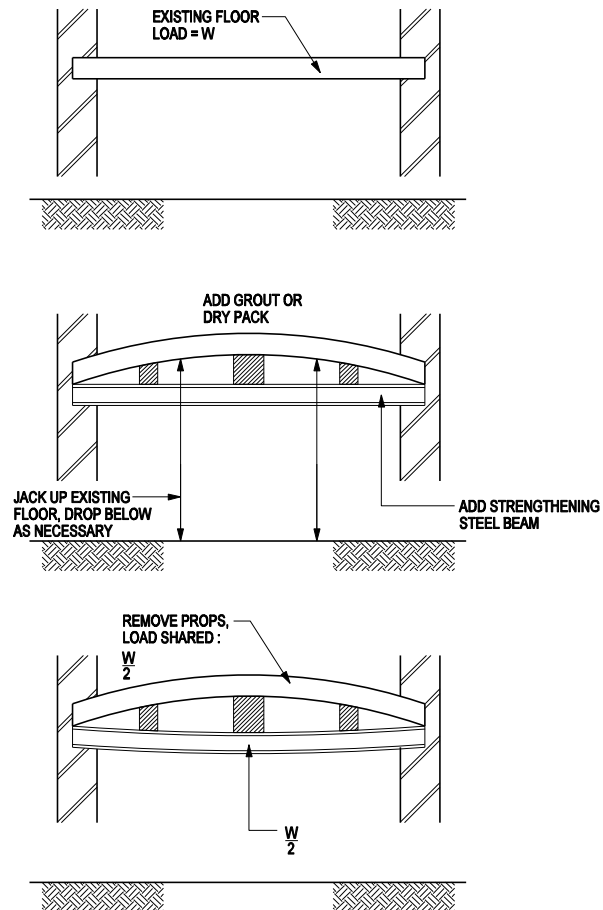


Figure E17.4 - Detail for new steel beam to strengthen existing slab, showing springing procedure to load new beam

References:-

Appraisal of existing structures, a guide published by the Institution of Structural Engineers, U.K.

Structural aspects of building conservation, a book written by Poul Beckmann and published by McGraw-Hill.

Appraisal of existing iron and steel structures, written by Michael Bussell and published by the Steel Construction Institute, U.K. (1997).

The refurbishment of the Royal Hong Kong Jockey Club, by D. G. Vesey, K. O. Yeung and P. W. Suett, Journal of the Hong Kong Institution of Engineers, April 1990.

ANNEX A REFERENCES

E.A1 ACCEPTABLE STANDARDS AND REFERENCES

Lists of acceptable standards and references for use in conjunction with the Code are given in Annex A1 of the Code. The term “acceptable” is defined in the Code as acceptable to the Building Authority and has the same meaning as “Normative” in other codes such as the Eurocodes or Australian codes.

Designers may use other standards or technical references if it can be demonstrated that they can achieve a performance equivalent to the acceptable standards given in the Code.

The acceptable references are dated in the Code and more recent versions are not acceptable until authorized by the Buildings Department and placed on their acceptable reference list. The current version of this list can be accessed from the Buildings Department’s website at “www.bd.gov.hk”.

Other informative references provide more detailed guidance on particular aspects of design. Annex D of the Code contains abstracted essentials of some standards where appropriate. The abstracted essentials are for guidance and ease of use of the Code; however, compliance with the acceptable standards and references is mandatory and takes precedence over guidance given in the abstracted essentials.

The Code accepts materials, that is hot rolled steel plates and sections, cold formed steel plates and sections, forgings, castings, bolts, shear studs, welding consumables to acceptable international steel product standards from the five regions. These are standards from Australia, China, Japan, North America and United Kingdom versions of European Union standards.

In order to provide a single consistent set of standards for workmanship, any testing of materials required in Hong Kong, testing and certification of workers and Quality Assurance procedures, these aspects shall generally be as defined in the Code or as acceptable to the Building Authority.

The reference standards are updated with current versions whereas the obsolete versions are deliberately retained for ease of reference. Another reason for such retention was to enable HOKLAS laboratories to have ample time to apply for accreditation with conformance to current version of standards.

E.A1.1 Steel materials

The list of standards for steel materials is self explanatory. The list refers to steel materials standards from Australia, China, Japan, North America and UK versions of European standards.

Various standards are produced jointly by Australia and New Zealand. Lists of these can be found on the relevant web sites “www.standards.com.au” or “www.standards.co.nz”.

E.A1.2 Castings and forgings

The list of standards for steel castings and forgings is self explanatory. The list refers to standards from Australia, China, Japan, North America and UK versions of European standards.

E.A1.3 Bolts

The list of standards for bolts, nuts and washers is self explanatory. The list refers to standards from Australia, China, Japan, North America and UK versions of European standards.

E.A1.4 Welding

References on workmanship and testing of welds and on certification for welders and weld testing personnel are based on UK versions of European standards and on North American ASTM standards only in order to avoid ambiguities. These references are

given in Annex A1.4 of the Code. The abstracted essentials for typically used welding symbols are given in Annex C of the Code. In the reference to AWS D1.1 2004, it is clarified that further references to other AWS are also acceptable.

E.A1.5 Materials for composite design

Materials for steel-concrete composite design i.e. concrete and steel reinforcement are taken from the acceptable standards in the Hong Kong Code of Practice for the Structural Use of Concrete.

E.A1.6 Shear studs

The list of standards for shear studs is self-explanatory. The list refers to standards from Australia, China, Japan, North America and UK versions of European standards.

E.A1.7 Cold-formed steel materials

The list of standards for cold formed steel materials is self-explanatory. The list refers to steel materials standards from Australia, China, Japan, North America and UK versions of European standards.

E.A1.8 Dimensions and Tolerances of sections

The list of standards for dimensions and tolerances of sections is self-explanatory. The list refers to standards from Australia, China, Japan, North America and UK versions of European standards.

E.A1.9 Protective Treatment

The list of standards for protective treatment for steel is self-explanatory and refers to UK and European standards only.

E.A1.10 Other acceptable references

Annex A.10 of the Code contains acceptable references for various specialist topics including the design of cranes and runway beams and a comprehensive and up to date fatigue design code: BS 7608. It also lists other Hong Kong codes relevant to building structures. These are: the Code of Practice on Wind Effects, the Code of Practice for the Structural Use of Concrete 2004, the Code of Practice for the Precast Concrete Construction: 2003 and the Code of Practice for Fire Safety in Buildings 2011.

E.A2 INFORMATIVE REFERENCES

The Practice Notes which are written and issued by the Buildings Department for Authorized Persons, Registered Structural Engineers and Registered Geotechnical Engineers provide a wealth of practical information and guidance on design and construction issues in steel and other materials. They can be accessed from the Buildings Department's website at "www.bd.gov.hk".

Various other design guides are referenced here, for example, the UK British Steel Construction Association guides on Simple and Moment connection design, website address: "<http://www.steelconstruction.org/>", the Steel Construction Institute guides, website address: "<http://www.steel-sci.org/>" and guides on appraising existing structures.

ANNEX B RELATIVE STRENGTH COEFFICIENT

Annex B of the Code describes how to calculate the relative strength coefficient referred to in clause 16.3.5 of the Code and is self-explanatory.

The derivation of design capacity for a failure test may be found in the Informative Reference of Annex A2.3, BS5950: Part 1: 2000, Clause 7.7.3.

ANNEX C TYPICAL WELDING SYMBOLS

Annex C of the Code contains typical welding symbols used. These are in general use by international steel designing and fabricating organizations and are self-explanatory.

ANNEX D NOTES ON TESTING TO ESTABLISH STEEL GRADE AND ESSENTIALS OF SOME PRODUCT STANDARDS

E.D1 TESTING TO ESTABLISH STEEL GRADE

The situations in which testing may be required could be very different. The exact details of the testing required should reflect the quantity of steel in a project, the origin and the number of different section types.

Class 2 steel:

The Code allows the use of Class 2 steel, that is steel not being supplied in accordance with one of the recognised standards from the five regions: Australia, China, European Union (British versions i.e. BS EN) Japan, North America, to be tested for compliance with one of these acceptable product standards.

The tests shall include tensile strength, notch ductility and chemical composition. The Code requires a minimum of one test in each category to be made for every 20 tonnes of steel or part thereof of the same product form, the same range of thickness or diameter, and the same cast. The results of each test and the characteristic value obtained by statistical analysis shall not be less than the value required by the standard.

Table D1a of the Code lists the essential performance requirements for hot rolled structural steel sections, flats, plates and hot rolled and cold formed structural hollow sections and similarly Table D1b of the Code lists the essential performance requirements for structural sections cold formed from sheet steel. These tables are reproduced here for easy reference.

Even after successfully passing these tests, Table 4.1 of the Code requires increased material factors to be used for Class 2 steel; thus it is preferable for a supplier to comply with the requirements for Class 1 steel, i.e. steel manufactured to one of the five regions' acceptable standards.

Class 3 steel:

For class 3, uncertified steel, the Code provides that tests for tensile strength and ductility are required to demonstrate that it has a yield strength of at least 170 N/mm^2 , an elongation of at least 15% and an ultimate tensile strength of at least 300 N/mm^2 . The Code requires a minimum of one test in each category for every 20 tonnes of steel or part thereof of the same product form and the same thickness or diameter. Alternatively, it allows the test frequency to be as directed by the Responsible Engineer. Given that tests are generally required for all use of Class 3 steel, its use is unlikely to be economical unless large batches of the same category are available.

If the steel is to be welded, the Responsible Engineer may additionally require tests for weldability as described in clause D1.1 of the Code.

E.D1.1 Abstracted essential requirements for structural steel

The essentials of the basic requirements for strength, resistance to brittle fracture, ductility and weldability of hot rolled structural steel are listed in clause 3.1.2 of the Code and repeated in clause D1.1 of the Code. Where it is required to weld class 3 steel, then it shall also comply with these requirements.

For commonly used grade 43C steel, the maximum contents for sulphur and phosphorous should not exceed 0.05% as stipulated in BS 4360: 1986. For equivalent grade S275J0 steel, the maximum contents for sulphur and phosphorous are reduced to 0.04% as stipulated in BS EN 10025: 1993. These maximum contents are further reduced to 0.03% as stipulated in BS EN 10025: 2004. Hence, the maximum contents for sulphur and phosphorous are set at 0.03% in clause 3.1.2 of the Code. While there is no intention to make the Code more stringent than the current reference standards, Class 1 steel products conforming to the materials reference standards from the five regions in Annex A1.1 are deemed to satisfy the chemical composition requirements. For Class 2 and Class 3 steel products, the chemical composition requirements as stipulated under 'Weldability' in clause 3.1.2 of the Code should be strictly observed.

For Class 1H steel products, the maximum contents for sulphur and phosphorous do not exceed 0.015% and 0.025% as stipulated in BS EN 10025-6: 2004. Hence, the maximum contents for sulphur and phosphorous are set at 0.025% in clause 3.1.3 of the Code. While there is no intention to make the Code more stringent than the current reference standards, Class 1H steel products conforming to the materials reference standards from the five regions in Annex A1.1 are deemed to satisfy the chemical composition requirements. Otherwise, the chemical composition requirements as stipulated in clause 3.1.3 of the Code should be strictly observed.

Table E.D1a - Performance requirements for hot-rolled structural steel and cold formed structural hollow sections

Performance requirement	Specified by	Additional requirements for steel in structures designed by the plastic theory
Minimum yield strength	Upper yield strength (R_{eH})	$R_m/R_{eH} \geq 1.2$
Minimum tensile strength	Tensile strength (R_m)	
Notch toughness	Minimum average Charpy V-notch impact test energy at specified temperature	None
Ductility	Elongation in a specified gauge length And bend test	Stress-strain diagram to have a plateau at yield stress extending for at least six times the yield strain. The elongation on a gauge length of $5.65 \sqrt{S_0}$ should not be less than 15% where S_0 is the cross sectional area of the section
Weldability	Maximum carbon equivalent value, Carbon content, Sulphur and Phosphorus contents	None
Quality on external/internal surface	See the relevant standard from the list in A1.8	None
Through thickness property (only for certain situations, see 3.1.5 and 14.3.3.4)	Elongation to failure in the through thickness direction	None

Table E.D1b - Performance requirements for cold-formed steel

Performance requirement	Specified by	Specific requirements
Minimum yield strength	Upper yield strength (R_{eH})	$R_m / R_{eH} \geq 1.08$ (min) ~ 1.2 (max)
Minimum tensile strength	Tensile strength (R_m)	
Notch toughness	None	None
Ductility	Elongation in a specified gauge length	The total elongation should not be less than: 10% for a 50mm gauge length, or 7% for a 200mm gauge length.
Weldability	None	None
Quality on external/internal surface	See the relevant standards in A1.8	None
Through thickness property	None	None

E.D1.2 Additional requirements for high strength steel

Steel with yield strengths above 460N/mm² is generally proprietary products and not produced to particular standards. More reliance must therefore be made on the manufacturers' QA system when specifying such steel. Otherwise, clause D.1.2 of the Code is self-explanatory.

E.D1.3 Design strength for high strength steel

Clause D.1.3 of the Code is self-explanatory.

E.D1.4 Quality control of testing

Clause D.1.4 of the Code is generally self-explanatory. In Table D2, the Code provides yield and ultimate strength values for a range of commercially available high strength steel, which is typically proprietary products manufactured by individual steel manufacturers. It is, therefore, difficult to produce specific design requirements for these types of steel. Annex D of the Code highlights issues that a competent designer should consider when using high strength Class 1H steel: e.g. design strength, buckling characteristics, weldability requirements, welding procedure and consumable requirements etc.

E.D2 ABSTRACT OF ESSENTIAL REQUIREMENTS FOR BOLTS

Clause D2 of the Code is self-explanatory. In addition to giving abstracted essential requirements for bolts, see also clauses E14.4.2 and E14.4.3, while clauses D2 of the Code provides a Table D4 with a range of normally used bolt strengths.