COMPOSITE STEEL AND CONCRETE DESIGN

COMPARISON OF INTERNATIONAL CODES

HALF THESIS
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CONTENTS

INTRODUCTION

The theme of this thesis is the comparison of design methodology for composite steel concrete beam members as permitted by four selected international design Codes. The investigation was necessitated by the considerable difference in complexity of and results obtained in the application of methods specified by the latest international design Codes for determining the design moment and moment capacity of composite beams.

The selected Codes are:

SABS 0162 - 1984 - SOUTH AFRICA

BS 5950 : Part 3.1 - 1990 - BRITAIN

EUROCODE No.4 - (DRAFT 1985) - COMMISSION OF THE EUROPEAN COMMUNITIES

CAN/CSA - S16.1 - M (DRAFT 1988) - CANADA

These Codes have been selected for comparison because:

1) They represent the latest research and design methods and/or are relevant to the South African design Code situation,
2) They are based on the limit state philosophy and
3) They are essentially for the design of building structures.

The comparison is confined to GLOBAL and CROSS-SECTION ANALYSIS at the ultimate limit state, and two topics which have a profound influence on these analyses, CROSS-SECTION CLASSIFICATION and EFFECTIVE WIDTH OF CONCRETE SLAB.

The comparison of the allowed methods of global analysis is presented in a descriptive manner.

The limits and recommendations for cross-section classification, effective width of concrete slab and cross-section analysis are compared in a series of parametric studies.

In order to gauge the correctness of the specified limits and methodology in the treatment of these topics, the theories are tested against the data from specimens used in independent laboratory investigations on various aspects of composite beams in flexure. The ultimate moment capacity of sections predicted by the four Codes are compared to the moments at failure obtained from the tests.

The application of the Codes to the design of composite beams in flexure yield considerably different results in hogging moment regions but, compare well in sagging moment regions. Significant differences in the limits of cross-section classification are shown to exist, and the influence of the concrete slab effective width on cross-section classification, and hence global and section analysis, is highlighted.

2. HISTORIC DEVELOPMENT OF PRESENT CODES

Introduction

Steel beams encased in concrete were used in construction as early as the 1920's with research on the topic dating back to 1922 in Canada followed by U.S.A., England, Switzerland, France, Belgium and Germany. (19)

The benefits of composite over bare steel beams in stabilising the top flange were taken advantage of in bridge design as early as 1935, when beam and slab composite bridges with spans up to 26 m were being designed and built in Australia. (17)

Rules for composite beam design in buildings were published in Canada in 1941 and the first formal Code for composite construction in the United States came about with adoption of the AASHO Specification in 1944. The need for economic methods of construction to rebuild bridges and buildings destroyed during World War II prompted the development of the German Code. It was not until the 1960's that composite beams became a popular form of bridge construction in the United Kingdom.

The growth of composite construction in buildings was slow in comparison with bridges, partly due to the initial high cost of shear connectors and the fact that early research was biased towards bridge construction. Global and section analysis were based on elastic design methods.

Design rules for buildings were included in the AISC Specifications in the 1950's and in 1965 the first British Code on simply supported beams in buildings was published.

The 1970's saw a marked increase in the use of composite construction in buildings due to improved welding techniques for shear connectors in combination with the introduction of profiled steel sheeting. It was also a period during which a considerable amount of research was done on composite design resulting in design rules for:

1) limit state design
2) control of deflection by limiting span/depth ratios
3) partial shear connection
4) effective breadth of the concrete slab
5) global analysis by plastic and elastic methods with moment redistribution
6) section analysis and solution of problems associated with local instability.

These design rules have been incorporated in subsequent Codes to varying degrees.

The history and development of design Codes from selected countries, viz. Britain, South Africa, Canada and Europe, are reviewed briefly.

The Australian and New Zealand Codes are not dealt with in depth in subsequent sections, but a description of their development and adopted method of global and section analysis is included here.

2.1 CODE : BS 5950: Part 3.1 - 1990 (Buildings)

BS 5400: Part 5 - 1979 (Bridges)

COUNTRY OF ORIGIN : BRITAIN

A brief history of the development of British composite steel and concrete design Codes is reviewed in chronological order.

1965 : CP117 - Part 1 issued with design rules for simply supported beams in buildings and allowed plastic design of beams.

1967 : BS449 - Part 2 dealt with the design of continuous beams for bridges by elastic analysis.

1976 : BS449 - Part 5 - DRAFT. When limit state design philosophy was introduced with CP110, Code of practice for the structural use of concrete, it became the policy of the BSI that future Codes of practice should be in limit state form, thus the format of this Code. As a result of a considerable amount of research on composite construction, this Code was to contain a number of new approaches:

a) Improved economical use of shear connectors by introducing 'partial shear connection',

b) Deflection controlled by limiting span/depth ratios

c) Two methods of GLOBAL ANALYSIS were allowed;

  i) A restricted plastic analysis,

  ii) An elastic analysis allowing redistribution of bending moments by considering sections over supports cracked or uncracked.

d) CROSS-SECTION ANALYSIS was given in terms of a reduction factor x the plastic moment of resistance. This draft, however, was never published.

The bridge design Code then became more urgent than the building Code, resulting in the latest bridge Code, BS 5400, which comprises ten parts and replaces CP117 : Part 2. BS 5400 : Part 5 - 1979 deals with the design of composite bridges with recommendations for simply supported and continuous beams, composite columns and composite box beams.

BS 5950, 'structural use of steelwork in building' now replaces BS449 and CP117, Part 1. It is a document combining Codes of practice to cover various aspects of steel construction and will comprise of nine parts.

BS 5950 - Part 4 was the first to be issued and deals with the design of concrete slab and profiled steel sheeting.

BS 5950 - Part 3 consists of two parts; Section 3.1, Code of practice for design of simple and continuous composite beams was issued in draft form for public comment in 1976 and as a formal Code in December 1990. This dissertation was originally based on the draft version and was in an advanced stage when the final version was issued. In view of the significant difference in result obtained by the use of the two versions, both have been included in the detailed investigation which follows. Where requirements are the same, the Code is referred to as BS 5950 but when they differ, the postscript (D) indicates the draft version and (F) the final.

2.2 CODE : SABS 0162 - 1984

COUNTRY OF ORIGIN : SOUTH AFRICA

This Code is a revision of SABS 0162 - 1980, an allowable stress design Code for the structural use of steel. It was based on BS449 and Chapter 6 of the South African Standard Building Regulations.

In the revised version the allowable stress design philosophy has been retained. Along with revisions concerning bare structural steel design, sections on plastic design and composite construction of beams based on ultimate limit state design philosophy have been added.

Chapter 13, Composite Construction, incorporates parts of the British, European and United States Code recommendations based on research in the 1970's. It is applicable to the design of simply supported and continuous uncased structural steel beams with rigid joints, compositely connected to a reinforced concrete slab. Design is based on two classes of section classification, compact or slender. Design rules are too restrictive, making the Code conservative by comparison to codes from other countries.

The design of encased steel beams is included but limited to elastic conditions due to problems associated with plastic strains. The use of profiled steel sheeting as permanent shuttering is also dealt with.

2.3 CODE : CAN/CSA - S16.1 - M (Draft 1988)

COUNTRY OF ORIGIN : CANADA

As noted previously, the Canadians were one of the forerunners in the study of composite interaction between steel beams and concrete, dating back to the early 1920's. Design tables for the partial encasement of the top flange and part of the web of beams in concrete were published in 1937 in the CISC Steel Handbook.

The first clauses covering composite beam design in buildings were published in the 1941 version of the National Building Code of Canada and in 1944, the first specification for composite bridge girder design was issued.

Design Codes up to the 1969 Standard were based on working stress design methods. The first United States design edition was published in 1974 in imperial units followed in 1978 with the first SI (metric) unit Code and revised in 1984. The 1988 version incorporated changes reflecting the latest research developments and will supersede the 1984 version.

The draft Code provides design rules for steel structures for buildings with composite steel and concrete design treated as a subsection.

2.4 CODE : EUROCODE 4 - COMMON UNIFIED RULES FOR COMPOSITE STEEL AND CONCRETE STRUCTURES

ORIGIN : COMMISSION OF THE EUROPEAN COMMUNITIES

In recent years there has been a drive to harmonise international design Codes and Standards. The two most influential bodies involved with this harmonization are the European Economic Community (ECC) and the International Standards Organisation. (ISO)

In 1971 the "Joint Committee on Composite Structures" was formed under the auspices of the Liaison Committee of International Associations for Civil Engineering to prepare a Draft Model Code for Composite Structures.

The Code was published in 1981 and deals with the design and construction of simple and continuous composite beams, composite floors with profiled steel sheet and composite columns.

The Commission of the European Communities is at present producing eight European Codes - the EUROCODES - for the design and execution of buildings and civil engineering structures. Based on the Model Code for Composite Structures discussed above and Eurocodes 2 and 3, the drafting panel has produced EUROCODE No.4, issued in 1985 in draft form for comment.

The EUROCODES are intended mainly for use in buildings and only includes small bridges. The scope of EUROCODE 4 has been chosen to include most common applications where "elements consisting of a steel component and a reinforced or prestressed concrete part mechanically interconnected so as to act together to resist load."

2.5 CODE : AS 2327, Part 1 - 1980

COUNTRY OF ORIGIN : AUSTRALIA

The Australian Standard 2327, Part 1 - 1980 is a revision of and supersedes supplement No. 1 to As 1480 - 1974.

The Code is divided into four sections. Part 1 deals with rules for the use of simply supported steel beam and concrete slab construction in structures; other Parts are as follows:

Part 2 - Continuous Beams
Part 3 - Slabs
Part 4 - Columns

Parts 2, 3, and 4 are in course of preparation. AS 2327, Part 1 is to be read in conjunction with the Standards Association of Australia steel structures Code AS 1250 - 1981.

The flexural capacity of a section may be assessed by the use of either the load factor method of design or the working stress method of design.

The LOAD FACTOR METHOD is used to determine the ultimate moment of resistance (M'r). There is no limit on the position of the neutral axis, but when the axis falls within the depth of the concrete slab, a capacity reduction factor ϕ = 0,95 where M'r = ϕ. M. When the neutral axis (NA) falls in the Steel Section, ϕ = 0,9.

There is also no limit on the steel section slenderness, but in order to comply with ductility requirements at the ultimate limit state, the depth of NA is limited to 0,16 x overall depth of the section i.e. depth of slab and steel section). When the NA falls within the concrete slab, the depth of the compression stress block is limited to 0,85 x 0,16 x overall depth of section.

The moment capacity by the WORKING STRESS METHOD is found by elastic analysis using the transformed section method assuming full interaction. It is more laborious than the load factor method and more complex than that for reinforced concrete beams.

2.6 CODE : NZS 3403 - 1977

COUNTRY OF ORIGIN : NEW ZEALAND

The New Zealand Standard NZS 3404 - 1977 comprises of the Australian Standard AS 1250 - 1975 : SAA steel structures Code with amendments. A section on composite structures has been added since it was not dealt with in As 1250 - 1975 (AS 1250 was revised in 1981 and in addition, a composite section has been incorporated).

The section on composite design covers the design of:

1) structural steel sections encased in reinforced concrete acting as a beam
2) structural steel sections supporting a reinforced concrete slab acting together to resist bending and shear
3) steel encased reinforced concrete member acting as a column.

The conventional concrete slab connected to a steel section by welded shear connectors is designed by the 'strength' or plastic design method as required for the design of bare steel sections.

Global analysis is done by elastic methods and 35% redistribution of bending moments due to dead and live load is allowed (only 15% redistribution is allowed for load combinations which include seismic loading).

The moment capacity of the section is based on a fully plastic distribution of stress across the section. The stress distribution in the concrete section is in accordance with the appropriate strength design rules for reinforced concrete and the effective slab width is taken as that assumed in reinforced concrete design.

In negative moment regions of continuous beams, the ultimate moment capacity may be taken as either that of the steel beam alone or the composite section taking the slab reinforcement within the effective width into account. In both cases, the steel section must comply with slenderness requirements as given for bare steel sections to ensure sufficient ductility for the formation of plastic hinges.

Should the section not comply with the slenderness limits, the maximum permissible compressive stress in bending, obtained from a section in the Code on allowable stress design of bare steel sections, is multiplied by a factor of 1,67 to convert the allowable stress to ultimate values.

3.0 GLOBAL ANALYSIS

The strict application of limit state design rules for the design of simple buildings has the disadvantage that two global analyses are required, using different factored loads on different structural models. The desired objective is a simple analysis at one of the limit states with correction factors for the other limit state.

The approach adopted by the four codes considered is to conduct the main analysis at the ultimate limit state, and to provide simple corrective measures and checks to ensure compliance at the serviceability limit state. Design at the ultimate limit state is often simpler to perform and provides a uniform distribution of safety.

The two methods generally employed for the global analysis of composite beams at the ultimate limit state are "plastic" or "collapse" analysis and elastic analysis with redistribution of bending moments.

These methods will be dealt with in Sections 3.1 and 3.2, but in order to appreciate the approach adopted by the Codes it is necessary to consider the behaviour of a loaded continuous composite beam.

3.01 Behaviour of Continuous Composite Beams

When a continuous composite beam of uniform section is gradually loaded, it behaves essentially elastically in the early stages with the hogging moments up to twice as large as the sagging moments. The ultimate sagging moment capacity may range from being less than to up to three times the ultimate hogging moment capacity. This poor match between the elastic moments at hinge locations and the ultimate moment capacity available at these locations points requires a large rotation capacity at the hogging regions to allow redistribution of bending moments.

Redistribution away from the hogging region is initiated by cracking of the concrete slab, followed by yielding of the slab reinforcement and then by yielding of the joist. The rare situation of redistribution to the support occurs as the joist yields at midspan, followed by deformation and crushing of the concrete slab.

3.1 PLASTIC GLOBAL ANALYSIS

Simple plastic theory is an attractive design approach because of its efficiency and ease of application, but it has limitations.

In order that the ultimate moment of resistance of the critical sections be achieved, it is necessary that the section exhibit sufficient rotation capacity at the hogging (negative) moment regions to allow redistribution of moment in the structure before collapse (4,12). These sections must also be capable of developing their full plastic moment of resistance and maintaining them while the plastic hinges rotate.

Johnson and Hope-Gill (12) investigated the applicability of simple plastic theory to continuous composite beams and recommended limitations on spans, loads and steel section slenderness to guarantee that plastic theory would give a safe estimate of the failure load.

The recommendations were:

a) Steel members to be of grade 43 or grade 50 steel.
b) Concrete slab to be of in situ normal density concrete with characteristic cube strengths 22,5 - 45 MPa.
c) Plastic neutral axis of positive moment regions of each span to lie within the concrete slab or the compression flange of the steel member, but not in the web.
d) The length of an end span simply supported at one end not to exceed that of the adjacent span by more than 15% .
e) Not more than half of the design ultimate load for any span to be concentrated within a length of span/5.
f) In regions where the concrete slab is in tension, the web and compression flange of the steel member are to be compact or to be suitably stiffened.

These recommendations have been adopted by most of the codes with some modification.

3.1.1 Methods of Presentation by Design Codes

BS 5950

Two methods of plastic global analysis are allowed:

METHOD 1:

A simple method to be used for CLASS 1 plastic sections where only nominal tension reinforcement in negative moment regions is used. (This reinforcement is excluded in the calculation of the section ultimate moment capacity, M~~ult~~ $_{ult}$) There are no limits on (*) adjacent span length ratios or on the position of the plastic neutral axis, but the following conditions must be met:

i) The steel beam should be a uniform section.

ii) The same section should be used in all spans.

iii) The loading should be uniformly distributed.

iv) The imposed load (SLS) < 2.5 x dead load (SLS)-

METHOD 2:

Should any of the above conditions not be met, this second more general method may be used with limitations on:

i) Adjacent span length ratios,

ii) Position of PNA (when beam is subjected to significant concentrated load, PNA must lie within o,15 x total depth of the composite section below the top of the concrete flange).

iii) Cross section slenderness. At plastic hinge locations, both the compression flange and the web should be CLASS 1 plastic.

It is then not necessary to limit the position of the PNA as in (c) above. (*)

refer to paragraph and page number (previous page)

EUROCODE 4

One method of global plastic analysis is allowed with limitations similar to BS 5950, METHOD 2 above. The additional requirements are:

1) At plastic hinge locations, the cross-section should be CLASS 1 plastic but elsewhere it could be CLASS 1 plastic or CLASS 2 compact.
2) A composite cross-section with a concrete slab in compression and a steel web CLASS 1 plastic, should be placed in CLASS 1 plastic only if the PNA of the section lies in the concrete slab or steel flange attached to it.

The cross-section slenderness limits of EC 4 are more conservative than BS 5950 (F) reducing the number of sections available for plastic analysis.

CSA S16.1

Here no distinction is made between the requirements for plain steel stuctures and composite steel and concrete structures. The conditions for global plastic analysis are similar to the simple method (METHOD 1) adopted by BS 5950 in that the steel section is limited to CLASS 1 plastic throughout and is independent of span configuration and position of the PNA.

BS 5950 (METHOD 1) limits the amount of tension reinforcement in hogging regions to nominal amounts and excludes its use in the calculation of the section plastic moment capacity. CSA S16.1 places no limits on the amount of reinforcement in hogging regions. As noted previously, composite beams in Canada and

America have traditionally been designed as simply supported flexural members [Chein and Richie (18)] and, where necessary, nominal anticrack steel is provided. This is not covered in the code and no guidance is given on the effects of tension reinforcement on the section plastic moment capacity.

The classification of steel sections for use in composite design is given as for plain steel sections and is independent of reinforcement in the concrete slab. The effects of this tension reinforcement on the cross section rotation capacity has been well documented (4, 14, 16) but is not dealt with in this Code.

SABS 0162

The conditions specified in the code, under which global plastic analysis may be performed, are as proposed by Johnson and Hope-Gill (12) with one exception that it is only necessary for the steel section in hogging regions to possess CLASS 1 plastic dimensions. No limit is placed on section slenderness in the sagging regions. Ductility for the development of fully plastic moment in the sagging regions is ensured by limiting the position of the PNA in these regions to the concrete slab or the steel compression flange.

This is a somewhat conservative approach. Later codes ensure ductility by specifying CLASS 1 plastic sections throughout the beam, or at least where plastic hinges form, with no limit on the position of the PNA under normal load.

TABLE: 3.1 <u>CONDITIONS FOR THE USE OF PLASTIC HINGE ANALYSIS</u>

CRITERIA	BS 5950 METHOD 1	BS 5950 METHOD 2	EURO - CODE 4	SABS 0162	CSA-S16
1. SPAN LENGTH RATIOS					
i) Adjacent span should not differ in length by more than 33% of the larger		■	■	■	
ii) End span not to exceed 115% of the length of the adjacent span.		■	■	■	
2. SECTION SLENDERNESS					
i) At the plastic hinge, the compression flange and web should be PLASTIC.		■	■		
ii) At <u>hogging moment</u> regions, the section should be plastic.				■	
iii) Elsewhere the web and compression flange should be plastic or compact.			■		
iv) Steel beam with uniform section in <u>all</u> spans with PLASTIC web and flange.	■				■
3. POSITION OF PLASTIC NEUTRAL AXIS (PNA)					
i) The plastic neutral axis in the sagging moment region should lie within the concrete slab or the compression flange of the steel section.				■	
ii) The PNA at sagging moment plastic hinges when beam is loaded with high concentrated loads is limited to 0,15 times the overall depth below the top of the concrete flange.		■	■		
4. NOMINAL TENSION REINFORCEMENT	■				
5. LOADING					
i) Loading to be uniformly distributed and imposed load < 2.5 times dead load.	■				
ii) Not more than ½ of imposed load on 20% of the span.				■	
iii) No repeat or impact loads allowed.			■		■
6. At plastic hinge locations compression flange should be laterally restrained.	■	■	■		■

3.1.2 Summary

Conditions for the use of plastic global analysis as specified by the relevant codes are compared in Table 3.1.

From the above, it is a general requirement that sections in the hogging moment regions be classed as CLASS 1 plastic. The acceptability of universal beams for continuous composite plastic beam design as required by the four codes is compared in Table 4.2.

The acceptance limits are based on compression flange and web slenderness ratios (see cross section classification SECTION 4.0) in the hogging moment region. The limits of CSA-16.1 are independent of tension reinforcement and are the least conservative. Eurocode 4 limits are the most conservative. It is evident that the use of a minimum amount of reinforcement permits greater freedom of choice of section, minimising the hogging moment capacity.

3.2 ELASTIC GLOBAL ANALYSIS

Elastic analysis is given as an alternative method of global analysis for beams in building structures because of the following advantages:

a) It is generally more applicable,

b) It may be applied to beams of slender cross-section that cannot be designed plastically,

c) The results of an analysis for one limit state and load combination may easily be scaled by the ratio of the relevant safety factors to obtain values for another limit state or load combination.

The main uncertainty associated with elastic global analysis of continuous composite beams is in accounting for the loss of stiffness due to cracking of the concrete slab near internal supports. This is achieved by the redistribution of bending moments.

The degree of redistribution, however, cannot be predicted accurately because of the effects of span proportions, sequence of casting and temperature and creep effects.

Modified versions of two methods of elastic analysis are generally used. The first is an "uncracked" analysis in which support moments are redistributed to midspan regions by a percentage dependent on the classification of the section and the second is a "cracked" analysis in which the stiffening effect of the concrete is neglected at internal supports over 15% of each relevant span length.

A computer study of redistribution of bending moments in composite beams has been done by Hope-Gill (8) where certain recommendations have been made and redistribution limits given.

3.2.1 Methods of Presentation by Design Codes

CSA - S16.1

The rules for elastic global analysis of continuous composite beams given by this code are as for bare steel beams with no allowance for redistribution of bending moments. An uncracked section is assumed throughout.

The influence of cracking of concrete near internal supports of continuous composite beams on the distribution of bending moments has been investigated in depth (17). It was concluded that in continuous composite beams of proportions used in buildings, the bending moment at internal supports under serviceability loads may be 15-30% lower than that given by an elastic analysis where cracking is not considered. Local yielding at the support may cause further redistribution of support moments.

The midspan ultimate moment capacity may be up to three times the support section moment capacity. By not allowing redistribution of support moments, the full moment capacity of the span sections in continuous beams cannot be utilised.

In Canada, composite steel and concrete beams are traditionally designed as simply supported flexural members (18), with allowance for nominal reinforcement to limit crack widths over supports.

The method of analysis specified would be suitable for this situation, but may be inadequate in the analysis of continuous beams.

SABS 0162

This codes allows elastic analysis of continuous composite beams with either "uncracked" section throughout or "cracked" sections over 15% of the span adjacent to each support and uncracked sections elsewhere.

Ultimate limit state bending moments obtained by one of the above methods may be redistributed, provided certain conditions are complied with. These are the same conditions under which conventional plastic analysis may be used and their purpose is to ensure that adequate rotation capacity exists.

Combining conditions for plastic and elastic global analysis place unnecessary severe restrictions on the choice of steel sections, resulting in uneconomic design.

No limits are given for the amount of redistribution when it is allowed. If the actual redistribution is less than that assumed in the design, the steel web or compression flange may be overstressed and buckle prematurely. Redistribution from sagging to hogging regions is not allowed.

EUROCODE 4

The methods of global elastic analysis and degree of redistribution of bending moments at the ultimate limit state given in Eurocode 4 are dependent on the lowest (most restrictive) classification of the support cross-section and are independent of the classification of the mid-span cross-section.

For CLASS 1 plastic and CLASS 2 compact sections, up to 30% redistribution is allowed when using the "uncracked" method. Cross-section design is then by plastic methods. Redistribution of up to 30% is as recommended by Hope-Gill (8) for CLASS 2 compact sections but is conservative for CLASS 1 plastic sections for which the recommended limit on distribution is of the order of 45%.

Two methods of global elastic analysis are given for CLASS 3 semi-compact members:

METHOD (a)

The flexural stiffness over 15% of the span on each side of an internal support is taken as the cracked value by ignoring the contribution of the concrete slab. The rest of the section is assumed uncracked and up to 20% redistribution is allowed on the results of this analysis. The method is however subject to two restrictions.

i) All mid-span sections must be designed elastically. Had the section been designed by plastic section analysis, the resultant ultimate moment capacity would have been approximately 1.35 times greater.

ii) The method is only applicable to "conventional" span ratios which is not defined by the Code.

METHOD (b)

This method uses the gross cross-section throughout without redistribution of bending moments, making it more generally applicable than method (a). The midspan regions of CLASS 1 or 2 sections may be designed plastically, but the full plastic sagging moment capacity will seldom be fully utilised because redistribution to midspan regions is disallowed.

BS 5950

The methods of continuous beam analysis at the ultimate limit state permitted by BS 5950 are considerably more straightforward than those of EUROCODE 4 and allow better utilisation of

sections because of larger amounts of redistribution permitted. The redistribution limits given are based on the findings of Hope-Gill (8).

"Cracked" or "uncracked" analysis is permitted for all classes of sections. The degree of moment redistribution depends on the classification of the compression flange only at the support and whether the section is considered "cracked" or "uncracked". Considering ease of use, section classification based on compression flange slenderness only has a distinct advantage over full section classification as adopted by Eurocode 4, since the latter method is dependent on the amount of tension reinforcement in the concrete flange, making the design process an iterative one.

TABLE 3.2 MAXIMUM REDISTRIBUTION OF SUPPORT MOMENTS FOR ELASTIC GLOBAL ANALYSIS

DESIGN CODE	METHOD OF ANALYSIS		CROSS SECT. DESIGN	COMMENTS
	UNCRACKED	CRACKED		
BS 5950				
	(*)	(*)		
CLASS 1	40%-50%	30%-40%	PLASTIC	* For non-reinforced sections.
CLASS 2	30%	20%	PLASTIC	. Section classification based on compression flange at supports.
CLASS 3	20%	10%	ELASTIC	
CLASS 4	10%	0%	ELASTIC	. The uncracked method of analysis is preferred and the cracked method is given as an alternative
EUROCODE 4				
CLASS 1 & 2	0-30%	-	PLASTIC	** Plastic cross-section design over supports and elastic in span
CLASS 3 (a)	-	20%	**ELASTIC	
CLASS 3 (b)	0%	-	ELASTIC	
CLASS 4	0%	-		
SABS 0162				
CLASS 1	UNSPECIFIED	UNSPECIFIED	PLASTIC	. Limits on min. and max. amounts of reinforcement. . Limits on position of PNA, adjacent span ratios, etc.
CLASS 2	0%	-	ELASTIC	
CSA-S16.1				
CLASS 1	0%	-	PLASTIC	. Requirements as for bare steel section.
CLASS 2	0%	-	PLASTIC	
CLASS 3	0%	-	PLASTIC	
CLASS 4	0%	-	PLASTIC	

3.2.2 Summary

The allowed methods of analysis and degree of redistribution permitted are summarised in Table 3.2.

The methods allowed by BS 5950 are clearly the least restrictive and permit better utilisation of sections by allowing relatively large amounts of redistribution.

4.0 CROSS SECTION CLASSIFICATION

In recent design codes, cross section classification has become an important criteria in determining the methods to be used for global analysis of the structure and ultimate moment capacity of the cross section and hence, load carrying capacity of the structure. This is due to reasons associated with web and flange instability and section rotation capacity.
Three levels of rotation capacity are identified by BS 5950, EUROCODE 4 and CSA-S16.1 shown gramatically below. SABS 0162 uses just two classes of cross-section, compact and slender, with a borderline between CLASS 1 plastic and CLASS 2 compact. refer to figure number

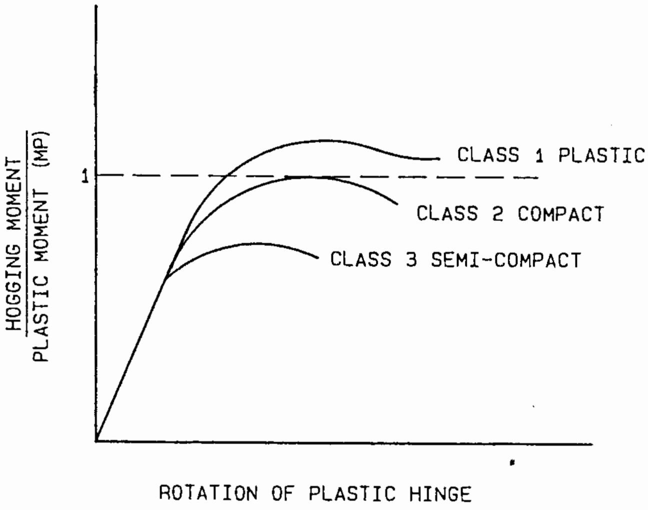


FIGURE ?

The classification of a section is taken as the more adverse of the classes of its web and compression flange.

Slenderness limits given by a design code will have a bearing on its economy and ease of use since the limits will dictate:

1) The method of global analysis - (elastic analysis, elastic analysis with moment redistribution or plastic analysis.)

2) The method of cross-section analysis (either elastic or plastic design)

3) The steel sections suitable for use.

4.1 METHOD OF PRESENTATION BY DESIGN CODES

CSA - 216.1

In this standard, cross-section analysis is by plastic methods, but no limits are placed on the slenderness of the steel section. Clause 13.5, which deals with bending of laterally supported plain steel members, allows plastic section analysis to be used on CLASS 1 plastic and CLASS 2 compact sections and one must assume that the same limits apply to composite sections. This implies, however, that the section slenderness is independent of the amount of tension reinforcement in hogging regions. The flange and web slenderness are given in the form of $b/t \; \alpha \; 1/\sqrt{Fy}$ and $h/w \; \alpha \; 1/\sqrt{Fy}$ respectively where Fy is the specified minimum yield strength of the steel member.

refer to paragraph in code

SABS 0162

Limits on the compression flange width to thickness and the clear web depth to thickness ratios are given for one class of section only, namely CLASS 1 plastic. The clear web depth to web thickness ratio is given by:

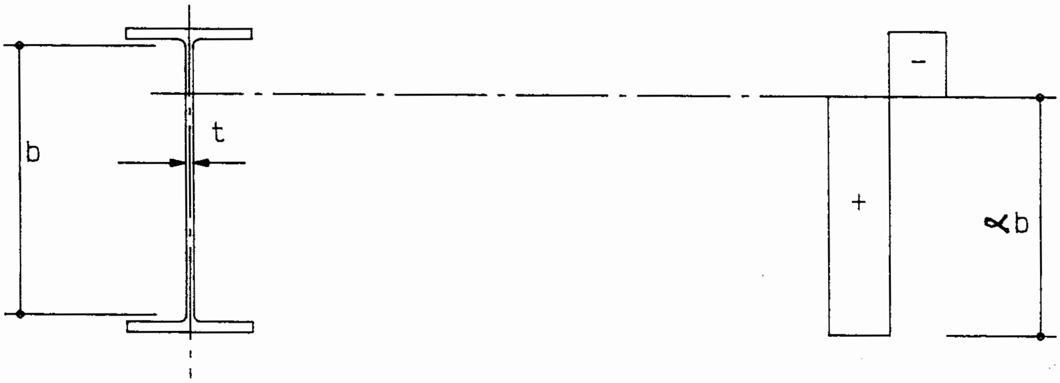
refer to paragraph and table in code.

$\frac{h_w}{t_w} \le \frac{66}{1+N_w}\sqrt{\frac{240}{f_{sd}}}$ where N_w, a factor representing the extent to which the compressive stress block will exceed half of the depth of web at the section of maximum hogging moment, is proportional to the ratio of the force in the slab reinforcement to the force in the clear web.

There are further limits placed on the area of tension reinforcement in calculating the ultimate hogging moment capacity dealt with in Section 6.

EUROCODE 4

Slenderness limits are given for four classes of cross-section. Web limits are given in the form $b/t \leq 33\epsilon/\alpha$, where α is the ratio of the depth of web in compression to the full clear depth.



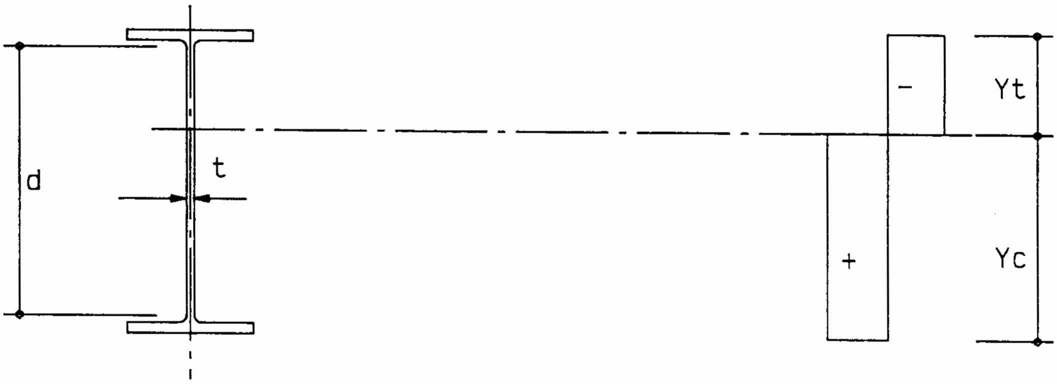
$$\epsilon = \sqrt{\frac{235}{p_y}}$$

Figure

The slenderness limits in hogging regions are therefore dependent on the area of tension reinforcement in the concrete slab.

BS 5950

Four classes of cross-section are considered. Limiting width to thickness ratio for webs is given in the form $d/t \leq 80\epsilon/1 + r$, where r is the ratio of the mean longitudinal stress in the web to p_y, the steel section design strength.



$r = (Y_c - Y_t)/d$ and $\epsilon = \sqrt{\frac{275}{p_y}}$

FIGURE

The limits specified in the draft of BS 5950 vary significantly from the final version and both have been included for comparison.

In the above, cross-section properties and slenderness limits have been specified in various forms with different dimensions. To facilitate meaningful comparison, the cross section limits and dimensions have been reduced to the form employed by BS 5950 Part 3.1.

Slenderness limits for webs are summarised in their original form in Table 4.1 and in their modified form in Table 4.2.

The modified form of slenderness limits of compression flanges written in terms of BS 5950 symbols is given in Table 4.3.

TABLE 4.1: CROSS-SECTION CLASSIFICATION FOR WEBS IN ORIGINAL FORM

CODE	ORIGINAL CRITERIA	CROSS-SECTION CLASSIFICATION			COMMENTS
		CLASS 1 PLASTIC	CLASS 2 COMPACT	CLASS 3 SEMI-COMPACT	
BS 5950 PART 1	d/t	$\frac{79\epsilon}{0.4+0.6\alpha}$	$\frac{98\epsilon}{\alpha}$	SEE 3.5.4 BS 5950 Part 1	$\epsilon = (275/p_y)^{\frac{1}{2}}$
BS 5950 PART 3 (DRAFT)	d/t	$\frac{64\epsilon}{1+0.6r}$	$\frac{80\epsilon}{1+r}$	$\frac{120\epsilon}{1+1.5r}$	
BS 5950 PART 3 (FINAL)	d/t	$\frac{64\epsilon}{1+r}$	$\frac{76\epsilon}{1+r}$	$\frac{114\epsilon}{1+2r}$	
EC 3 (PLAIN STEEL SECTION)	d/t	$\frac{62.42\epsilon}{\alpha}$	$\frac{72.59\epsilon}{\alpha}$	$19.54\sqrt{K\sigma}\ \epsilon$	$K\sigma$ = The buckling value obtained from linear buckling theory -
EC 4	d/t	$\frac{55.84\epsilon}{\alpha}$	$\frac{61.42\epsilon}{\alpha}$	$19.54\sqrt{K\sigma}\ \epsilon$	Table 5.2.3 - EC 3. $\epsilon = (235/f_y)^{\frac{1}{2}}$
SABS 0162	h_w/t_w	$\frac{66}{1+N_w}\sqrt{\frac{240}{f_{sd}}}$	-	-	N_w See Note 3 $N_w \leq 1$
CAN/CSA-S16.1	h/w	$\frac{1100}{\sqrt{Fy}}$	$\frac{1700}{\sqrt{Fy}}$	$\frac{1900}{\sqrt{Fy}}$	As for plain steel section with web in flexural compression.

NOTES:

1) d/t ratios which exceed CLASS 3 semi-compact limits are considered CLASS 4 slender.

2) For a plastic stress distribution in the web, $r = (Y_c - Y_t)/d$

For BS 5950 PART 1, $\alpha = 2\ Y_c/d$
For EC 3 and 4, $\alpha = Y_c/d$
Y_c = depth of web in compression
Y_t = depth of web in tension
d = clear web depth

3) $N_w = A_r . f_{rd} / h_w . t_w . f_{sd}$

TABLE: 4.2 CROSS-SECTION CLASSIFICATION FOR WEBS WRITTEN IN TERMS OF BS 5950,PART 3 SYMBOLS

CODE	CRITERIA	CROSS-SECTION CLASSIFICATION			COMMENT
		CLASS 1 PLASTIC	CLASS 2 COMPACT	CLASS 3 SEMI-COMPACT	
BS 5950 PART 1 (PLAIN STEEL SECTION)	d/t	$\frac{79\epsilon}{1 + 0.6r}$	$\frac{98\epsilon}{1 + r}$	See 3.4.5 BS 5950 Part 1	
BS 5950 PART 3 (DRAFT)	d/t	$\frac{64\epsilon}{1 + 0.6r}$	$\frac{80\epsilon}{1 + r}$	$\frac{120\epsilon}{1 + 1.5r}$	
BS 5950 PART 3.1 (FINAL)	d/t	$\frac{64\epsilon}{1 + r}$	$\frac{76\epsilon}{1 + r}$	$\frac{114\epsilon}{1 + 2r}$	When $r < 0$, $\frac{d}{t} = \frac{114\epsilon\ (1 + r)}{(1 + 2r)3/2}$
EC 3 (PLAIN STEEL SECTION)	d/t	$\frac{61\epsilon}{1 + r}$	$\frac{72.1\epsilon}{1 + r}$	$18.06\sqrt{K\sigma}\ \epsilon$	$K\sigma$ = The buckling value obtained from linear buckling theory.
EC 4 (COMPOSITE SECTION)	d/t	$\frac{55.5\epsilon}{1 + 1.099r}$	$\frac{61\epsilon}{1 + 1.099r}$	$18.06\sqrt{K\sigma}\ \epsilon$	TABLE: 5.2.3 - EC 3
SABS 0162	d/t	$\frac{63.8\epsilon}{1 + 1.07r}$	-	-	For SABS 0162, r is applicable to -ve moment only since position of PNA in +ve moment regions is limited to the depth of the compression flange.
CAN/SCA - S16.1	d/t	$63.3\epsilon - 1.667$	$102.5\epsilon - 1.667$	$114.6\epsilon - 1.667$	

NOTES:

1) For positive moments $r = -\frac{\text{Compressive force in concrete flange}}{\text{Resistance of clear web depth}}$, BUT $r \leq 1$

2) For negative moments $r = +\frac{\text{Resistance of reinforcement}}{\text{Resistance of clear web depth}}$, BUT $r \leq 1$

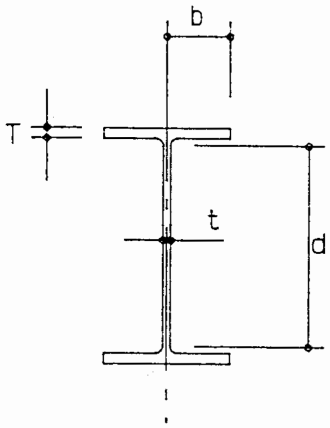
3) d/t ratios which exceed CLASS 3 semi-compact limits are considered CLASS 4 slender.

4) $\epsilon = (275/Py)^{\frac{1}{2}}$

TABLE: 4.3 CROSS-SECTION CLASSIFICATION FOR FLANGES WRITTEN IN TERMS OF BS 5950 PART 3 SYMBOLS

CODE	CRITERIA	CROSS-SECTION CLASSIFICATION		
		CLASS 1 PLASTIC	CLASS 2 COMPACT	CLASS 3 SEMI-COMPACT
BS 5950 PART 1 (PLAIN STEEL SECTION)	b/T	8.5ϵ	9.5ϵ	15ϵ
BS 5950 PART 3.1 (DRAFT)	b/T	8.5ϵ	9.5ϵ	15ϵ
BS 5950 PART 3.1 (FINAL)	b/T	8.5ϵ	9.5ϵ	15ϵ
EC3	b/T	9.24ϵ	10.17ϵ	13.87ϵ
EC4	b/T	7.40ϵ	9.24ϵ	13.87ϵ
SABS 0162	b/T	8.7ϵ	-	-
CAN/CSA - S16.1	b/T	8.74ϵ	10.25ϵ	-

$$\epsilon = \sqrt{\frac{275}{p_y}}$$



4.2 COMPARISON OF SLENDERNESS LIMITS

4.2.1 Class 1 Plastic Limits

As noted previously, all the codes require that the web and compression flange of a continuous composite beam analysed plastically (i.e. Global analysis) be classed as CLASS 1 plastic. This is to ensure that:

1) at the first plastic hinge to form, the ultimate moment capacity is in excess of the simple plastic moment capacity (Mp) and
2) that sufficient rotation capacity exists for redistribution of moment to the midspan section, while the plastic moment capacity at the support is maintained.

A comparison of the variation of web slenderness limits, d/t, for CLASS 1 plastic sections with the force ratio r is shown in Figure 4.1.

r = Force in reinforcement/Force in clear web

P_y = 300 MPa; Fst = 450 MPa

The limits of BS 5950 (F) are considerably more conservative than BS 5950 (D) for large values of the force ratio.

The slenderness classification of a section however, is taken as the more restrictive of the flange and web classification. Table 4.4 shows locally available universal beams which would comply with the various code slenderness requirements for CLASS 1 plastic sections in hogging regions and may therefore be

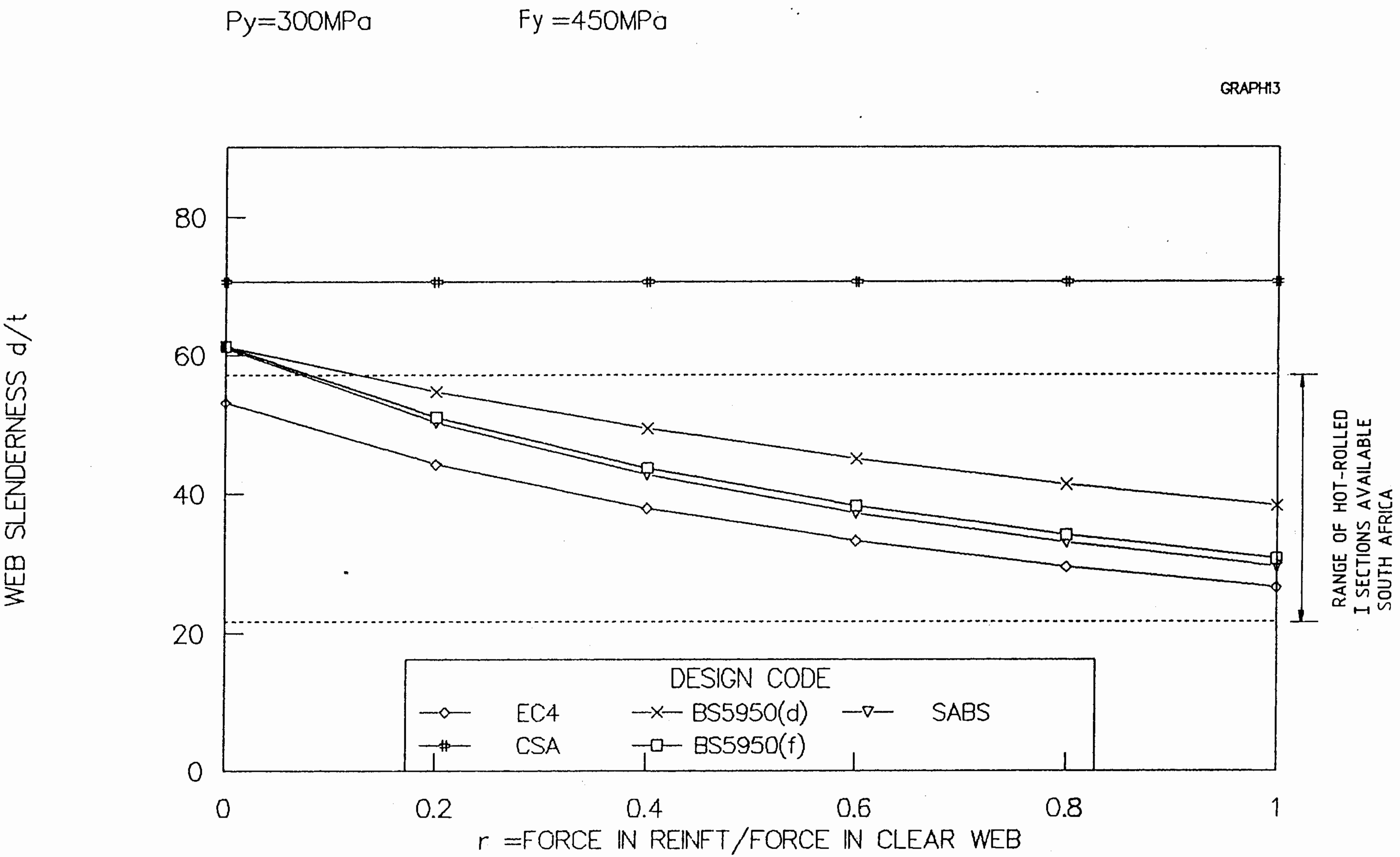


FIGURE 4.1 WEB SLENDERNESS LIMITS FOR PLASTIC GLOBAL ANALYSIS

TABLE 4.4 ACCEPTABILITY OF UNIVERSAL BEAMS FOR PLASTIC GLOBAL ANALYSIS BASED ON FLANGE AND WEB SLENDERNESS LIMITS

I SECTION mm x mm x kg/m	AREA OF REINFORCEMENT														
	250 mm²					1000 mm²					2000 mm²				
	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
152 x 89 x 16	■	■	■	■	■					■					■
178 x 102 x 19	■	■	■	■	■					■					■
203 x 133 x 25															
203 x 133 x 30	■	■	■	■	■	■	■			■					■
254 x 146 x 31															
254 x 146 x 37	■	■	■	■	■	■				■					■
254 x 146 x 43	■	■	■	■	■	■	■		■	■	■				■
254 x 203 x 49							■	■	■						
254 x 203 x 58	■	■	■	■	■	■	■		■	■	■				■
254 x 203 x 67	■	■	■	■	■	■	■	■	■	■	■	■		■	■
305 x 102 x 25	■	■		■	■					■					■
305 x 102 x 29	■	■		■	■					■					■
305 x 102 x 33	■	■	■	■	■	■				■					■
305 x 165 x 41					■					■					■
305 x 165 x 46	■	■	■	■	■	■				■					■
305 x 165 x 54	■	■	■	■	■	■	■		■	■					■
305 x 203 x 60	■	■		■	■	■	■		■	■	■				■
305 x 203 x 67	■	■	■	■	■	■	■	■	■	■	■				■
305 x 203 x 74	■	■	■	■	■	■	■	■	■	■	■	■		■	■
356 x 171 x 45															
356 x 171 x 54	■	■		■	■	■				■					■
356 x 171 x 57	■	■	■	■	■	■	■		■	■					■
256 x 171 x 67	■	■	■	■	■	■	■	■	■	■	■				■
406 x 140 x 39					■					■					■
406 x 140 x 46	■	■		■	■					■					■
406 x 178 x 54				■	■					■					■
406 x 178 x 60	■	■	■	■	■	■				■					■
406 x 178 x 67	■	■	■	■	■	■	■		■	■	■				■
406 x 178 x 75	■	■	■	■	■	■	■	■	■	■	■				■
457 x 152 x 52	■	■		■	■					■					■
457 x 152 x 60	■	■		■	■					■					■
457 x 152 x 67	■	■	■	■	■	■	■			■					■
457 x 152 x 74	■	■	■	■	■	■	■		■	■	■				■
457 x 152 x 82	■	■	■	■	■	■	■	■	■	■	■	■			■
457 x 191 x 67	■	■		■	■	■				■					■
457 x 191 x 75	■	■	■	■	■	■	■			■					■
457 x 191 x 82	■	■	■	■	■	■	■		■	■	■				■
457 x 191 x 90	■	■	■	■	■	■	■	■	■	■	■				■
457 x 191 x 98	■	■	■	■	■	■	■	■	■	■	■	■		■	■
533 x 210 x 82	■	■		■	■	■				■					■
533 x 210 x 93	■	■	■	■	■	■	■		■	■					■
533 x 210 x 101	■	■	■	■	■	■	■		■	■	■				■
533 x 210 x 109	■	■	■	■	■	■	■	■	■	■	■	■			■
533 x 210 x 122	■	■	■	■	■	■	■	■	■	■	■	■		■	■
610 x 229 x 101	■	■		■	■	■				■					■
610 x 229 x 113	■	■	■	■	■	■	■		■	■					■
610 x 229 x 125	■	■	■	■	■	■	■		■	■	■				■
610 x 229 x 140	■	■	■	■	■	■	■	■	■	■	■	■		■	■

1) BS 5950 (D)
2) BS 5950 (F)
3) EC 4
4) SABS 0162
5) CSA - S16.1

used for plastic global analysis. The table is based on p_y = 250 MPa and f_y = 450 MPa and areas of tension reinforcement (250 mm^2, 1000 mm^2 and 2000 mm^2) which would correspond to 0.1%, 0.4% and 0.8% of a representative 1900 x 130 thick concrete slab.

4.2.2 Class 2 Compact

Compact sections are defined as sections which have sufficient plasticity to attain their ultimate plastic moment capacity, but not necessarily to exceed it, nor to be able to sustain any plastic rotation at this moment.

The limiting value given in BS 5950 draft, $80\epsilon/(1+r)$ was modified to $76\epsilon/(1+r)$ in the final code. This is less conservative than the Eurocode 4 limit, $61\epsilon/(1+1.099r)$. *

In SABS 0162, no limits are given for CLASS 2 compact sections, but the definition for compact sections given above is deemed to apply if, in addition to the slenderness limits given for CLASS 1 plastic sections, the following are complied with:

1) The minimum area of longitudinal reinforcement within the effective width of the slab shall be not less than 7.5% of the area of the steel section or 0.3% of the area of the concrete slab and
2) The maximum value of this area of reinforcement shall be not greater than 30% of the area of the steel section, nor greater than 1.5% of the area of the concrete slab.

Py=275MPa Fy =460MPa

GRAPH 22

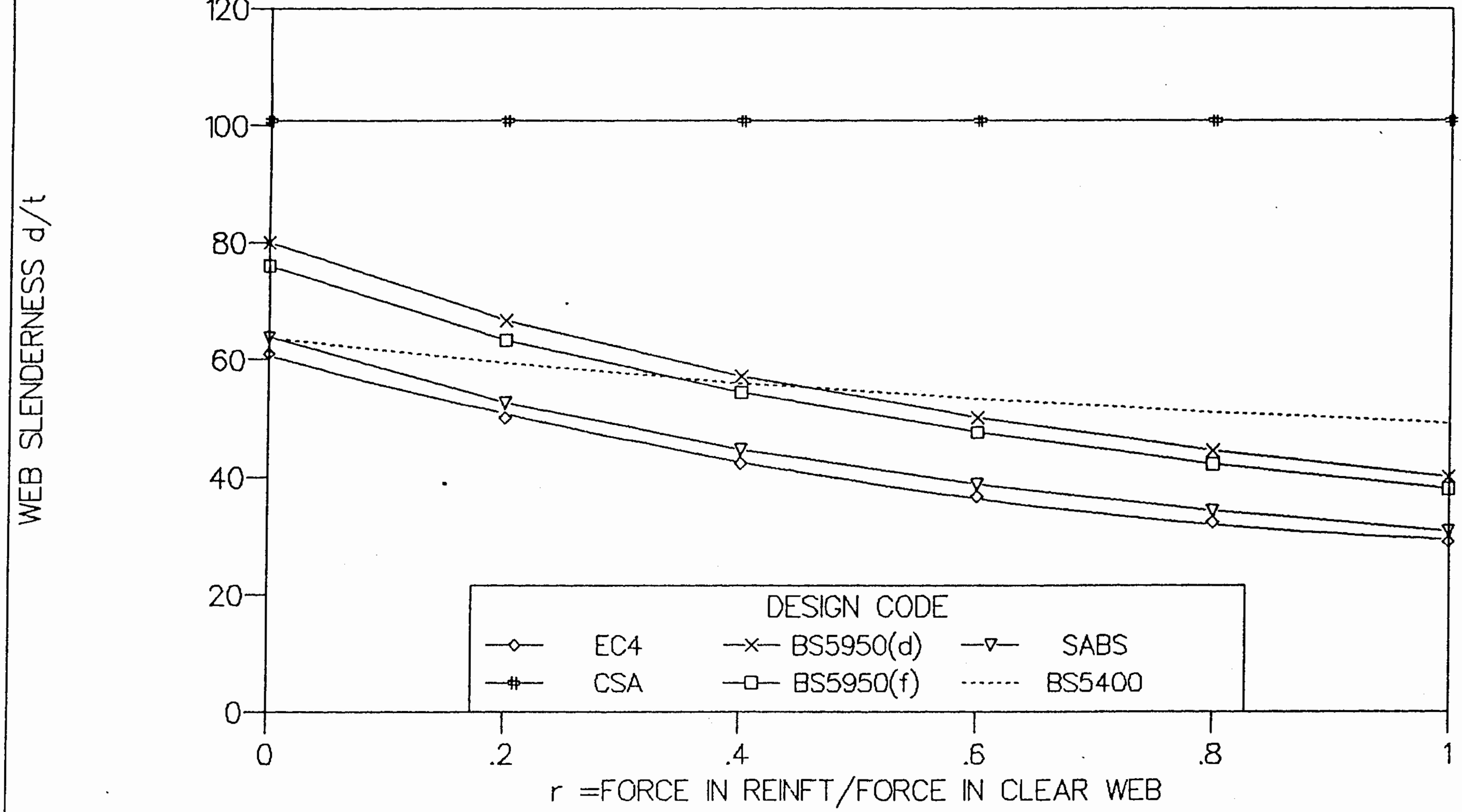


FIGURE 4.2 COMPARISON OF WEB SLENDERNESS LIMITS – CLASS 2

The variation of web slenderness limit, d/t, for CLASS 2 compact sections, with the force ratio r is shown in figure 4.2. For SABS 0162, Eurocode 4 and both versions of BS 5950, the depth of web in compression is based on the plastic neutral axis.

Slenderness limits given by BS 5400 (code of practice for the design of composite bridges) have been included for comparison.

The depth of web in compression to BS 5400 is based on the elastic neutral axis and is of the form $d/t = 31.818\ \epsilon/\alpha_{el}$ where α_{el} = depth of web in compression/clear depth of web for an elastic stress distribution, the concrete flange assumed cracked.

Eurocode 4 and SABS 0162 are the most conservative for the full range of force ratio values. Both versions of BS 5950 show moderate conservatism compared to BS 5400 for values of r>0.4, but become less conservative for r<0.4.

As defined previously, a section classed as CLASS 2 compact should at least be able to reach its simple plastic moment capacity before failure. Using the results of a series of composite beams tested to failure under laboratory conditions (See Appendix B for details of tests and material and section properties), the slenderness limits specified by the Codes are tested for accuracy by plotting the ratio of test moment capacity (M_t) to the simple plastic moment capacity (M_p) against the more restrictive of either the web slenderness limits (d/t) allowable/(d/t) actual or the flange slenderness limits (b/t_f) allowable/(b/t_f) actual. See figures 4.3 a, b, c & d.

CRITICAL RATIOS
□ d/t
◆ b/tf

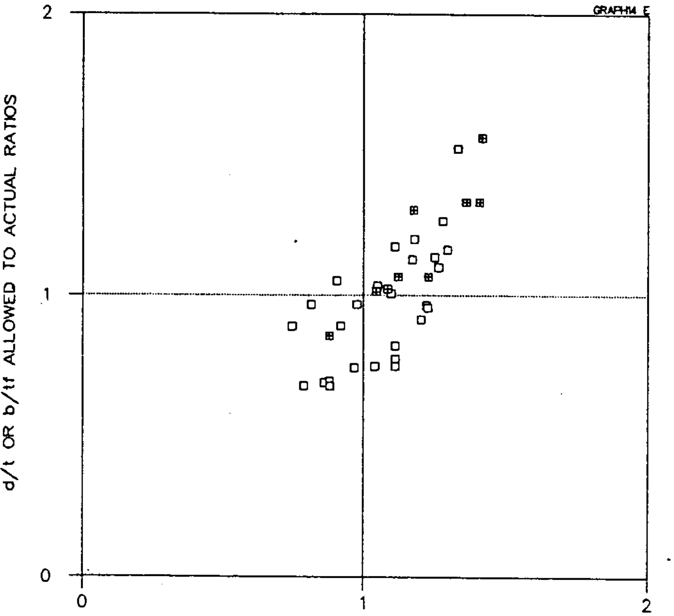


FIGURE 4.3a SLENDERNESS LIMITS FOR PLASTIC SECTION ANALYSIS – BS5950(F)

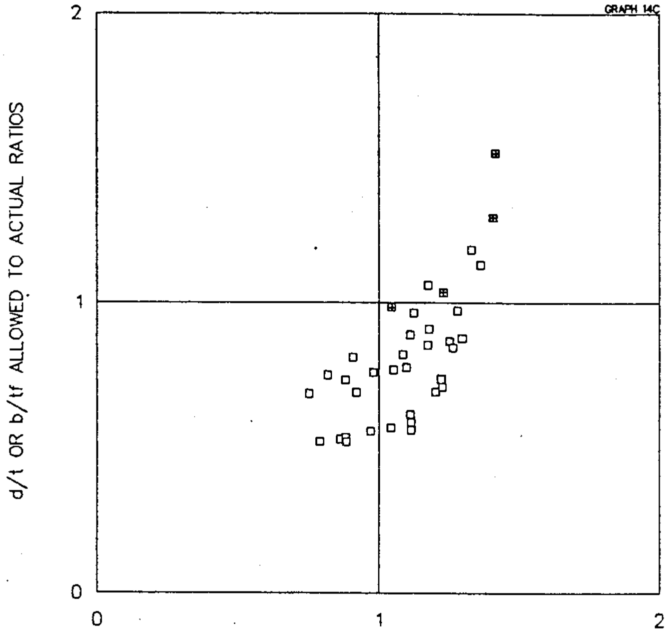


FIGURE 4.3b SLENDERNESS LIMITS FOR PLASTIC SECTION ANALYSIS – EUROCODE 4

CODE UNSAFE	CODE COMPLIANCE PLASTIC ANALYSIS ALLOWED
CODE COMPLIANCE PLASTIC ANALYSIS DISALLOWED	CODE CONSERVATIVE

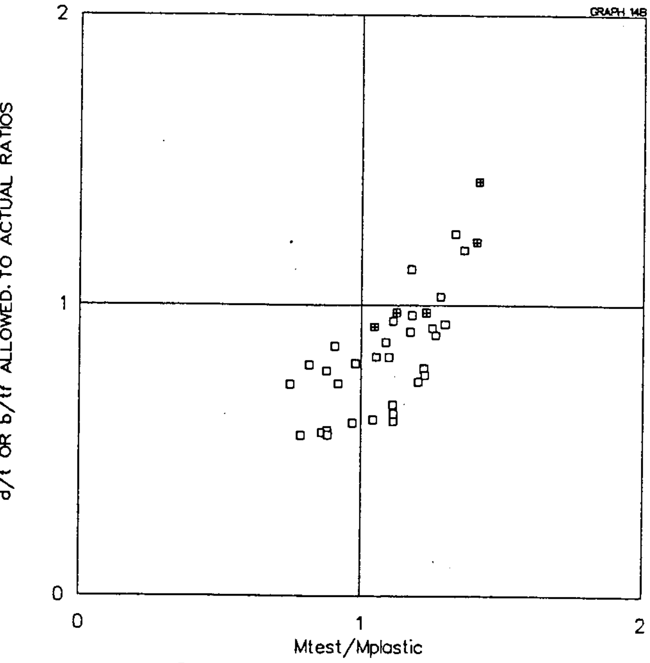


FIGURE 4.3c SLENDERNESS LIMITS FOR PLASTIC SECTION ANALYSIS – SABS0162

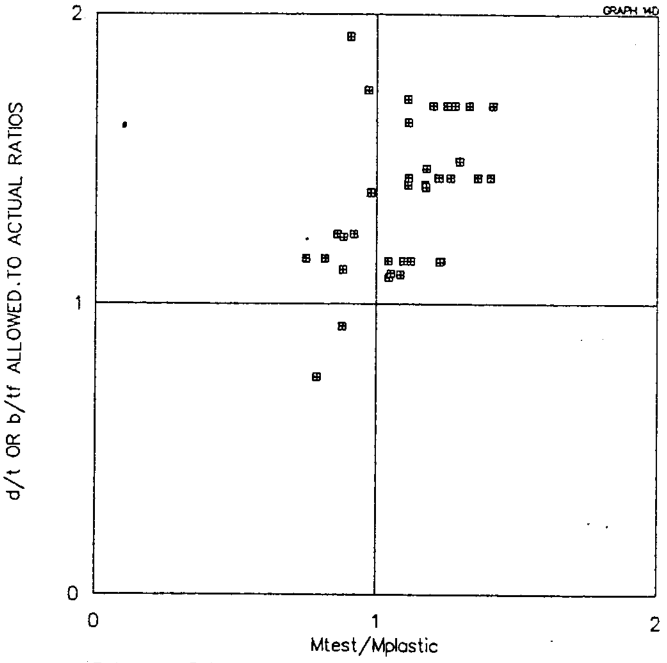


FIGURE 4.3d SLENDERNESS LIMITS FOR PLASTIC SECTION ANALYSIS – CSA–S16.1

The figures are divided into four quadrants by the lines through the point (1,1). Plotted results in the top right and bottom left quadrants are therefore consistent with the limits required by the respective code i.e. either allowing or disallowing full plastic section analysis.

Results which fall in the bottom right quadrant suggest that the code limits are conservative and results in the top left quadrant are considered unconservative.

The results of EUROCODE 4 and SABS 0162 are similar but conservative compared to the grouping of the BS 5950 (F) results which indicates a better and less conservative code compliance. Slenderness limits of CSA-S16.1 are independent of concrete slab tension reinforcement, with the result that the flange slenderness limits are more critical. A large percentage of the points fall within the unsafe quadrant.

4.2.3 Class 3 Semi-Compact

A class 3 semi-compact section is defined as one in which failure occurs before the full plastic resistance is reached. Beyond this slenderness limit, the section yield strength cannot be reached before the onset of buckling of the steel section.

In BS 5950 (D), it was proposed that plastic section analysis be used to determine the ultimate moment capacity of all sections, provided the compression flange is compact. For sections more slender than CLASS 2 compact, part of the web is disregarded when assessing the moment of resistance. The limiting criteria would then be the slenderness of the compression flange only.

This approach was significantly modified in the final version of BS 5950. Part of the web is disregarded when assessing the moment of resistance of CLASS 3 semi-compact sections, but elastic analysis is used to determine the ultimate moment capacity of sections which are more slender.

Eurocode 4 is more conservative and requires that the ultimate moment capacity of CLASS 3 semi-compact sections be determined by elastic methods. Slenderness limits for CLASS 3 sections, adopted from Eurocode 3, are given as $d/t \le 18.06 \sqrt{K\sigma\epsilon}$ where $K\sigma$ is defined as the buckling value obtained from linear buckling theory.

In view of the significant difference in ultimate moment capacity of a section obtained by plastic analysis compared to that obtained by elastic analysis, the slenderness limit beyond which a section is required to be analysed elastically is of considerable importance. This limit specified by BS 5950 (F) is CLASS 3 semi compact, by Eurocode 4 is CLASS 2 compact and by SABS 0162, CLASS 1 plastic.

In order to gauge the accuracy of these limits, a method similar to that used to compare CLASS 2 slenderness limits will be employed. The same set of test beam described in Appendix B will be used, but the ratio of the test moment capacity (M_t) to the elastic moment capacity (M_e) is plotted against the more restrictive of the web slenderness limits (d/t) allowable/(d/t) actual or the flange slenderness limits (b/t_f) allowable /(b/t_f) actual for the cases cited above. See Figures 4.4 a, b, c & d.

CRITICAL RATIOS

□ d/t or b/tf ratio – MT>MP

⊞ d/t or b/tf ratio – MT<MP

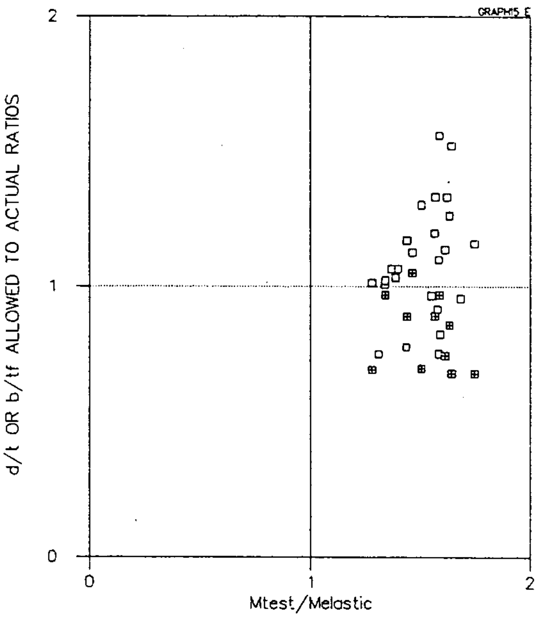


FIGURE 4.4 a SLENDERNESS LIMITS FOR ELASTIC SECTION ANALYSIS – BS5950(F)

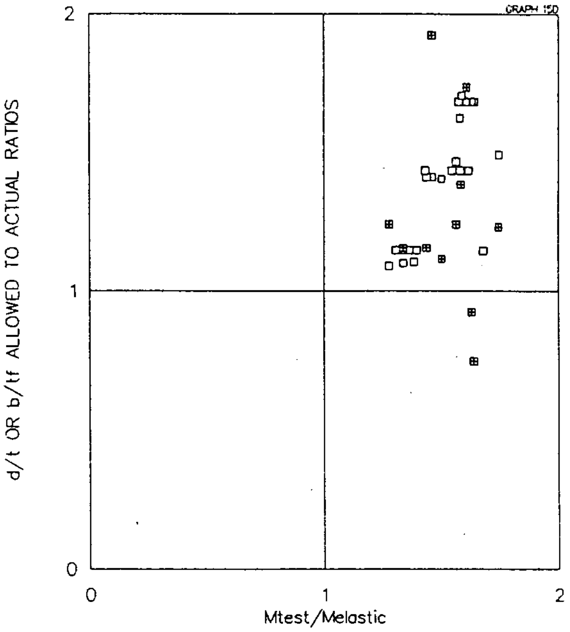


FIGURE 4.4 b SLENDERNESS LIMITS FOR ELASTIC SECTION ANALYSIS – CSA–S16.1

CODE UNCONSERVATIVE	CODE COMPLIANCE
CODE COMPLIANCE	CODE CONSERVATIVE

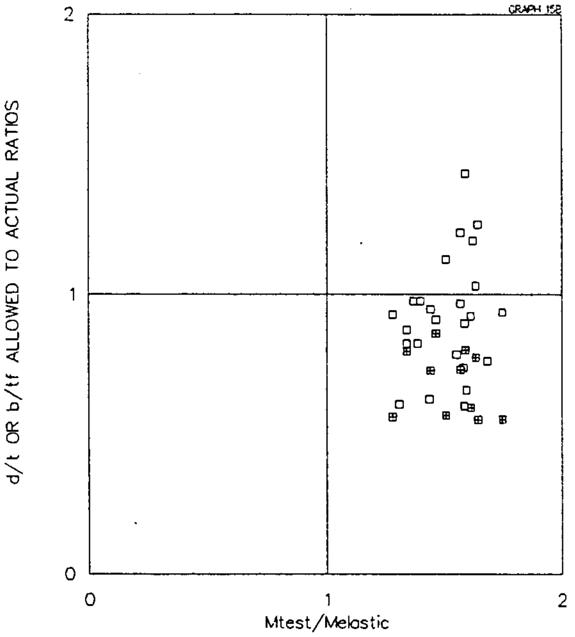


FIGURE 4.4 c SLENDERNESS LIMITS FOR ELASTIC SECTION ANALYSIS – SABS0162

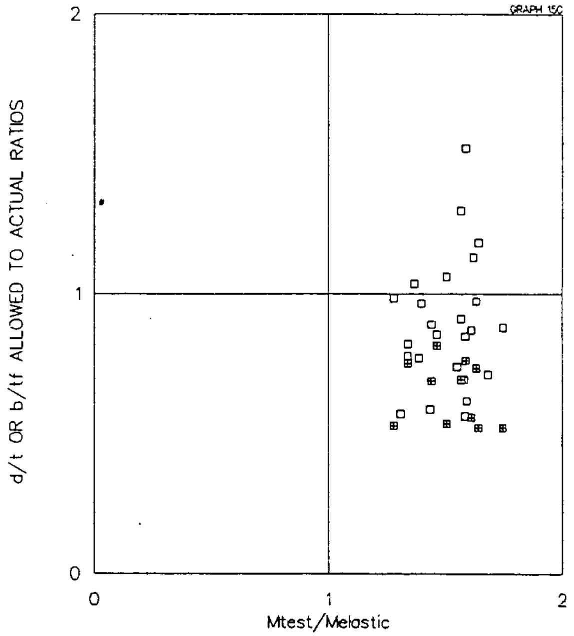


FIGURE 4.4 d SLENDERNESS LIMITS FOR ELASTIC SECTION ANALYSIS – EUROCODE 4

5.0 EFFECTIVE WIDTH OF CONCRETE FLANGE

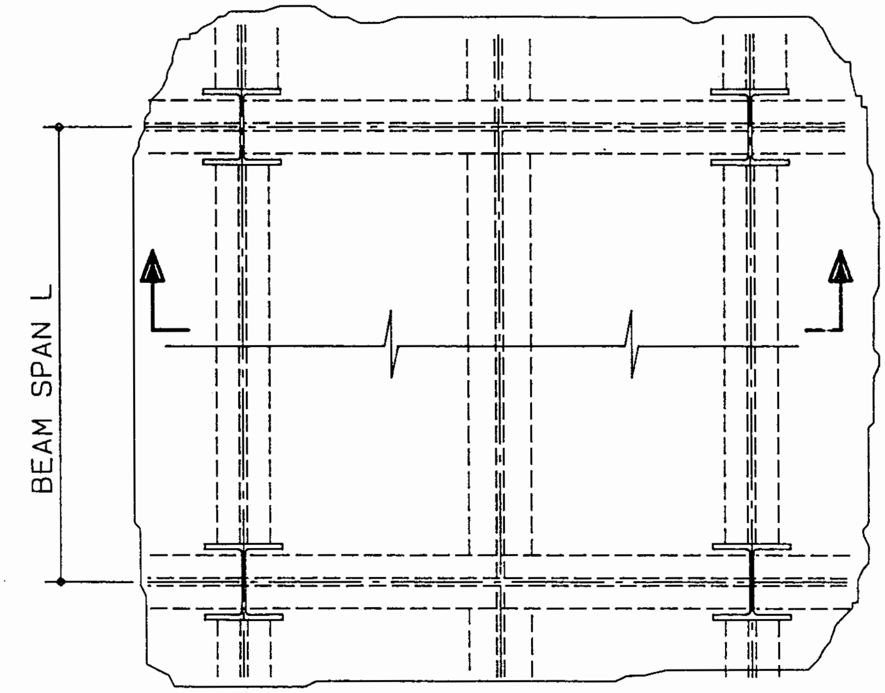
In reinforced concrete design, the effective width concept in T-beam design is a practical method of dealing with a non-uniform distribution of compressive stress in the flange.

A similar approach is adopted in composite design to a far more complex situation in which the effective width of the concrete flange is directly linked to the cross-section classification which in turn dictates the method of section and global analyses which may be employed.

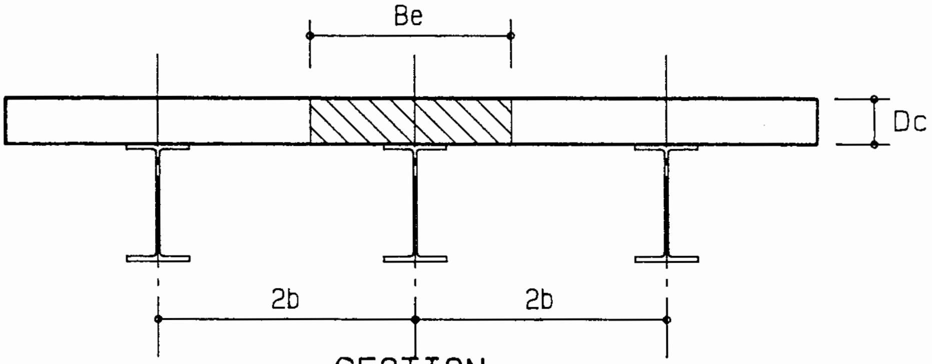
The lack of agreement on values and diversity of presentation indicates that differences in opinion exist on the effects of the value of the effective width as well as the complexity of the problem.

5.1 METHODS OF PRESENTATION BY DESIGN CODES

In the Codes considered, the total effective width B_e of concrete flange is taken as the lesser of the sum of the effective widths b_e, of the portions of flange each side of the centreline of the steel beam or the actual width b taken as half the distance to the adjacent beams, measured to the centreline of the web.



TYPICAL PART PLAN



SECTION

EFFECTIVE FLANGE WIDTH FOR INTERIOR BEAMS

The methods of determining B_e as specified by each Code are discussed below.

SABS 0162

A simple approach has been adopted considering simply supported beams with B_e equal to 0,2 L and continuous beams with B_e

equal to 0,15 L. A uniform effective flange width is used throughout the span.

CSA-S16.1

One value for B_e = 0,125 L is given for all spans, whether

* simply supported or continuous. A value of B_e = 0,1 L is given for the free edge overhanging a spandrel beam. A uniform effective flange width is used throughout.

BS 5950

The approach here is more sophisticated than the above cases, but still easy to use. Effective widths may be calculated at ends spans, internal spans, internal supports and cantilevers.

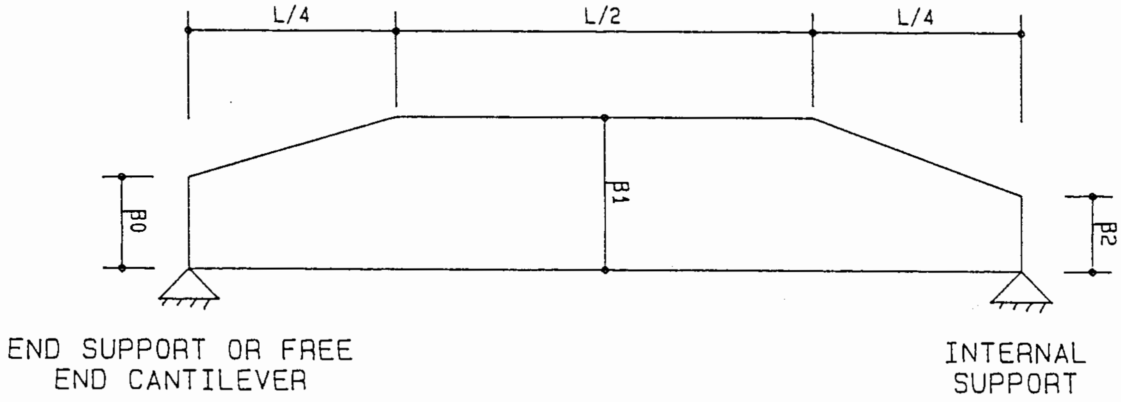
The effective width on each side of the centreline of the steel web B_e, is given as Lz/8, where Lz is the distance between points of zero moment. Value of Lz for different span situations are provided.

EC4

Two methods of determining effective widths are allowed. The first is a simple method to determine effective widths for global analysis of all sections and cross-section design of CLASS 1 and 2 cross-sections. The effective width on each side of the centreline of the steel web, B_e, is given by L/8 where L is the distance between the centres of support, for a beam, or the distance from the face of the support to the free end, for a cantilever.

The second method is much more sophisticated for general application to all classes of cross-sections. The effective width/actual width, ß, is given in relation to the ratio of the half width b to the distance between the points of contraflexure, Lz. Values of Lz for various span situations are given.

The effective width ratio, ß, is calculated separately for each discontinuous end of a span, each midspan region and each continuous end of a span, denoted $ß_0, ß_1$ and $ß_2$ respectively as shown below:



VARIATION OF EFFECTIVE BREADTH RATIOS ALONG A SPAN

This enables the effective flange width to be calculated at any point in the span, which would be necessary when the beam is subjected to heavy point loads.

5.2 COMPARISON OF LIMITS

5.2.1 Global Analysis

Research has shown that shear lag has little effect on the moments obtained in an elastic global analysis of continuous beams, so the full widths may be used in calculations of bending moments and shears. The effective flange widths allowed in global analysis are compared in Table 5.1.

The effective widths differ considerably, but the bending moments obtained from elastic global analysis will not be affected.

TABLE 5.1 COMPARISON OF EFFECTIVE FLANGE WIDTHS FOR GLOBAL ANALYSIS

DESIGN CODE	B_e INTERNAL SPAN (1)	B_e END SPAN
BS 5950	0.175L	0.2L
EC4	0.25L	0.25L
SABS 0162	0.15L	0.15L
CSA-S16.1	0.25L	0.25L

(1) at internal supports, B_e is equal to the average of the two adjacent spans.

5.2.2 Section Analysis

TABLE 5.2 SUMMARY OF EFFECTIVE FLANGE WIDTHS FOR SECTION ANALYSIS

DESIGN CODE	FULL EFFECTIVE FLANGE WIDTH B_e		
	END SPAN REGION	INTERNAL SPAN REGION	INTERNAL SUPPORT
BS 5950	0.2L	0.175L	0.125L
EC4 (CLASS 1 & 2)	0.25L	0.25L	0.25L
EC4 (CLASS 3 & 4)	$2b\beta_1$	$2b\beta_1$	$2b\beta_2$
SABS 0162	0.15L	0.15L	0.15L
CSA-S16.1	0.25L	0.25L	0.25L

In the above, the effective flange width should be taken as the lesser of B_e and 2b

$\beta_1 = 1/(1+6.4(b/L_z)^2)$

$\beta_2 = 1/1+6(b/L_z) + 1.6(b/L_z)^2)$

L_z = effective span

L_z (end span) = 0,85L

L_z (internal span) = 0.7L

L_z (over support) = 0.3L (Assuming adjacent spans equal)

The effective concrete flange widths for section analysis in the internal span and internal support regions are compared graphically in figures 5.1 and 5.2 respectively. The graphs show the variation of effective width B_e with clear span L, both axes made non-dimensional with division by b.

The graphs show a significant variation in allowable effective flange widths. The effects of the concrete flange width on cross-section classification and ultimate moment capacity of the composite section are discussed below.

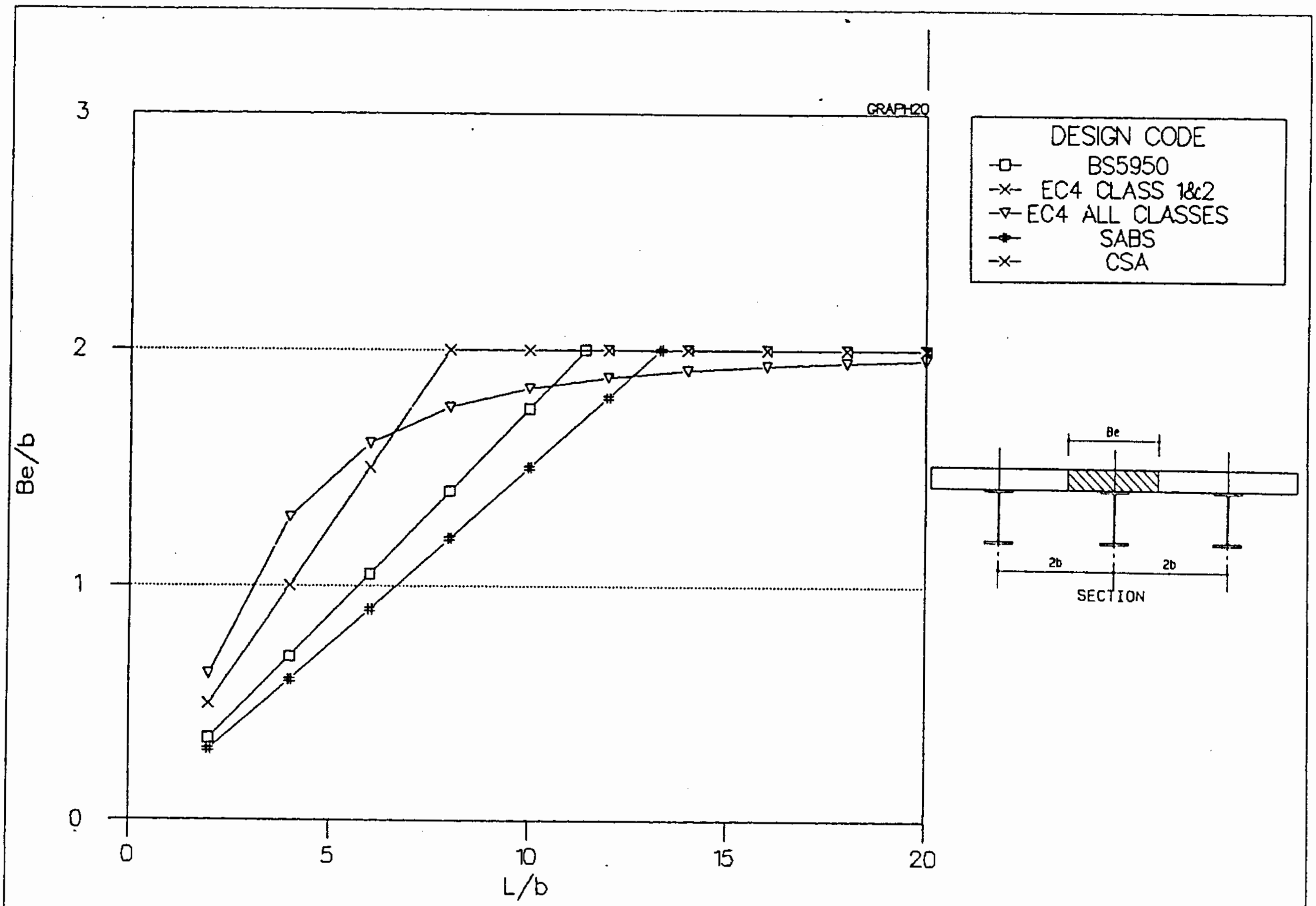


FIGURE 5.1 COMPARISON OF EFFECTIVE FLANGE WIDTHS
INTERNAL SPAN

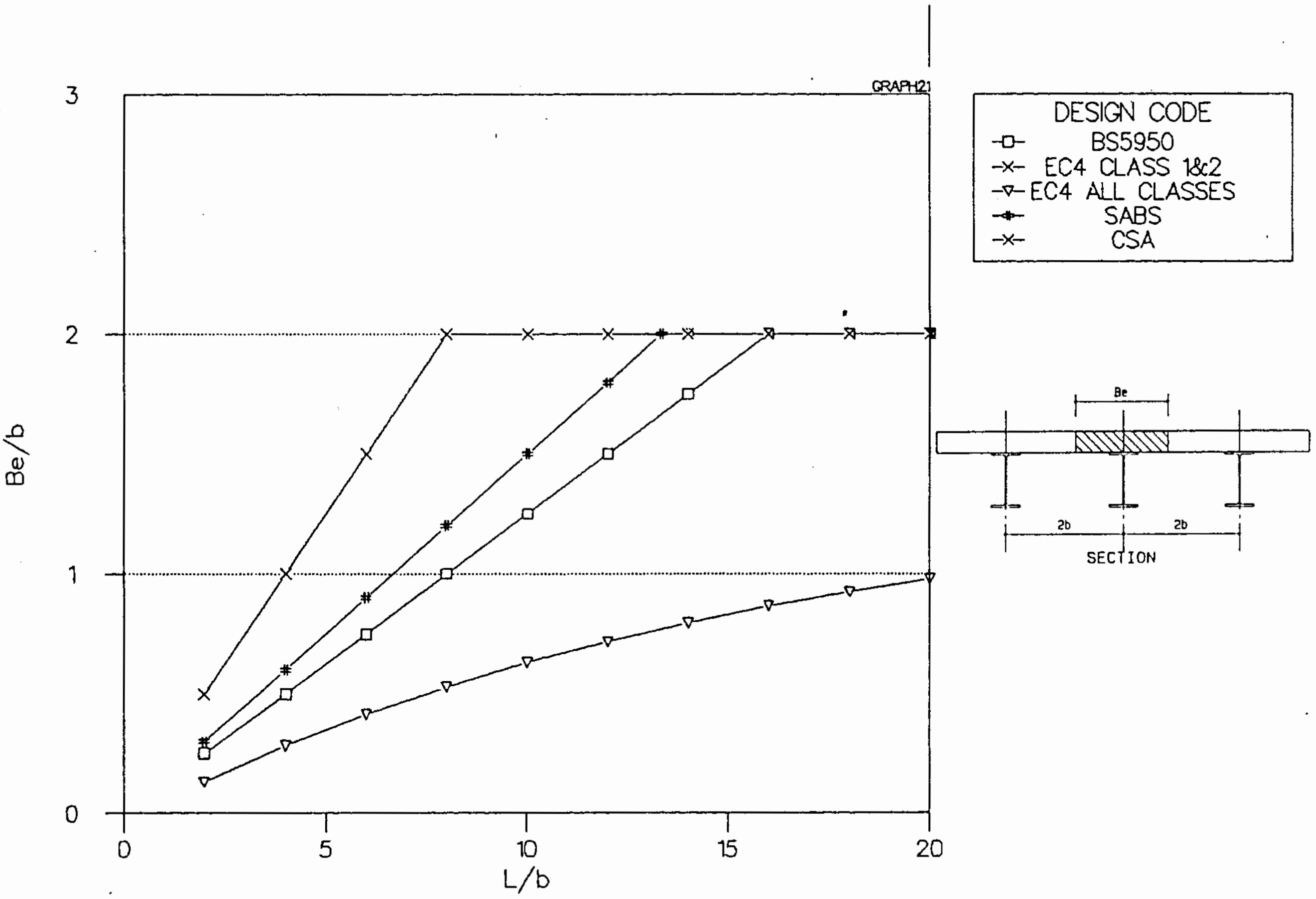


FIGURE 5.2 COMPARISON OF EFFECTIVE FLANGE WIDTHS
INTERNAL SUPPORT

5.3 EFFECT ON CROSS-SECTION CLASSIFICATION

The influence of cross-section classification on the allowed methods of global and section analysis has been demonstrated previously. It was shown that the slenderness limit concept is an important criteria in composite design. Limits given by each Code were compared, assuming equal flange widths in sagging regions and equal areas of tension reinforcement in hogging regions.

For all the Codes, excluding CSA-S16.1, the web slenderness limits are of the form $d/t \propto 1/(1 + r)$ where,
for hogging moment regions, r = Resistance of reinforcement/Resistance of clear web depth and
for sagging moment regions, r = Compressive force in concrete flange/Resistance of clear web depth.

In the sagging region, the compression force in the concrete flange is proportional to B_e and in the hogging region, the area of tension reinforcement considered is that which falls within the effective flange width.

In both instances, $r \propto B_e$ which has been show to vary significantly from Code to Code. (Fig 5.1 and 5.2)

From Figure 5.2, for an L/b ratio = 8, representing a 10 m span with parallel beams at 2,5 m centres, the effective flange width given by each Code and corresponding relative area of reinforcement is shown in the table below.

DESIGN CODE	B_e	Effective Area of Reinforcement
BS 5950	1.25	1.89A
EC4 (METHOD 1)	2.5	3.78A
EC4 (METHOD 2)	0.661	1.0A
SABS 0162	1.5	2.27A

5.4 EFFECT ON HOGGING ULTIMATE MOMENT CAPACITY

Figures 6.9 and 6.10 (Section 6) show the variation in hogging moment capacity predicted by each of the Codes for a chosen steel section with a constant flange width and varying area of tension reinforcement.

To demonstrate the influence of the effective flange width on the ultimate hogging moment capacity of a section, two typical cross-sections have been analysed and the results shown graphically in Figures 5.3 and 5.4. In each case, the predicted hogging moment capacity has been plotted against the span length, assuming a fixed area of tension reinforcement per metre of flange width. The effective area of reinforcement is proportional to the flange width which varies with span length.

Figure 5.3: A 203 x 133 x 30 kg/m I section is analysed assuming parallel beams at 2.0 m c/c. Reinforcement in hogging regions is taken as Y10 @ 200. The section is well behaved over the full range of spans considered and plastic section analysis is allowed throughout.

The predicted results do not differ too significantly and Eurocode 4 is found to be the most conservative.

SECTION : 203X133X30 kg/m
Py = 300MPa Fy = 450MPa Fcu = 25MPa
CONCRETE SLAB : 100mm thick ; 2b=2.0m ; REINFORCEMENT : Y10 @ 200

GRAPH 24

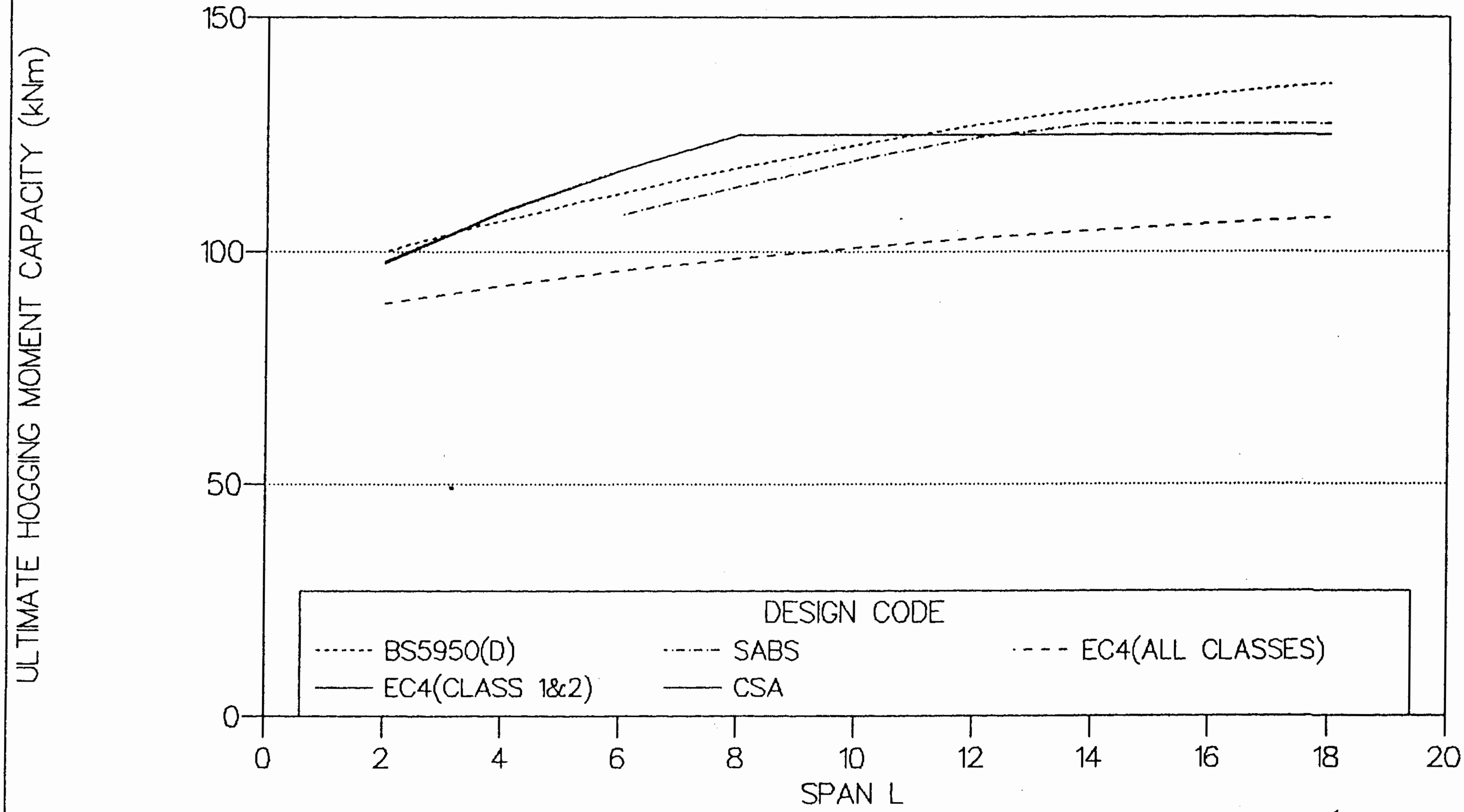


FIGURE 5.3 EFFECT OF Be ON ULTIMATE MOMENT CAPACITY (HOGGING)

Figure 5.4(a): A more slender section, 305 x 102 x 33 kg/m I section, is analysed with parallel beams at 4.0 m c/c and reinforcement in hogging regions taken as Y12 @ 200.

For SABS 0162, Eurocode 4 and BS 5950 (F) plastic analysis is allowed up to a limit beyond which the section is considered liable to fail by local instability. The predicted results differ significantly.

Figure 5.4(b): Shows the same section analysed in Fig 5.4(a) assuming equal areas of reinforcement.

Three individual sections are analysed below to highlight some of the inconsistencies that exist and to show the lack of agreement between Codes.

CASE 1

Consider the section analysed in Figure 6.9

SECTION 203 x 133 x 30 kg/m Py = 300 MPa

$D_{(s)}$ = 100 mm Fy = 450 MPa

d* = 30 mm Fcu = 25 MPa

Assuming a constant full flange width and area of tension reinforcement = 700 mm², the ultimate hogging moment capacity predicted by each Code ($M_{(c)}$) is shown in Table 5.1. Plastic section analysis was used for all the Codes.

Assume the beam is continuous with adjacent spans 7.0 m long and parallel beams at 1.75 m c/c. The allowable effective flange

SECTION 305 X 102 X 33kg/m
Fy =450MPa Py=300MPa Fcu=25MPa

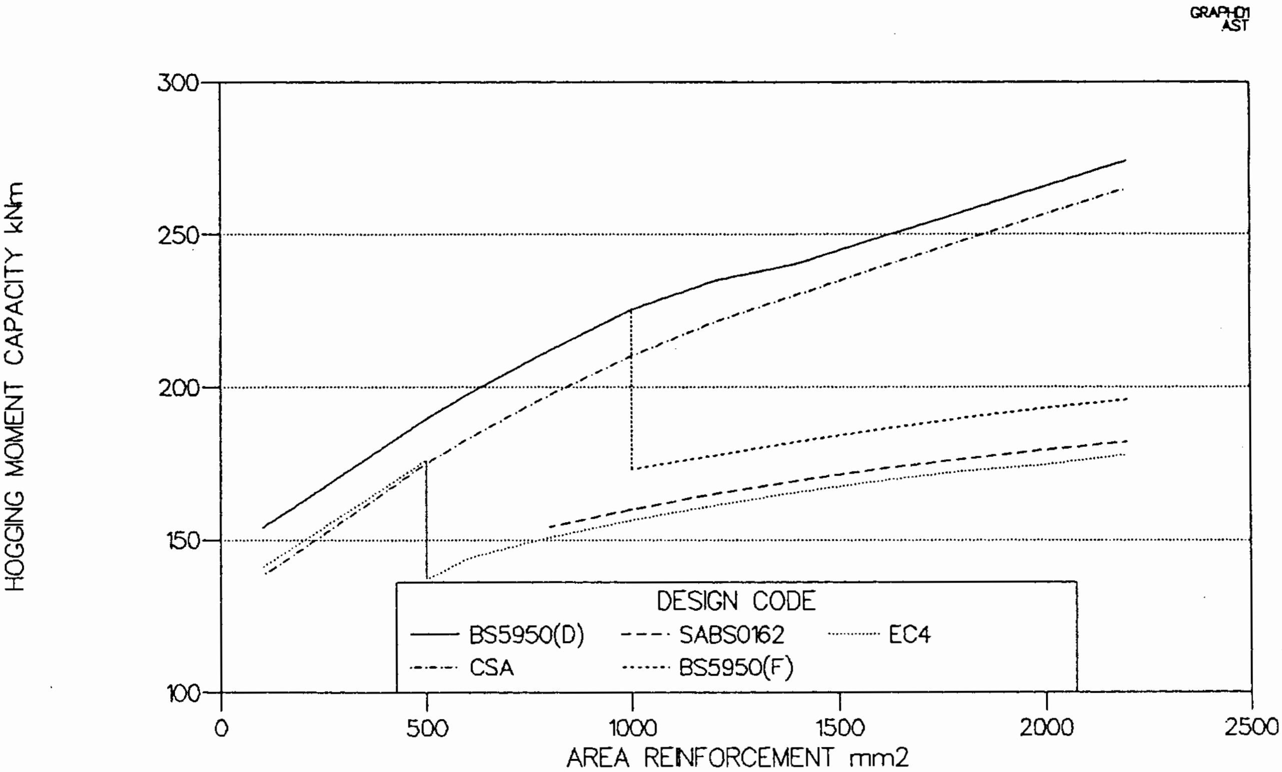


FIGURE: 5.4a COMPARISON OF PREDICTED HOGGING MOMENT CAPACITY

SECTION : 305X102X33 kg/m
Py = 300MPa Fy = 450MPa Fcu = 25MPa
CONCRETE SLAB : 130mm thick ; 2b=4.0m ; REINFORCEMENT : Y12 @ 200

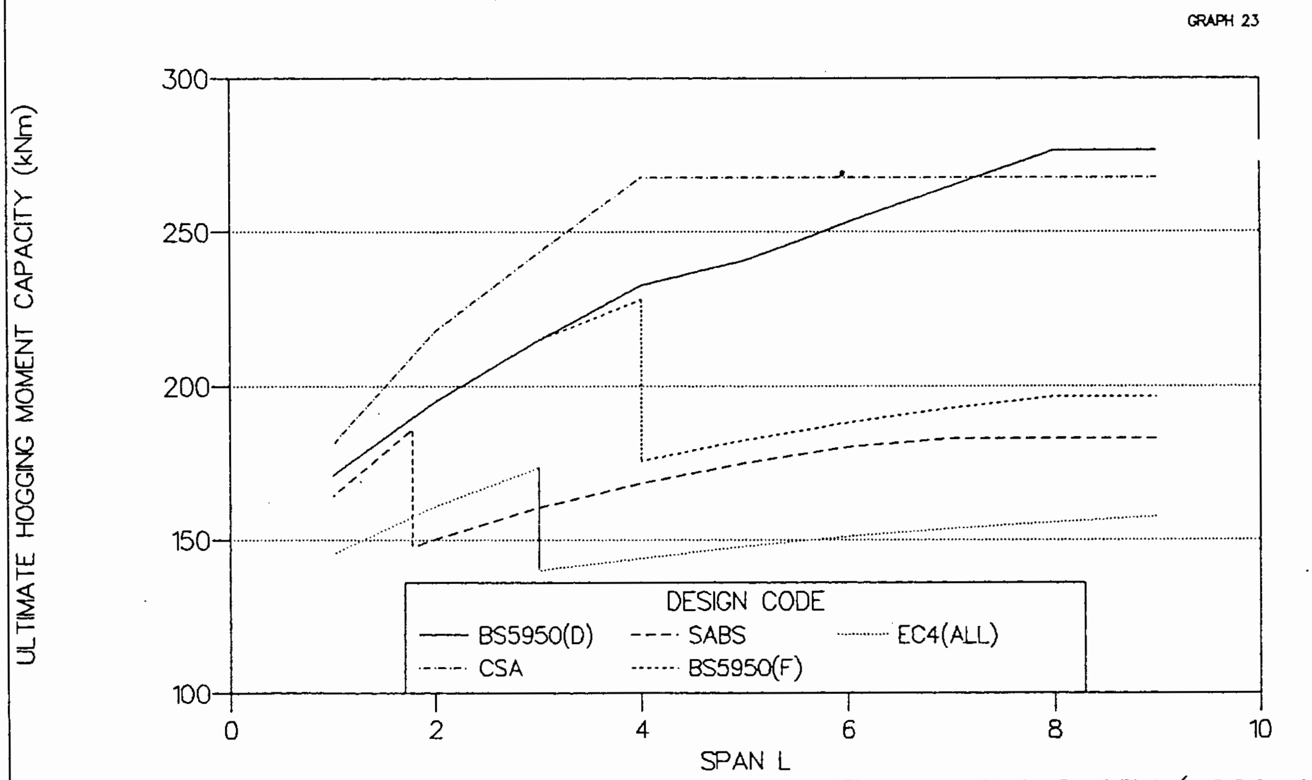


FIGURE 5.4b EFFECT OF Be ON ULTIMATE MOMENT CAPACITY (HOGGING)

width and corresponding predicted ultimate hogging moment capacity (Mv) is shown. The same reinforcement area per 1 m width as used above has been assumed i.e. Ast = $^{700}/1.75$ = 400 mm²/m.

All the sections were again found to be CLASS 1 or 2 and plastic section analysis was used throughout.

TABLE 5.1 SUMMARY OF CASE 1

DESIGN CODE	B_e mm	Ast mm²	Mv kNm	Mc kNm	Mv/ Mc
BS 5950	875	350	115.3	131.6	0.88
EC 4 (CLASS 1 OR 2)	1750	700	122.0	122.0	1
EC 4 (General)	1) 796	318	104.8	122.0	0.86
SABS 0162	1050	420	111.4	124.8	0.90
CSA-S16.1	1750	700	121.8	121.8	1

Mc is the predicted hogging moment capacity assuming a constant flange width and area of reinforcement

Mv is the predicted hogging moment capacity assuming the effective flange width and corresponding area of reinforcement.

1) Since the cross-section is classed as 1 or 2, the designer has the option of using B_e = 1750 or 796 mm and plastic section analysis is used.

CASE 2

Consider the section analysed in Figure 6.10

SECTION 305 x 102 x 33 kg/m

Ds = 130 mm

d* = 30 mm

Py = 300 MPa

Fy = 450 MPa

Fcu = 25 MPa

not consistent with list of Notation (~~page~~ parag. 10.1)

Assuming a constant full flange width and area of tension reinforcement = 800 mm^2, the ultimate hogging moment capacity predicted by each Code (Mc) is shown in Table 5.2. Due to slenderness limits, values indicated by * were obtained by elastic section analysis as required by the relevant Code.

As for Case 1, a continuous beam with adjacent spans 7.0 m long and parallel beams at 1.75 m c/c are assumed. Area of tension reinforcement assumed constant at 800/1.75 = 457 mm^2/m.

Consider the EC4 situation:

Assuming B_e = 0.25 L = 1750 mm, area of tension reinforcement = 800 mm^2

The web slenderness $^d/_t = 61\epsilon/(1 + 1.099)r = 35.83$

$^d/_t$ (actual) = 41.8 > 35.83

which implies that the section is not CLASS 1 OR 2 and hence elastic analysis was used to determine the ultimate moment capacity Mc.

However, in terms of Clause 4.5.3 (Effective area of concrete flange), since the cross-section is not CLASS 1 OR 2, the effective width is given by $B_e = 2\ b\beta z$

2b = average parallel beam spacing = 1750

$\beta z = 1/(1 + 6\ (b/Lz) + 1.6\ (b/Lz)^2)$

= 0,2647

Therefore B_e = 1750 x 0,2647 = 463 mm

and the effective area of tension reinforcement is equal to 212 mm^2.

The web slenderness $^d/_t$ = $61\epsilon/(1 + 1.099r)$ is now equal to 42,8 which is > $^d/_t$ actual.

This implies that the section now qualifies as CLASS 2 and plastic section analysis is allowed. It is unclear from clauses 4.5.3.2 and 4.5.3.3 whether this is correct or whether the section should be treated as class 3 and 4 with elastic section analysis.

TABLE 5.2 SUMMARY OF CASE 2

DESIGN CODE	B_e mm	Ast mm²	Mv kNm	Mc kNm
BS 5950	875	400	181.3	212
EC4 (CLASS 1 & 2)	463	212	151.4	-
EC4 (CLASS 3 & 4)	463	212	*125.6	*150.7
SABS 0162	1050	480	- (1)	*154.2
CSA-S16.1	1750	800	197.4	197.4

1) Section may not be used in terms of SABS 0162 limits

* elastic section analysis employed due to section slenderness

Mc : Ultimate moment capacity with constant flange width

Mv : Ultimate moment capacity with effective flange width B_e.

CASE 3

Consider the section analysed in Figure 6.11.

SECTION 406 x 140 x 39 kg/m p_y = 300 MPa

Ds = 130 mm f_y = 450 MPa

d* = 30 mm f_{cu} = 25 MPa

A constant effective flange width is assumed with area of tension reinforcement 800 mm². The ultimate hogging moment capacity (Mc) predicted by each Code is shown in Table 5.3.

As before a continuous beam span of 7.0 m is assumed with parallel beams at 1.75 m c/c giving an effective area of tension reinforcement = 457 mm²/m.

TABLE 5.3 SUMMARY OF CASE 3

DESIGN CODE	B_e mm	Ast mm²	Mv kNm	Mc kNm
BS 5950 (D)	875	400	259	285
BS 5950 (F)	875	400	259	*231 (1)
EC4 (CLASS 3 & 4)	673	308	* 189	*210 (2)
SABS 0162	1050	480	-	*215. (3)
CSA-S16.1	1750	800	273	273

Mc : ultimate moment capacity with constant flange width

Mv : ultimate moment capacity with effective flange width B_e

(1) The results of BS 5950 (D) and BS 5950 (F) are shown. For BS 5950 (F) with full flange width, the cross-section is considered slender and elastic section analysis is used. With the reduction in effective area of tension reinforcement, due to the reduced effective flange width, the section is no longer considered slender and plastic section analysis may now be used. The plastic moment capacity with less reinforcement (Mv = 259 kNm) is greater than the elastic ultimate moment capacity with 100% more reinforcement.

(2) Although there is a 60% reduction in the area of effective tension reinforcement, the section is still classed as CLASS 3 OR 4 and elastic section analysis must still be used.

(3) With the full Ast and concrete flange width, the ultimate moment capacity is found by elastic analysis. The reduced effective flange width results in a reduced area of tension

reinforcement which does not comply with the minimum Code requirements and the section may consequently not be used.

5.5 EFFECT ON ULTIMATE SAGGING MOMENT CAPACITY

Consider the cross-section analysed in Figure 6.7

SECTION: 203 x 133 x 30 kg/m p_y = 300 MPa

Ds = 100 mm f_y = 25 MPa

Assuming a constant full concrete flange width = 2000 mm, the ultimate hogging moment capacity predicted by each Code (Mc) is shown in Table 5.4.

With adjacent spans each 8,0 m and parallel beams at 2,0 m c/c, the predicted ultimate sagging moment capacities using the allowable effective flange widths are shown below.

TABLE 5.4

DESIGN CODE	B_e mm	Mv kNm	Mc kNm
BS 5950	1400	191	203
SABS 0162	1200	169	188
EC4	2000	204	204
CSA-S16.1	2000	184	184

The predicted sagging moment capacity is affected by the effective flange width, but is not as sensitive as the hogging moment region.

6.0 CROSS-SECTION ANALYSIS

The ultimate moment capacity of a composite section is determined by analysing a simplified theoretical model of the composite beam, comprising the steel section connected to an effective width of concrete slab.

As shown previously, the width of slab considered effective varies considerably from Code to Code and is dependent on the span and support beam spacing. The contribution of the steel member to the strength of the composite section is dependent on the slenderness of its web and flanges, the classification of which varies from Code to Code.

The object of this section is to compare the approach adopted by the Codes, the ease of use and accuracy of predicting the ultimate moment capacity of sections tested to failure under laboratory conditions.

The following restraints and constraints have been introduced to limit the number of variables and allow meaningfull comparison:

1) In tests conducted on composite sections (Appendix B), the beam and slab proportions of test specimens were such that the full concrete flange width was always considered effective. In this section, the effective slab width of a test specimen will be assumed constant for all Codes.
2) It will be assumed that full shear connection has been provided and that the section is not influenced by reduction in moment capacity due to shear.

3) The effect of load factors is excluded, but material factors will be accounted for.

4) The effects of lateral torsion buckling is not considered a constraint.

5) The Codes considered are used primarily for the design of building structures and consequently the sections, span proportions and loads are assumed to be suitable for this application.

6.01 Materials

Concrete:

Concrete strengths in Eurocode 4 and CSA-S16.1 are given in terms of f_c', the cylinder crushing strength. ISO 3893 gives an approximate relationship between cylinder and cube strengths shown below.

CUBE STRENGTH f_{cu} (MPa) 150 mm x 150 mm	CYLINDER STRENGTH f_c' (MPa) 150 mm x 300 mm	f_c'/f_{cu}
20.0	16.0	0.80
25.0	20.0	0.80
30.0	25.0	0.833
35.0	30.0	0.857

This relationship is not universally accepted, but for the range of concrete strengths used in this study, a value of $f_c' = 0.833\ f_{cu}$ will be assumed.

Structural Steel

The characteristic strength of structural steel is given in various grades with different limiting criteria. For ease of comparison, this study is based on yield strengths with corresponding material factors.

Material Design Strengths

Taking all resistance and material factors into account, the material design strengths at the ultimate limit state are shown below.

TABLE 6.1 MATERIAL DESIGN STRENGTHS (APPLICABLE TO ELASTIC AND PLASTIC ANALYSIS)

DESIGN METHOD OR CODE	CONCRETE MPa	STEEL SECTION MPa	REINFORCEMENT MPa
BS 5950	*$0.45\ f_{cu}$	$1.0\ p_y$	$0.87\ f_y$
EC4	$0.472\ f_{cu}$	$0.91\ p_y$	$0.87\ f_y$
SABS 0162	$0.40\ f_{cu}$	$0.93\ p_y$	$0.87\ f_y$
CSA-S16.1	$0.425\ f_{cu}$	$0.9\ p_y$	$0.9\ f_y$
SIMPLE PLASTIC ANALYSIS	$0.67\ f_{cu}$	$1.0\ p_y$	$1.0\ f_y$

f_{cu} : characteristic 28 day cube strength (MPa)

p_y : specified minimum yield strength of structural steel (MPa)

f_y : characteristic strength of reinforcement (MPa)

* : 0,05 f_{cu} for elastic moment capacity with linear strain distribution

6.1 ULTIMATE STRENGTH OF FULLY COMPOSITE SECTIONS

The cross-section classification or combination of section element classification will determine whether the ultimate moment capacity is calculated by elastic or plastic methods.

6.1.1 SAGGING (POSITIVE) MOMENT REGIONS

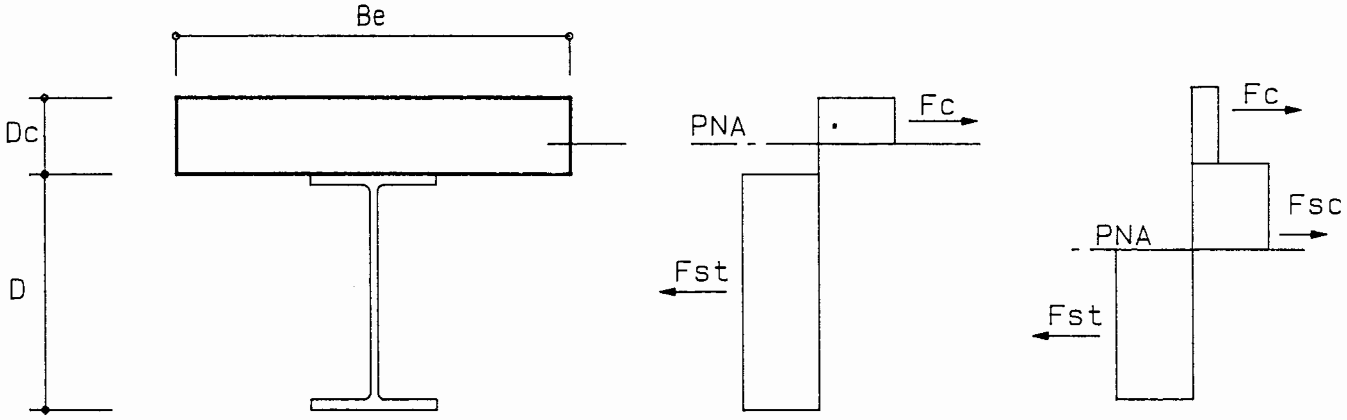
Plastic Ultimate Moment Capacity

The calculation of the plastic ultimate moment capacity of a composite section in sagging regions is a simple application of the rectangular-stress-block theory. The section strength depends on the compressive resistance of the effective concrete slab (the steel in compression normally neglected), and the yield strength of the steel section in tension or compression.

Assuming full shear connection between the concrete slab and steel section, three modes of failure are possible:

1) crushing of concrete,
2) yielding of the steel section and,
3) if the neutral axis lies low in the web, buckling of the web above this axis is possible.

Two typical stress block arrangements are shown, depending on the position of the plastic neutral axis (PNA).



PNA IN CONCRETE FLANGE

PNA IN STEEL SECTION

Fc = COMPRESSIVE FORCE IN CONCRETE FLANGE
Fsc = COMPRESSIVE FORCE IN STEEL SECTION
Fst = TENSILE FORCE IN STEEL SECTION

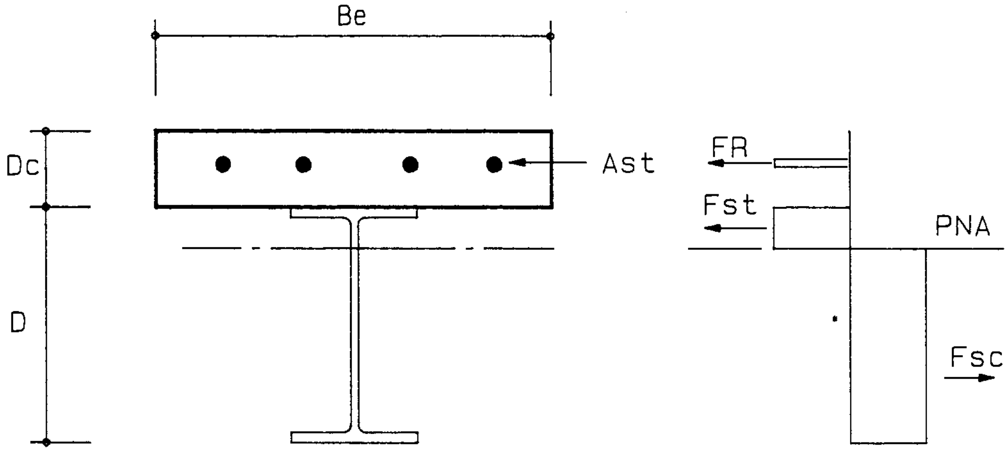
Elastic Ultimate Moment Capacity

Elastic section analysis is normally used when the requirements for plastic analysis are not complied with. The elastic moment capacity is taken as the largest moment that can be accommodated without exceeding the design strength (See Table 6.1) of the concrete slab or steel section, based on a linear strain distribution in the effective cross-section.

6.1.2 HOGGING (NEGATIVE) MOMENT REGIONS

Plastic Ultimate Moment Capacity

Provided the steel section satisfies the relevant Code requirements for section slenderness, the hogging moment capacity may be determined by plastic methods. The cross-section considered is shown below.



FR = TENSILE FORCE IN REINFORCEMENT
Fst = TENSILE FORCE IN STEEL SECTION
Fsc = COMPRESSIVE FORCE IN STEEL SECTION

The limits on section slenderness should ensure that the plastic moment capacity of the composite section is developed without premature local buckling of the steel beam.

Elastic Ultimate Moment Capacity

When required, the elastic moment capacity is taken as the largest moment that can be accommodated without exceeding the design strength (See Table 6.1) of the reinforcement or steel section, based on a linear strain distribution in the effective cross-section.

6.2 PRESENTATION OF DESIGN METHODS

6.2.1 SABS 0162

Sagging Regions

The full plastic sagging moment capacity may be utilised, provided that the PNA falls within the concrete flange or the steel section compression flange. This applies to all classes of steel section because it is considered that with the PNA in the concrete slab or steel compression flange, there is no risk of local buckling of the web.

Should the PNA fall in the web, the ultimate moment capacity must be found by assuming an elastic stress distribution across the section.

Consider a composite section comprising of a 305 x 165 x 54 kg/m UB and a 100 mm thick concrete slab. Assume f_{cu} = 25 MPa and p_y = 300 MPa.

With a slab effective width Be = 700 mm, the PNA lies within the steel flange and the ultimate plastic moment capacity = 324 kNm. Should Be now be reduced to 600 mm, the PNA falls in the steel web and elastic analysis must be used. The ultimate capacity by elastic analysis = 242.2 kNm. The corresponding plastic moment capacity = 317.7 kNm, representing a 29% increase.

The rule limiting the PNA to the steel or concrete flange was based on recommendations by Johnson and Hope-Gill (12). Later research by Ansourian (14) indicated that the full plastic moment capacity of a composite section may be utilised, provided the section is CLASS 2 compact and has an 'adequate' slab with a cross-section ductility ratio χ_u 1.4. (χ_u is defined as the ratio of the limiting neutral axis depth to the conventional neutral axis depth at ultimate strength).

The present rule is clearly conservative in restricting the position of the PNA to the compression flange for all classes of sections since the majority of sections in building structures will fall within the CLASS 2 compact category as defined by Ansourian (14).

Hogging Regions

The ultimate plastic moment capacity may be used provided the steel section compression flange and web slenderness comply with the limits given for CLASS 1 plastic sections. The limits for classifying a cross-section as CLASS 1, $d/t \leq 63.8\epsilon/(1+1.07r)$.

and $^{b}/_{tf} \le 8.7\epsilon$ are not conservative, but limiting the attainment of the plastic ultimate moment capacity to a CLASS 1 section is.

In addition to the slenderness requirement, the area of longitudinal reinforcement, Ast, within the effective width of the slab must comply with the following:-

a) $75\%\ A \le Ast \le 30\%\ A$

b) $0{,}3\%\ Ac \le Ast \le 1.5\%\ Ac$

Where A = Area of steel section

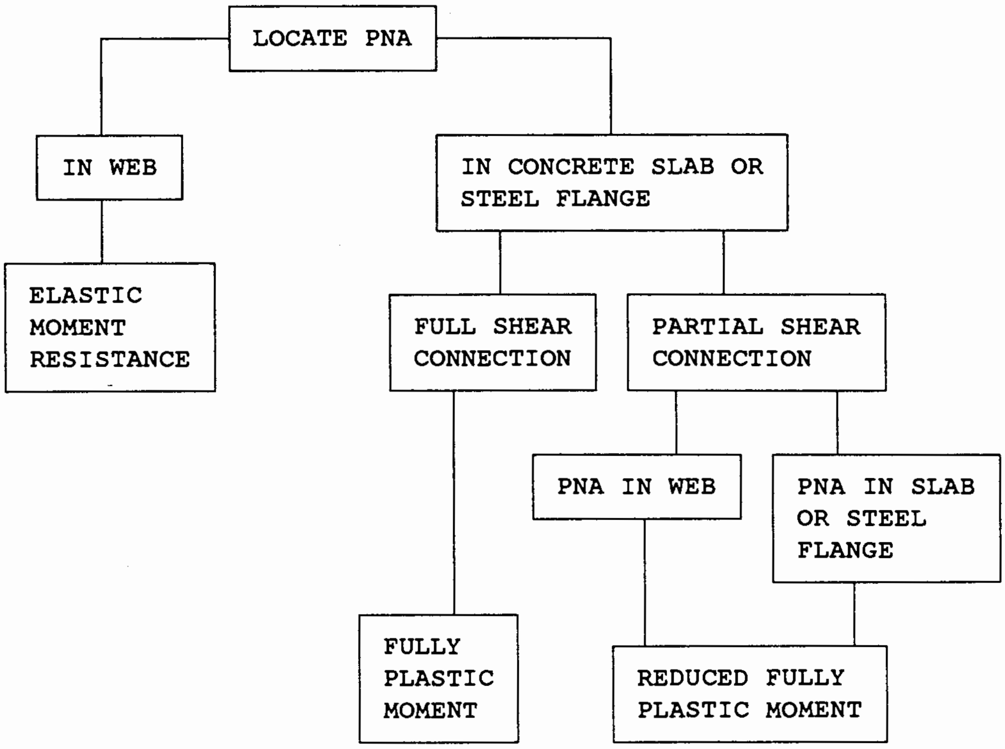
Ac = Area of concrete flange

The object of the above is to limit the crack width and to prevent premature buckling of the steel section. It has been assumed in the sections that follow that, if the above limits are not complied with, elastic section analysis must be used to calculate the ultimate moment capacity of the section.

It will be shown later that this method is very conservative due to the rules limiting the attainment of full plastic capacity to CLASS 1 compact sections and limits on area of reinforcement.

The design procedures for sagging and hogging regions are shown diagrammatically in Figure 6.1.

A. SAGGING REGIONS



B. HOGGING REGIONS

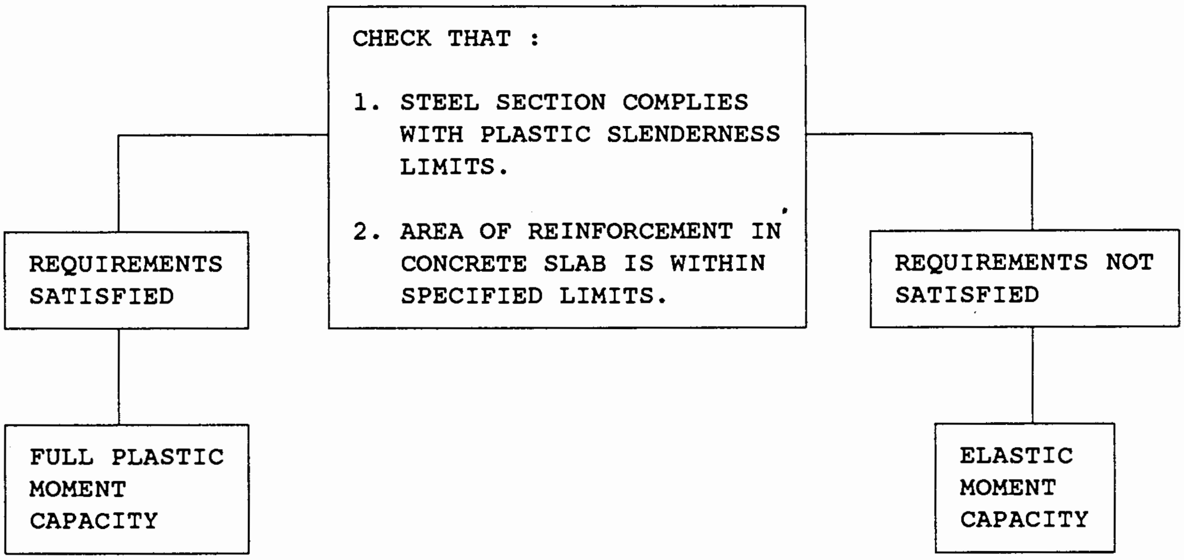


FIGURE : 6.1 - SABS 0162 - SCHEMATIC PRESENTATION OF DESIGN PROCEDURES FOR DETERMINING MOMENT CAPACITY IN SAGGING AND HOGGGING REGIONS

6.2.2 CSA-S16.1

The factored, or ultimate moment capacity, of composite beams in both sagging and hogging regions must be determined by plastic analysis. No allowance is made for elastic section analysis.

The recommendations for composite steel and concrete design is presented as a section of a general Code for the design of steel structures for buildings. No specific recommendations on section slenderness limits for composite design is provided and one must assume that since plastic section analysis is used, it is implied that the steel section is classed as CLASS 1 plastic or CLASS 2 compact as would be the case for plain steel sections.

In discussion with the supervisor on interpretation of the Code, it was agreed that the web section classification be based on "webs in flexural compression" and not "webs in combined flexure and axial compression" as given in Table 1 of CSA-S16.1. The web slenderness limits are then effectively independent of concrete flange proportions in sagging moment regions and tension reinforcement in hogging moment regions.

The general approach to section analysis is similar to the AISC and AASHTO methods where it has been traditional to design the sagging moment region on continuous beams as a composite section and the hogging moment region as a non-composite section (18).

In terms of the Code, reinforcement which extends parallel to the beam span and that is contained within the concrete slab effective width may be considered if sufficient shear connectors are provided. It is not mandatory to take the reinforcement into account and no limits on area of reinforcement are given.

Insufficient reinforcement over internal supports may result in excessive cracking, but more important, large areas of tension reinforcement may result in premature web instability due to induced compression in the web and compression flange. These topics have been well researched and documented (14, 16) resulting in recommendations which have been included in the other design Codes, but omitted here.

In sagging moment regions, no limit~~s~~ is placed on the position of the PNA. It was shown in Section 4.0 that the section slenderness limits specified by the Code are very unconservative and would be insufficient to preclude the combination of slender steel section and narrow effective flange width which may result in the PNA falling deep within the steel web resulting in possible web instability.

It could be argued that for the combination of rolled steel section, span and effective flange width utilised in normal building structures, the PNA will lie in the concrete or steel flange. The full plastic moment capacity should then be reached prior to section instability and the design rules may be considered satisfactory.

6.2.3 EUROCODE 4

Ultimate plastic section analysis may be used in both sagging and hogging regions on CLASS 1 plastic and CLASS 2 compact sections only. All other cross-sections must be analysed by elastic methods.

Sagging Region

In sagging regions, the compression steel flange attached to the concrete flange is considered CLASS 1 compact, irrespective of its actual slenderness. The degree of conservatism or otherwise of the method is then only dependent on the limit of web slenderness. The sagging plastic moment capacity of a typical section is 50 - 60% more than the elastic moment capacity, so the classification limit concept is an important criteria as far as the economy of the method is concerned.

No limit is placed on the position of the PNA, in keeping with Ansourian (14) who concluded that local buckling will occur well into the strain-hardening range when the flange and web are considered CLASS 2 compact.

For composite sections which comply with CLASS 3 semi-compact limits, the ultimate moment capacity is based on an elastic section analysis.

The ultimate moment capacity of CLASS 4 slender sections is determined by elastic methods as for CLASS 3 sections, except that for structural steel in compression, either:

a) the design yield stress is replaced by the limiting buckling stress as determined for plain steel sections, (clause 5.2.4 - EUROCODE 3); or

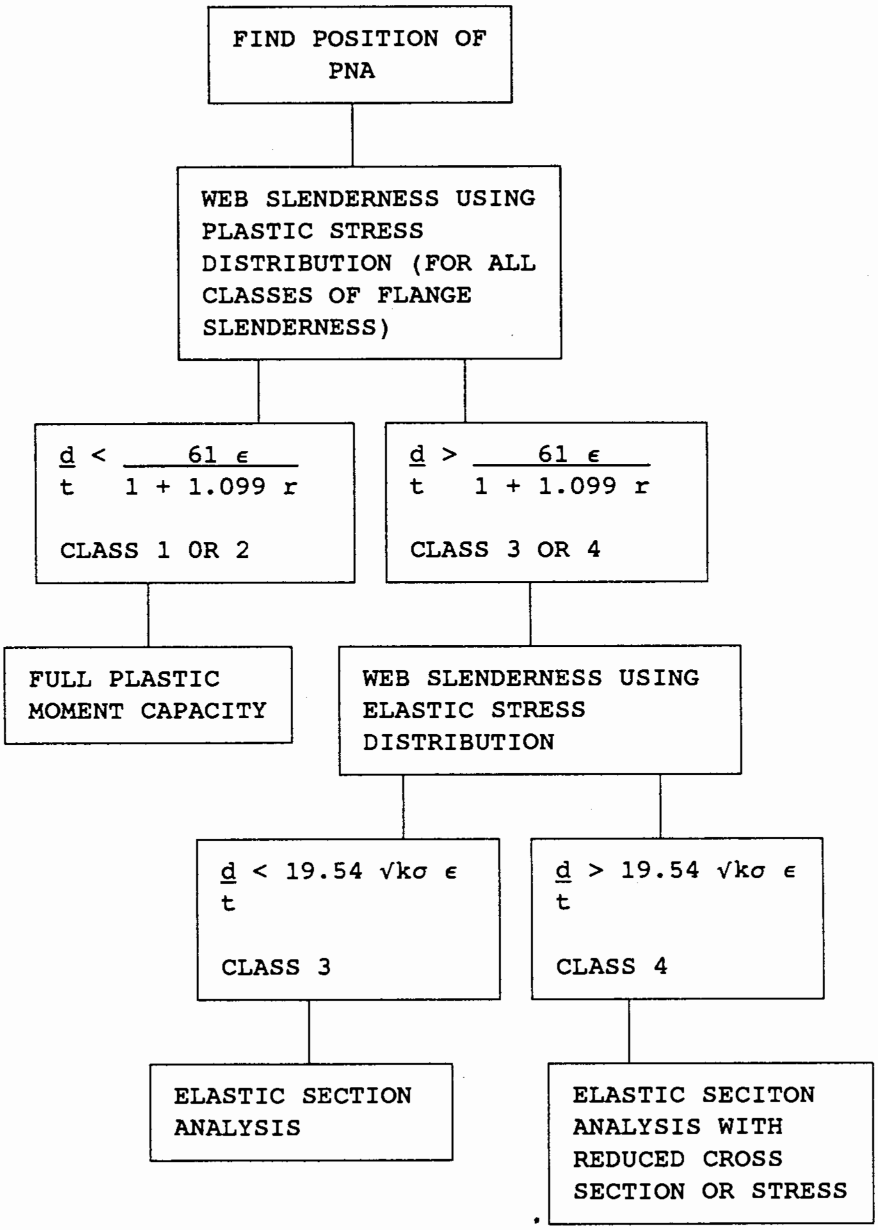


FIG: 6.2 EUROCODE 4 - SCHEMATIC PRESENTATION OF SECTION DESIGN METHODS AS DICTATED BY WEB AND FLANGE SLENDERNESS - SAGGING REGIONS

b) a reduced cross-section is used to account for the buckling (clause 5.2.6 - EUROCODE 3)

The design procedure as dictated by the section slenderness is shown in Figure 6.2.

Hogging Regions

Limiting the use of ultimate plastic moment capacity to CLASS 1 plastic and CLASS 2 compact sections is in keeping with the recommendations of Ansourian (14) but the slenderness limits for CLASS 2 compact sections,

$d/t < 61\epsilon/(1+1.099r)$ are shown to be conservative when the predicted moment capacity is compared to the test moment capacity.

Should it be found that the section slenderness falls outside the limits of CLASS 1 and 2, the elastic neutral axis (ENA) must be used to place the section in CLASS 3 semi-compact or CLASS 4 slender. The ultimate moment of resistance is then determined by elastic analysis as described above.

The web slenderness limit for CLASS 3 sections is given as

$d/t \leq 18.06 \sqrt{K\sigma}\, \epsilon$ where $K\sigma$ is the buckling value obtained by the linear buckling theory for the actual stress distribution.

The design procedure is shown in Figure 6.3.

6.2.4 BS 5950 (F)

In this Code, four methods of cross-section analysis are utilised, depending on the combination of the steel section compression flange and web classification.

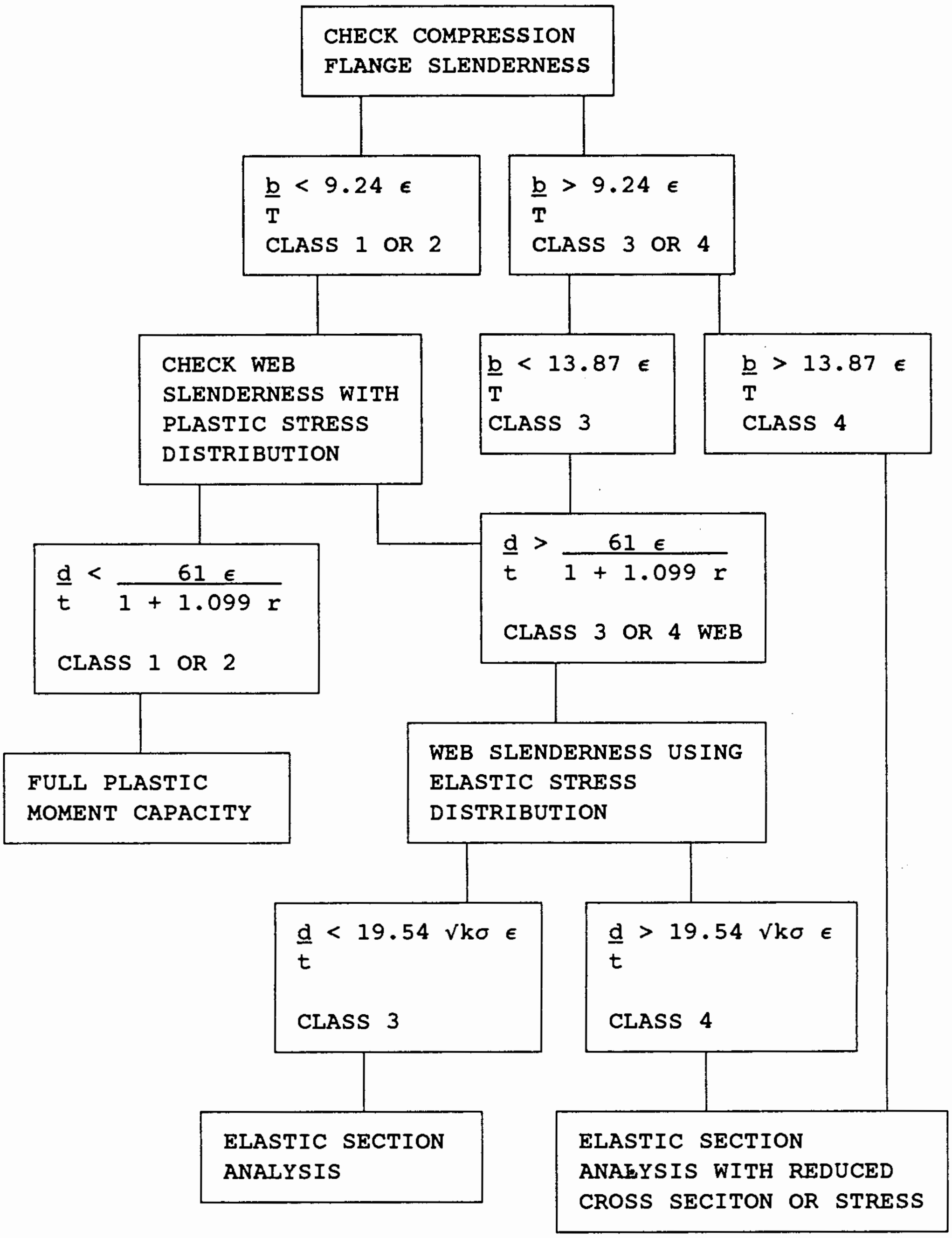


FIG: 6.3 EUROCODE 4 - SCHEMATIC PRESENTATION OF SECTION DESIGN METHODS AS DICTATED BY WEB AND FLANGE SLENDERNESS - HOGGING REGIONS

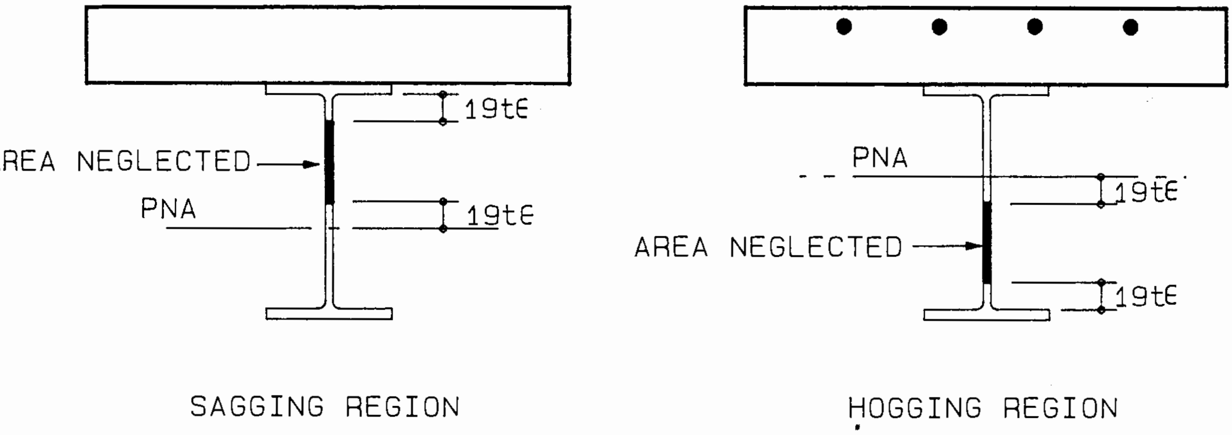
6.2.4.1 Design Methods

Full Ultimate Plastic Moment Capacity

Where full ultimate plastic section analysis is allowed, it is calculated by the normal rectangular stress-block method, using the effective width of the concrete flange and assuming the compressive stress in the concrete to be 0.45 f_{cu} (neglected in tension) and the tensile or compressive stress in the structural steel and reinforcement to be the design yield strength p_y and 0,87 f_y respectively.

Reduced Plastic Ultimate Moment Capacity

When required, the reduced plastic moment capacity is determined using the effective section shown below.



EFFECTIVE SECTION FOR REDUCED
PLASTIC MOMENT CAPACITY

The method is used when the web is classed as CLASS 3 semi-compact. Should the PNA lie deep in the web, buckling of the web on the compression side of the PNA is possible. This portion of web is considered not to contribute to the moment of resistance of the section. The position of the revised PNA is located, based on an effective depth of web limited to $19t\epsilon$ adjacent to the compression flange and $19t\epsilon$ adjacent to the PNA. This is an important innovation resulting in a more efficient utilisation of a section which would otherwise be analysed by elastic methods.

Elastic Ultimate Moment Capacity

The limiting slenderness for composite sections analysed elastically are those beyond which the yield stress cannot be reached before buckling occurs in the steel section. A linear strain distribution across the section is assumed with the following limiting stresses:

concrete : 0,5 f_{cu} - with a linear stress distribution or

: 0,45 f_{cu} - with a parabolic rectangular stress distribution

steel : p_y

reinforcement : 0,87 f_y

Reduced Elastic Moment Capacity

Where the section slenderness exceeds the limits described above, the ultimate moment capacity is determined as above, but the steel stress Py is reduced as recommended in BS 5950 : Part 1 for bare steel sections.

6.2.4.2 Selection of Design Method

Hogging Region

It is stated that when the compression flange is CLASS 1 plastic or CLASS 2 compact, the plastic moment capacity should be used, provided that the web is not CLASS 4 slender. One must assume then that when the flange is CLASS 1 or 2 and the web CLASS 4, elastic analysis must be used without reducing the allowable stress.

Where the compression flange is CLASS 3 semi-compact, the elastic moment capacity should be used with the normal allowable stress provided the web is not more slender than CLASS 3 semi-compact.

When the flange is CLASS 3 semi-compact and the web CLASS 4 slender or the flange CLASS 4 slender irrespective of the web slenderness, elastic moment capacity should be reduced as recommended in BS 5950: Part 1.

The combinations of flange and web slenderness and allowed method of section analysis described above is summarised in Table 6.2 and shown schematically in Figure 6.4.

TABLE 6.2 METHOD OF SECTION ANALYSIS CORRESPONDING TO COMBINATION OF WEB AND FLANGE CLASSIFICATION

FLANGE CLASSIFICATION	WEB CLASSIFICATION			
	CLASS 1 PLASTIC	CLASS 2 COMPACT	CLASS 3 SEMI-COMP.	CLASS 4 SLENDER
CLASS 1	P	P	P_{RED}	E
CLASS 2	P	P	P_{RED}	E
CLASS 3	E	E	E	E_{RED}
CLASS 4	E_{RED}	E_{RED}	E_{RED}	E_{RED}

P - PLASTIC MOMENT CAPACITY
E - ELASTIC MOMENT CAPACITY
P_{RED} - REDUCED PLASTIC MOMENT CAPACITY
E_{RED} - REDUCED ELASTIC MOMENT CAPACITY

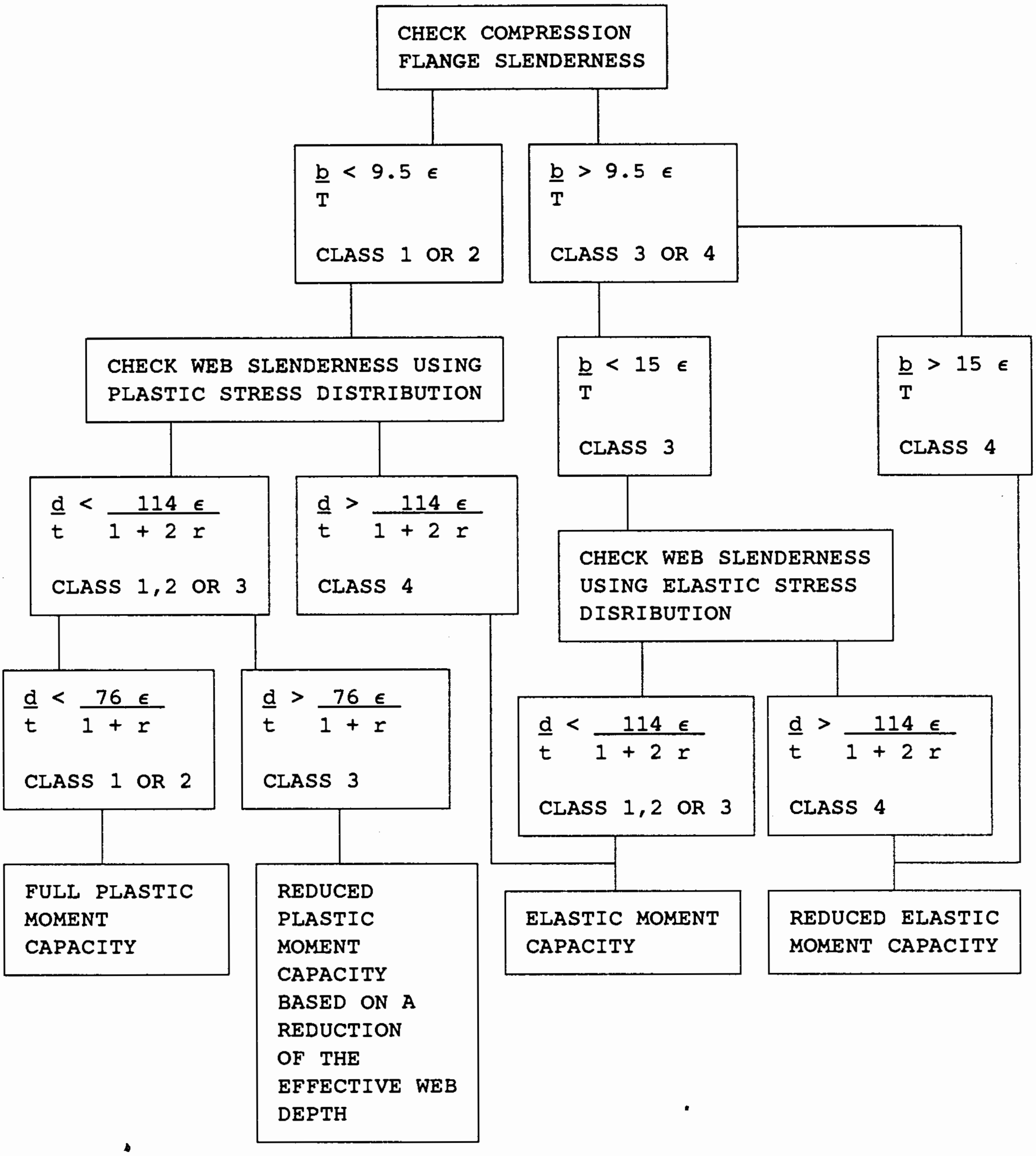


FIG: 6.4 A SCHEMATIC PRESENTATION OF HOGGING MOMENT SECTION DESIGN METHODS WITH VARIOUS COMBINATIONS OF WEB AND FLANGE SLENDERNESS AS REQUIRED BY BS 5950 (F).

Sagging Region

The allowed methods of section analysis corresponding to combinations of web and flange slenderness is as for hogging regions given above. Where the compression steel flange is effectively attached to the concrete slab, the steel flange may be assumed to comply with CLASS 1 plastic. This will be the general case.

Simple plastic analysis is used for CLASS 1 plastic and CLASS 2 compact webs and a reduced effective web depth is used for CLASS 3 semi-compact webs.

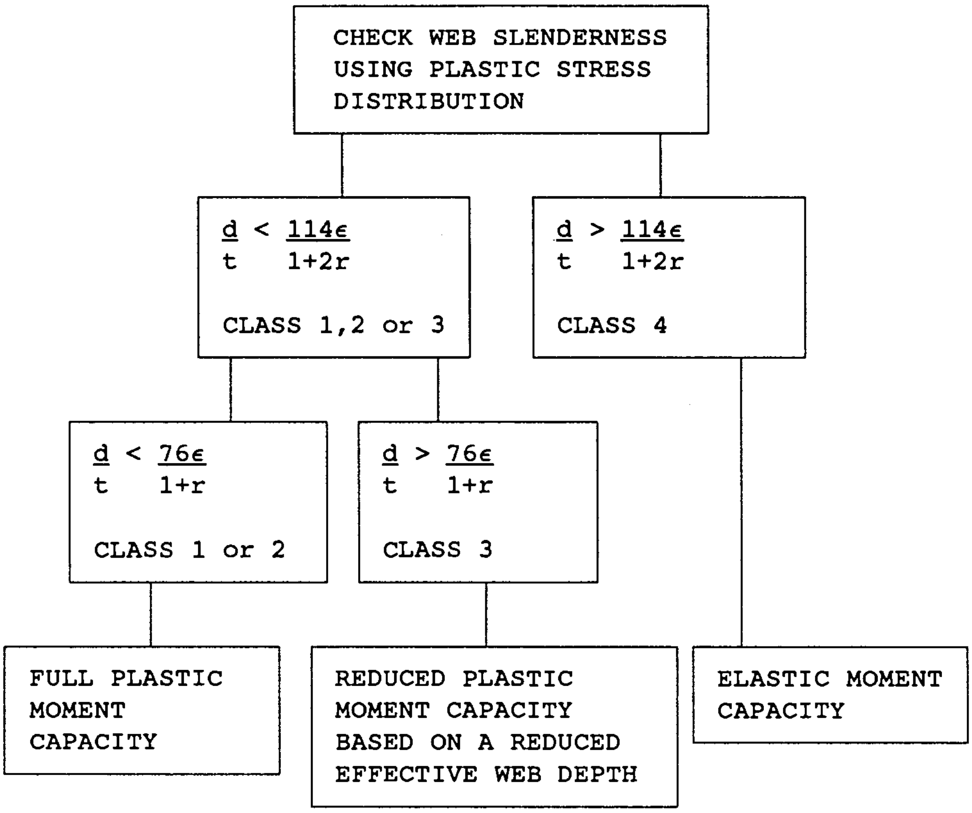
When a web is classed as CLASS 4 slender, the elastic moment capacity must be used. The web slenderness limit for CLASS 3 semi-compact sections however is given by $\frac{d}{t} \leq \frac{114\epsilon\ (1+r)}{(1 + 2r)\ ^3/_2}$

where $r = - Fc/Rv$ $(r \geq - 1)$

Fc = Compressive force in the concrete flange

Rv = the resistance of the clear web depth $dt.p_y$

For the equation to have a real value, the denominator 1 + 2r must have a positive value which implies that $r > - 0,5$. It is unclear whether the denominator should read $(1 + 2r)^{3/2}$ or $(1 + 2r)\ ^3/_2$. The latter gives a more realistic result and will be used for this exercise. As r tends to zero, the limit of $^d/_t$ tends to 72.76. For a realistic value of $r = -0.10$, $^d/_t = 81.9$. This represents a very slender member and assuming a web thickness t = 15 mm, d = 1228,5 mm.



(THE STEEL COMPRESSION FLANGE IS ASSUMED RESTRAINED BY THE CONCRETE FLANGE AND HENCE COMPLIES WITH CLASS 1 PLASTIC)

FIGURE 6.5 A SCHEMATIC PRESENTATION OF SAGGING MOMENT SECTION DESIGN METHODS WITH VARIOUS COMBINATIONS OF WEB AND FLANGE SLENDERNESS AS REQUIRED BY BS 5950 (F)

For f_{cu} = 25 MPa, p_y = 300 MPa, the corresponding effective slab would be 490 wide x 100 mm thick. A composite, section of these proportions would be unusual in building structures.

In the vast majority of cases, therefore, the ultimate moment capacity in sagging regions, would be found by plastic section analysis.

The flow chart for the selection of design method dictated by section slenderness is shown in Figure 6.5.

6.2.5 COMPARISON OF BS 5950 (F) and BS 5950 (D) METHODS

The predicted ultimate moment capacities of composite beams based on limits and methods proposed in the draft version of BS 5950 compare very well with test results. Revisions adopted in the final version yield considerably more conservative results in certain situations. In view of this, it was deemed necessary to highlight the main changes.

From Table 6.2 above, for CLASS 1 and 2 compression flanges, when the web is more slender than CLASS 3 semi-compact, the ultimate moment capacity is found by elastic analysis to comply with BS 5950 (F). The original requirement in BS 5950 (D) was that provided the compression flange was CLASS 1 or 2, the ultimate moment capacity was based on the reduced plastic moment capacity for ALL classes of web more slender than CLASS 2 compact. This, coupled with the more stringent slenderness limits given by BS 5950 (F) make it considerably more conservative than the draft version.

In Figure 6.6 a typical plot of ultimate hogging moment capacity v/s area of tension reinforcement predicted by both versions of BS 5950 is shown.

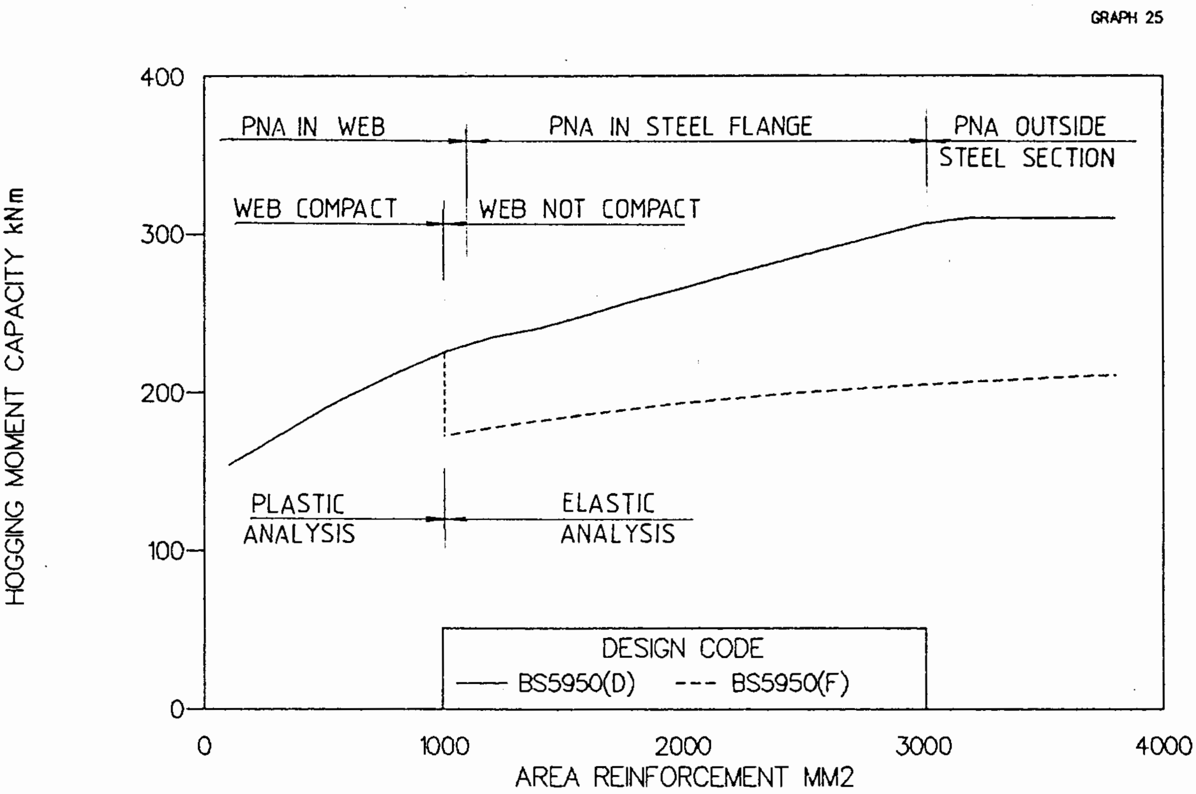


FIGURE: 6.6 TYPICAL MOMENT V/S Ast CURVES FOR BS5950 (D) & (F)

6.5 COMPARISON OF PREDICTED ULTIMATE MOMENT CAPACITIES

The aim of this section is:

1. To compare the performance of the Codes in predicting the ultimate moment capacity of a composite section with the effective slab width as a variable in sagging moment regions and the area of tension reinforcement as a variable in hogging moment regions, and
2. To compare the accuracy of the Codes in predicting the ultimate moment capacity of a composite section in sagging or hogging bending. This is achieved by comparing the moment at failure obtained from beams tested under laboratory conditions, described in Appendix B,

to the ultimate moment capacity of the section, predicted by each of the Codes using the actual material strengths and section properties.

In both cases, the predicted ultimate moment capacity was calculated with the aid of programmes developed for a Hewlett Packard 41CV programmable calculator described in Appendix A.

6.5.1 SAGGING MOMENT REGIONS

The methods of section analysis recommended by the Codes corresponding to the steel web classification is summarised in the table below.

TABLE 6.3

DESIGN CODE	WEB CLASSIFICATION				
	CLASS 1	CLASS 2	CLASS 3	CLASS 4	
BS 5950	P	P	P_{RED}	E	
EC 4	P	P	E	E_{RED}	
CSA-S16.1	P	P	-	-	
SABS 0162	P/E	P/E	P/E	P/E	1)

P = PLASTIC MOMENT CAPACITY P_{RED} = REDUCED PLASTIC MOMENT CAPACITY

E = ELASTIC MOMENT CAPACITY E_{RED} = REDUCED ELASTIC MOMENT CAPACITY

1) Plastic analysis may be used provided the PNA falls within the concrete slab or steel compression flange otherwise elastic analysis is used.

For the common range of beam and slab proportions used in buildings, the steel section will be classed as CLASS 2 compact and the PNA will fall within the concrete slab or steel compression flange. This is demonstrated graphically in Figures 6.7 and 6.8.

Two sections have been analysed; a compact 203 x 133 x 30 kg/m I section (Figure 6.7) and a more slender 305 x 102 x 33 kg/m I section (Figure 6.8). The material properties are constant and the slab thickness fixed at 100 mm and 130 mm respectively.

For a range of effective flange widths, the ultimate moment capacity predicted by each design Code has been calculated. The predicted ultimate moment capacities compare very well and differ in magnitude due only to the different material factors. The step in the SABS 0162 curves is due to the change from elastic to plastic analysis as the neutral axis moves from the web to the steel compression flange. The approximate position of the PNA within the section corresponding to a range of effective flange widths is indicated on the graphs.

From Figure 6.8, for a slab Be = 1800 mm, the ultimate moment capacity predicted by each Code, normalised with respect to the simple plastic moment capacity (Mp) is given below. These figures are representative of the relative magnitude of moment capacities predicted by the Codes.

DESIGN CODE	BS 5950	SABS 0162	EUROCODE 4	CSA-S16.1
M/Mp	0,962	0,890	0,880	0,870

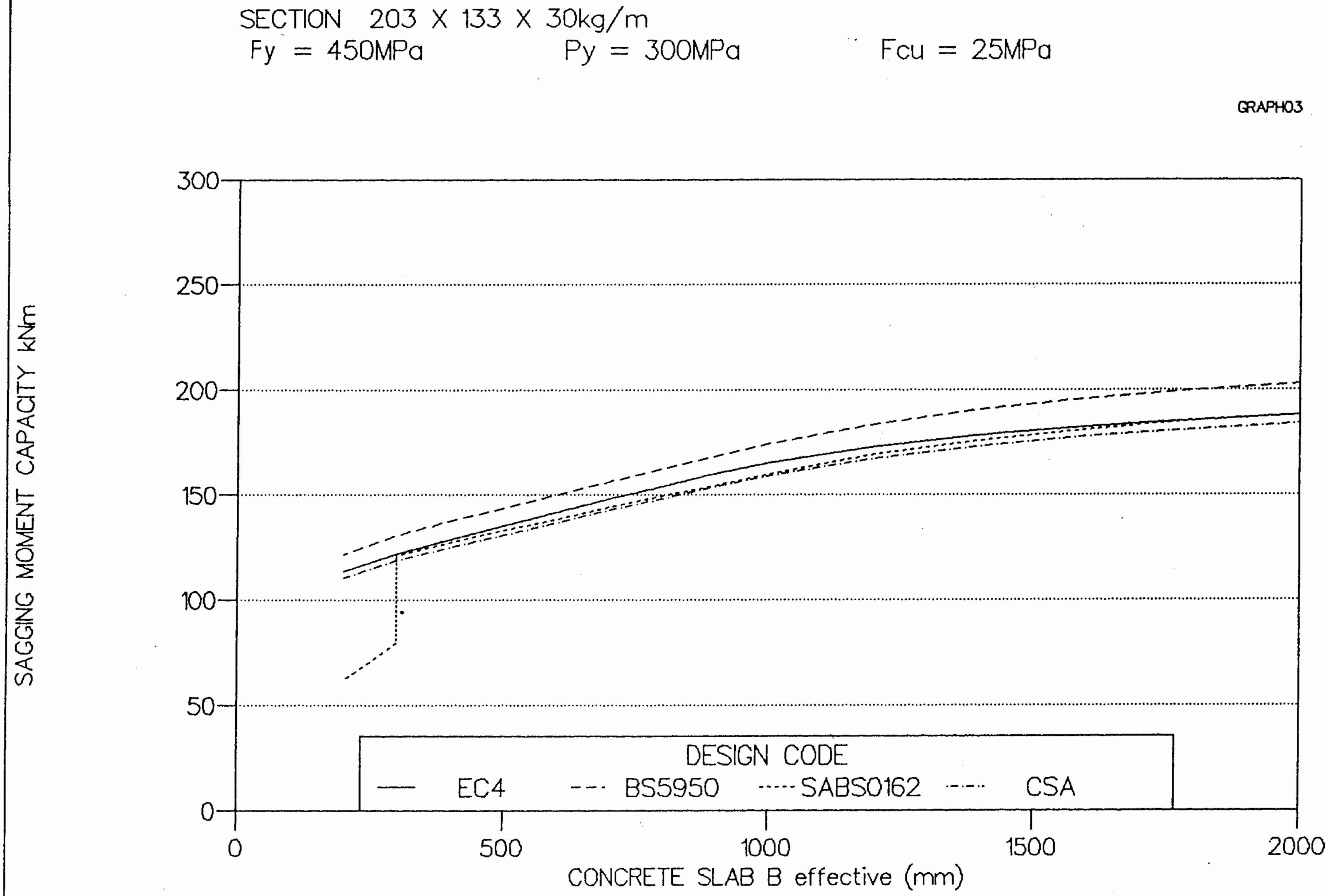


FIGURE: 6.7 COMPARISON OF PREDICTED SAGGING MOMENT CAPACITY

SECTION 305 X 102 X 33kg/m
Fy = 450MPa Py = 300MPa Fcu = 25MPa

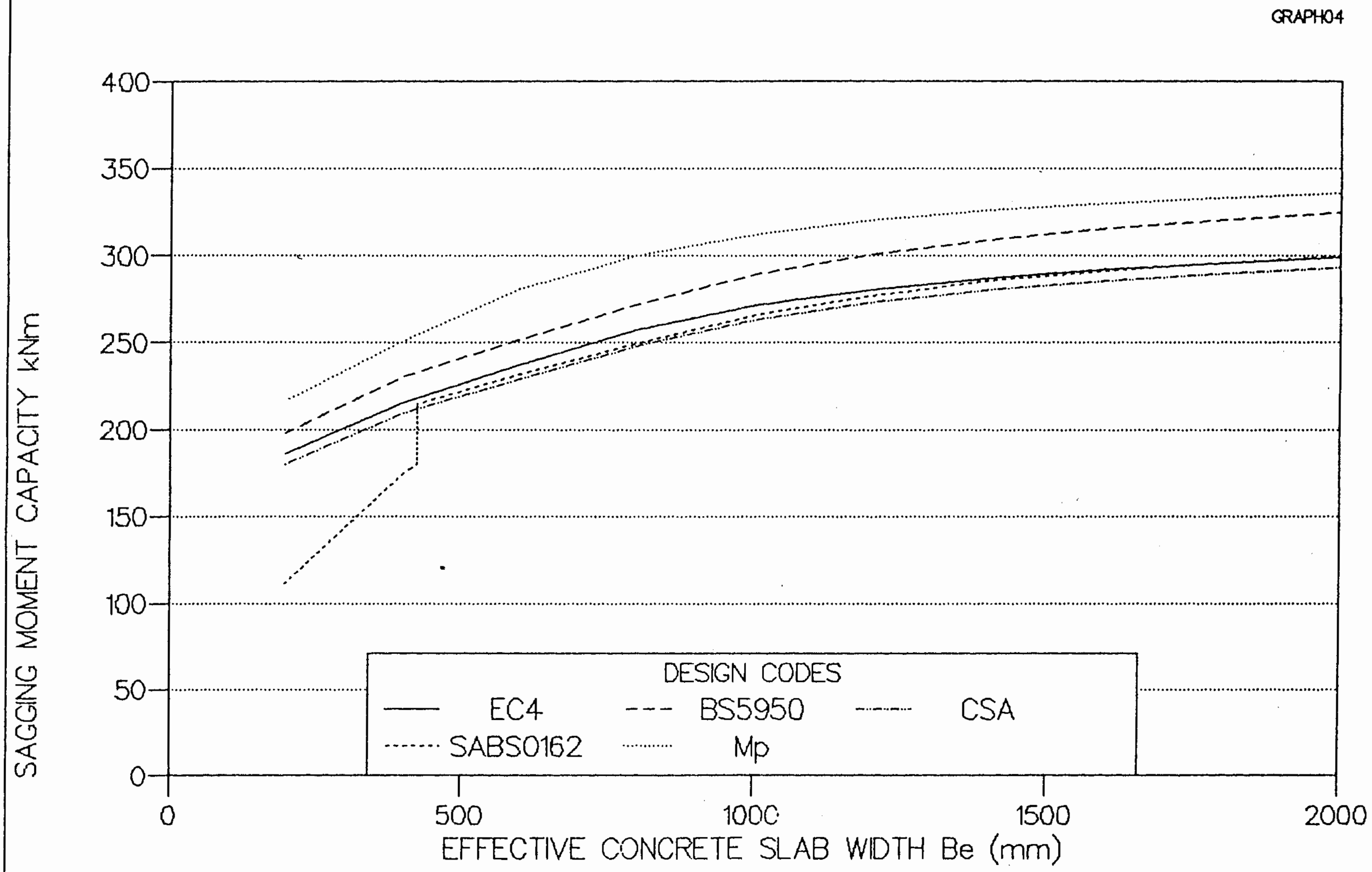


FIGURE: 6.8 COMPARISON OF PREDICTED SAGGING MOMENT CAPACITY

Test Results

Composite beams in sagging bending have been tested to failure by Hope-Gill & Johnson (13) and Ansourian (14). The testing procedure and relevant section and material details of specimens tested are described in Appendix B.

Using actual material stresses and section properties, the predicted ultimate sagging moment capacity of each beam was calculated using the different design Code methods.

Tests results and predicted ultimate moment capacities are tabulated below. The maximum moments achieved in some of the tests are significantly greater than the predicted ultimate moments, due mainly to the effect of strain-hardening of the steel section which is dependent on the beam geometry and material properties. It cannot, however, be relied upon. (11)

TABLE 6.4 COMPARISON OF TEST AND PREDICTED SAGGING MOMENT CAPACITIES (kNm)

REF	SPECIMEN	M TEST	MP	ULTIMATE MOMENT CAPACITY PREDICTED BY:			
				BS 5950	SABS 0162	EUROCODE 4	CSA
14	CTB 1	166	138.5	129.0	118.8	120.8	117.4
14	CTB 2	184	146.3	147.2	136.5	135.1	133.0
14	CTB 3	250	212	203.0	187.6	187.9	183.8
14	CTB 4	217	203	181.8	166.5	173.2	166.4
14	CTB 5	232	206.7	196.3	181.2	182.3	178.0
14	CTB 6	254	232.9	224.0	207.2	206.9	202.8
13	CB 10	200	162.5	151.9	139.7	142.6	138.2
13	CB 11	185	147.5	138.8	128.0	130.0	126.1
13	CB 12	196	189.5	183.1	169.2	169.5	165.9

TABLE 6.5 RATIO OF PREDICTED MOMENT NORMALISED WRT MTEST

REF	SPECIMEN	M TEST	MP	BS 5950	SABS 0162	EUROCODE 4	CSA
14	CTB 1	1.00	0.83	0.78	0.72	0.73	0.71
14	CTB 2	1.00	0.80	0.80	0.74	0.73	0.72
14	CTB 3	1.00	0.85	0.81	0.75	0.75	0.74
14	CTB 4	1.00	0.94	0.84	0.77	0.80	0.77
14	CTB 5	1.00	0.89	0.85	0.78	0.79	0.77
14	CTB 6	1.00	0.92	0.88	0.82	0.81	0.80
13	CB 10	1.00	0.81	0.76	0.70	0.71	0.69
13	CB 11	1.00	0.80	0.75	0.69	0.70	0.68
13	CB 12	1.00	0.97	0.93	0.86	0.86	0.85
MEAN			0.866	0.822	0.758	0.766	0.746
STD. DEV			0.06	0.056	0.053	0.051	0.051
MEAN + STD. DEV.			0.926	0.878	0.811	0.816	0.797

6.5.2 HOGGING REGIONS

The methods of section analysis allowed by each Code is summarised below.

TABLE 6.6

DESIGN CODE	METHOD OF ANALYSIS			
	PLASTIC	REDUCED PLASTIC	ELASTIC	REDUCED ELASTIC
BS 5950 (D)	■	■	■	–
BS 5950 (F)	■	■	■	■
EUROCODE 4	■	–	■	■
SABS 0162	■	–	■	–
CSA-S16.1	■	–	–	–

Should the same method of analysis (e.g. plastic analysis) be employed by each Code to calculate the ultimate moment capacity of a section, the values obtained would differ only as a result of the material factors. What sets the Codes apart, are the conditions under which a particular method of analysis becomes applicable. The main deciding factor is the member

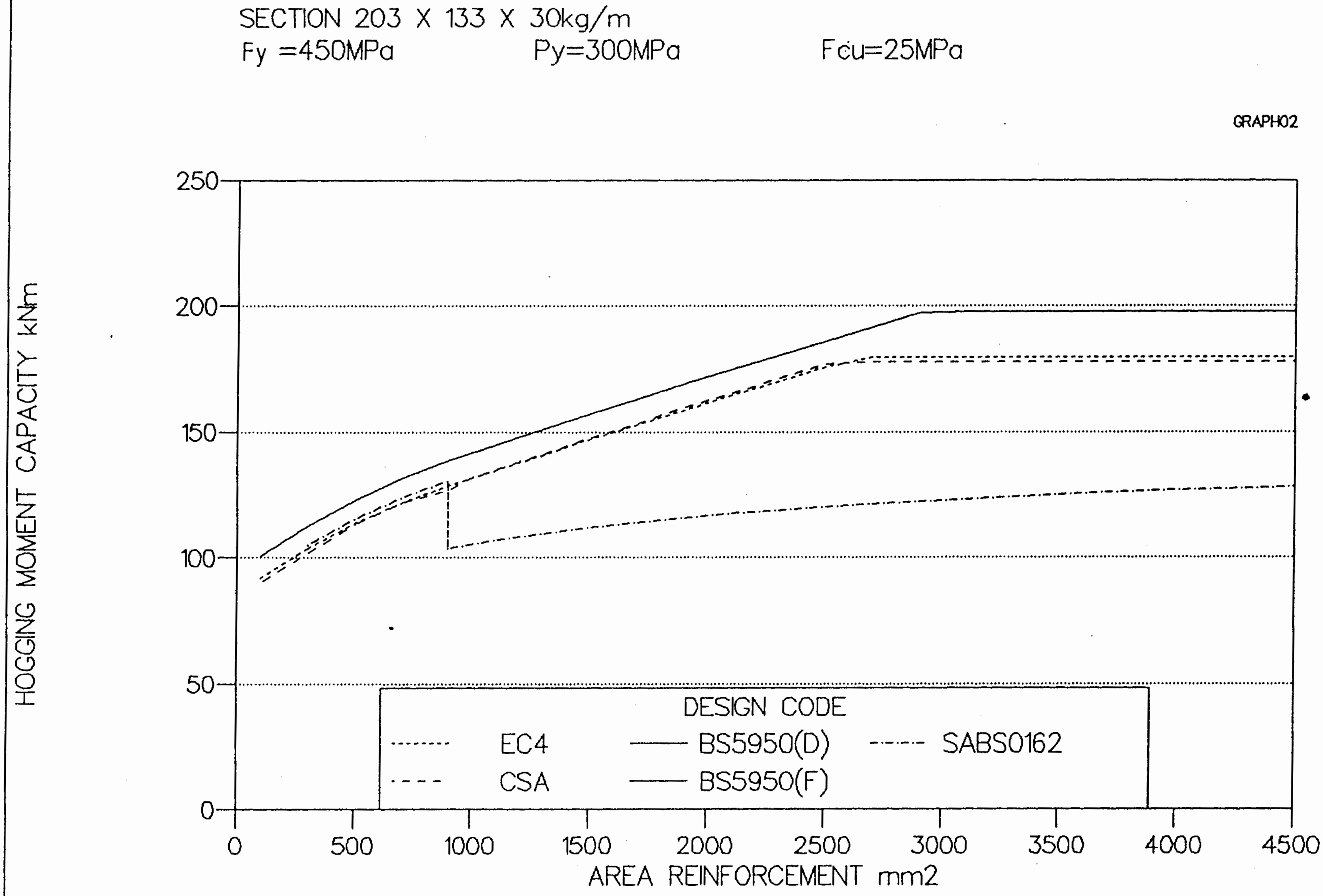


FIGURE: 6.9 COMPARISON OF PREDICTED HOGGING MOMENT CAPACITY

cross-section classification or the combination of element classifications (i.e. web and flange).

Three typical composite sections are analysed below to demonstrate some situations which may occur in practice and to compare the relative performance of each Code in these situations. In each composite section, the only variable is the area of longitudinal reinforcement in the effective concrete flange width.

The results are shown graphically in Figures 6.9, 6.10 & 6.11.

Comment on Graphs

FIGURE 6.9

SECTION : 203 x 133 x 30 kg/m ; p_y = 300 MPa

CONCRETE SLAB: 1000 x 100 mm f_{cu} = 25 MPa

REINFORCEMENT: Ast variable f_y = 450 MPa

BS 5950, EUROCODE 4, CSA-S16.1	Plastic analysis may be employed throughout the range of area of reinforcement (Ast). The difference in predicted values is due to the variation in material factors.
SABS 0162	Plastic analysis may be used up to Ast = 900 mm², where the allowable slenderness limit exceeds the section slenderness. At this stage, the moment predicted by BS 5950 is 34% more and increases to 58% more than the value predicted by SABS 0162 for the same Ast.

SECTION 305 X 102 X 33kg/m
Fy =450MPa Py=300MPa Fcu=25MPa

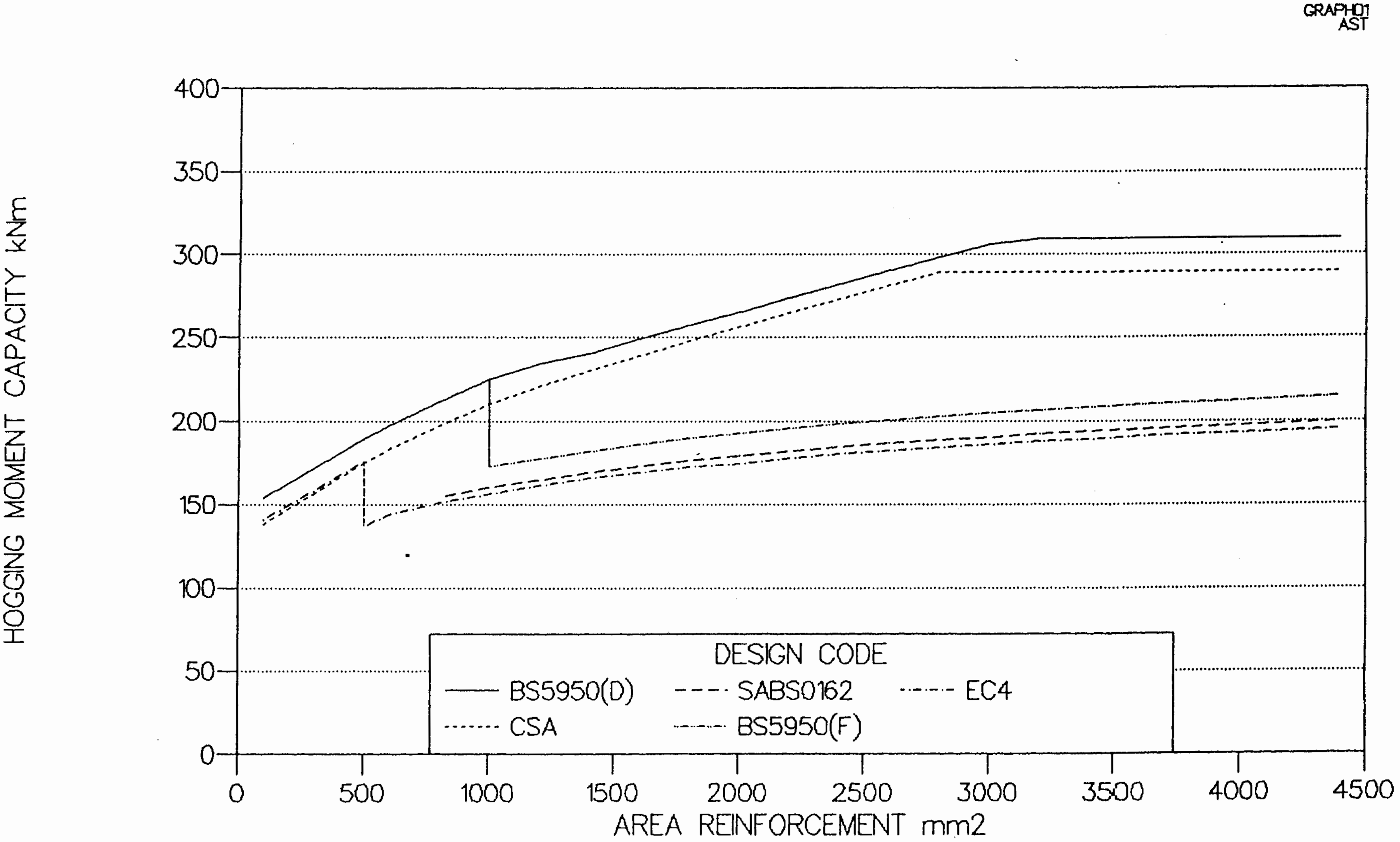


FIGURE: 6.10 COMPARISON OF PREDICTED HOGGING MOMENT CAPACITY

FIGURE 6.10

SECTION : 305 x 102 x 33 kg/m ; p_y = 300 MPa

CONCRETE SLAB: 1800 x 130 f_{cu} = 30 MPa

REINFORCEMENT: Ast variable f_y = 450 MPa

BS 5950 (D)	Plastic analysis may be used throughout the range of Ast, but for Ast > 1160 mm², the effective web depth is reduced due to section slenderness.
BS 5950 (F)	Plastic analysis may be used up to the point where d/t > 75 $\epsilon/(1+r)$, corresponding to Ast = 1103 mm². Elastic analysis must now be employed resulting in a significantly reduced moment capacity compared to BS 5950 (D).
CSA-S16.1	Plastic analysis is used throughout, yielding a range of moment capacities more conservative than BS 5950 (D).
EUROCODE 4	Plastic analysis is used for a relatively small amount of tension reinforcement (˜500 mm²) at which point the web is classed as CLASS 2 semi-compact and elastic analysis is introduced. In this case, EC 4 is the most conservative of the Codes. Note that at Ast = 2100 mm², the elastic ultimate moment capacity is equal to the plastic moment capacity with Ast = 500 mm², and increasing.
SABS 0162	The minimum amount of tension reinforcement allowed is 0,3% x effective concrete flange area. At this point the section slenderness exceeds plastic requirements and elastic analysis is used throughout.

SECTION 406 X 140 X 39kg/m
Fy = 450MPa Py = 300MPa Fcu = 25MPa

GRAPH05

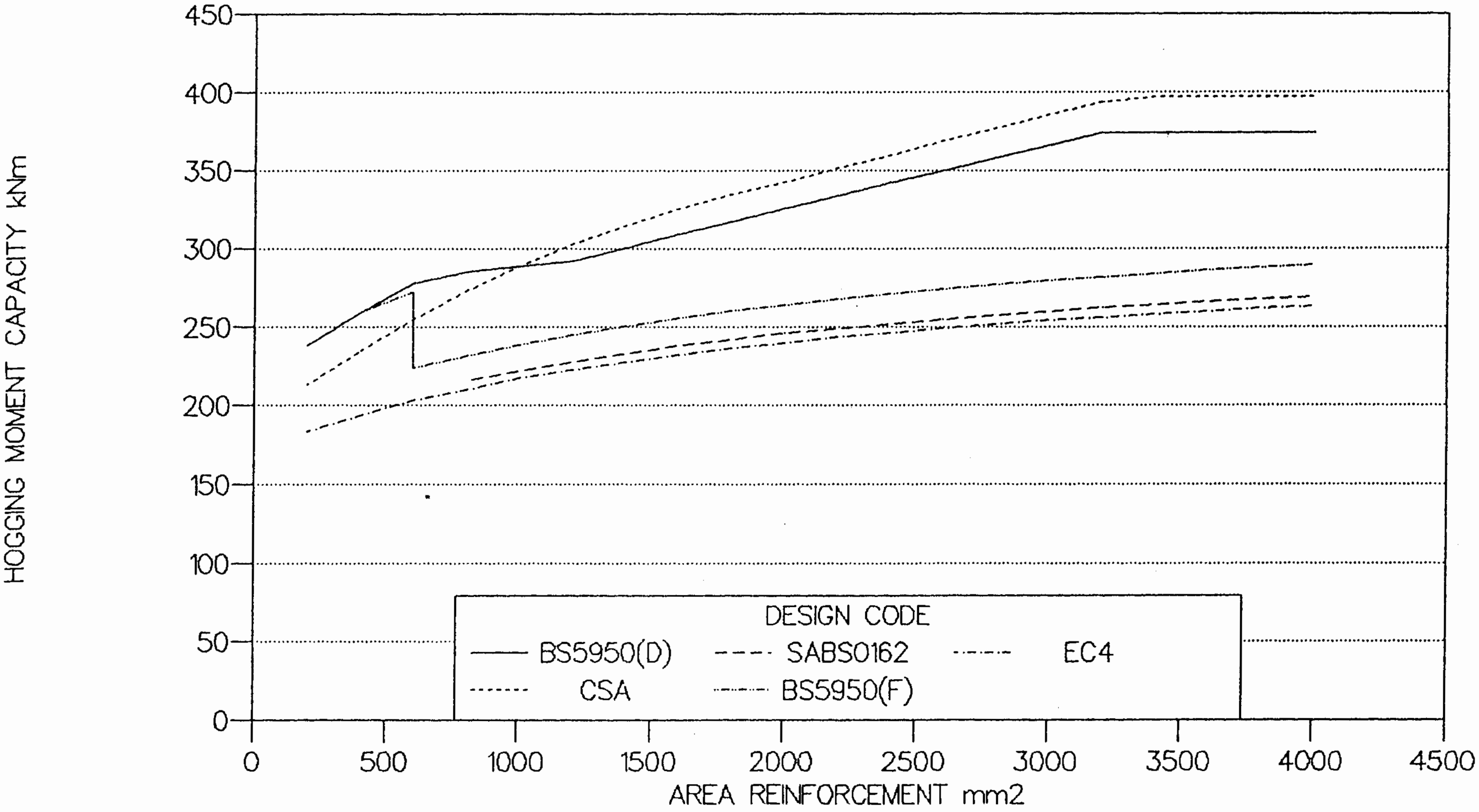


FIGURE: 6.11 COMPARISON OF PREDICTED HOGGING MOMENT CAPACITY

FIGURE 6.11

SECTION : 406 x 140 x 39 kg/ m p_y = 300 MPa

CONCRETE SLAB: 1800 x 130 f_{cu} = 30 MPa

REINFORCEMENT: Ast variable f_{st} = 450 MPa

BS 5950 (D)	Plastic analysis may be used throughout with a reduction in effective web depth at Ast > 592 mm^2. Note that the moment capacity is initially greater than that predicted by CSA-S16.1, but as Ast increases, the value predicted by BS 5950 (D) becomes less than the CSA-S16.1 value.
BS 5950 (F)	Full plastic analysis may be utilised up to Ast = 475 mm^2. At this point plastic analysis with a reduced web depth is introduced, but only up to Ast = 791 mm^2 where the section is classed as slender and elastic analysis is used.
CSA-S16.1	This method is the least conservative in this case and plastic analysis is used throughout.
EUROCODE 4	For Ast > 35 mm^2, the section is classed as CLASS 3 semi-compact requiring elastic analysis throughout.
SABS 0162	To comply with the minimum slab area requirement, Ast min = 700 mm^2. The section slenderness exceeds plastic requirements and elastic analysis is used throughout.

Comparison of Test Results

It is clear from the above that unlike the sagging moment region, there is little agreement on the predicted ultimate moment capacity in the hogging moment region. The less compact the section, the greater the scatter of predictions.

The results of composite beams, tested to failure in hogging bending during five independent investigations, will be compared to predicted ultimate moment capacities calculated using actual material stresses and section properties. The test procedure and relevant material and section properties are described in Appendix B. Thirty three of the beams tested were considered suitable for inclusion in this report. Results tabulated in tables 6.7 (a) and (b) are divided in two groups; GROUP 1 represents specimens in which the moment at failure exceeds the simple plastic moment capacity of the section, and GROUP 2 represents specimens which failed by local instability before the bending moment reached the plastic moment capacity.

Table 6.8 (a) and (b) show the analytical moment capacities normalised with respect to the test moment capacities. An examination of the mean and standard deviation of these values indicate that BS 5950 (D) is the most accurate in predicting the ultimate moment capacity, followed by BS 5950 (F). A measure of the consistency of the Codes in predicting the ultimate moment capacity of the full range of sections, i.e. groups 1 and 2 combined, is given by the percentage difference between the mean values obtained in tables 6.8 (a) and (b). Results shown below indicate that BS 5950 (F) is the most consistent.

	BS 5950 (D)	SABS 0162	EURO CODE 4	CSA	BS 5950 (F)
% DIFFERENCE BETWEEN MEAN VALUES OF TABLES 6.8 (a) & (b)	18.8	20.7	17.9	36.3	15.2

TABLE 6.7(a) TEST AND ANALYTICAL RESULTS - HOGGING BENDING (kNm)
GROUP 1 - TEST MOMENT > PLASTIC MOMENT CAPACITY

BEAM REF	M TEST	MP	BS 5950 (D)	SABS 0162	EURO CODE 4	CSA	BS 5950 (F)
SB3	140	120	118	89	108	108	118
HB40	154	139	135	100	125	125	135
SB2	118	108	107	82	80	98	107
HB41	121	116	113	88	86	104	113
CB10	150	127	124	89	115	115	124
CB11	155	132	129	121	112	119	118
CB12	179	138	133	95	124	124	133
18	328	232	232	216	211	209	232
11	375	282	277	258	255	254	277
12	389	304	298	279	217	274	298
13	393	314	308	227	222	283	308
14	401	334	326	236	231	300	326
21	288	205	205	191	186	184	205
22	323	237	234	217	215	214	234
23	338	268	262	199	196	241	262
24	340	278	273	204	202	251	220
25	328	295	289	213	210	265	229
31	257	209	209	195	190	188	209
32	271	242	238	169	169	218	238
33	292	266	261	203	198	240	261
34	305	293	275	217	212	264	233
CTB3	190	155	149	105	139	140	149
CTB4	160	152	148	107	105	137	148
CTB5	187	168	162	109	151	152	162
CTB6	205	184	178	121	118	166	178

TABLE 6.7 (b) TEST AND ANALYTICAL RESULTS - HOGGING BENDING (kNm)
GROUP 2 - TEST MOMENT < PLASTIC MOMENT CAPACITY

BEAM REF	M TEST	MP	BS 5950 (D)	SABS 0162	EURO CODE 4	CSA	BS 5950 (F)
SB14	196	216	211	156	152	194	211
SB4	167	223	163	129	126	201	138
SB10	174	213	164	123	120	192	160
SB5	218	253	219	180	176	228	193
SB11	212	231	220	169	165	208	215
SB6	292	331	279	235	230	298	252
RS3	3 752	4 249	3 495	2 812	2 749	3 824	3 024
PG1	39 081	39 960	38 935	30 948	30 252	35 964	38 387
PG5	3 355	4 257	3 045	2 832	2 768	3 831	2 841
PG7	37 815	43 020	36 119	33 590	32 835	38 718	35 904
CTB2	119	123	117	75	110	111	117

TABLE 6.8 (a) ANALYTICAL RESULTS NORMALISED WRT MTEST
GROUP 1 - TEST MOMENT > PLASTIC MOMENT CAPACITY

BEAM REF	MP	BS 5950 (D)	SABS 0162	EURO CODE 4	CSA	BS 5950 (F)
SB3	0.85	0.85	0.64	0.77	0.77	0.85
HB40	0.90	0.88	0.65	0.81	0.81	0.88
SB2	0.92	0.91	0.69	0.68	0.83	0.91
HB41	0.96	0.93	0.73	0.71	0.86	0.93
CB10	0.85	0.83	0.59	0.77	0.76	0.83
CB11	0.85	0.83	0.78	0.72	0.77	0.76
CB12	0.77	0.74	0.53	0.69	0.69	0.74
18	0.71	0.71	0.66	0.64	0.64	0.71
11	0.75	0.74	0.69	0.68	0.68	0.74
12	0.78	0.77	0.72	0.56	0.70	0.77
13	0.80	0.78	0.58	0.56	0.72	0.78
14	0.83	0.81	0.59	0.58	0.75	0.81
21	0.71	0.71	0.66	0.65	0.64	0.71
22	0.73	0.72	0.67	0.66	0.66	0.72
23	0.79	0.78	0.59	0.58	0.71	0.78
24	0.82	0.80	0.60	0.59	0.74	0.65
25	0.90	0.88	0.65	0.64	0.81	0.70
31	0.81	0.81	0.76	0.74	0.73	0.81
32	0.89	0.88	0.62	0.62	0.80	0.88
33	0.91	0.89	0.70	0.68	0.82	0.89
34	0.96	0.90	0.71	0.69	0.87	0.76
CTB3	0.82	0.78	0.55	0.73	0.73	0.78
CTB4	0.95	0.92	0.67	0.66	0.86	0.92
CTB5	0.90	0.87	0.59	0.81	0.81	0.87
CTB6	0.90	0.87	0.59	0.57	0.81	0.87
MEAN		0.824	0.648	0.672	0.759	0.802
STD. DEV		0.067	0.063	0.073	0.067	0.077
MEAN+STD. DEV		0.891	0.711	0.746	0.826	0.879

TABLE 6.8 (b) ANALYTICAL RESULTS NORMALISED WRT MTEST
GROUP 2 - TEST MOMENT < PLASTIC MOMENT CAPACITY

BEAM REF	MP	BS 5950 (D)	SABS 0162	EURO CODE 4	CSA	BS 5950 (F)
SB14	1.10	1.08	0.80	0.78	0.99	1.08
SB4	1.34	0.97	0.77	0.75	1.20	0.83
SB10	1.23	0.94	0.70	0.69	1.10	0.92
SB5	1.16	1.00	0.83	0.81	1.04	0.89
SB11	1.09	1.04	0.80	0.78	0.98	1.01
SB6	1.14	0.96	0.81	0.79	1.02	0.87
RS3	1.13	0.93	0.75	0.73	1.02	0.81
PG1	1.02	1.00	0.79	0.77	0.92	0.98
PG5	1.27	0.91	0.84	0.83	1.14	0.85
PG7	1.14	0.96	0.89	0.87	1.02	0.95
CTB2	1.03	0.98	0.63	0.92	0.93	0.98
MEAN		0.979	0.782	0.792	1.034	0.923
STD. DEV		0.047	0.066	0.061	0.082	0.082
MEAN+STD. DEV		1.026	0.848	0.853	1.117	1.006

The results predicted by SABS 0162 and EUROCODE 4 are similar, but conservative compared to BS 5950. This is due mainly to the introduction of elastic analysis for relatively small areas of tension reinforcement, indicating conservative section slenderness limits.

Values predicted by CSA-S16.1 for sections in group 1 are less conservative than SABS 0162 and EC 4 with a standard deviation of 0,067. The predicted ultimate moments of sections in group 2, however, are overestimated by up to 20% with a high value of mean + standard deviation = 1.117.

The accuracy of a Code in predicting the ultimate moment capacity of a particular section may vary considerably as the area of tension reinforcement and hence the effective section slenderness varies.

This is well demonstrated by utilising results obtained from tests on continuous composite sections by Climenhaga and Longworth (17). The object of the original investigation was to test the effect of local flange buckling on the ultimate moment capacity. Three sets of beams were tested, using the same steel section within a set, but with increased section slenderness per set. The main variable was the area of tension reinforcement.

The ultimate test moments of each set of beams is plotted against the ultimate moment capacities predicted by the design Codes. (Ref. Figures 6.12, 6.13 & 6.14)

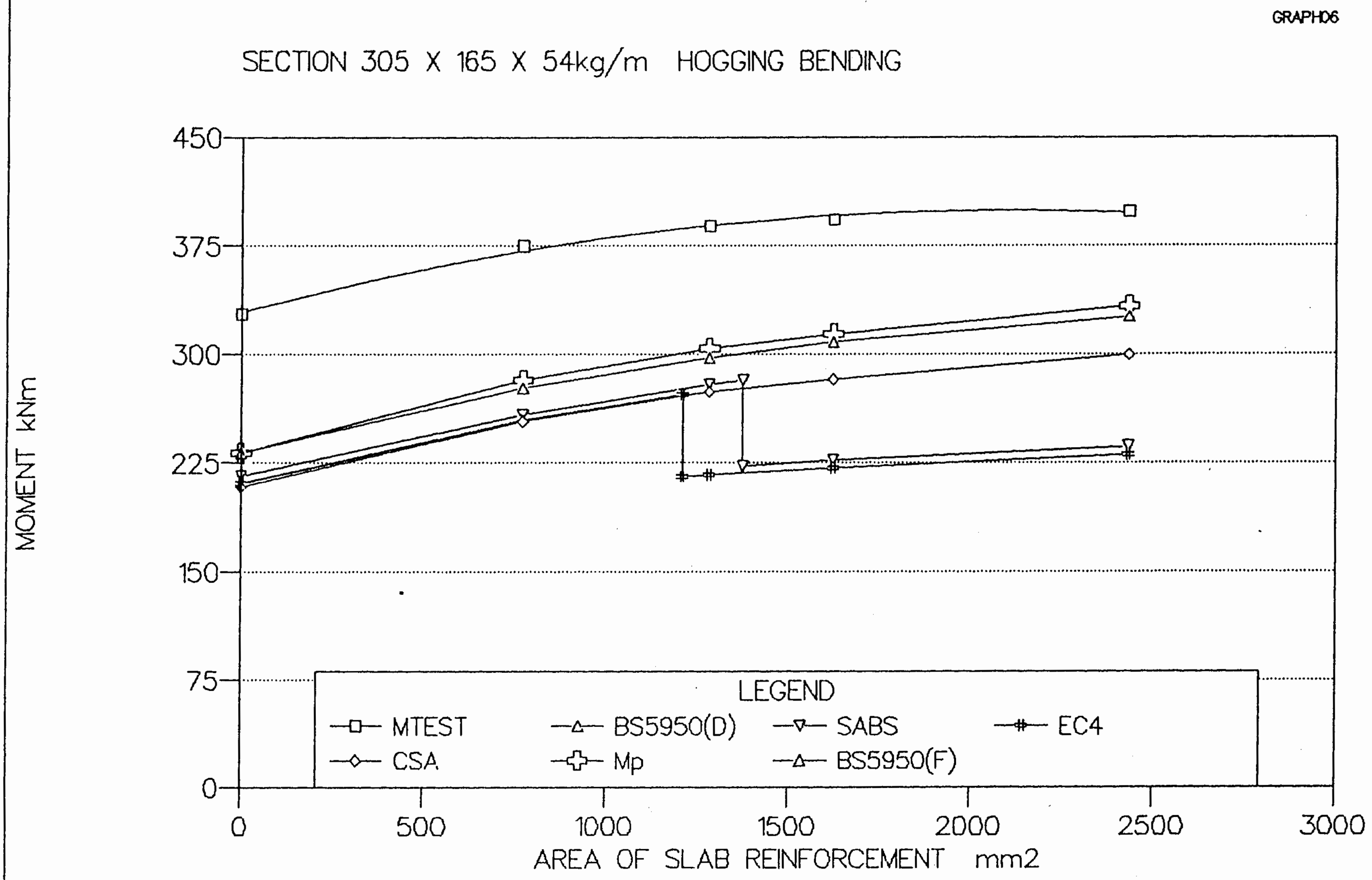


FIGURE: 6.12 COMPARISON OF TEST AND PREDICTED RESULTS – REF16

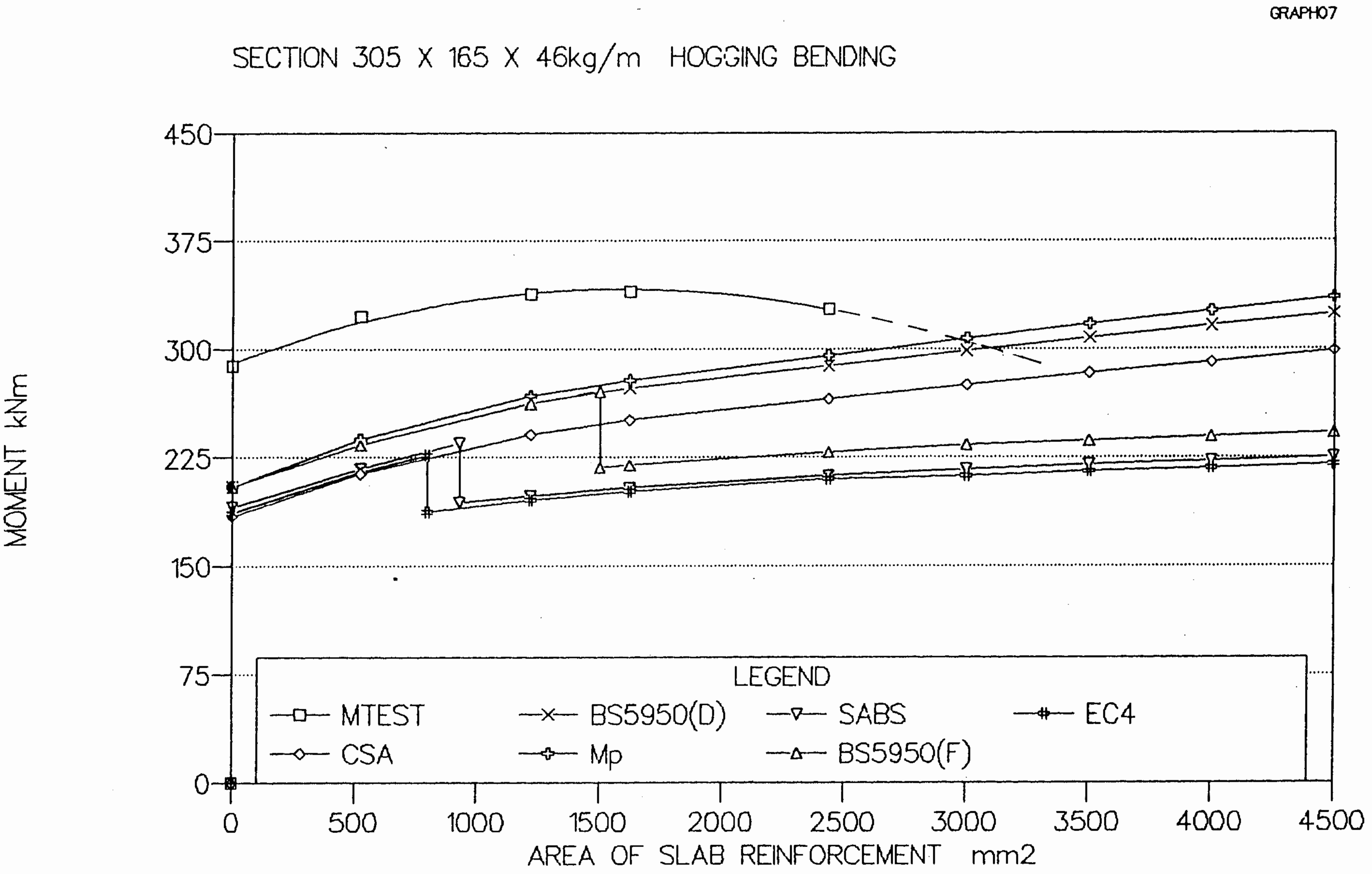


FIGURE:6.13 COMPARISON OF TEST AND PREDICTED RESULTS – REF16

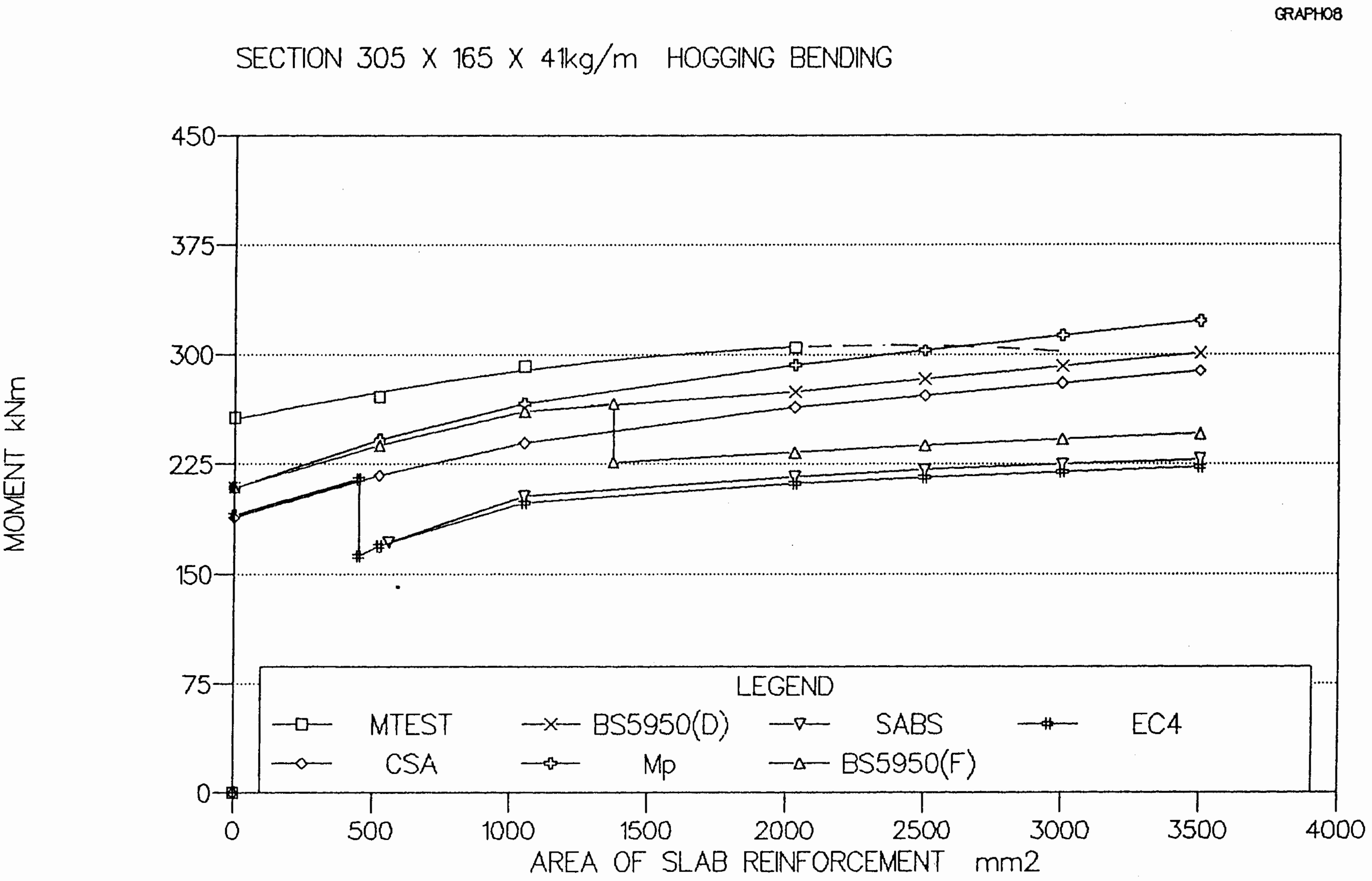


FIGURE: 6.14 COMPARISON OF TEST AND PREDICTED RESULTS – REF16

Generally, the largest difference between test and predicted moments occurs for the less slender sections and lower amounts of reinforcement. This is contrary to what one would expect since the performance of the more stable sections should be more predictable.

In the tests, the maximum amount of longitudinal slab reinforcement was limited to the web area of the steel section since it was believed that "increasing the longitudinal slab reinforcement above this amount does not significantly increase the theoretical ultimate moment capacity." For the two more slender sections, Figures 6.13 & 6.14, theoretical ultimate moments have been calculated for areas of slab reinforcement beyond these limits and test curves extrapolated.

The extrapolated curves of the test results intersect the predicted moment curves of BS 5950 (D) and CSA-S16.1 indicating that both Codes may overestimate the section moment capacity for large areas of tension reinforcement.

The revision brought about in BS 5950 (F) appear to be justified although the change over from plastic to elastic analysis, ie. the limiting slenderness, is somewhat conservative.

7.0 CONCLUSION

This dissertation has reviewed aspects of four selected international Codes for the design of composite steel and concrete buildings. Code comparisons have been based on the methods of global and section analysis, with special emphasis on the influence of cross-section classification and effective width of concrete flange on these methods.

7.1 GLOBAL ANALYSIS

The Codes allow plastic or elastic global analysis with variable redistribution of bending moments. The principal requirement of PLASTIC GLOBAL ANALYSIS is the assurance that the section possesses sufficient rotation capacity for the hinges to form. A simple but conservative method of achieving this is to limit the section slenderness to CLASS 1 plastic. This is the method adopted by CSA-S16 and BS 5950 (METHOD 1).

EUROCODE 4, BS 5950 (METHOD 2) and SABS 0162 have adopted the more general approach with restraints on span lengths, section slenderness, position of PNA and types of loading. Compliance with these restrictions result in longer design times, but a wider range of steel sections may be used.

The BS 5950 approach is the best, allowing the designer to take advantage of a simple method when suitable conditions exist, but making provision for the more general situation.

ELASTIC GLOBAL ANALYSIS is in more general use in design offices than plastic analysis and is given as an alternative method by the four Codes. The success of its application relies on the fact that the section is utilised to the extent of its rotational capacity and does not require a hinge to form. The effect of cracking of concrete over the supports is simulated by redistribution of bending moments. The main code restraints and consequences are highlighted below:

1. CSA-S16-1 - No redistribution of bending moments is allowed with the result that the sagging moment capacity cannot be fully utilised.

2. SABS 0162 - There are no limits on the amount of redistribution but other restraints are as for plastic global analysis, making the method conservative.

3. EUROCODE 4 - There is a variety of elastic analysis, with an increasing amount of redistribution the more compact the cross-section, making it less conservative than the above methods.

4. BS 5950 - Cracked or uncracked analysis is allowed for all classes of section with amount of redistribution dependent on whether the section is cracked or uncracked and on the class of section which is based on the flange slenderness. It is much more straightforward than EUROCODE 4 and less conservative.

7.2 CROSS-SECTION CLASSIFICATION

This is a much researched subject and is shown to have a bearing on a number of aspects of composite design. The slenderness limits are clearly defined in three of the Codes, but it was necessary to make assumptions about the implied slenderness limits applicable to the fourth code, CSA-S16.1, in which limits are specified for plain steel sections only.

The limits specified in CSA-S16.1 are independent of tension reinforcement in the concrete slab and are shown to be considerably optimistic. The effect of the limits of SABS 0162 are very similar to EUROCODE 4 and compare well in parametric studies. There are however additional limits in SABS 0162 on area of tension reinforcement which make it more conservative than EUROCODE 4.

The slenderness limits of BS 5950 compare best with test results and are shown to be least conservative in parametric studies.

7.3 EFFECTIVE WIDTH OF CONCRETE FLANGE

It has been demonstrated that the assumed effective width of concrete flange has a considerable influence on cross-section classification and consequently, section and global analysis, and yet such diversity exists in the codes in its presentation and magnitude.

In research on many topics related to composite steel and concrete design, eg. cross-section classification, ultimate moment capacity, etc., tests have been based on individual beams in which the full slab width is considered effective. The effects of varying flange widths on section classification and ultimate moment capacity has been demonstrated and shown to have a considerable influence on the performance of the Codes.

This is clearly a topic requiring clarification.

7.4 CROSS-SECTION ANALYSIS

For the purposes of this dissertation, cross-section analysis entailed calculating the ultimate moment capacity of a section in sagging or hogging bending.

Three of the Codes allow plastic or elastic section analysis. Plastic analysis is the preferred method yielding an ultimate hogging moment capacity some 30% more than that obtained in an elastic analysis. It may be employed up to a point where it is anticipated that the section will fail by local instability, and elastic analysis is then introduced.

The fourth Code, CSA-S16.1, allows only plastic section analysis in both hogging and sagging regions.

SAGGING REGIONS

Sagging moment capacities predicted by the Codes compare very well, differing only due to material factors.

HOGGING REGIONS

The accuracy of a Code in predicting the ultimate hogging moment capacity is shown to be largely dependent on the limits at which elastic section analysis is introduced. Parametric studies indicate considerable differences in the hogging moment capacity predicted by Codes. BS 5950 (F) values relate best to actual test results and has the highest overall consistency. CSA-S16.1 predicted moments exceed test results in a number of cases and is considered unsafe for sections where the moment at failure is less than the plastic moment capacity.

7.5 GENERAL CONCLUSION

The methods of BS 5950 (F) have been found to give the best all-round performance as far as ease of use is concerned as well as comparison of predicted to test results.

The approach of EUROCODE 4 is similar to that of BS 5950 (F), but more conservative and certain calculations tend to be more laborious.

The more theoretical approach adapted by these two Codes result in long hand calculations, but all the methods given are of a type suitable for computer aided design.

The methods of SABS 0162 are easier to apply but are considerably more conservative in global analysis and in estimating the ultimate hogging moment capacity.

CSA-S16.1 is simple to apply and well suited to the analysis of simply supported beams, but is found to be lacking in the analysis of continuous beams.

APPENDIX A

DESIGN PROGRAMS

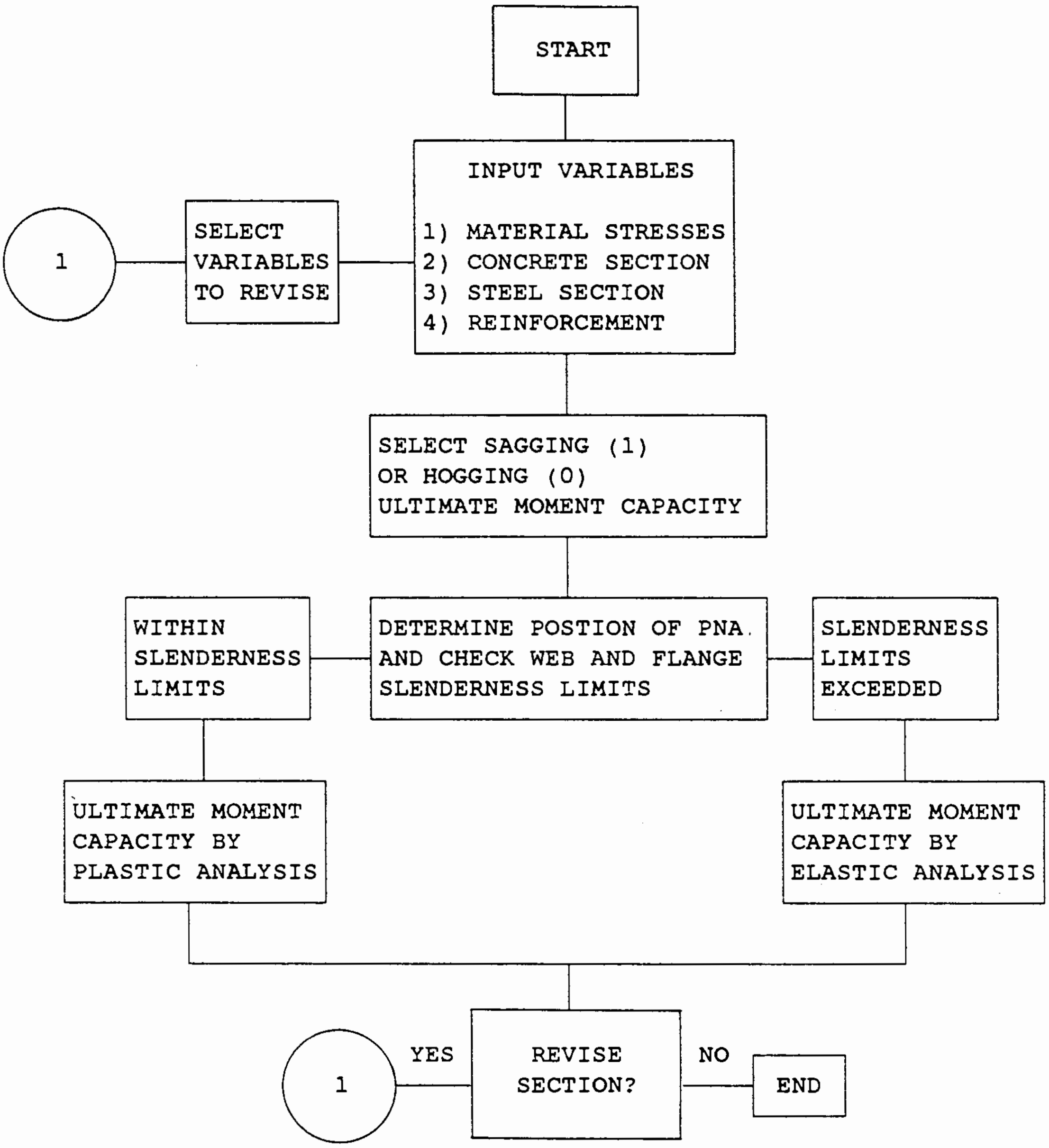
Programmes to calculate the ultimate bending moment capacity of composite sections in sagging and hogging bending have been developed for the four design Codes considered. They have been written for a hand-held programmable calculator, the Hewlett Packard 41CV.

The programmes have been written in such a manner that once the input data, viz. material properties and steel and concrete section dimensions, have been entered, selected variables may be changed and the programme rerun without affecting the other input. Checks on section slenderness limits are performed and it is determined within the programme whether elastic or plastic analysis may be used. For BS 5950 and EUROCODE 4, seperate programmes are used to calculate elastic and plastic moment capacities due to the length of the programmes and limitations of the calculator memory capacity.

The general format has been retained for all the Codes. A typical flow chart is shown below.

OPERATING INSTRUCTIONS:

1. When executed, the programme will prompt for variables. A prompt ending with an = sign, e.g. CONC: Be =, requires a data input.



BASIC FLOW CHART OF CALCULATOR PROGRAMS TO CALCULATE THE ULTIMATE MOMENT CAPACITY OF COMPOSITE SECTIONS.

2. A prompt ending with a ? represents an option requiring a figure 1 input for YES and 0 (zero) for NO.

3. When changes to variables are required, the string M:1, C:2, S:3, A:29 will be displayed where:

M represents material properties,

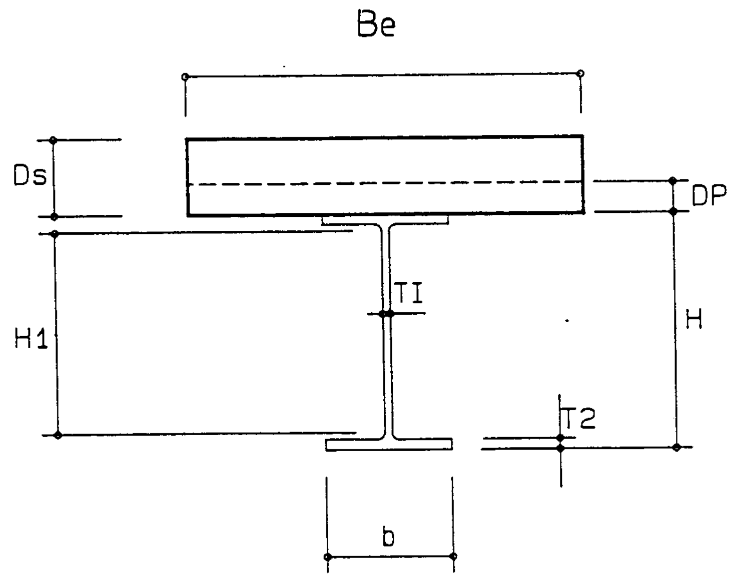
C - the concrete section dimensions,

S - the structural steel section dimensions,

A - the steel reinforcement data.

By entering the number adjacent to the relevant letter representing the variable to be changed will result in new values for that variable being prompted for. The programme may then be rerun.

INPUT VARIABLES



		UNITS	STORAGE REGISTER
MATERIAL STRESSES			
Fcu	Characteristic cube strength of concrete	MPa	01
Fy	Characteristic strength of reinforcement	MPa	03
Py	Yield strength of structural steel	MPa	02
αe	Modular ratio		29
CONCRETE SECTION			
Be	Total effective width of concrete flange	mm	04
Ds	Overall depth of slab	mm	05
Dp	Overall depth of profiled steel sheet	mm	06
STEEL SECTION			
H1	Clear depth of web	mm	07
b	Width of steel flange	mm	08
T_2	Steel flange thickness	mm	09
T_1	Steel web thickness	mm	12
H	Overall depth of steel beam	mm	14
A	Area of steel section	mm^2	10
I	Second moment of area of steel section about major axis	mm^4	19
Zpl	Plastic modulus of steel section	mm^3	17
REINFORCEMENT			
Ast	Area of tension reinforcement within effective width	mm^2	11
d*	top of concrete flange to centroid of reinforcement	mm	

PART SHRα Partial shear as a fraction: 1 = full shear connection

P R O G R A M   L I S T I N G

DESIGN CODE: BS 5950 - PLASTIC DESIGN **SHEET**

```
01♦LBL "BS5950"
02 "FINAL"
03 "COMP DESIGN MOM"
04 "⊢CAPACITY"
05 CLRG
06 FIX 2
07 CF 04
08 XEQ 28
09♦LBL 01
10 "FcU="
11 PROMPT
12 STO 01
13 "FY REINFT="
14 PROMPT
15 STO 03
16 "PY JOIST="
17 PROMPT
18 STO 02
19 RTN
20♦LBL 02
21 "CONC SECT"
22 AVIEW
23 PSE
24 "CONC: Be="
25 PROMPT
26 STO 04
27 "CONC: DS="
28 PROMPT
29 STO 05
30 "CONC: DP="
31 PROMPT
32 STO 06
33 RTN
34♦LBL 03
35 "STEEL SECT"
36 AVIEW
37 PSE
38 "STEEL: H1="
39 PROMPT
40 STO 07
41 "STEEL: b="
42 PROMPT
43 STO 08
44 "STEEL: T2="
45 PROMPT
46 STO 09
47 "STEEL: T1="
48 PROMPT
49 STO 12
50 "STEEL: A="
51 PROMPT
52 STO 10
53 "STEEL: H="
54 PROMPT
55 STO 14
56 RCL 07
57 RCL 12
58 /
59 STO 15
60 "ZPL="
61 PROMPT
62 STO 17
63 RCL 02
64 *
65 STO 16
66 RTN
67♦LBL 29
68 "AST="
69 PROMPT
70 STO 11
71 "d*="
72 PROMPT
73 RCL 05
74 -
75 CHS
76 STO 18
77 RTN
78♦LBL 04
79 "RESISTANCE"
80 SF 04
81 "RC"
82 RCL 05
83 RCL 06
84 -
85 RCL 04
86 *
87 RCL 01
88 *
89 .45
90 *
91 STO 20
92 "RF"
93 RCL 08
94 RCL 09
95 *
96 RCL 02
97 *
98 STO 21
99 "RS"
100 RCL 10
101 RCL 02
102 *
103 STO 25
104 "RV"
105 RCL 07
106 RCL 12
107 *
108 RCL 02
109 *
110 STO 26
111 "$"
112 RCL 02
113 1/X
114 275
115 *
116 .5
117 Y↑X
118 STO 13
119 "RO"
120 RCL 13
121 38
122 *
123 RCL 12
124 X↑2
125 *
126 RCL 02
127 *
128 STO 28
129 "RN"
130 RCL 25
131 RCL 26
132 -
133 RCL 28
134 +
135 STO 22
136 "RR"
137 RCL 11
138 RCL 03
139 *
140 .87
141 *
142 STO 24
143 "RW"
144 RCL 21
145 2
146 *
147 CHS
148 RCL 25
149 +
150 STO 27
151 "FLANGE CLASS"
152 "SIMPLY SUP?"
153 PROMPT
154 1
155 X=Y?
156 GTO A
157 RCL 08
158 2
159 /
160 RCL 09
161 /
162 STO 00
163 RCL 13
164 8.5
165 *
```

```
166 RCL 00
167 X<=Y?
168 GTO "S"
169 RCL 13
170 9.5
171 *
172 RCL 00
173 X<=Y?
174 GTO "T"
175 RCL 13
176 15
177 *
178 RCL 00
179 X<=Y?
180 GTO "U"
181 "SLENDER"
182 SF 01
183 GTO "V"
184◆LBL "S"
185 "PLASTIC"
186 GTO "V"
187◆LBL "T"
188 "COMP"
189 GTO "V"
190◆LBL "U"
191 "SEMI COMP"
192 SF 01
193◆LBL "V"
194 AVIEW
195 PSE
196 FS?C 01
197 GTO D
198◆LBL A
199 "+VE SAG MOM?"
200 PROMPT
201 X=0?
202 GTO C
203 "FULL SHEAR?"
204 PROMPT
205 X=0?
206 GTO B
207◆LBL 05
208 RCL 27
209 RCL 20
210 X>Y?
211 GTO 06
212 "PNA IN WEB"
213 AVIEW
214 PSE
215 RCL 13
216 76
217 *
218 RCL 15
219 X<=Y?
220 GTO 07
221 RCL 20
222 RCL 26
223 /
224 1
225 X<>Y
226 X>Y?
227 X<>Y
228 1
229 -
230 CHS
231 1/X
232 RCL 13
233 *
234 76
235 *
236 RCL 15
237 X<=Y?
238 GTO 07
239 GTO 08
240◆LBL 07
241 "WEB COMP"
242 AVIEW
243 PSE
244 RCL 14
245 RCL 05
246 +
247 RCL 06
248 +
249 2
250 /
251 RCL 20
252 *
253 RCL 20
254 X↑2
255 RCL 26
256 /
257 RCL 07
258 *
259 4
260 /
261 -
262 RCL 16
263 +
264 GTO 10
265◆LBL 08
266 "WEB NOT COMP"
267 AVIEW
268 PSE
269 RCL 14
270 RCL 05
271 +
272 RCL 06
273 +
274 2
275 /
276 RCL 20
277 *
278 RCL 26
279 RCL 20
280 -
281 RCL 28
282 2
283 *
284 -
285 RCL 26
286 RCL 20
287 -
288 *
289 RCL 20
290 X↑2
291 +
292 RCL 26
293 /
294 RCL 07
295 *
296 4
297 /
298 -
299 RCL 16
300 +
301 GTO 10
302◆LBL 06
303 RCL 20
304 RCL 25
305 X>Y?
306 GTO 09
307 "PNA CON FLNG"
308 AVIEW
309 PSE
310 RCL 05
311 RCL 06
312 -
313 2
314 /
315 RCL 25
316 *
317 RCL 20
318 /
319 RCL 05
320 -
321 CHS
322 RCL 14
323 2
324 /
325 +
326 RCL 25
327 *
328 GTO 10
329◆LBL 09
330 "PNA STL FLNG"
```

P R O G R A M   L I S T I N G

DESIGN CODE: BS 5950 - PLASTIC DESIGN **SHEET**

```
496 *
497 2
498 /
499 RCL 05
500 RCL 06
501 -
502 RCL 23
503 *
504 RCL 20
505 /
506 2
507 /
508 RCL 05
509 -
510 CHS
511 RCL 23
512 *
513 +
514 RCL 25
515 RCL 23
516 -
517 X↑2
518 RCL 21
519 /
520 RCL 09
521 *
522 4
523 /
524 -
525 GTO 10
526♦LBL C
527 RCL 24
528 RCL 26
529 /
530 1
531 X<>Y
532 X>Y?
533 X<>Y
534 2
535 *
536 1
537 +
538 1/X
539 114
540 *
541 RCL 13
542 *
543 RCL 15
544 X>Y?
545 GTO D
546 RCL 13
547 38
548 *
549 RCL 15
550 X<=Y?
551 GTO 15
552 RCL 24
553 RCL 26
554 /
555 1
556 X<>Y
557 X>Y?
558 X<>Y
559 1
560 +
561 1/X
562 76
563 *
564 RCL 13
565 *
566 RCL 15
567 X>Y?
568 GTO 14
569♦LBL 15
570 "WEB COMP"
571 AVIEW
572 PSE
573 RCL 24
574 RCL 27
575 X<=Y?
576 GTO 13
577 "PNA IN WEB"
578 AVIEW
579 PSE
580 RCL 14
581 2
582 /
583 RCL 18
584 +
585 RCL 24
586 *
587 RCL 16
588 +
589 RCL 24
590 X↑2
591 RCL 26
592 /
593 RCL 07
594 *
595 4
596 /
597 -
598 GTO 10
599♦LBL 14
600 "WEB NOT COMP"
601 AVIEW
602 PSE
603 RCL 28
604 RCL 24
605 X<Y?
606 GTO 16
607 "RR>RO"
608 AVIEW
609 PSE
610 GTO 13
611♦LBL 16
612 "PNA IN WEB"
613 AVIEW
614 PSE
615 RCL 26
616 RCL 24
617 +
618 RCL 28
619 2
620 *
621 -
622 RCL 26
623 RCL 24
624 +
625 *
626 RCL 24
627 X↑2
628 +
629 RCL 26
630 /
631 RCL 07
632 *
633 4
634 /
635 CHS
636 RCL 14
637 2
638 /
639 RCL 18
640 +
641 RCL 24
642 *
643 +
644 RCL 16
645 +
646 GTO 10
647♦LBL 13
648 "PNA IN FLNG"
649 AVIEW
650 PSE
651 RCL 13
652 38
653 *
654 RCL 15
655 X<=Y?
656 GTO 17
657 "WEB NOT COMP"
658 AVIEW
659 PSE
660 RCL 22
```

```
661 RCL 24
662 X>Y?
663 GTO 18
664 "PNA ST FLNG"
665 AVIEW
666 PSE
667 RCL 22
668 RCL 14
669 *
670 2
671 /
672 RCL 24
673 RCL 18
674 *
675 +
676 RCL 22
677 RCL 24
678 -
679 X↑2
680 RCL 21
681 /
682 RCL 09
683 *
684 4
685 /
686 -
687 GTO 10
688♦LBL 18
689 "PNA OUT ST FLNG"
690 AVIEW
691 PSE
692 RCL 14
693 2
694 /
695 RCL 18
696 +
697 RCL 22
698 *
699 GTO 10
700♦LBL 17
701 "WEB COMP"
702 AVIEW
703 PSE
704 RCL 25
705 RCL 24
706 X>Y?
707 GTO 19
708 "PNA IN ST FL"
709 AVIEW
710 PSE
711 RCL 25
712 RCL 14
713 *
714 2
715 /
716 RCL 24
717 RCL 18
718 *
719 +
720 RCL 25
721 RCL 24
722 -
723 X↑2
724 RCL 21
725 /
726 RCL 09
727 *
728 4
729 /
730 -
731 GTO 10
732♦LBL 19
733 "PNA OUT ST BEAM"
734 AVIEW
735 PSE
736 RCL 14
737 2
738 /
739 RCL 18
740 +
741 RCL 25
742 *
743 GTO 10
744♦LBL 28
745 FS? 04
746 GTO 30
747 XEQ 01
748 XEQ 02
749 XEQ 03
750 XEQ 29
751 "   RUN?"
752 PROMPT
753 1
754 X=Y?
755 XEQ 04
756♦LBL 30
757 "VARIABLES"
758 AVIEW
759 PSE
760 "M:1 C:2 S:3 A:2"
761 "⊢9"
762 PROMPT
763 X>0?
764 XEQ IND X
765 "MORE CHANGE?"
766 PROMPT
767 1
768 X=Y?
769 GTO 30
770 XEQ 04
771♦LBL D
772 "USE ELASTIC ANA"
773 "⊢LYSIS"
774 AVIEW
775 GTO 30
776 .END.
```

PROGRAM LISTING

DESIGN CODE: BS 5950 - ELASTIC DESIGN **SHEET**

```
01♦LBL "BSMELAS"
02 "ELASTIC ANAL OF"
03 "⊦COMPOSITE BEAM"
04 "⊦ TO BS5950"
05 CLRG
06 GTO 28
07♦LBL 01
08 "FcU="
09 PROMPT
10 STO 01
11 "FY="
12 PROMPT
13 STO 03
14 "PY="
15 PROMPT
16 STO 02
17 RTN
18♦LBL 02
19 "CONC SECT"
20 AVIEW
21 PSE
22 "CONC: Be="
23 PROMPT
24 STO 04
25 "CONC:DS="
26 "CONC: DS="
27 PROMPT
28 STO 05
29 "CONC: DP="
30 PROMPT
31 STO 06
32 RTN
33♦LBL 03
34 "STEEL SECT"
35 AVIEW
36 PSE
37 "STEEL: H1="
38 PROMPT
39 STO 07
40 "STEEL: b="
41 PROMPT
42 STO 08
43 "STEEL: T2="
44 PROMPT
45 STO 09
46 "STEEL: T1="
47 PROMPT
48 STO 12
49 "STEEL: A="
50 PROMPT
51 STO 10
52 "STEEL: I="
53 PROMPT
54 STO 19
55 "∡E="
56 PROMPT
57 STO 29
58 "STEEL: H="
59 PROMPT
60 STO 14
61 RCL 07
62 RCL 12
63 /
64 STO 15
65 RTN
66♦LBL 29
67 "AST="
68 PROMPT
69 STO 11
70 "d*="
71 PROMPT
72 RCL 05
73 -
74 CHS
75 STO 18
76 RTN
77♦LBL 28
78 FS? 04
79 GTO 30
80 XEQ 01
81 XEQ 02
82 XEQ 03
83 XEQ 29
84 "   RUN?"
85 PROMPT
86 1
87 X=Y?
88 XEQ 04
89♦LBL 30
90 "VARIABLES"
91 AVIEW
92 PSE
93 "M:1 C:2 S:3 A:2"
94 "⊦9"
95 PROMPT
96 X>0?
97 XEQ IND X
98 "MORE CHANGS?"
99 PROMPT
100 1
101 X=Y?
102 GTO 30
103♦LBL 04
104 "RS"
105 RCL 10
106 RCL 02
107 *
108 STO 25
109 "$"
110 RCL 02
111 1/X
112 275
113 *
114 .5
115 Y↑X
116 STO 13
117 RCL 14
118 RCL 05
119 +
120 RCL 06
121 +
122 X↑2
123 RCL 05
124 RCL 06
125 -
126 *
127 RCL 04
128 *
129 RCL 10
130 *
131 4
132 /
133 RCL 05
134 RCL 06
135 -
136 RCL 04
137 *
138 RCL 10
139 RCL 29
140 *
141 +
142 /
143 RCL 05
144 RCL 06
145 -
146 3
147 Y↑X
148 RCL 04
149 *
150 12
151 /
152 RCL 29
153 /
154 +
155 RCL 19
156 +
157 STO 30
158 "IN"
159 RCL 18
160 2
161 *
162 RCL 14
163 +
164 X↑2
165 RCL 11
```

PROGRAM LISTING

DESIGN CODE: BS 5950 - ELASTIC DESIGN **SHEET**

```
166 *
167 RCL 10
168 *
169 4
170 /
171 RCL 10
172 RCL 11
173 +
174 /
175 RCL 19
176 +
177 STO 31
178 "Ye"
179 RCL 05
180 2
181 *
182 RCL 14
183 +
184 STO 00
185 RCL 04
186 *
187 RCL 10
188 /
189 RCL 29
190 /
191 1
192 +
193 SQRT
194 1
195 +
196 1/X
197 RCL 00
198 *
199 STO 33
200 "IP"
201 RCL 05
202 RCL 33
203 -
204 RCL 14
205 2
206 /
207 +
208 X↑2
209 RCL 10
210 *
211 RCL 33
212 3
213 Y↑X
214 RCL 04
215 *
216 3
217 /
218 RCL 29
219 /
220 +
221 RCL 19
222 +
223 STO 32
224 "+VE MOM? 1/0"
225 PROMPT
226 X=0?
227 GTO 20
228 "ELAST NA"
229 AVIEW
230 RCL 05
231 RCL 06
232 -
233 X↑2
234 RCL 04
235 *
236 RCL 14
237 RCL 06
238 2
239 *
240 +
241 RCL 29
242 *
243 /
244 RCL 10
245 X<Y?
246 GTO 21
247 GTO 22
248♦LBL 21
249 "ENA IN CONC FLN"
250 RCL 32
251 RCL 29
252 *
253 RCL 33
254 /
255 STO 34
256 RCL 14
257 RCL 05
258 +
259 RCL 33
260 -
261 1/X
262 RCL 32
263 *
264 STO 35
265 GTO 23
266♦LBL 22
267 "ENA IN STEEL"
268 RCL 05
269 RCL 06
270 -
271 X↑2
272 RCL 04
273 *
274 RCL 05
275 2
276 *
277 RCL 14
278 +
279 RCL 29
280 *
281 RCL 10
282 *
283 +
284 2
285 /
286 RCL 05
287 RCL 06
288 -
289 RCL 04
290 *
291 RCL 10
292 RCL 29
293 *
294 +
295 /
296 STO 00
297 RCL 30
298 RCL 29
299 *
300 RCL 00
301 /
302 STO 34
303 RCL 14
304 RCL 05
305 +
306 RCL 00
307 -
308 1/X
309 RCL 30
310 *
311 STO 35
312♦LBL 23
313 RCL 34
314 "Fc CONCRETE?"
315 RCL 01
316 *
317 .5
318 *
319 RCL 35
320 RCL 02
321 *
322 X>Y?
323 X<>Y
324 GTO 27
325♦LBL 20
326 "-VE MOM"
327 "YR"
328 RCL 18
329 2
330 *
```

```
331 RCL 14
332 +
333 RCL 10
334 *
335 2
336 /
337 RCL 10
338 RCL 11
339 +
340 /
341 STO 37
342 RCL 31
343 RCL 37
344 /
345 STO 34
346 RCL 14
347 RCL 18
348 +
349 RCL 37
350 -
351 1/X
352 RCL 31
353 *
354 STO 35
355 RCL 14
356 RCL 05
357 +
358 RCL 18
359 -
360 RCL 37
361 -
362 STO 00
363 RCL 02
364 RCL 37
365 *
366 RCL 00
367 /
368 RCL 03
369 .87
370 *
371 X<>Y
372 X>Y?
373 X<>Y
374 RCL 11
375 *
376 STO 38
377♦LBL 25
378 RCL 38
379 RCL 25
380 /
381 1
382 X<>Y
383 X>Y?
384 X<>Y
385 STO 00
386 1.5
387 *
388 1
389 +
390 1/X
391 120
392 *
393 RCL 13
394 *
395 RCL 15
396 X>Y?
397 GTO 26
398 "WEB NOT SLENDER"
399 AVIEW
400 PSE
401 RCL 07
402 RCL 12
403 /
404 63
405 X<>Y
406 X>Y?
407 GTO 32
408 "CHECK FLNGE"
409 AVIEW
410 PSE
411 RCL 13
412 26
413 *
414 RCL 08
415 RCL 09
416 /
417 X>Y?
418 GTO 31
419 "FLNGE NOT SLNDR"
420 AVIEW
421 PSE
422 RCL 34
423 RCL 03
424 *
425 .87
426 *
427 RCL 35
428 RCL 02
429 *
430 X>Y?
431 X<>Y
432♦LBL 27
433 1 E-6
434 *
435 "Me="
436 "⊦"
437 ARCL X
438 "⊦KN.M"
439 AVIEW
440 STOP
441 GTO 30
442♦LBL 26
443 "WEB SLENDER: RE"
444 "⊦DUCE ALLOW STR"
445 "⊦ESS"
446 GTO 28
447♦LBL 31
448 "FLANGE SLENDER:"
449 "⊦RED STRESS"
450 AVIEW
451 STOP
452 GTO 28
453♦LBL 32
454 "d/TW >63"
455 AVIEW
456 STOP
457 GTO 28
458 .END.
```

PROGRAM LISTING

DESIGN CODE: SABS 0162

SHEET

```
01♦LBL "SABS016"
02 "SABS 0160"
03 "MOM CAP - PLAS"
04 "⊦TIC ANALYSIS"
05 CLRG
06 GTO 14
07♦LBL 01
08 "FcU="
09 PROMPT
10 .4
11 *
12 STO 01
13 "PY JOIST="
14 PROMPT
15 .93
16 *
17 STO 02
18 "FRY REINFT="
19 PROMPT
20 1.15
21 /
22 STO 03
23 "EC= (GPa)"
24 PROMPT
25 STO 34
26 RTN
27♦LBL 02
28 "CONC SECT"
29 AVIEW
30 PSE
31 "CONC: Be="
32 PROMPT
33 STO 04
34 "CONC: DS="
35 PROMPT
36 STO 05
37 "CONC: DP="
38 PROMPT
39 STO 06
40 RTN
41♦LBL 03
42 CF 04
43 "STEEL SECT"
44 AVIEW
45 PSE
46 "STEEL: H="
47 PROMPT
48 STO 07
49 "STEEL: H1="
50 PROMPT
51 STO 33
52 "STEEL: b="
53 PROMPT
54 STO 08
55 "STEEL: T2="
56 PROMPT
57 STO 09
58 "STEEL: T1="
59 PROMPT
60 STO 20
61 "STEEL: A="
62 PROMPT
63 STO 10
64 "STEEL: I="
65 PROMPT
66 STO 24
67 "PART SHR ∡="
68 PROMPT
69 STO 17
70 RTN
71♦LBL 04
72 RCL 17
73 1
74 X>Y?
75 SF 04
76 RCL 01
77 RCL 04
78 *
79 STO 14
80 RCL 02
81 RCL 10
82 *
83 STO 15
84 RCL 07
85 2
86 /
87 RCL 05
88 +
89 RCL 06
90 +
91 STO 13
92 RCL 14
93 RCL 05
94 *
95 RCL 15
96 X>Y?
97 GTO 06
98♦LBL 05
99 "PNA IN CONC"
100 AVIEW
101 PSE
102 FS? 04
103 GTO 07
104 RCL 15
105 RCL 14
106 /
107 .5
108 *
109 RCL 13
110 -
111 CHS
112 RCL 15
113 *
114 STO 11
115 RCL 15
116 STO 12
117 GTO 10
118♦LBL 06
119 RCL 14
120 RCL 05
121 *
122 RCL 15
123 -
124 CHS
125 2
126 /
127 RCL 02
128 /
129 STO 16
130 RCL 08
131 RCL 09
132 *
133 RCL 16
134 X>Y?
135 GTO 11
136 "PNA IN TOP FLAN"
137 "⊦GE"
138 AVIEW
139 PSE
140 FS? 04
141 GTO 07
142 RCL 16
143 2
144 /
145 RCL 08
146 /
147 RCL 06
148 +
149 RCL 05
150 2
151 /
152 +
153 RCL 16
154 *
155 RCL 02
156 *
157 2
158 *
159 RCL 05
160 2
161 /
162 RCL 13
163 -
164 CHS
165 RCL 10
```

```
166 *
167 RCL 02
168 *
169 -
170 CHS
171 STO 11
172 RCL 14
173 RCL 05
174 *
175 STO 12
176 GTO 10
177♦LBL 07
178 "PARTIAL SHEAR C"
179 "⊦ONNECTION"
180 AVIEW
181 PSE
182 RCL 14
183 RCL 05
184 *
185 RCL 15
186 X>Y?
187 X<>Y
188 STO 21
189 RCL 17
190 *
191 RCL 14
192 /
193 STO 18
194 RCL 14
195 RCL 18
196 *
197 RCL 15
198 -
199 CHS
200 2
201 /
202 RCL 02
203 /
204 STO 16
205 RCL 08
206 RCL 09
207 *
208 RCL 16
209 X>Y?
210 GTO 08
211 "PNA IN TOP FLAN"
212 "⊦GE :PART SHEAR"
213 AVIEW
214 PSE
215 RCL 16
216 2
217 /
218 RCL 08
219 /
220 STO 19
221 GTO 09
222♦LBL 08
223 "PNA IN WEB"
224 "⊦:PART SHEAR"
225 RCL 08
226 RCL 09
227 *
228 RCL 16
229 -
230 CHS
231 RCL 20
232 /
233 ENTER↑
234 ENTER↑
235 2
236 /
237 RCL 09
238 +
239 *
240 RCL 20
241 *
242 RCL 09
243 X↑2
244 RCL 08
245 *
246 .5
247 *
248 +
249 RCL 16
250 /
251 STO 19
252♦LBL 09
253 RCL 18
254 2
255 /
256 RCL 06
257 -
258 CHS
259 RCL 05
260 +
261 RCL 19
262 +
263 RCL 16
264 *
265 RCL 02
266 *
267 2
268 *
269 RCL 18
270 2
271 /
272 RCL 13
273 -
274 CHS
275 RCL 15
276 *
277 -
278 CHS
279 STO 11
280 RCL 21
281 RCL 17
282 *
283 STO 12
284 GTO 12
285♦LBL 10
286 RCL 11
287 1 E-6
288 *
289 "MU="
290 "⊦"
291 ARCL X
292 "⊦KN.M"
293 AVIEW
294 STOP
295 RCL 12
296 1 E-3
297 *
298 "SHV="
299 "⊦"
300 ARCL X
301 "⊦KN"
302 AVIEW
303 STOP
304 CF 04
305 GTO 17
306♦LBL 12
307 RCL 11
308 1 E-6
309 *
310 "MUR="
311 "⊦"
312 ARCL X
313 "⊦KN.M"
314 AVIEW
315 STOP
316 RCL 12
317 1 E-3
318 *
319 "SHV="
320 "⊦"
321 ARCL X
322 "⊦KN"
323 AVIEW
324 STOP
325 GTO 17
326♦LBL 11
327 "PNA IN WEB - US"
328 "⊦E ELASTIC MOM "
329 "⊦OF RESIST"
330 GTO 15
```

PROGRAM LISTING

DESIGN CODE: SABS 0162

SHEET

```
331♦LBL 14
332 FS? 01
333 GTO 17
334 XEQ 01
335 XEQ 02
336 XEQ 03
337 GTO 17
338 "RED SHEAR CAP"
339 PROMPT
340 X>0?
341 XEQ 07
342 "ELASTIC ANALYSI"
343 "├S?"
344 PROMPT
345 X>0?
346 GTO 15
347♦LBL 15
348 "ELAST M :SAG"
349 AVIEW
350 PSE
351 RCL 34
352 206
353 /
354 1/X
355 STO 22
356 RCL 05
357 X↑2
358 RCL 04
359 *
360 2
361 /
362 RCL 22
363 /
364 RCL 10
365 RCL 13
366 *
367 +
368 RCL 04
369 RCL 05
370 *
371 RCL 22
372 /
373 RCL 10
374 +
375 /
376 STO 23
377 RCL 05
378 2
379 /
380 RCL 23
381 -
382 X↑2
383 RCL 05
384 *
385 RCL 04
```

```
386 *
387 RCL 22
388 /
389 RCL 05
390 3
391 Y↑X
392 RCL 04
393 *
394 12
395 /
396 RCL 22
397 /
398 +
399 RCL 13
400 RCL 23
401 -
402 X↑2
403 RCL 10
404 *
405 +
406 RCL 24
407 +
408 STO 25
409 RCL 22
410 RCL 01
411 *
412 .6
413 /
414 RCL 25
415 *
416 RCL 23
417 /
418 RCL 02
419 RCL 25
420 *
421 RCL 07
422 2
423 /
424 RCL 05
425 +
426 RCL 06
427 +
428 RCL 23
429 -
430 /
431 X>Y?
432 X<>Y
433 1 E-6
434 *
435 "Me="
436 "├"
437 ARCL X
438 "├KN.M"
439 AVIEW
440 STOP
```

```
441♦LBL 17
442 "REV,M:1 C:2 S:3"
443 PROMPT
444 XEQ IND X
445 1
446 "MORE CHANGS?"
447 PROMPT
448 X=Y?
449 XEQ 17
450♦LBL 00
451 "SAG MOM?"
452 PROMPT
453 X>0?
454 XEQ 04
455 XEQ B
456♦LBL B
457 "HOG MOM"
458 AVIEW
459 PSE
460♦LBL 16
461 RCL 07
462 2
463 /
464 RCL 05
465 +
466 RCL 06
467 +
468 STO 13
469 "d*=?"
470 PROMPT
471 STO 29
472 "AR=?"
473 PROMPT
474 STO 30
475 RCL 10
476 .075
477 *
478 X>Y?
479 GTO 22
480 RCL 30
481 RCL 04
482 RCL 05
483 *
484 .003
485 *
486 X>Y?
487 GTO 23
488 RCL 10
489 .3
490 *
491 RCL 30
492 X>Y?
493 GTO 18
494 RCL 04
495 RCL 05
```

```
496 *
497 .015
498 *
499 RCL 30
500 X>Y?
501 GTO 18
502 GTO 19
503♦LBL 22
504 "AST<,075AS"
505 AVIEW
506 GTO 16
507♦LBL 23
508 "AST<,003AC"
509 AVIEW
510 PSE
511 1
512 "REV. AST? 0/1"
513 PROMPT
514 X=Y?
515 GTO 16
516 "MELAST? 0/1"
517 PROMPT
518 X=0?
519 GTO 19
520 GTO C
521♦LBL 18
522 "AR>,3ASTEEL"
523 AVIEW
524 GTO C
525♦LBL 24
526 "AR>,015AC"
527 AVIEW
528 1
529 "REV. AST? 0/1"
530 PROMPT
531 X=Y?
532 GTO 16
533 GTO 19
534♦LBL 19
535 240
536 RCL 02
537 /
538 SQRT
539 18
540 *
541 RCL 08
542 RCL 09
543 /
544 X>Y?
545 GTO C
546 240
547 RCL 02
548 /
549 SQRT
550 66
551 *
552 RCL 30
553 RCL 03
554 *
555 RCL 02
556 /
557 RCL 20
558 /
559 RCL 33
560 /
561 1
562 +
563 /
564 RCL 33
565 RCL 20
566 /
567 X>Y?
568 GTO C
569 RCL 02
570 RCL 10
571 *
572 RCL 17
573 RCL 03
574 *
575 RCL 30
576 *
577 -
578 2
579 /
580 RCL 02
581 /
582 STO 31
583 RCL 08
584 RCL 09
585 *
586 RCL 31
587 X>Y?
588 GTO 20
589 "PNA IN FLNG"
590 AVIEW
591 PSE
592 RCL 31
593 2
594 /
595 RCL 08
596 /
597 STO 32
598 GTO 21
599♦LBL 20
600 "PNA IN WEB"
601 AVIEW
602 PSE
603 RCL 31
604 RCL 08
605 RCL 09
606 *
607 -
608 RCL 20
609 /
610 STO 00
611 2
612 /
613 RCL 09
614 +
615 RCL 20
616 *
617 RCL 00
618 *
619 RCL 09
620 X↑2
621 .5
622 *
623 RCL 08
624 *
625 +
626 RCL 31
627 /
628 STO 32
629♦LBL 21
630 RCL 13
631 RCL 05
632 -
633 RCL 06
634 -
635 RCL 32
636 -
637 RCL 31
638 *
639 RCL 02
640 *
641 2
642 *
643 RCL 13
644 RCL 29
645 -
646 RCL 30
647 *
648 RCL 03
649 *
650 RCL 17
651 *
652 +
653 1 E-6
654 *
655 "MUH="
656 "⊢"
657 ARCL X
658 "⊢KN.M"
659 AVIEW
660 STOP
```

R7

PROGRAM LISTING

DESIGN CODE: SABS 0162

SHEET

```
661 GTO 17
662♦LBL C
663 "ELAST M:HOG"
664 AVIEW
665 RCL 34
666 206
667 /
668 1/X
669 STO 22
670 RCL 07
671 2
672 /
673 RCL 05
674 +
675 RCL 06
676 +
677 STO 13
678 RCL 10
679 RCL 13
680 *
681 RCL 30
682 RCL 29
683 *
684 +
685 RCL 10
686 RCL 30
687 +
688 /
689 STO 23
690 RCL 23
691 RCL 29
692 -
693 X↑2
694 RCL 30
695 *
696 RCL 13
697 RCL 23
698 -
699 X↑2
700 RCL 10
701 *
702 +
703 RCL 24
704 +
705 STO 25
706 RCL 07
707 RCL 05
708 +
709 RCL 06
710 +
711 RCL 23
712 -
713 RCL 25
714 /
715 RCL 02
716 /
717 1/X
718 RCL 23
719 RCL 29
720 -
721 RCL 25
722 /
723 RCL 03
724 /
725 1/X
726 X>Y?
727 X<>Y
728 1 E-6
729 *
730 "MeHOG="
731 "⊢"
732 ARCL X
733 "⊢KN.M"
734 AVIEW
735 STOP
736 GTO 17
737 .END.
```

PROGRAM LISTING

DESIGN CODE: EUROCODE 4 - PLASTIC DESIGN **SHEET**

```
01♦LBL "EC4"
02 "COMP DESIGN TO "
03 "⊢EUROCODE 4"
04 CLRG
05 GTO 28
06♦LBL 01
07 "FcU="
08 PROMPT
09 STO 01
10 "FY REINFT="
11 PROMPT
12 STO 03
13 "PY JOIST="
14 PROMPT
15 STO 02
16 RTN
17♦LBL 02
18 "CONC SECT"
19 AVIEW
20 PSE
21 "CONC: Be="
22 PROMPT
23 STO 04
24 "CONC:DS="
25 PROMPT
26 STO 05
27 "CONC:DP="
28 PROMPT
29 STO 06
30 RTN
31♦LBL 03
32 "STEEL SECT"
33 AVIEW
34 PSE
35 "STEEL: H1="
36 PROMPT
37 STO 07
38 "STEEL: b="
39 PROMPT
40 STO 08
41 "STEEL: T2="
42 PROMPT
43 STO 09
44 "STEEL: T1="
45 PROMPT
46 STO 12
47 "STEEL: A="
48 PROMPT
49 STO 10
50 "STEEL: H="
51 PROMPT
52 STO 14
53 RCL 07
54 RCL 12
55 /
```

```
56 STO 15
57 "ZPL="
58 PROMPT
59 STO 17
60 RCL 02
61 *
62 1.1
63 /
64 STO 16
65 RTN
66♦LBL 29
67 "AST="
68 PROMPT
69 STO 11
70 "d*="
71 PROMPT
72 RCL 05
73 -
74 CHS
75 STO 18
76 RTN
77♦LBL 04
78 "RESISTANCE"
79 "RC"
80 RCL 05
81 RCL 06
82 -
83 RCL 04
84 *
85 RCL 01
86 *
87 .472
88 *
89 STO 20
90 "RF"
91 RCL 08
92 RCL 09
93 *
94 RCL 02
95 *
96 1.1
97 /
98 STO 21
99 "RS"
100 RCL 10
101 RCL 02
102 *
103 1.1
104 /
105 STO 25
106 "RY"
107 RCL 07
108 RCL 12
109 *
110 RCL 02
```

```
111 *
112 1.1
113 /
114 STO 26
115 "$"
116 RCL 02
117 1/X
118 275
119 *
120 .5
121 Y↑X
122 STO 13
123 "RO"
124 RCL 13
125 40
126 *
127 RCL 12
128 Y↑X
129 *
130 RCL 02
131 *
132 1.1
133 /
134 STO 28
135 "RN"
136 RCL 25
137 RCL 26
138 -
139 RCL 28
140 +
141 STO 22
142 "RR"
143 RCL 11
144 RCL 03
145 *
146 .87
147 *
148 STO 24
149 "RW"
150 RCL 21
151 2
152 *
153 CHS
154 RCL 25
155 +
156 STO 27
157 "+VE SPAN MOM?"
158 PROMPT
159 1
160 X=Y?
161 GTO A
162 RCL 08
163 2
164 /
165 RCL 09
```

```
166 /
167 STO 00
168 RCL 13
169 7.4
170 *
171 RCL 00
172 X<=Y?
173 GTO "S"
174 RCL 13
175 9.24
176 *
177 RCL 00
178 X<=Y?
179 GTO "T"
180 RCL 13
181 13.87
182 *
183 RCL 00
184 X<=Y?
185 GTO "U"
186 "CLASS 4"
187 SF 01
188 GTO "V"
189♦LBL "S"
190 "CLASS 1"
191 GTO "V"
192♦LBL "T"
193 "CLASS 2"
194 GTO "V"
195♦LBL "U"
196 "CLASS 3"
197 SF 01
198♦LBL "V"
199 "⊢"
200 AVIEW
201 PSE
202 FS?C 01
203 GTO D
204 GTO C
205♦LBL A
206 RCL 27
207 RCL 20
208 X>Y?
209 GTO 06
210 "PNA IN WEB"
211 AVIEW
212 PSE
213 .999
214 RCL 20
215 RCL 26
216 /
217 X>Y?
218 X<>Y
219 RCL X
220 1
```

```
221 -
222 CHS
223 1/X
224 RCL 13
225 *
226 61
227 *
228 RCL 15
229 X<=Y?
230 GTO 07
231 GTO D
232♦LBL 07
233 "WEB COMP"
234 AVIEW
235 PSE
236 RCL 14
237 RCL 05
238 +
239 RCL 06
240 +
241 2
242 /
243 RCL 20
244 *
245 RCL 20
246 X↑2
247 RCL 26
248 /
249 RCL 07
250 *
251 4
252 /
253 -
254 RCL 16
255 +
256 GTO 10
257♦LBL 06
258 RCL 20
259 RCL 25
260 X>Y?
261 GTO 09
262 "PNA CONC FLNG"
263 AVIEW
264 PSE
265 RCL 05
266 RCL 06
267 -
268 2
269 /
270 RCL 25
271 *
272 RCL 20
273 /
274 RCL 05
275 -
```

```
276 CHS
277 RCL 14
278 2
279 /
280 +
281 RCL 25
282 *
283 GTO 10
284♦LBL 09
285 "PNA STL FLNG"
286 AVIEW
287 PSE
288 RCL 05
289 RCL 06
290 +
291 2
292 /
293 RCL 20
294 *
295 RCL 25
296 RCL 20
297 -
298 X↑2
299 RCL 21
300 /
301 RCL 09
302 *
303 4
304 /
305 -
306 RCL 25
307 RCL 14
308 *
309 2
310 /
311 +
312♦LBL 10
313 1 E-6
314 *
315 "Mc="
316 "⊢"
317 ARCL X
318 "⊢KN.M"
319 AVIEW
320 STOP
321 GTO 30
322♦LBL C
323 2
324 RCL 24
325 RCL 26
326 /
327 1
328 +
329 X>Y?
330 X<>Y
```

PROGRAM LISTING

DESIGN CODE: EUROCODE 4 - PLASTIC DESIGN SHEET

```
331 1/X
332 61
333 *
334 RCL 13
335 *
336 RCL 15
337 X<=Y?
338 GTO 15
339 "WEB NOT COMP"
340 AVIEW
341 PSE
342 GTO D
343♦LBL 15
344 RCL 27
345 RCL 24
346 X>Y?
347 GTO 17
348 "WEB COMP AND"
349 "⊢ PNA IN WEB"
350 AVIEW
351 PSE
352 RCL 14
353 2
354 /
355 RCL 18
356 +
357 RCL 24
358 *
359 RCL 16
360 +
361 RCL 24
362 X↑2
363 RCL 26
364 /
365 RCL 07
366 *
367 4
368 /
369 -
370 GTO 10
371♦LBL 17
372 RCL 25
373 RCL 24
374 X>Y?
375 GTO 19
376 "WEB COMP AND"
377 "⊢PNA IN ST FLG"
378 AVIEW
379 PSE
380 RCL 25
381 RCL 14
382 *
383 2
384 /
385 RCL 24
386 RCL 18
387 *
388 +
389 RCL 25
390 RCL 24
391 -
392 X↑2
393 RCL 21
394 /
395 RCL 09
396 *
397 4
398 /
399 -
400 GTO 10
401♦LBL 19
402 "WEB COMP AND PN"
403 "⊢A OUTSIDE STBM"
404 AVIEW
405 PSE
406 RCL 14
407 2
408 /
409 RCL 18
410 +
411 RCL 25
412 *
413 GTO 10
414♦LBL 28
415 FS? 04
416 GTO 30
417 XEQ 01
418 XEQ 02
419 XEQ 03
420 XEQ 29
421 "RUN?"
422 PROMPT
423 1
424 X=Y?
425 XEQ 04
426♦LBL 30
427 "VARIABLES"
428 AVIEW
429 PSE
430 "M:1 C:2 S:3 A:2"
431 "⊢9"
432 PROMPT
433 X>0?
434 XEQ IND X
435 "MORE CHANGS?"
436 PROMPT
437 1
438 X=Y?
439 GTO 30
440 XEQ 04
441♦LBL D
442 "USE ELASTIC ANA"
443 "⊢LYSIS"
444 AVIEW
445 STOP
446 GTO 30
447 .END.
```

PROGRAM LISTING

DESIGN CODE: EUROCODE 4 - ELASTIC DESIGN **SHEET**

```
01♦LBL "EC4ELAS"
02 "ELASTIC ANAL OF"
03 "⊢COMPOSITE BEAM"
04 "⊢ TO EC4"
05 CLRG
06 GTO 28
07♦LBL 01
08 "FcU="
09 PROMPT
10 STO 01
11 "FY REINFT="
12 PROMPT
13 STO 03
14 "PY JOIST="
15 PROMPT
16 1.1
17 /
18 STO 02
19 RTN
20♦LBL 02
21 "CONC SECT"
22 AVIEW
23 PSE
24 "CONC: Be="
25 PROMPT
26 STO 04
27 "CONC: DS="
28 PROMPT
29 STO 05
30 "CONC: DP="
31 PROMPT
32 STO 06
33 RTN
34♦LBL 03
35 "STEEL SECT"
36 AVIEW
37 PSE
38 "STEEL: H1="
39 PROMPT
40 STO 07
41 "STEEL: b="
42 PROMPT
43 STO 08
44 "STEEL: T2="
45 PROMPT
46 STO 09
47 "STEEL: T1="
48 PROMPT
49 STO 12
50 "STEEL: A="
51 PROMPT
52 STO 10
53 "STEEL: I="
54 PROMPT
55 STO 19
56 "STEEL: H="
57 PROMPT
58 STO 14
59 RCL 07
60 RCL 12
61 /
62 STO 15
63 "∡e MOD RAT.="
64 PROMPT
65 STO 29
66 RTN
67♦LBL 29
68 "AST="
69 PROMPT
70 STO 11
71 "d*="
72 PROMPT
73 RCL 05
74 -
75 CHS
76 STO 18
77 RTN
78♦LBL 28
79 FS? 04
80 GTO 30
81 XEQ 01
82 XEQ 02
83 XEQ 03
84 XEQ 29
85 "   RUN?"
86 PROMPT
87 1
88 X=Y?
89 XEQ 04
90♦LBL 30
91 "VARIABLES"
92 AVIEW
93 PSE
94 "M:1 C:2 S:3 A:2"
95 "⊢9"
96 PROMPT
97 X>0?
98 XEQ IND X
99 "MORE CHANGS?"
100 PROMPT
101 1
102 X=Y?
103 GTO 30
104♦LBL 04
105 "RS"
106 RCL 10
107 RCL 02
108 *
109 STO 25
110 "$"
111 RCL 02
112 1/X
113 275
114 *
115 .5
116 Y↑X
117 STO 13
118 RCL 14
119 RCL 05
120 +
121 RCL 06
122 +
123 X↑2
124 RCL 05
125 RCL 06
126 -
127 *
128 RCL 04
129 *
130 RCL 10
131 *
132 4
133 /
134 RCL 05
135 RCL 06
136 -
137 RCL 04
138 *
139 RCL 10
140 RCL 29
141 *
142 +
143 /
144 RCL 05
145 RCL 06
146 -
147 3
148 Y↑X
149 RCL 04
150 *
151 12
152 /
153 RCL 29
154 /
155 +
156 RCL 19
157 +
158 STO 30
159 "IN"
160 RCL 18
161 2
162 *
163 RCL 14
164 +
165 X↑2
```

PROGRAM LISTING

DESIGN CODE: EUROCODE 4 - ELASTIC DESIGN **SHEET**

```
166 RCL 11
167 *
168 RCL 10
169 *
170 4
171 /
172 RCL 10
173 RCL 11
174 +
175 /
176 RCL 19
177 +
178 STO 31
179 "Ye"
180 RCL 05
181 2
182 *
183 RCL 14
184 +
185 STO 00
186 RCL 04
187 *
188 RCL 10
189 /
190 RCL 29
191 /
192 1
193 +
194 SQRT
195 1
196 +
197 1/X
198 RCL 00
199 *
200 STO 33
201 "IP"
202 RCL 05
203 RCL 33
204 -
205 RCL 14
206 2
207 /
208 +
209 X↑2
210 RCL 10
211 *
212 RCL 33
213 3
214 Y↑X
215 RCL 04
216 *
217 3
218 /
219 RCL 29
220 /
221 +
222 RCL 19
223 +
224 STO 32
225 "+VE MOM? 1/0"
226 PROMPT
227 X=0?
228 GTO 20
229 "ELAST NA"
230 AVIEW
231 RCL 05
232 RCL 06
233 -
234 X↑2
235 RCL 04
236 *
237 RCL 14
238 RCL 06
239 2
240 *
241 +
242 RCL 29
243 *
244 /
245 RCL 10
246 X<Y?
247 GTO 21
248 GTO 22
249♦LBL 21
250 "ENA IN CONC FLN"
251 RCL 32
252 RCL 29
253 *
254 RCL 33
255 /
256 STO 34
257 RCL 14
258 RCL 05
259 +
260 RCL 33
261 -
262 1/X
263 RCL 32
264 *
265 STO 35
266 GTO 23
267♦LBL 22
268 "ENA IN STEEL"
269 RCL 05
270 RCL 06
271 -
272 X↑2
273 RCL 04
274 *
275 RCL 05
276 2
277 *
278 RCL 14
279 +
280 RCL 29
281 *
282 RCL 10
283 *
284 +
285 2
286 /
287 RCL 05
288 RCL 06
289 -
290 RCL 04
291 *
292 RCL 10
293 RCL 29
294 *
295 +
296 /
297 STO 00
298 RCL 30
299 RCL 29
300 *
301 RCL 00
302 /
303 STO 34
304 RCL 14
305 RCL 05
306 +
307 RCL 00
308 -
309 1/X
310 RCL 30
311 *
312 STO 35
313♦LBL 23
314 RCL 34
315 "Fc CONCRETE?"
316 RCL 01
317 *
318 .472
319 *
320 RCL 35
321 RCL 02
322 *
323 X>Y?
324 X<>Y
325 GTO 27
326♦LBL 20
327 "-VE MOM"
328 "YR"
329 RCL 18
330 2
```

```
331 *
332 RCL 14
333 +
334 RCL 10
335 *
336 2
337 /
338 RCL 10
339 RCL 11
340 +
341 /
342 STO 37
343 RCL 31
344 RCL 37
345 /
346 STO 34
347 RCL 14
348 RCL 18
349 +
350 RCL 37
351 -
352 1/X
353 RCL 31
354 *
355 STO 35
356 RCL 14
357 RCL 05
358 +
359 RCL 18
360 -
361 RCL 37
362 -
363 STO 00
364 RCL 02
365 RCL 37
366 *
367 RCL 00
368 /
369 RCL 03
370 .87
371 *
372 X<>Y
373 X>Y?
374 X<>Y
375 RCL 11
376 *
377 STO 38
378♦LBL 25
379 RCL 14
380 RCL 37
381 -
382 RCL 18
383 +
384 CHS
385 1/X
386 RCL 37
387 RCL 18
388 -
389 *
390 STO 00
391 -1
392 X>Y?
393 GTO 32
394 0
395 ENTER↑
396 RCL 00
397 X>Y?
398 GTO 33
399 RCL 00
400 X↑2
401 10
402 *
403 RCL 00
404 6.26
405 *
406 -
407 7.64
408 +
409 STO 00
410 GTO 34
411♦LBL 33
412 RCL 00
413 1.1
414 +
415 1/X
416 8.4
417 *
418 STO 00
419♦LBL 34
420 RCL 00
421 SQRT
422 19.54
423 *
424 RCL 13
425 *
426 RCL 15
427 X>Y?
428 GTO 26
429 "WEB NOT SLENDER"
430 AVIEW
431 RCL 13
432 2
433 *
434 13.87
435 *
436 RCL 08
437 RCL 09
438 /
439 X>Y?
440 GTO 31
441 "FLNG NOT SLNDR"
442 AVIEW
443 RCL 34
444 RCL 03
445 *
446 .87
447 *
448 RCL 35
449 RCL 02
450 *
451 X>Y?
452 X<>Y
453♦LBL 27
454 1 E-6
455 *
456 "Me="
457 "⊢ "
458 ARCL X
459 "⊢KN.M"
460 AVIEW
461 STOP
462 GTO 30
463♦LBL 26
464 "WEB SLENDER: RE"
465 "⊢DUCE ALLOW STR"
466 "⊢ESS"
467 AVIEW
468 GTO 30
469♦LBL 31
470 "FLNG SLENDER: R"
471 "⊢REDUCE ALLOW S"
472 "⊢TRESS"
473 AVIEW
474 GTO 30
475♦LBL 32
476 "STRESS RATIO>-1"
477 AVIEW
478 GTO 30
479 .END.
```

APPENDIX B: TEST BEAM DATA

TITLE: INELASTIC BUCKLING OF COMPOSITE BRIDGE GIRDERS NEAR INTERNAL SUPPORTS **REF 3**

AUTHORS M.A. BRADFORD, R.P. JOHNSON

DATE OF PUBLICATION: MARCH 1987

In this paper a finite strip method of inelastic analysis is used to predict the distortional and local buckling, bending moments of eleven composite beams of differing cross-sections under uniformly distributed load. The cross-section geometry and lengths of the beams used in the parametric study were selected to be within the range of beams that would be more suitable for bridge girders than beams in buildings.

The predictions by the finite strip method of analysis were compared to tests on locally buckling composite beams conducted by Hamada and Longworth (16) and by Ansourian (14) and good agreement between theory and test results were found.

Although the dimensions of the sections used in the parametric study are more suitable for bridge girders, it would test the different design codes performance with sections bordering on or in the slender range. Only local buckling moments, Mc, have been compared since distortional buckling would not be applicable to beams in building structures.

Four of the sections analysed were considered suitable for inclusion in this paper.

MATERIAL PROPERTIES

CONCRETE SLAB : 1000 X 220 : f_{cu} = 25 MPa
STEEL SECTION : p_y = 338 MPa
REINFORCEMENT : f_y = 400 MPa

GEOMETRIC PROPERTIES

SPECIMEN	h mm	b mm	T_1 mm	T_2 mm	H_1 mm	A $x10^n mm^2$	Z PLASTIC $x10^6 mm^n$	I $x10^9 mm^4$
RS 3	903	303	15.2	20.2	863	25.4	8.2	3.202
PG 1	1800	800	30.0	60.0	1680	146.4	103.2	84.55
PG 5	899	390	15.2	15.8	867	25.5	8.29	3.227
PG 7	1854	1000	30.0	50.0	1754	152.9	112.3	94.87

REINFORCMENT

SPECIMEN	Ast mm^2	d* mm
RS 3	9900	110
PG 1	12960	150
PG 5	9900	110
PG 7	12960	150

SUMMARY OF HOGGING MOMENT CAPACITIES (kNm)

BEAM REF	M TEST	MP	ULTIMATE MOMENT CAPACITY PREDICTED BY:				
			BS 5950 (D)	SABS 0162	EUROCODE 4	CSA	BS 5950 (F)
RS 3	3752	4232	3495	*2812	*2749	3809	*3024
PG 1	39081	39248	38935	*30748	*30060	35323	38387
PG 5	3355	4255	*2841	*2832	*2768	3830	*2841
PG 7	37815	42463	*35904	*33590	*32640	38217	*35904

* ELASTIC ANALYSIS AS REQUIRED BY DESIGN CODE

In all the above cases, the sections failed to reach the simple plastic moment capcity due to section slenderness.

BS 5950: For RS 3, BS 5950 (D) allows plastic analysis with a reduced effective web depth and the predicted ultimate moment capacity is well within the MTEST. BS 5950 (F) requires that elastic analysis be used.

For PG 1, full plastic analysis is allowed by BS 5950 (D) whereas BS 5950 (F) requires elastic analysis with reduced effective web depth.

SABS 0162 & Elastic moment capacity used throughout.
EUROCODE 4

CSA-S16.1 The full plastic moment capacity is used throughout, exceeding the test moment capacity in three cases.

TITLE: LOCAL BUCKLING IN CONTINUOUS COMPOSITE BEAMS **REF 4**

AUTHORS J. CLIMENHAGA
R. JOHNSON

DATE OF PUBLICATION: SEPTEMBER 1972

The object of the tests described in this paper was to provide more information on LOCAL BUCKLING and its effects on rotational capacity in the negative moment regions of continuous composite beams, and more specifically, (to determine whether limits of slenderness ratios of flanges & webs of normal steel sections in the negative moment regions are applicable to continuous composite beams).

For simplicity, the hogging moment region in a continuous beam was simulated by testing a specimen in a double cantilever fashion with point loads applied at the ends and centre only. A total of eighteen hot-rolled I-sections were tested, fifteen of which had simulated reinforced concrete slabs by replacing the slab with a rolled steel flat attached to the joist by intermittent welds. Excluding beams with stiffened webs and duplicated sections, only 9 specimens tested were suitable for inclusion in this investigation.

The principal variables were the web slenderness, the flange slenderness and the force ratio ϕ = Ast.Fy/A.Py. Buckling is more severe for large values of ϕ, so values used in the tests were close to the upper end of the practical range, which is from zero to about 0,4.

Behaviour of specimens could be divided roughly into two groups, those in which the moment at failure exceeded the fully plastic moment capacity Mp and those in which elastic-plastic web buckling led to a very sudden unloading before the bending moment reached Mp.

Results predicted by the four design codes followed a trend of increased accuracy with an increase in member slenderness for members in group A. As can be seen, the moments predicted by BS 5950 were the most consistent and least conservative.

MATERIAL AND SECTION PROPERTIES

BEAM REF	SECTION	CONCRETE SLAB		REINF.	d*	ALLOWABLE STRESSES		
		b eff mm	thickness mm	mm²	mm	f_{cu} MPa	p_y' MPa	f_{st} MPa
SB3	203 X 133 X 30	#114	19	2177	9.5	N/A	298	234
HB40	203 X 133 X 30	762	105	1144	53	45	298	416
SB2	203 X 133 X 25	#114	19	2177	9.5	N/A	325	202
HB41	203 X 133 X 25	762	105	674	53	45	331	427
SB14	305 X 102 X 33	#89	19	1694	9.5	N/A	348	232
SB4	305 X 102 X 25	#89	25.4	2258	12.7	N/A	386	197
SB10	305 X 102 X 25	#76	19	1453	9.5	N/A	386	240
SB5	14" X 5" UB22	#114	25.4	2903	12.7	N/A	343	217
SB11	14" X 5" UB22	#114	12.7	1452	6.4	N/A	343	227
SB6	406 X 140 X 39	#114	25.4	2903	12.7	N/A	340	219

ROLLED STEEL FLATS WELDED TO TOP FLANGE SIMULATING REINFORCED CONCRETE SLABS

SUMMARY OF HOGGING MOMENT CAPACITIES (kNm)

BEAM REF	M TEST	MP	ULTIMATE MOMENT CAPACITY PREDICTED BY:				
			BS 5950 (D)	SABS 0162	EUROCODE 4	CSA	BS 5950 (F)
SB3	139.6	119.5	118.3	*89.2	108.3	107.6	118.3
HB40	154.3	139.1	135.4	*99.7	125.4	125.2	135.4
SB2	117.5	108.4	107.4	*81.6	*79.74	97.6	107.4
HB41	121	115.9	112.8	*88.0	*86.00	104.3	112.8
SB14	195.5	215.7	211.3	*155.6	*152.1	194.1	211.3
SB4	167	223.1	162.5	*128.5	*125.6	200.8	*138.1
SB10	174	213.2	164.0	*122.6	*120.0	191.9	159.5
SB5	218	253.1	218.7	*179.9	*176.0	227.8	*193.4
SB11	212	230.5	219.7	*168.8	*165.0	207.5	214.9
SB6	291.5	331.4	279.2	*234.7	*229.5	298.3	*252.41

* MOMENT CAPACITY BY ELASTIC ANALYSIS AS REQUIRED BY DESIGN CODE

TITLE: BUCKLING OF COMPOSITE BEAMS IN NEGATIVE BENDING **REF 16**

AUTHORS S. HAMADA
J. LONGWORTH

DATE OF PUBLICATION: NOVEMBER 1974

Climenhaga and Johnson (4) studied local buckling of continuous composite beams in the negative moment region. In this paper, local flange buckling and lateral buckling were investigated.

Nineteen composite beams were tested. Again the hogging moment region in a continuous beam was simulated by testing specimens in a double cantilever fashion with point loads applied at ends and centre only. Each beam consisted of a wide-flange steel section, a concrete slab with longitudinal and transverse reinforcement and shear connectors. Slab width varied from 800 to 1800 mm, but the major variable was the longitudinal reinforcement (0 -> 2400 mm^2). The amount of longitudinal slab reinforcement, however, was limited to the effective web area of the steel section. It was felt that increasing the longitudinal slab reinforcement above this amount would not significantly increase the ultimate moment capacity.

It was concluded from the results that the ultimate moment capacity of composite beams in negative bending is affected by local flange buckling unless the compression flange is stiffened by a cover plate.

For the sets of beams 21-25 and 31-34, the area of tension reinforcement was limited to 2433 mm^2 and 2028 mm^2 respectively.

To test the performance of the codes with areas of reinforcement beyond these limits, hypothetical sections were analysed with increased area of reinforcement.

MATERIAL AND SECTION PROPERTIES

BEAM	SECTION	CONCRETE SLAB		REINFT.	d*	ALLOWABLE STRESSES		
REF		b eff mm	thickness mm	mm²	mm	f_{cu} MPa	p_y MPa	F_{st} MPa
18	305 X 165 X 54	-	-	-	-	-	275	373
11	305 X 165 X 54	1219	102	772	50	36	275	373
12	305 X 165 X 54	1219	102	1282	50	33	275	373
13	305 X 165 X 54	1219	102	1622	50	37	275	373
14	305 X 165 X 54	1219	102	2433	50	29	275	373
21	305 X 165 X 46	-	-	-	-	-	284	341
22	305 X 165 X 46	1219	102	523	50	28.5	284	341
23	305 X 165 X 46	1219	102	1217	50	22.0	284	341
24	305 X 165 X 46	1219	102	1622	50	32.0	284	341
25	305 X 165 X 46	1219	102	2433	50	23.0	284	341
H	305 X 165 X 46	1219	102	3000	50	23.0	284	341
H	305 X 165 X 46	1219	102	3500	50	23.0	284	341
H	305 X 165 X 46	1219	102	4000	50	23.0	284	341
H	305 X 165 X 46	1219	102	4500	50	23.0	284	341
31	305 X 165 X 41	-	-	-	-	-	-	-
32	305 X 165 X 41	1219	102	523	50	18	334	341
33	305 X 165 X 41	1219	102	1047	50	28	334	341
34	305 X 165 X 41	1219	102	2028	50	24	334	341
H	305 X 165 X 41	1219	102	2500	50	24	334	341
H	305 X 165 X 41	1219	102	3000	50	24	334	341
H	305 X 165 X 41	1219	102	3500	50	24	334	341

H - Hypothetical section

SUMMARY OF TEST AND PREDICTED HOGGING MOMENT CAPACITIES (kNm)

BEAM REF	M TEST	MP	ULTIMATE MOMENT CAPACITY PREDICTED BY:				
			BS 5950 (D)	SABS 0162	EUROCODE 4	CSA	BS 5950 (F)
18	328	231.9	231.9	215.7	210.8	208.7	231.9
11	375	281.9	276.5	258.3	254.7	253.7	276.5
12	389	304.1	297.8	279.4	* 216.9	273.7	297.8
13	393	314.2	308.4	* 226.9	* 221.8	282.8	308.4
14	399	333.7	326.2	* 236.4	* 231.0	300.3	326.2
21	288	204.9	204.9	190.6	186.3	184.4	204.9
22	323	237.4	233.6	217.3	214.7	213.7	233.6
23	338	267.6	262.0	* 198.6	* 195.7	240.8	262.0
24	340	278.4	273.0	* 204.2	* 201.5	250.6	* 219.5
25	328	295.7	288.5	212.6	* 210.1	265.4	* 228.6
-	-	307.3	298.8	* 217.1	* 212.2	274.9	* 233.4
-	-	317.2	307.6	* 220.3	* 215.3	283	* 236.9
-	-	326.8	316.2	* 223.0	* 218.0	291	* 239.8
-	-	336.0	324.5	* 225.3	* 220.3	299	* 242.3
31	257	209.2	209.2	194.6	190.2	188.3	209.2
32	271	241.7	237.9	* 168.6	* 168.7	217.5	237.9
33	292	266.4	260.7	* 203.0	* 198.4	239.8	260.7
34	305	293.2	274.7	* 216.7	* 211.8	263.9	* 233.0
-	-	302.9	283.2	* 221.2	* 216.3	272.1	* 237.9
-	-	313.0	292.0	* 225.2	* 220.1	280.5	* 242.1
-	-	322.9	300.6	* 228.4	* 223.4	288.8	* 245.6

* ELASTIC ANALYSIS AS REQUIRED BY DESIGN CODE

\- NO TEST RESULTS AVAILABLE HYPOTHETICAL SECTIONS ANALYSED

TITLE: TESTS ON THREE THREE-SPAN CONTINUOUS BEAMS **REF 13**

AUTHORS M.C. HOPE-GILL
R.P. JOHNSON

DATE OF PUBLICATION: JUNE 1976

In this study, three full scale, three span, continuous composite beams have been tested to failure to assess the limitations on the use of plastic design methods for composite beams.

Previous studies were concerned largly with the ultimate strength of critical sections in flexure, here the continuous beam as a whole is examined. Two of the beams, CB10 and CB11, were chosen to fail prematurely by crushing of concrete in mid-span (ie. insufficient rotation capacity) and the third, CB12, to fail by local buckling at the internal supports.

The specimens failed as predicted yielding ultimate moment capacities greater than the calculated plastic moment capacities.

Sections tested were considered compact to all the codes of practice, except SABS 0162 which considers CB10 and CB12 non-compact due to reinforcement limitations over supports.

BEAM REF	SECTION	CONCRETE SLAB		REINF.	d*	ALLOWABLE STRESSES		
		b eff mm	thickness mm	mm²	mm	f_{cu} MPa	p_y MPa	f_{st} MPa
CB10	203 X 133 X 30	915	76	993	29	37	276	457
CB11	203 X 133 X 30	915	76	792	25	19	301	436
CB12	203 X 133 X 30	1220	102	1018	28	37	267	469

SUMMARY OF HOGGING MOMENT CAPACITY (kNm)

BEAM REF	M TEST	MP	ULTIMATE MOMENT CAPACITY PREDICTED BY:				
			BS 5950 (D)	SABS 0162	EUROCODE 4	CSA	BS 5950 (F)
CB10	150	127	124	*89	115	115	124
CB11	155	132	129	121	112	119	118
CB12	179	138	133	*95	124	124	133,0

*Elastic analysis - section slenderness exceeds limits for use of plastic analysis.

SUMMARY OF SAGGING MOMENT CAPACITY (kNm)

BEAM REF	M TEST	MP	ULTIMATE MOMENT CAPACITY PREDICTED BY:			
			BS 5950	SABS 0162	EUROCODE 4	CSA
CB10	200	162.5	151.9	139.7	142.6	138.2
CB11	185	147.5	138.8	128.0	130.0	126.1
CB12	196	189.5	183.1	169.2	169.5	165.9

TITLE: EXPERIMENTS ON CONTINUOUS COMPOSITE BEAMS **REF 14**

AUTHORS P. ANSOURIAN

DATE OF PUBLICATION: DECEMBER 1981

The object of experiments described in this paper was to give further information on the ultimate behaviour of continuous composite beams when the steel joist is compact, and to assess the effect of cross-section ductility on the ultimate rotation of hinges.

Tests were conducted on six, two span continuous composite beams, each beam with a total length of 9,0 m. The major variables were the load configuration the slab width and geometry of the steel joist.

Two beams (CTB1 and CTB2) were tested to provide information on sagging rotation. The spans were 4 m and 5 m long with a single point load at the centre of the 4 m span. For beam CTB1, the midspan plastic moment was exceeded, but insufficient rotation capacity prevented the plastic hogging moment from developing. This accounts for the discrepancy between the predicted ultimate moment capacity and the actually obtained in the test. CTB2 was similar to CTB1 but with a wider slab and hence greater rotation capacity allowing fully plastic sagging moment capacity and in excess of 97% plastic hogging moment capacity to be reached.

Beams CTB3 and CTB4, selected for the slenderness of their steel compression flange and CTB5 and CTB6, selected for their web proportions, were symmetric and loaded by concentrated loads at the centre of both spans in order to investigate hogging hinges.

In the calculation of Mp, the characteristic compressive stress for concrete is taken as 0.67 fcu. For the steel section, the yield stress is taken as the yield stress of the flange in keeping with tests by other authors. In the comparison of results by Ansourian (14), an average of the web and flange lower yield stress was used to calculate the simple plastic moment.

BEAM REF	SECTION	CONCRETE SLAB		REINF.	d*	ALLOWABLE STRESSES		
		b eff mm	thickness mm	mm²	mm	f_{cu} MPa	p_y MPa	f_{st} MPa
CTB 1	IPE 200	800	100	1116	40	30	277	430
CTB 2	IPE 200	1300	100	1700	40	50	277	430
CTB 3	IPBL 200	1300	100	1700	40	43	220	430
CTB 4	IPBL 200	800	100	1571	50	34	236	430
CTB 5	IPE 240	1300	100	1730	40	29	265	430
CTB 6	IPE 240	1300	100	2027	45	41	292	430

SUMMARY OF HOGGING MOMENT CAPACITY : (kNm)

BEAM REF	M TEST	MP	ULTIMATE MOMENT CAPACITY PREDICTED BY:				
			BS 5950 (D)	SABS 0162	EUROCODE 4	CSA	BS 5950 (F)
CTB 2	119	122.8	116.9	* 75.3	109.9	110.5	116.9
CTB 3	190	155.1	148.9	*105.1	139.2	139.5	148.9
CTB 4	160	152.5	147.5	*107.3	*104.9	137.3	147.5
CTB 5	187	168.3	162.0	*109.4	151.1	151.5	162.0
CTB 6	205	184.4	177.7	*120.5	*117.8	166.0	177.7
CTB 1	(74)	106.9	102.8	* 71.0	95.9	96.2	102.8

* ELASTIC ANALYSIS

SUMMARY OF SAGGING MOMENT CAPACITY (kNm)

BEAM REF	M TEST	MP	ULTIMATE MOMENT CAPACITY PREDICTED BY:			
			BS 5950	SABS 0162	EUROCODE 4	CSA
CTB 1	166	138.5	129.0	118.8	120.8	117.4
CTB 2	184	146.3	147.2	136.5	135.1	133.0
CTB 3	250	212	203.0	187.6	187.9	183.8
CTB 4	217	203	181.8	166.5	173.2	166.4
CTB 5	232	206.7	196.3	181.2	182.3	178.0
CTB 6	254	232.9	224.0	207.2	206.9	202.8

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10.1 NOTATION

For the purpose of this thesis, the following symbols apply.

A	Area of steel beam
A_c	Area of concrete flange
A_r, A_{st}	Area of reinforcement in the effective cross-section
b	Half width of steel flange
B_e	Total effective width of concrete flange
b_e	Effective width of concrete flange, one side of steel beam
d	Clear depth of web
d^*	Distance from top of concrete slab to centroid of reinforcement
D_p	Overall depth of profiled steel sheet
D_s	Overall depth of slab
f_{cu}	Characteristic cube strength of concrete (MPa)
f_y, f_{st}	Characteristic strength of reinforcement (MPa)
I	Second moment of area of steel beam about major axis
L	Length of span
L_z	Distance between points of zero moment
M_p	Simple plastic moment capacity
M_T	Moment capacity at failure established in tests
p_y	Specified minimum yield strength of structural steel (MPa)
r	Ratio of mean longitudinal stress in web to p_y

t, T_1	Web thickness
T, T_2	Flange thickness
Z_{pl}	The plastic section modulus of the steel section
β	The concrete flange effective breadth ratio
k_σ	the buckling value obtained from linear buckling theory
ϵ	Constant, equal to $(275/p_y)^{\frac{1}{2}}$

10.2 DEFINITIONS

For the purposes of this dissertation, the following definitions apply.

Composite section :A steel beam which acts compositely with a concrete slab due to shear interconnection between the beam and slab.

Concrete flange :The reinforced concrete slab forming part of a floor or roof of a structure acting compositely with the steel beam.

Global analysis :Analysis of the structure to determine the bending moments and shear forces in the members.

Section or sectional analysis :Analysis of an effective composite cross-section to determine the moment capacity of the section.

Effective slab width :The width of concrete flange, measured perpendicular to the span of the beam, considered effective in the composite section.

Hogging moment :Moment causing compression at the bottom of the beam.

Sagging moment :Moment causing tension in the bottom of the beam.

Moment redistribution :The redistribution of bending moments in hogging and sagging regions in relation to the distribution of moments expected from elastic analysis.

Plastic hinge :A point in a beam at which the section is fully plastic under a bending moment and any attempt to increase this moment will cause the member to act as if hinged at that point.

Simple plastic moment :Moment capacity allowing for redistribution of stress within a cross-section and based on the measured strength of the materials as the design strength, ie. the material partial safety factor = 1.

Class 1 Plastic cross-section :A cross-section which can develop a plastic hinge with sufficient rotation capacity to allow redistribution of bending moments within the structure.

Class 2 Compact cross-section :A cross-section which can develop the plastic moment capacity of the section but in which local buckling prevents rotation at constant moment.

Class 3 Semi-compact cross-section :A cross-section in which the stress in the extreme fibres should be limited to yield because local buckling would prevent development of the plastic moment capacity in the section.

Class 4 Slender cross-section :A cross-section in which yield of the extreme fibres cannot be attained because of premature local buckling.

Shear connector :A mechanical device providing interconnection between the steel section and concrete slab.

Full shear connection :Sufficient shear interconnection between the steel section and concrete slab to provide full composite action and allowing full moment capacity of the composite section to develop.