

Imperial College of Science, Technology and Medicine

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VOLUME 1

LONG SPAN COMPOSITE FLOOR SYSTEM DESIGN

by

THOMAS M. LEIPER BSc(Hons) MSc DIC

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ABSTRACT

LONG SPAN COMPOSITE FLOOR SYSTEM DESIGN

Design procedures for long span composite floor and roof beams, which incorporate a number of novel steel sections, are developed and explained. These design procedures have been developed to the best available industry practice, and are assembled from information scattered throughout the literature on composite construction which has been critically reviewed by the writer, using not only the findings of the modern literature but also by exercising his engineering judgement from thirty years of practice in steel design and fabrication.

The procedures are computerized in spreadsheet formats which allow easy adjustment of the many input variables and some special options peculiar to composite construction. The computer solutions, which can generate quick and comprehensive designs for the floor and roof framing of multi-storey buildings and other structures, use alternative framing systems and a number of novel steel shapes.

The results of three hundred design studies on composite beams are reported and compared. The studies, which are made at beam spans ranging from 9 to 24 metres and steel section span-depth ratios of 10 to 25, use material properties, loading values, and floor framing arrangements commonly employed in British multi-storey building practice.

The work is presented in two volumes.

Volume 1: In this section; earlier design studies at Imperial College and their shortcomings are described; the new studies and the method of assessing their adequacy at the Ultimate and Serviceability Limit States are explained; the use made of codes and other literature is described; the design formulae used in member analysis are developed; the results from the design studies are discussed and conclusions drawn.

Volume 2: This volume is presented in eight parts. Part 1 is a synopsis of the volume and gives rules for interpreting the spreadsheet design procedure. The format of Parts 2 to 8 is the same and each part includes: a set of manual design calculations; the cell formulae used in the spreadsheet computer programme; the results of the design studies; and a designated study in spreadsheet display format which is used as an example to demonstrate the manual calculations and spreadsheet cell formulae.

The steel sections investigated in Parts 2 to 8 are:

Part 2: Universal Rolled Sections.

Part 3: Light Plate Girders of Constant Depth.

Part 4: Light Plate Girders with a Constant Taper.

Part 5: Light Plate Girders with a Modified Taper.

Part 6: Universal Beams with Unreinforced Web Openings.

Part 7: Universal Beams with Reinforced Web Openings.

Part 8: Warren Trusses with Hollow Structural Chords.

The studies reported here use criteria and design procedures which are similar to those adopted for the new British code of practice on composite construction (BS 5950: Part 3.1). They have also made a substantial contribution to the Steel Construction Institute's recent publication "Design of Fabricated Composite Beams in Buildings".

FOREWORD

LONG SPAN COMPOSITE FLOOR SYSTEM DESIGN

This is a work of design in which the writer, having critically reviewed and analysed the readily available and relevant information on composite construction, has tried to assemble the best of current industry practice and design theory, and incorporate this knowledge in the design procedures developed within this work.

While most of the design criteria and formulae, used in the procedures, are taken from modern design codes and recent literature on composite construction, there are aspects of the work in which the writer has imposed his engineering judgement and an experience based on thirty years of steel design and fabrication. After all, design is often as much an art as a science!

The central objective of the design procedures, which are presented in a computer spreadsheet format, is to give the user freedom of choice in the design process. The procedures allow the designer to investigate quickly the use of span lengths, alternative steel sections, and floor framing arrangements which are not as yet commonly applied in British structural practice.

The procedures could also be used as teaching tools at the under-graduate and post-graduate levels where time does not always allow member refinement and the use of options such as partial shear connection, variations in stud distribution, asymmetry of the cross-section, economic span-depth ratio, and other variables. They could also be used as mathematical models for experimental work in composite construction.

However, the cogent argument for the work is the movement of the steel construction industry to buildings with long span, column free framing systems. It is the writer's contention that this work will go some way to meet this development.

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SYMBOLS

UNITS

A_{bt}	area of tee below web opening	mm ²
A_{cc}	area of concrete in compression	mm ²
A_{cf}	transformed area of concrete	mm ²
A_{cs}	area of concrete in shear plane	mm ²
A_{ctc}	area of composite top chord.	mm ²
A_{em}	equivalent area of generalized mass	m ²
A_f	area of flange of steel section	mm ²
A_o	peak acceleration	%g
A_{ot}	permissible peak acceleration.	%g
A_r	area of reinforcing flat bars above and below web opening	mm ²
A_{ro}	area of steel removed at web opening	mm ²
A_s	area of steel section.	mm ²
A_{so}	net area of steel section at web opening	mm ²
A_{sc}	area of steel section in compression	mm ²
A_{se}	effective area of steel section	mm ²
A_{tt}	area of tee above web opening	mm ²
A_v	shear area of steel section	mm ²
A_1	factor used to calculate V_{1t}	--
B_b	width of steel flange	mm
B_{dy}	effective flange width of composite section in dynamic analysis	mm
B_e	effective flange width of composite section	mm
B_r	width of reinforcing flat bars at web opening	mm
B_1	factor used to calculate V_{1t}	--
C_{lp}	centres of load points	mm
C_{pb}	centres of primary beams	mm
$\%C_o$	percentage shear stud connection	%
D_b	depth of steel section	mm
D_c	length of web with axial load	mm
D_{ca}	calculated camber	mm
D_{cc}	distance between centroids of chords of steel truss	mm
D_{co}	calculated camber due to web opening	mm
D_{ct}	distance between centroids of chords of composite truss	mm
D_{fn}	deflection for SLS case n	mm
D_{fno}	deflection for SLS case n due to web opening	mm
D_{fs}	shrinkage deflection	mm

$D_{\max}$	service deflection	mm
D_{pc}	deflection due to partial shear connection	mm
D_s	diameter of shear stud.	mm
D_t	depth of steel truss	mm
D_{vo}	deflection at web opening due to Vierendeel action	mm
D_w	depth of web between flanges, fillets for UBs	mm
d_1	length of web in compression	mm
d_2	distance from top of steel beam to centroid of compression steel stress block	mm
d_3	distance from u/s of steel beam to centroid of tension steel stress block	mm
E_{bc}	eccentricity at bottom chord panel points	mm
E_c	modulus of elasticity of concrete	kN/mm ²
E_{lt}	Perry co-efficient	--
E_s	modulus of elasticity of steel	kN/mm ²
E_{tc}	eccentricity at top chord panel points	mm
e	eccentricity of web opening from mid-depth of web	mm
F_{bcr}	lateral torsional buckling design strength	N/mm ²
F_{btn}	tensile stress for SLS case n	N/mm ²
F_{chs}	design strength of concrete in longitudinal shear	N/mm ²
F_{cu}	cube strength of concrete	N/mm ²
F_{dy}	design strength of profiled steel deck	N/mm ²
F_{ry}	design strength of reinforcement	N/mm ²
F_y	design strength of steel	N/mm ²
F_{yf}	design strength of top flange of steel section	N/mm ²
F_{yr}	design strength impaired by web buckling	N/mm ²
F_1	fundamental frequency of primary floor beam	Hz
F_2	fundamental frequency of secondary floor beams	Hz
F_s	fundamental frequency of system	Hz
H_o	height of web opening	mm
H_s	distance between shear centres of flanges	mm
H_{st}	length of shear stud	mm
H_w	warping constant	mm ⁶
h	ordinate interval used in numerical integration	mm
I_{bt}	second moment of area of tee below web opening	mm ⁴
I_{cd}	second moment of area of concrete slab	mm ⁴
I_{cf}	second moment of area of transformed concrete slab	mm ⁴
I_{ct}	second moment of area of composite top tee about the major axis	mm ⁴

I_t	second moment of area of composite section	mm
I_{tc}	second moment of area of composite section for long term loading	mm ⁴
I_{to}	second moment of area of perforated composite section about the major axis	mm ⁴
I_{tt}	second moment of area of tee above web opening	mm ⁴
I_x	second moment of area of steel section about the major axis	mm ⁴
I_{xo}	second moment of area of perforated steel section about the major axis	mm ⁴
I_y	second moment of area of steel section about the minor axis	mm ⁴
J_i	torsion index	--
J_t	torsion constant	mm ⁴
K_b	factor used to calculate V_{lb}	--
K_{cr}	reduction factor applied to the critical axial strength of the web	--
K_{df}	deflection factor	--
K_f	stress reduction factor for slender outstands	--
K_p	loaded point factor	--
K_s	reduction factor applied to design strength of steel	--
K_t	factor used to calculate V_{lt}	--
L_b	span of beam	mm
L_d	length of web diagonal	mm
L_e	effective length of beam	mm
L_{lt}	equivalent slenderness factor	--
L_{lo}	limiting equivalent slenderness	--
L_{max}	distance to max. bending moment from end of beam	mm
L_o	length of web opening	mm
L_p	length of truss panel	mm
L_r	length of reinforcing flat bars at web opening	mm
L_{sp}	length of shear plane.	mm
L_t	length of taper	mm
L_w	equivalent slenderness of web.	--
M_b	lateral torsional buckling resistance moment	kNm
M_{cr}	elastic critical moment	kNm
M_{ecn}	factored moment due to panel point eccentricity for ULS case n	kNm

M_i	virtual moment due unit load at mid-span	m
M_{lb}	factored local bending moment	kNm
M_o	moment of resistance of perforated section, under pure bending at centreline of web opening, corresponding to point "0" on the interaction diagram	kNm
M_p	plastic moment capacity of steel section	kNm
M_{rc}	moment of resistance of composite section	kNm
M_{rco}	moment of resistance of composite section at web opening	kNm
M_{rct}	moment of resistance of composite truss	kNm
M_{rsn}	moment of resistance of steel section for ULS case n	kNm
M_{rso}	moment of resistance of steel section at web opening	kNm
M_{sh}	moment on composite section due to shrinkage strain	kNm
M_{sn}	unfactored bending moment for SLS case n	kNm
M_{un}	factored bending moment for ULS case n	kNm
M_{wcr}	buckling resistance moment of the web	kNm
M_{wn}	bending moment in the web for ULS case n	kNm
M_y	elastic moment capacity of steel section	kNm
M_1	moment of resistance of perforated section at centreline of web opening, corresponding to point "1" on the interaction diagram	kNm
m	equivalent uniform moment factor	--
m_1	modular ratio for short term loading	--
m_2	modular ratio for long term loading	--
N	top flange stiffness factor	--
N_h	number of shear studs within the length of web opening	#
N_{lp}	number of loaded points	#
N_p	number of panels at top chord of truss	#
N_{pl}	number of point loads to left of point considered	#
N_{pp}	number of panel points at top chord of truss	#
N_s	number of shear studs between sections considered	#
N_{s+}	N_s + allowance for minimum stud spacing	#
N_{sb}	number of secondary beam connections	#
N_{sp}	number of spaces	#
N_t	number of shear studs on beam	#
PCD	percentage critical damping	%
P_{cn}	compressive resistance of steel section for ULS case n	kN
P_e	Euler strength	kN
P_{ecr}	critical compressive strength for edge loading	kN

P_{en}	factored edge loading for ULS case n	kN
P_i	virtual force due to unit load at mid-span	--
P_n	panel number	#
P_p	panel point number	#
P_{sc}	force in chords due to shrinkage strain	kN
P_{sd}	design strength of shear stud	kN
P_{sh}	force on concrete slab due to shrinkage strain	kN
P_{sn}	unfactored force for SLS case n	kN
P_{tn}	tensile resistance of steel section for ULS case n	kN
P_{un}	factored axial load for ULS case n	kN
P_{wcr}	critical axial strength of the web	kN
P_{wn}	axial load in the web for ULS case n	kN
P_y	ultimate axial strength of steel section	kN
P_{yrn}	ultimate axial strength of steel section impaired by shear interaction for ULS case n (Von Mises criterion)	kN
Q_{cr}	critical shear strength of an unstiffened web	N/mm^2
Q_e	elastic critical shear strength of an unstiffened web	N/mm^2
R_c	resistance of concrete stress block	kN
$R_{c(max)}$	available resistance of concrete reduced by $\%C_o$	kN
$R_{c(min)}$	available resistance of concrete above profiled deck	kN
R_{ch}	resistance of shear studs within length of web opening	kN
R_r	ratio of reinforcement in shear plane	%
R_r	resistance of reinforcing flat bars above and below web opening	kN
R_s	resistance of steel stress block	kN
$R_{s(max)}$	available resistance of steel section	kN
R_{sc}	resistance of steel stress block in compression	kN
R_{sn}	structural performance ratio for SLS case n	--
R_{so}	available resistance of steel section at web opening	kN
R_{tf}	resistance of top flange in compression	kN
R_{un}	structural performance ratio for ULS case n	--
R_v	radius of gyration of steel section about the "V" axis	mm
R_{wbr}	resistance of web of bottom tee impaired by shear (Von Mises criterion)	kN
R_{wtr}	resistance of web of top tee impaired by shear (Von Mises criterion)	kN
R_x	radius of gyration of steel section about the major axis	mm
R_y	radius of gyration of steel section about the minor axis	mm
r	root radius of rolled steel section	mm

S_b	depth of tee-section web below opening	mm
S_{sh}	shrinkage strain of concrete	--
S_t	depth of tee-section web above opening	mm
S_v	plastic modulus of shear area	mm ³
S_x	plastic modulus of steel section about the major axis	mm ³
S_{xo}	plastic modulus of perforated steel section about the major axis	mm ³
S_{xw}	plastic modulus of web	mm ³
T_{bf}	thickness of bottom flange of steel section	mm
T_c	thickness of concrete above steel deck	mm
T_{co}	thickness of concrete in compression at web opening	mm
T_d	depth of profiled steel deck	mm
T_{fc}	thickness of steel flange in compression	mm
T_o	total depth of composite deck	mm
T_r	thickness of reinforcing flat bars at web opening	mm
T_{sd}	thickness of profiled steel deck	mm
T_{tf}	thickness of top flange of steel section	mm
T_w	thickness of web	mm
T_{we}	effective thickness of web	mm
u	buckling parameter	--
V_{cr}	critical shear strength of an unstiffened web	kN
V_{hs}	longitudinal force on shear plane	kN
V_{sn}	unfactored vertical shear for SLS case n	kN
V_{sbn}	unfactored vertical shear at bottom tee for SLS case n	kN
V_{uc}	longitudinal shear capacity of slab	kN
V_{un}	factored vertical shear for ULS case n	kN
V_{ur}	longitudinal shear capacity of reinforcement	kN
V_{usd}	longitudinal shear capacity of profiled steel deck	kN
V_{utn}	factored vertical shear at top tee for ULS case n	kN
V_y	ultimate vertical shear capacity of steel section	kN
V_{yb}	ultimate vertical shear capacity of bottom tee	kN
V_{yo}	ultimate vertical shear capacity of steel section at web opening	kN
V_{yt}	ultimate vertical shear capacity of top tee	kN
V_1	shearing force at centreline of web opening, corresponding to point "1" on the interaction diagram	kN
V_{1b}	portion of V_1 carried by bottom tee	kN
V_{1t}	portion of V_1 carried by top tee	kN

v	slenderness factor	--
W_{cd}	unfactored dead load of dry concrete	kN/m^3
W_{cdd}	unfactored dead load of dry composite deck	kN/m^2
W_{cdw}	unfactored dead load of wet composite deck	kN/m^2
W_{cln}	unfactored live load of construction for ULS case n	kN/m^2
W_{cw}	unfactored dead load of wet concrete	kN/m^3
W_{fs}	unfactored dead load of floor structural steel	kN/m^2
W_l	unfactored live load of floor occupancy	kN/m^2
W_{ms}	unfactored dead load of mechanical services and finishes	kN/m
W_{pa}	unfactored live load of partitions	kN/m^2
W_{rs}	unfactored dead load of roof structural steel	kN/m^2
W_s	unfactored live load of snow	kN/m^2
W_{sd}	unfactored dead load of profiled steel deck	kN/m^2
W_{sn}	total unfactored load on beam for SLS case n	kN
W_{un}	total factored load on beam for ULS case n	kN
x	distance from end of beam	mm
X'	ratio of x to the half-span length	--
X''	$2X' - (X')^2$	--
Y_{bt}	distance from flange to ENA of bottom tee	mm
Y_{cd}	distance from top to centroid of concrete slab	mm
Y_{cc}	distance from top to centroid of concrete stress block	mm
Y_{cs}	distance from top to centroid of continuous slab	mm
Y_{eo}	distance from top to ENA of perforated steel section	mm
Y_{es}	distance from top to ENA of composite section	mm
Y_{et}	distance from top to ENA of steel section	mm
Y_{po}	distance from top to PNA of perforated steel section	mm
Y_{pt}	distance from top to PNA of steel section	mm
Y_{te}	distance from top to ENA of effective steel section	mm
Y_{tt}	distance from flange to ENA of top tee	mm
Y_1	lever arm to compression steel stress block	mm
Y_2	lever arm to concrete stress block	mm
Z_{bt}	elastic modulus of bottom tee at web opening	mm^3
Z_t	elastic modulus at bottom of composite section about the major axis	mm^3
Z_{xb}	elastic modulus at bottom of steel section about the major axis	mm^3
Z_{xo}	elastic modulus at top of perforated steel section about the major axis	mm^3

Z_{xt}	elastic modulus at top of steel section about the major axis	mm^3
Z_{xw}	elastic modulus of web	mm^3
ψ	mono-symmetry index	--
λ	slenderness ratio	--

CHAPTER 1: BACKGROUND TO THESIS

1.0 Nature of the Design Problem

Ideally, in a work such as this, the design procedures would be developed by the consistent application of a comprehensive design code on composite structures, which has broad acceptance and regular usage in both the construction and academic communities. Unfortunately, such a published code does not exist as yet in British practice. In this work, to overcome the absence of a suitable code, design procedures have been developed from draft codes of practice, research and technical literature with a proven record, and the writer's engineering background.

Even the design process for the standard composite beam, which incorporates the commonly used unpropped and unperforated Universal Section, may be quite complex. The complexity arises from the number of staged construction load cases which must be satisfied at both the Ultimate and Serviceability Limit States, the selection of suitable criteria for justifying the section, and the refinement of the composite member with design options such as partial shear connection and non-uniform stud distribution.

In this work, while the difficulties described for Universal Sections still apply, the complexity is greatly compounded by the use of several novel steel sections in the composite member design. These novel sections include:

i) **Light Plate Girders:** These sections may have thin webs, slender flanges, and cross-sections which are both asymmetric and non-prismatic. The variable member geometry, when coupled with the interactive effects of shear, compression, bending, and edge loading on the thin webs, greatly complicates the design process, and usually requires that design checks be made at intervals along the member for each stage of construction.

ii) **Perforated Universal Sections:** These are rolled sections with large rectangular openings which may, or may not, be reinforced and are cut in the web to accommodate mechanical services. The primary concern is to establish the resistance of the perforated section for each stage of construction, and to determine the increase in deflections due to the web penetrations. The analytical methods must take account of the interactive effects of shear and Vierendeel bending

at the opening, and assume a failure mechanism which has been justified by research.

iii) **Composite Trusses:** These are trusses with a Warren configuration which have hollow structural chords, unbattened angle diagonals, and, when the chord dimensions allow, are designed without panel point gussets. The "gussetless" panel points usually require that the web diagonals be connected eccentrically at the panel points. While much of the truss design is straight forward and conventional, the absence of gussets requires that the resistance of the truss at all top and bottom panel points be checked for the interactive effects of axial load, eccentric and local bending moments, and the shear transfer between diagonals.

A meaningful comparison of the structural performance of the novel steel sections used in the design studies with that of the commonly used rolled sections can only be made, if the measures of structural adequacy used in the design procedures are applied consistently and with precision. These requirements are met in two ways. Firstly, a common system of Structural Performance Ratios, which correspond to the staged construction load cases at both the Ultimate and Serviceability Limit States, are applied to all the design studies. Secondly, to assure arithmetic accuracy, the design procedure for each type of steel section has been embodied in a spreadsheet computer format.

It is a prerequisite of the design process that it produces a structure fit for its task, a requirement which precludes the undue scrutiny of one aspect of the design and a superficial treatment of the others. To this end, it is intended that the design procedures developed here will satisfy conservatively all the normal strength and serviceability limits imposed by practising engineers on composite beams in building structures.

1.1 Origin of the Design Studies

Over the period January to May 1984, Prof.P.J.Dowling and Dr.G.W.Owens of the Department of Civil Engineering, Imperial College, made several submissions to the British Steel Corporation on new design methods and forms of construction in multi-storey buildings.

The central point of the Imperial College submission was to question the dominant United Kingdom practice of framing the floor plans

in low and medium-rise multi-storey buildings with internal columns on a 6 to 9 metre grid; an industry practice highly suitable to solid slab and waffle floor construction, but a practice which totally under values the potential of structural steel.

The Imperial College proposal suggested that the work be carried out in two phases. In the first phase, several forms of primary long span beams using novel steel shapes would be developed and rigorously designed to the best industrial practice and theoretical knowledge available. Ideally, the objective was to achieve a composite floor system which would eliminate internal columns, accommodate the air ducts and other services in a minimum floor zone, produce frame weights comparable to conventional designs, and use unpropped construction to speed erection.

The work on the first phase was to be appraised and guided by an interdisciplinary Advisory Committee whose primary function was to ensure that the Imperial College designs took full account of the many constraints placed on the steel frame of a modern multi-storey building.

In the second phase, a research programme would be initiated to investigate areas of uncertainty in the design procedures adopted for phase one.

1.2 "Design Development of Multi-Storey Steel Frames"

On the 4th of June 1984, Imperial College was awarded a research grant by the British Steel Corporation Trust Fund for the first phase of their proposal. Work on the project formally commenced on the 1st January 1985, and at that time the project was given the above title. Dr.Owens directed the project and acted as chairman of the Advisory Committee. The writer of this work joined the project as a Research Fellow in Feb.1985 and worked at Imperial College on design procedures for long span composite beams until June 1986.

The original studies on the new types of primary beams and floor plan arrangements were appraised on a lump sum cost basis within the context of a "Standard Building". At that time, it was deemed advisable to compare the new systems with a "conventional" floor plan within the context of a complete building rather than in component parts, such as bays and single members. The dimensions of the "Standard Building" were fixed at six storeys high, fifteen metres wide, and sixty metres long. These proportions were chosen by the Advisory Committee at its

inaugural meeting, and were thought to be most representative of the market demand in the Greater London area. The floor plans used in the cost comparisons are shown in Fig 1 and 2. The "conventional" floor plan (Fig 1: Plan A) was taken as the basis for the cost comparisons.

The design calculations were made manually with a hand calculator, and the results recorded by word processor in prepared tables. While the prepared tables helped to make the process systematic, the total elapsed time to calculate, check, and refine each design was between three and four days. The calculations were made by the writer and Mr.M.Balsara, a Research Assistant, who joined the project in June 1985. Dr.Owens has given a full account of the conceptual thinking, and some results from the design studies, in his paper to the International Conference on Steel and Aluminium Structure, Cardiff, 1987. [31]

1.3 The Advisory Committee

The Advisory Committee, which met three times during 1985, gave careful consideration to the central objective of designing open plan, column free, composite floor system, as well as guidance on specific design values. A review of the minutes of those meetings, which are a matter of record, shows that some of the points discussed were: minimizing floor zone depth; ceiling allowance; loading values for occupancy and moveable partitions; camber and service deflection; load factors for partitions and construction loads; dynamic response; and a number of important design values which are discussed below. The names and professional disciplines of the members of the Advisory Committee for the period of the writer's involvement, January 1985 to June 1986, are given in Appendix 1.

1.4 Past Design Studies

As mentioned above the Imperial College studies were made manually and each study required the dilligent efforts of two engineers for several days. In the end, most of the studies were deficient in some respects.

The design time consumed, and the ever pervasive sloth, made refinement of much of the work unattractive. While considerable efforts were made to obtain efficient and balanced members, design time often limited the refinement of the steel cross-section and the use of

several important options, such as: section mono-symmetry; optimum span-depth ratio; partial shear connection; and non-uniform stud distribution.

In 1985, a modern code of practice for the design of composite beams in buildings was not available. This void made it necessary for the design team to select design values, which were thought to be appropriate to building construction, from a variety of sources. The important values and the sources which were considered appropriate are discussed below in the Chapter 3. The contentious values included items such as: effective breadth of the concrete flange; effective thickness of the concrete flange; design strength of concrete in compression and longitudinal shear; shrinkage strain in concrete; compactness rules for the steel flange and web in the composite section; design strength of shear stud connectors; and temporary construction loading. Since the values chosen for these early studies tended to be conservative, the full potential of the composite systems was seldom realized.

BS 5950: Part 1: "Structural use of steelwork in building" was published in 1985 [1]. Wherever practicable, it was intended that the design procedures for the steel and composite sections should apply the provisions of this standard. However, a major difficulty was found in the application of the standard's procedure for calculating the strength of a steel section with thin unstiffened webs, and extending the method to the composite section [BS 5950: Appendix H.3]. A design procedure, which fully satisfied the method of BS 5950: Part 1:, was never quite achieved at that time.

The absence of firm rules for the compactness of the steel flange and web in the composite section also made systematic design difficult.

1.5 Aims of this Thesis

The compelling reason for this work, which is a continuance of the Imperial College studies, is the firm conviction that there is in the construction community a need for reliable methods of quickly assessing the relative merits of alternative composite long-span framing systems in steel structures.

It is intended that the computerised design procedures developed in this work will afford engineers, and other users, the ability to select from a number of composite assemblies and floor framing

arrangements those which best satisfy their conditions.

It is further intended, that within each procedure the user should be free to set and adjust at will the input values of the many variables commonly used in composite design, and, when advantageous, to implement a number of special design options which will allow a high degree of member refinement.

CHAPTER 2: OUTLINE OF NEW DESIGN STUDIES

2.0 Composite Sections Studied

In this work, design procedures have been devised and studies conducted on long span composite beams using the steel sections which are amenable to systematic design and appear to offer the most immediate advantage over the short span conventional system.

The detailed design procedures for the steel sections used in the studies are given in Volume 2: Parts 2 to 8. Each of these parts is described in detail, with some explanation for the choice of member, later in this chapter (2.8). The steel sections studied in Parts 2 to 8 are:

Part 2: Universal Rolled Sections (Fig 5)

Part 3: Light Plate Girders of Constant Depth (Fig 11)

Part 4: Light Plate Girders with a Constant Taper (Fig 7)

Part 5: Light Plate Girders with a Modified Taper (Fig 8)

Part 6: Universal Beams with Unreinforced Web Openings (Fig 6)

Part 7: Universal Beams with Reinforced Web Openings (Fig 6)

Part 8: Warren Trusses with Hollow Structural Chords (Fig 9)

2.1 Important Advances in the New Design Studies

In this work, the design procedures are computerized in a spreadsheet format. Instead of several days by hand calculation, the elapsed time to complete a study and achieve a refined design is now about one hour. The proportions of the steel section can be optimized and a number of important options tried in a few minutes. The spreadsheet format used in the studies allows the user to run all, or any part of the programme. The programme running time for a complete study, the part of the study relating to the composite member, and an estimate of total design time are given later in this chapter (2.7).

To overcome the uncertainty associated with certain input values in composite construction, the spreadsheet format has been arranged to allow the user almost complete discretion in this regard. It is intended that the flexibility inherent in this approach will allow the user to tailor his designs to specific conditions. A list of forty five input variables, which are common to Parts 2 to 8 of the design studies, is given in Table 1.

Table 1: Common Input Variables

Variables	Symbol
Span-depth ratio of light plate girders and trusses	Any Value:
Beam type, Fig 6	1,2
Beam symmetry in light plate girders, Fig 11	1,2,3
Span of the beam	L_b
Centres of primary beams	C_{pb}
Number of secondary beams	N_{sb}
Effective breadth of concrete flange	B_e
Structural steel grades	50,43
Elastic moduli of steel and concrete	E_s, E_c
Cube strength of concrete	F_{cu}
Density of dry and wet concrete	W_{cd}, W_{cw}
Shrinkage strain of concrete	S_{sh}
Design strength of concrete in longitudinal shear	F_{chs}
Design strength of reinforcing fabric	F_{ry}
Type of profiled deck or solid slab, Fig 12	1,2,3
Depth and thickness of profiled deck	T_d, T_{sd}
Design strength of profiled deck	F_{dy}
Dead load of roof structural steel	W_{rs}
Dead load of floor structural steel	W_{fs}
Dead load of steel deck and mechanical services	W_{sd}, W_{ms}
Live load of snow	W_s
Live load of occupancy and partitions	W_1, W_{pa}
Live load of construction for stage 1	W_{cl1}
Live load of construction for stage 2	W_{cl2}
Load factors for Ultimate and Serviceability Limit States	LF
Depth of steel section	D_b
Width of steel section	B_b
Thickness of top and bottom flange	T_{tf}, T_{bf}
Thickness of web	T_w
Design strength of shear stud connector	P_{sd}
Percentage shear connection	$\%C_o$
Type of stud distribution, Fig. 20 & 21	1,2

The design procedures now include routines, which are in accordance with BS 5950: Part 1:, for checking the strength of steel and composite sections with thin unstiffened webs. Draft versions of Euro-code 4 [3] and BS 5950: Part 3.1: "Code of practice for design of composite beams" [2] both give guidance on compactness rules for steel flanges and thin webs in composite sections. The recommendations of these codes have been implemented, in part, in these design studies.

2.2 Important Nomenclature

The spreadsheet computer programme used in the design procedures can perform several Boolean functions which act on numeric input rather than string variables. These Boolean functions allow the user to choose from a variety of options which have been built into the procedures. The need to use numeric operators has made it necessary to develop a nomenclature suitable to this purpose.

Also, for simplicity and brevity, mnemonics have been used to describe the different steel sections studied in Parts 2 to 8. The important nomenclature and mnemonics are listed in Table 2.

Table 2: Important Nomenclature

:-----:-----:	
: Nomenclature :	Meaning :
:-----:-----:	
:Type 1 Beam :	:beam supporting uniformly distributed loading, :
:	:with the deck profile running transversely, Fig 6 :
:Type 2 Beam :	:beam with interconnected secondary members, :
:	:with the deck profile running parallel, Fig 6 :
:Type 1 Symmetry :	:symmetric light plate girder, $T_{tf} = T_{bf}$, Fig 11:
:Type 2 Symmetry :	:mono-symmetric light plate girder, $T_{tf} < T_{bf}$, Fig 11:
:Type 3 Symmetry :	:mono-symmetric light plate girder, $T_{tf} > T_{bf}$, Fig 11:
:Steel Grade 50 :	:structural steel grade 50 to BS 4360 :
:Steel Grade 43 :	:structural steel grade 43 to BS 4360 :
:Type 1 Deck :	:CF 46 profiled steel deck by Precision Metal Forming:
:	:Fig 12 :
:Type 2 Deck :	:Super-Holorib profiled steel deck by Richard Lees :
:	:Fig 12 :
:Type 3 Deck :	:a solid concrete slab :
:-----:-----:	

Table 2: Important Nomenclature (cont'd)

:-----:-----:		
:Roof/Floor Code 1:	loads, shears, and bending moments are calculated	:
:	:for a roof beam only (Parts 5 to 8)	:
:Roof/Floor Code 2:	loads, shears, and bending moments are calculated	:
:	:for a floor beam only (Parts 5 to 8)	:
:Type 1 Spacing	:uniform density stud distribution, Fig 20	:
:Type 2 Spacing	:triangular density stud distribution, Fig 20	:
:LPG	:a light plate girder built-up by welding with an	:
:	:unstiffened web and constant width, Fig 10 & 11	:
:LPG(C)	:a light plate girder of constant depth	:
:LPG(T)	:a light plate girder with a constant taper, Fig 7	:
:LPG(M)	:a light plate girder with a modified taper, Fig 8	:
:UB	:universal rolled sections, Fig 5	:
:UB(U)	:universal rolled sections with unreinforced web	:
:	:openings, Fig 6	:
:UB(R)	:universal rolled sections with reinforced web	:
:	:openings, Fig 6	:
:TRUSS(H)	:a Warren truss with hollow structural chords, Fig 9	:
:STDBLDG	:loadings and material values selected by the	:
:	:Advisory Committee	:
: R_{sn}	:load case and structural performance ratio for	:
:	:serviceability limit state (SLS) case n	:
: R_{un}	:load case and structural performance ratio for	:
:	:ultimate limit state (ULS) case n	:
:-----:-----:		

2.3: "Standard Building Properties" (STDBLDG)

To make the results of this work immediately relevant, it is important that the design studies use key input values which are representative of current industrial practice. In this work the key values, which were chosen by the Advisory Committee, are called the "Standard Building Properties" (STDBLDG). Although the computer design procedures now allow the user total freedom in the choice of input, all the studies in this work have been calculated using the standard properties listed below in Table 3.

Table 3: "Standard Building Properties" (STDBLDG)

Property	Value	Units	Symbol
Centres of primary beams (Fig 1,2: Plans A,B,C)	3000	mm	C_{pb}
Centres of primary beams (Fig 2: Plan D)	6000	mm	C_{pb}
Centres of secondary beams (Fig 2: Plan D)	3000	mm	-
Structural steel grade	50	-	-
Lightweight structural concrete - dry density	17.66	kN/m^3	W_{cd}
Cube strength of lightweight structural concrete	25	N/mm^2	F_{cu}
Profile decking, CF46 by Precision Metal Forming	-	-	-
Thickness of the composite deck	120	mm	T_o
Occupancy live load	4.0	kN/m^2	W_1
Moveable partition taken as live load	1.0	kN/m^2	W_{pa}
Mechanical services and ceiling	0.3	kN/m^2	W_{ms}
Stage 1: construction load taken as live load	0.3	kN/m^2	W_{c11}
Stage 2: construction load taken as live load	0.5	kN/m^2	W_{c12}
Limit on service deflection	$L_b/360$	mm	D_{max}
Limit on calculated camber	$L_b/250$	mm	D_{ca}

2.4 Floor Plans and Primary Beams

Plans A, B, C, D (Fig 1 & 2) are the floor framing arrangements used in the original studies to evaluate the cost effectiveness of several types of long span composite beams within the context of a six storey building (1.2).

Here, floor plans are simply used to demonstrate some suitable applications for the long span composite beams studied in Parts 2 to 8 of this work. The floor plans considered are shown in Fig 1 to 4, and cross-sections showing the method by which the long span beams cater for air ducts and other services are shown in Fig 5 to 9.

Floor plans C and F have the benefit of carrying the floor load directly to the columns, but are only efficient when coupled with a deep composite deck as used in Floor plan F, Sears Building, Chicago, where the decking spans 4570 mm and the trusses span 22850 mm [47].

The six floor plans with compatible long span composite beams, and some remarks and restrictions on their use are given next in Table 4.

Table 4: Floor Plans and Primary Beams

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:Floor:	Fig:	Beam:	Primary	:Part:	Fig:	Cross:	Remarks and Restrictions		
:Plan :	:Mark:	Beam	:	:	:Sect.:	:			
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
: A	: 1	: B _{f1} :UB	:	2	: 5	: A-A	:Duct under shorter span	:	
: B,C	: 1,2	: B _{f1} :UB(U)	:	6	: 6	: A-A	:Duct anywhere, unreinforced	:	
:	: 1,2	: B _{f1} :LPG(T)	:	4	: 7	: A-A	:Duct near perimeter	:	
:	: 1,2	: B _{f1} :LPG(M)	:	5	: 8	: A-A	:Duct at perimeter	:	
:	: 1,2	: B _{f1} :TRUSS(H)	:	8	: 9	: A-A	:Duct anywhere	:	
: D	: 2	: G _{f1} :UB(R)	:	7	: 6	: B-B	:Duct anywhere, reinforced	:	
:	: 2	: G _{f1} :LPG(T)	:	4	: 7	: B-B	:Duct near perimeter	:	
:	: 2	: G _{f1} :LPG(M)	:	5	: 8	: B-B	:Duct at perimeter	:	
:	: 2	: G _{f1} :TRUSS(H)	:	8	: 9	: B-B	:Duct anywhere	:	
: E	: 3	: B _{f1} :UB	:	2	: 5	: A-A	:Duct under	:	
:	: 3	: B _{f1} :LPG(C)	:	3	: 11	: -	:Duct under, span depth ratio 25:	:	
:	: 3	: G _{f1} :UB(R)	:	6	: 6	: B-B	:Few penetrations, reinforced	:	
: F	: 4	: B _{f1} :TRUSS(H)	:	8	: 9	: A-A	:Duct anywhere, deep comp.deck	:	
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									

2.5: Format of the Design Studies

Design procedures and studies for seven types of composite beam are included in Volume 2 as Parts 2 to 8. The text of the seven parts, which are documented in the same format, is sub-divided into four sections.

(1) **Manual Design Procedures:** These manual calculations, which follow the computer solution in a step by step fashion, are made by hand calculator on a Designated Design Study which is included at the end of each part in a spreadsheet display format. The hand calculations demonstrate to the user the design assumptions, formulae, and options built into the computer programme. They also check the accuracy of the computer solution.

(2) **Cell Formulae:** The cell formulae used in the computer design procedure are listed for cells which generate results. The input values used in the Designated Study are not given. The cell formulae clearly demonstrate how the programme handles the design assumptions and formulae. However, more importantly, they show how the loads, shears, bending moments, design check locations, etc., are generated by the pro-

gramme from a minimum of input data and option selection. The computer design procedures in this work run on the Hewlett Packard spreadsheet system "Context MBA". The computer hardware and software used in the design studies are given in Appendix 2.

(3) Results of Design Studies: The results are displayed in several tables. The parameters of the study, the structural performance ratios, and physical properties of the members studied are given in as many as eleven tables. The results are also given graphically in this volume in terms of mass, beam span, governing performance ratios, and vibration assessment in Fig 29 to 56. A catalogue of the 300 design studies completed in this work is give in Volume 2: Part 1 (1.3). A complete set of the studies, which are recorded on micro-fiche, is enclosed at the end of Volume 2.

(4) Designated Design Study with Cell Addresses: The Designated Study, which is used as the worked example for the Manual Design Procedure, is displayed in the computer output format with cell addresses at the end of each part in Volume 2. The Manual Calculations and Cell Formulae can only be fully understood if read in conjunction with a Designated Study.

2.6 Format of the Computer Output

As stated above, an example of the computer output format is given in Volume 2 in each of Parts 2 to 8. The logic of the computer solution will be best understood by taking the simple example of Study No.23: Volume 2: Part 2 (2.3).

Tables 1 to 5: the essential material properties, unfactored loads and preferred options are entered.

Table 6: factored loads for ultimate and serviceability limit states are calculated.

Table 7: the cross-sectional dimensions for roof and floor beam are input from universal section tables.

Table 8: the flexural properties are calculated and the sections are checked for compactness.

Table 9 to 11: the sections are checked for strength in lateral torsional buckling and the restrained condition.

Table 12: the effect of shear on the web of the composite sections is calculated.

Table 13: material values and options pertaining to the

composite sections are entered. Entering these values at this point slightly reduces the computer running time for options such as percentage shear connection and stud distribution.

Tables 14 to 16: the composite sections are checked for strength.

Tables 17 to 21: the steel and composite sections are checked for deflection and premature yield.

Tables 22 to 23: the number of shear studs and the amount of reinforcing steel in the deck slab is calculated.

Tables 24 to 25: floor vibrations are checked.

Table 26: the performance of the selected members at the ultimate and serviceability limit states is summarised. These results tell the user at a glance whether the selected member has passed or failed, and guide him to a refined design.

Table 27: the physical properties necessary for cost estimation and material procurement are tabulated.

The example chosen is a simple one. The computer format for some design procedures requires the use of iteration and considerably more tables. However, the principle is the same. The user controls the input and options, can inspect any part of the calculations, and can refine the design with the guidance of the performance ratios.

2.7 Elapsed Time for Computer Studies

As stated above (1.4), the pure slog and tedium associated with the manual calculations of the original studies produced designs which were seldom refined and quite limited in scope. At that time, the studies were only conducted on a span of fifteen metres. A perusal of Table 5 will show that design times are no longer a restriction on the work.

The spreadsheet system allows the user to calculate at his discretion all, or any part, of a design study. For example, a complete study can be run with 100% shear connection to find a steel section which satisfies the performance ratios, and then the composite section only run to find a balance between strength and increased deflection due to reduced shear connection. Similarly, the early part of a design study may be run to minimise the number of lines of floor bracing by changing the cross-sectional dimensions of the section. Perhaps, more usefully, the user might wish to consider using normal density concrete to lessen

floor vibrations, or the less expensive steel grade 43 in roof beams which tend to be dominated by deflection of the steel section (Fig 45).

Table 5: Computer Running Times and Design Time - Parts 2 to 8

:-----:-----:-----:-----:-----:-----:-----:-----:							
:	:Roof/:	Computer Running Times				: Estimate	:
:Part:	Floor:	Complete Study		:Composite Part of Study		: of	:
:	:Beam :	Tables	: Time	: Tables	: Time	:Design Time:	:
:-----:-----:-----:-----:-----:-----:-----:-----:							
:	2 :	Both:	1 to 27	:1.75 mins.:	13 to 27	:1.25 mins.:	0.50 hrs.:
:	3 :	Both:	1 to 32	:3.25 mins.:	15 to 32	:2.00 mins.:	1.00 hrs.:
:	4 :	Both:	1 to 32	:6.25 mins.:	15 to 32	:4.00 mins.:	2.50 hrs.:
:	5 :	One :	1 to 33	:3.25 mins.:	16 to 33	:2.00 mins.:	1.50 hrs.:
:	6 :	One :	1 to 28	:2.25 mins.:	15 to 28	:1.50 mins.:	1.00 hrs.:
:	7 :	One :	1 to 28	:2.25 mins.:	15 to 28	:1.50 mins.:	1.00 hrs.:
:	8 :	One :	1 to 27	:3.45 mins.:	13 to 27	:2.75 mins.:	1.75 hrs.:
:-----:-----:-----:-----:-----:-----:-----:-----:							

The elapsed time to complete the design of a long span composite beam with the computer depends on factors such as: the complexity of the procedure which may require iteration; the degree of refinement to the cross-section, and the number of options and special variables used in the design. The options and special variables available to some of the design procedures, Parts 2 to 8, are given below in Table 6.

Table 6: Additional Options and Variables - Parts 2 to 8

:	:	:	:	:	:	:	:	:	:								
:	Options and Variables	:Code/ :	Part Number							:							
:		:Symbol:	2	:	3	:	4	:	5	:	6	:	7	:	8	:	
:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	
:	Percentage connection	: %C _O	:	X	:	X	:	X	:	X	:	X	:	X	:	X	:
:	Stud distribution type	: 1,2	:	X	:	X	:	X	:	X	:	X	:	X	:	X	:
:	Symmetry type (Fig 11)	:1,2,3	:	:	:	X	:	X	:	X	:	:	:	:	:	:	:
:	Design of roof beam and floor beam	: -	:	X	:	X	:	X	:	:	:	:	:	:	:	:	:
:	Design of roof beam or floor beam	: 1,2	:	:	:	:	:	X	:	X	:	X	:	X	:	X	:
:	Degree of taper (depth ratio)	: -	:	:	:	X	:	X	:	:	:	:	:	:	:	:	:
:	Length of taper	: L _t	:	:	:	X	:	X	:	:	:	:	:	:	:	:	:
:	Location of web opening (Fig 23)	: X _O	:	:	:	:	:	X	:	X	:	:	:	:	:	:	:
:	Length of web opening (Fig 23)	: L _O	:	:	:	:	:	X	:	X	:	:	:	:	:	:	:
:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:

Table 6: Additional Options and Variables - Parts 2 to 8 (cont'd)

Options and Variables		Code/	Part Number							
		Symbol:	2	3	4	5	6	7	8	
Height of web opening (Fig 23)	H_o						X	X		
Eccentricity of opening (Fig 23)	e						X	X		
Area of reinforcement at opening	A_r							X		
Number of shear studs at opening	N_h						X	X		
Number of panels in truss (Fig 28)	N_p								X	
Eccentricity at top panel point	E_{tc}								X	
Eccentricity at btm panel point	E_{bc}								X	
Truss properties from data banks	PART								X	

2.8 Design Procedures and Studies - Parts 2 to 8 (Volume 2)

In this part of the text, the long span composite members studied in Parts 2 to 8 of Volume 2 are described, some possible applications are explored, and the design options and special input variables applied in the design procedures are discussed.

2.8.1 Part 2: Universal Rolled Sections [UB] (Fig 5)

Universal rolled beams and columns will usually be the sections considered first for the design of composite floor systems. The reasons are obvious: it is the long standing industrial practice, they are readily available, and all steel fabricators have the facilities and abilities to work them. They have several major disadvantages: the web is too thick for the shear values normally found in floor beams, the limited rolling schedules entail large jumps in beam mass and preclude sizing beams on a span depth basis, and the web must be perforated to accommodate air ducts.

Universal rolled sections are primarily included in these studies as a basis for comparison with other steel sections. However, there are applications where services are not a major consideration. Industrial buildings, warehouses, and hotels where services usually run in the corridors, are a few. The perimeter steel and secondary members in floor arrangements are usually fabricated from universal beams.

In this procedure a floor and roof beam are designed simultaneously. All section properties are derived from the minimum cross-section dimensions of depth, width, flange and web thickness, and root radius, which are entered at Table 7. The dimensions are taken from the Constrado Handbook [15]. The formulae used to calculate the section properties in the spreadsheet routine are given in Appendix 9.

The reduced resistance of the composite section due to shear yield interaction (Von Mises Criterion) is calculated at intervals along the beam.

The design studies were made to find the minimum mass and minimum depth beam or column which satisfied the imposed conditions. The results are plotted in terms of mass and depth for a span range of 9 to 24 metres in Fig 29 to 32.

2.8.2 Part 3: Light Plate Girders of Constant Depth [LPG(C)] (Fig 11)

These are prismatic sections which could replace universal sections in the applications suggested above (2.8.1), where services do not require web penetrations. The web thickness and span-depth ratio can be tailored to fit the conditions. Comparison with the results for universal rolled sections show that reductions in beam mass in the order of 30 to 40 per cent are obtainable (Fig 33 to 36).

For example in Floor Plan E (Fig 3), where the secondary members (B_{f1}) are long span, a prismatic light plate girder could be designed with a span-depth ratio of 25. This member would allow services to run under and still show a substantial saving in beam mass with respect to the alternative universal rolled section (Fig 34).

In Part 3, the design procedure allows for three types of beam symmetry (Fig 11). Floor and roof beams are designed simultaneously. All section properties are derived from the cross-section dimensions of depth, width, and flange and web thickness, which are entered in the spreadsheet at Table 7. The formulae used to calculate the properties of mono-symmetric sections are given in Appendix 10. The resistance of the steel and composite sections are calculated by an iterative process at the point of maximum bending moment and at intervals along the beam.

The design studies were made to find the minimum mass Light Plate Girder at span depth ratios of 15, 20 and 25 which would satisfy the conditions. The results are plotted for a span range of 9 to 24 metres in Fig 33 to 36 where the results for the comparable universal

beams are also displayed.

2.8.3 Part 4: Light Plate Girders with a Constant Taper [LPG(T)] (Fig 7)

These are tapered members designed primarily to accommodate air ducts and other services without perforating the web. Ideally, they would be used in floor systems where services can be constrained to run near the perimeter of the building. However, they are also mass efficient in their own right. The increased shear resistance due to the shallower web in the high shear areas, coupled with efficient web cutting, gives better results in some instances than the prismatic section (Part 3). However, they are prone to be governed by deflection and appear to be most efficient at span depth ratios of 15 to 20.

As with Part 3, the design procedure allows for three types of beam symmetry (Fig 11), and floor and roof beams are designed simultaneously.

All section properties are derived from the cross-section dimensions input to the spreadsheet at Table 7, and are calculated with the formulae given in Appendix 10 which are built into the procedure. The degree of taper (DEPTH RATIO) and the length of taper (L_t) are variables also entered at Table 7. The variable depth web greatly increases the arithmetic. Deflection calculations are made using virtual work and numerical analysis (Simpson's Rule), and, as in Part 3, the resistance of the steel and composite section are calculated by an iterative process at the point of maximum bending moment and at intervals along the beam. It is found that the governing section resistance is never at the point of maximum bending moment.

A full scale test on a tapered plate girder, 15 metres span and 800 mm deep, was made at Imperial College by Hargreaves and Hobbs, who concluded that the beam maintained its integrity during the test and probably had a reserve of strength greater than the calculated load factors [32]. The Imperial College test was also reported by Raven and Glover [30], who also described a role for tapered beams in the expanding market for structural steel. It is worth noting, that Design Study No.39 of Part 4 uses parameters almost identical to the those of the Imperial College test. The cross-section dimensions obtained from the study are given in Part 4: Results of Design Studies: Table 11, and are close to those used in the test.

The design studies were made to find the minimum mass Tapered Light Plate Girder which would satisfy the conditions with span depth ratios of 15, 20 and 25 and the degree of taper held constant at 0.5. The results are plotted for a span range of 9 to 24 metres in Fig 37 to 40 where the results for the comparable universal beams are also displayed.

2.8.4 Part 5: Light Plate Girders with a Modified Taper [LPG(M)] (Fig 8)

These are light plate girders partially tapered at each end. They have better deflection characteristics than the fully tapered member, and ideally would be used in floor systems where the services are constrained to run at the perimeter of the building.

As with Parts 3 and 4, the design procedure allows for three types of beam symmetry (Fig 11). However, the user must now select whether to design a roof or floor member. This additional option greatly reduces the running time of the programme, facilitates member refinement and allows more appropriate material values to be used in the design of roof beams.

All section properties are derived from the cross-section dimensions, degree of taper, and length of taper input to the spreadsheet at Table 7, and are calculated with the formulae given in Appendix 10 which are built into the procedure. Although it is cumbersome when applied to the prismatic section, the Part 5 design programme can be applied to the three types of light plate girder. Deflection calculations are made using virtual work and numerical analysis (Simpson's Rule), and, as in Parts 3 and 4, the resistance of the steel and composite section are calculated with iteration at the point of maximum bending moment and at intervals along the beam. An additional check of section strength is made at the half-length of the taper.

In Part 5, the design studies were made on floor beams only to find the minimum mass Modified Light Plate Girder. The studies were conducted at span-depth ratios of 15, 20 and 25 with the degree and length of taper held constant at 0.5 and 3000 mm respectively. The results are plotted for a span range of 9 to 24 metres in Fig 41 and 42 where the results for the comparable universal beams are also displayed.

2.8.5 Part 6: Universal Beams with Unreinforced Web Openings [UB(U)] (Fig 6 & 23:A and Appendices 11 & 13)

As argued above (2.8.1), for reasons of industrial practice and availability, universal beams will be the steel sections first considered for floor framing in multi-storey buildings. In long span floor systems these sections usually have reinforced web openings to accommodate the air ducts and to minimize the floor zone depth. The major disadvantage in this approach is the high expenditure of shop labour to install the reinforcement at the web opening (Fig 23:B). The major advantage is the unrestricted routing of the alignment of the air ducts.

Web openings are often reinforced in the most complicated manner, and, perhaps more importantly, reinforcement is used when it is not required.

In Part 6, the design procedure allows the user to design universal beams with unreinforced web openings at almost any location. Universal columns have not been considered in the studies. The design methods, which assume plastic failure, are those developed for steel and composite sections by Prof.R.G.Redwood and his collaborators [37 to 45], and are included for the steel section only in the Canadian Handbook of Steel Construction [46].

Each study is restricted to a roof or floor member, an option introduced in Part 5. The input variables pertaining to the web openings are entered in the spreadsheet at Table 8, and include: location of the web openings (X_o), the length of the web opening (L_o), the height of the web opening (H_o), the eccentricity of the web opening (e), and the number of shear studs in the length of the web opening (N_h). The procedure allows the user to investigate unreinforced web openings at four locations anywhere along the beam, but assumes that the openings will not be located under load points in Type 2 beams. The details at web openings used in the design studies are shown in Fig 23:A.

Design studies were made on floor beams with a span range of 12 to 24 metres, $H_o/D_b = 0.5$, $L_o/H_o = 2$, and the results are given in Part 6: Results of Design Studies: Tables 1 to 8 (Volume 2).

The unperforated rolled sections designed in Part 2 were used a starting point for the studies in Part 6. The mass of the unperforated (Part 2) and perforated (Part 6) floor beams are reported and

compared in Tables 12 and 13 (6.1.6). It can be seen from these results that the same section can be justified for Type 1 beams, and only a modest increase in mass is required for Type 2 beams.

2.8.6 Part 7: Universal Beams with Reinforced Web Openings [UB(R)] (Fig 6 & 23:B and Appendices 12 & 14)

As stated above (2.8.5), web openings are often reinforced in the most complicated manner. In Part 7, the design procedure assumes that the web openings are reinforced in the manner shown in Fig 23:B; namely, with a pair of flat bars at the top and bottom of the opening and each bar joined to the web with a single fillet weld. This is the arrangement usually used by Prof.Redwood et al [37,39,41,44], and also applied in the Canadian Handbook of Steel Construction [46].

In Part 7, the design procedure allows the user to design universal beams (universal columns are not considered in the studies) with reinforced web openings at any location. For the steel section, the design method is that developed by Prof.Redwood and also included in the Canadian Handbook of Steel Construction. The design method used for the composite section has been extrapolated by the writer from that used for the unreinforced opening, and is developed in Appendix 14.

Each study is restricted to a roof or floor member, an option introduced in Part 5. The input variables pertaining to the web openings are entered in the spreadsheet at Table 8, and include: location of the web opening (X_o), length of the web opening (L_o), height of the web opening (H_o), the eccentricity of the web opening (e), the number of shear studs in the length of the web opening (N_h), and the area (A_r) and thickness (T_r) of the reinforcing flat bars. The procedure allows the user to investigate reinforced web openings at four locations anywhere along the beam, but assumes that the openings will not be located under load points in Type 2 beams.

Design studies were made on floor beams with a span range of 12 to 24 metres, $H_o/D_b = 0.5$, $L_o/H_o = 2$, $A_r/A_f = 0.3$ to 0.5 , and the results are given in Part 7: Results of Design Studies: Tables 1 to 8 (Volume 2).

The unperforated rolled sections designed in Part 2 were used as a starting point for the studies in Part 7. The mass of the unperforated (Part 2) and perforated (Part 7) floor beams are reported and compared in Tables 15 and 16 (6.1.7). It can be seen from these

results that the same section can be justified for Type 1 and 2 beams.

2.8.7 Part 8: Trusses with Hollow Structural Chords [TRUSS(H)] (Fig 9)

Light plate girders are an attractive proposition to steel fabricators with highly automated plants and a sound background in thin web construction. Unfortunately, such plants are very few. On the other hand, all steel fabricators can construct trusses; a staple of the industry.

Composite trusses have been in regular use in North America for some time. Two of the world's taller buildings, the Sears Tower, Chicago (109 storeys, see Fig 4), and the World Trade Centre, New York (110 storeys), use composite trusses spanning 22.9 and 18.3 metres respectively. In the city of Edmonton, Alberta, there are three thirty storey buildings with floor plans similar to Plan E (Fig 3) which use trusses spanning 11 to 12 metres. It should also be realized that these trusses have been supplied from an industry where the shop labour costs are very much higher than those in the United Kingdom.

The two essential ingredients for an economic truss are "gussetless" construction and chord sections which allow easy assembly in jigs. The truss web framing configurations which have been used in composite construction are shown in Fig 26. The most commonly used is the Warren Truss which has fewer components and provides large openings for services. It is the form used in these studies.

Chord and web steel combinations, which can be used with Warren trusses, are shown in Fig 27 where they are arranged in descending order of usefulness. The combination of hollow structural chords and unbattened angle diagonals is used in the design studies. The hollow structural section does not require to be stripped and straightened as do structural tees. It has good resistance to: lateral buckling during erection, local and eccentric bending moments, and shear forces at the "gussetless" panel points.

A full scale test was carried out on a 12 metre truss of this form at the University of Alberta and is reported by Bjorhovde [48]. As a worked example, Chien and Ritchie use a truss of this type in their book on composite construction [27]. Chien also gives an account of the use of trusses in Canadian practice [28]. Composite truss design is included in the Canadian steel structures code [7]. The design and full scale test made on the composite Warren truss (tee chords) used

in the Sears Building (Fig 4) are described by Iyengar and Zils [47].

In Part 8, the section properties for chords and diagonals are called up from Data Banks. The Data Banks are an integral part of the computer routine and are displayed with Study No.5, the Designated Design Study of Part 8 (Volume 2). The section properties for HSS chords and equal leg angles are listed in the Data Banks by Part Numbers which the user inputs to the spreadsheet at Tables 7, 16 and 17.

The key diagram of the truss geometry is shown in Fig 28. The truss geometry is generated from the span-depth ratio and the number of panels (N_p) which are entered in the spreadsheet at Table 1, and the top and bottom chord eccentricities (E_{tc} and E_{bc}) which are entered at Table 8.

As for Parts 5 to 7, each study is restricted to a roof or floor beam. A summary of the truss components, which could be used for costing and material procurement, is displayed in the spreadsheet at Table 22. Design studies were made on floor trusses at span-depth ratios of 10 and 15. Results are plotted for a span range of 12 to 24 metres in Fig 43 and 44.

2.9 Limitations on the Design Studies

While the long span composite members studied in Parts 2 to 8 can be applied to a variety of floor plans and other applications, the design procedures are bound by a number of basic assumptions.

2.9.1 Simply Supported Beams and Symmetrical Loading

In the early studies, consideration was given to members such as Haunched Beams which require a rigid frame, and Vierendeel Girders which are redundant structures. Such members are beyond the scope of this work, and it is assumed that all the composite sections studied are simply supported and statically determinate in themselves.

The design procedures assume that all beams are equally spaced and symmetrically loaded about their centrelines.

2.9.2 Unpropped Construction

The use of unpropped construction was a firm objective of the Imperial College proposal (1.1) and the Advisory Committee. At the

Ultimate Limit State, the strength of the composite section is the same for propped and unpropped construction. The disadvantages of unpropped construction are that the strength of the steel section at the deck pouring stage might govern the design, and a check must be made for Premature Yield. The advantages are that the camber deflection is easily separated from the service deflection, and, since most of the long term loads are carried by the steel section, the effects of creep are small. However, the overriding consideration is speed of erection, a major selling point for steel. With the possible exception of Stub Girder floors, propped construction is seldom used in modern multi-storey building practice.

2.9.3 Light Plate Girders with Unstiffened Webs

In this work, Light Plate Girders are offered as structurally sound and economic alternatives to universal rolled sections and other forms of construction. If the girders were designed with intermediate stiffeners, the high shop labour costs to install these stiffeners on the girder webs would greatly reduce the economic advantage of the light plate girders. The exception to this is in tapered plate girders, where double stiffeners are added to resist the local compression in the web due to the change in direction of the flanges, and to forestall possible bottom flange tripping. (Fig 7,8)

2.9.4 Light Plate Girders Fabricated from Stripped Plate

It is assumed that the light plate girders are fabricated from flanges and webs which are stripped from wide plates, and not from universal flats which conform to a rolling schedule of widths. This approach, which was endorsed by the Advisory Committee, will allow the user to design a girder cross-section with flange and web dimensions which will just satisfy the imposed conditions. As a consequence of this approach, there has been no attempt in the design studies to schedulize girder sizes or restrict flange widths to rounded values such as 100, 125, 150 mm, etc.

Although, for purposes of comparison, the design studies have been made at span depth ratios of 15, 20 and 25 which give rounded beam depths, the user has total freedom to use a beam depth which will just satisfy the floor zone requirement.

2.9.5 Size Limitations in Light Plate Girders

With the exception of the web slenderness, which is limited by BS 5950: Part 1: to $D_w/T_w < 250$ (Fig 10), the size limitations are fixed for reasons of shop and field handling, and distortion during fabrication.

The minimum sizes used are:

Web thickness: 5 mm

Flange sizes: 6 x 150 mm, 7 x 200 mm, 8 x 300 mm

Flange thicknesses are limited by rolling schedules to:

6, 7, 8, 9, 10, 12.5, 15, 20, 25 and 30 mm.

2.9.6 Symmetry in Light Plate Girders

The three types of girder symmetry, which are shown in Fig 11, are of constant width (B_b) and may be used in Parts 3, 4 and 5.

2.9.7 Truss Configuration and Components

The design procedure of Part 8 is based on a Warren truss with the configuration shown in Fig 26:A. The chords are hollow structural sections and the diagonals are pairs of unbattened equal leg angles, as shown in section at Fig 27:A. The list of material, which is available to the design procedure for the chords and diagonals, is shown in Data Banks A, B and C: Part 8: Designated Study (Volume 2).

2.10 Dead Loads

The dead loads used in the design studies are always input or calculated in all spreadsheets at Table 4, and include: structural steel at roof (W_{rs}) and floor (W_{fs}), profiled steel deck (W_{sd}), composite deck assembly wet (W_{cdd}) and dry (W_{cdw}), and mechanical services (W_{ms}).

2.10.1 Dead Load of Structural Steel, Profiled Deck and Mechanical Services

The structural steel is applied as a uniformly distributed load and not as a line load. This is very slightly inaccurate for floors

and roofs with Type 2 beams, Plan D (Fig 2), but greatly simplifies the arithmetic. The structural steel load includes an allowance of 10% for detail and shear studs, and an additional allowance of 0.10 kN/mm^2 for secondary members connected to Type 2 beams. After the first run of the programme this input value can be refined from the beam mass which is calculated from the cross-section dimensions.

The load of the profiled steel deck is taken from proprietary literature [20, 21].

The load of mechanical services is taken as 0.3 kN/m^2 . This value is at best an estimate and is the value approved by the Advisory Committee (Table 3).

2.10.2 Dead Load of Wet and Dry Composite Deck Assembly

This includes the combined mass of: the wet or dry concrete, the profiled deck, the weight of reinforcing fabric, and an allowance for additional concrete due to "ponding".

The weight of wet and dry concrete included in the deck assembly is calculated from the concrete density and deck properties input to spreadsheets at Tables 2 and 3. The cross-sectional area (A_{cc}) and other properties of the profiled composite deck are derived by the programme from the input of Deck Type (1,2,3) and depth (T_o Fig 12) and the formulae given in Appendices 7 and 8. The concrete densities (W_{cw} , W_{cd}) are taken from BS 648 [14].

When concrete is poured, the unpropped profiled decking will deflect and in a level screeded floor an additional thickness of concrete will be formed between the beams. The additional concrete is called "ponding". The amount of "ponding" depends on the stiffness and continuity of the profiled deck. Formulae for different deck span conditions are given by Chien and Ritchie [27], and, with their work as a guide, a value of 6% has been used in these studies.

The weight of reinforcing fabric is taken as 0.04 kN/m^2 , and the mass of the wet and dry composite deck assembly is given by:

$$W_{cdw} \text{ (wet)} = (1.06W_{cw}A_{cc} + W_{sd} + 0.04) \text{ kN/m}^2$$

$$W_{cdd} \text{ (dry)} = (1.06W_{cd}A_{cc} + W_{sd} + 0.04) \text{ kN/m}^2$$

2.11 Imposed Loads

The imposed loads used in the design studies are always input to the spreadsheets at Table 5 of Parts 2 to 8, and include: occupancy (W_1), moveable partitions (W_{pa}), snow (W_s), stage 1 construction load (W_{c11}), and stage 2 construction load (W_{c12}).

2.11.1 Imposed Load of Occupancy, Moveable Partitions and Snow

In these studies, the occupancy load is 4.0 kN/m^2 and is the value suggested by the Advisory Committee (Table 3). It is worth noting that the original studies were directed to office space requirements in multi-storey buildings, and that BS 6399: Part 1 [8] gives a value of 2.5 kN/m^2 for "general office use". The value given by the code for corridors, hallways, stairs, etc., is 4.0 kN/m^2 . Many of the long span composite designs are governed by the deflection of the composite section where the occupancy load is 75% of the total load. (Fig 48, 50, and 51). The stiff, short span, concrete alternative is not governed in this way and it would seem prudent to ascertain a reasonable value for occupancy load before proceeding with a competitive design.

The imposed load due to moveable partitions is taken as 1.0 kN/m^2 and is the value suggested by the Advisory Committee (Table 3) and BS 6399: Part 1.

The snow load is taken as 1.50 kN/m^2 which is the value given in BS 6399: Part 1: for flat roofs where access is provided.

2.11.2 Stage 1 and 2 Construction Loads

The stage 1 construction load is taken as 0.3 kN/m^2 . It is an allowance for operatives, hand tools, welding equipment, and stacking of steel deck, when the steel deck is being installed but has not been secured to the floor steel.

The stage 2 construction load is taken as 0.5 kN/m^2 . It is primarily an allowance for the heaping of concrete during placing, but also the smaller type loads of stage 1.

When the values given in Table 3 were approved by the Advisory Committee, there was no clear guidance in British practice for Stage 1 and 2 construction loads. The values used were taken from Canadian practice, as given by Chien and Ritchie [27]. The Draft BS 5950:

Part 3.1: [2] gives a value for stage 2 of 0.5 kN/m^2 .

2.12 Load Factors

The load factors are taken from BS 5950: Part 1: [1], and in the design studies are always entered in the spreadsheets at Tables 4 and 5. The values are:

Ultimate Limit State: Dead load = 1.4, Imposed Load = 1.6

Serviceability Limit State: Dead load = 1.0, Imposed Load = 1.0

2.13 Load Combinations and Structural Performance Ratios

A major disadvantage of unpropped construction, which was not discussed above (2.9.2), is the lengthy design process which it entails. The steel section must be checked for stability and strength before and during the placing of concrete. The separation of working loads carried by the steel and composite section may cause premature yield in the bottom flange of the steel section. This is a condition which must also be checked. The load separation also makes it necessary to check the deflections of both the steel and composite sections.

In this work, a minimum of eight design checks are routinely made during each design procedure, four at the Ultimate and four at the Serviceability Limit States. A system of "Structural Performance Ratios" is used to identify the load case and assess member performance in the eight design checks. In Parts 2 to 8 (Volume 2), the performance ratios are displayed in a table at the end of the spreadsheet design procedure, which allows the user to see at a glance whether the section being studied has passed, or failed, and how to further refine the design. A typical set of ratios is shown in Part 2: Designated Design Study No.23: Table 26: Summary of Structural Performance Ratios. The mnemonics used for the ratios indicate whether they apply to the Ultimate (R_{u1}) or Serviceability (R_{s1}) Limit States.

In this section of the work (2.13.1 to .8), the meanings of the performance ratios for the solid webbed members studied in Parts 2 to 7 are explained. The performance ratios for trusses, which are somewhat more numerous and may involve interaction relationships, are explained in the truss design section (Chapter 5).

2.13.1 Structural Performance Ratio - R_{u1}

This check only applies to Type 1 Beams. In Floor Plans B and C the steel section is unrestrained between columns or floor bracing until the steel deck is secured to the floor steel. The performance ratio measures the resistance of the steel section in lateral torsion buckling (M_b) with the lines of bracing supplied (if any) to the maximum factored bending moment (M_{u1}), which is calculated from the total factored load (W_{u1}). The load combination for this design check includes: structural steel (W_{fs}), profiled steel deck (W_{sd}), and Stage 1 construction load (W_{c11}). The total factored load and the performance ratio are given by:

$$W_{u1} = L_b C_{pb} [1.4(W_{fs} + W_{sd}) + 1.6W_{c11}]$$

$$R_{u1} = M_{u1}/M_b$$

2.13.2 Structural Performance Ratio - R_{u2}

This design check is made at the construction stage when the steel deck has been secured to the floor steel, and the wet concrete has been poured but has not set. The load combination at this stage includes: structural steel (W_{fs}), wet composite deck (W_{cdw}), and Stage 2 construction load (W_{c12}).

The design checks are different for Type 1 and 2 Beams.

For Type 1 Beams, it is assumed that the steel deck gives continuous restraint to the top flange, and the performance ratio measures the resistance of the steel section (M_{rs2}) to the factored bending moment (M_{u2}) at the point considered.

For Type 2 Beams, it is assumed that a steel deck with the profile running parallel to the beam cannot restrain the beam and restraint is only provided at the secondary beam connections (Fig 12). This assumption is included in the Draft BS 5950: Part 1.3 [2]. The governing performance ratio is the greater of two values: Firstly, the resistance of the steel section in lateral torsion buckling (M_b) between the secondary members to the the maximum factored bending moment (M_{u2}). Secondly, the resistance of the steel section (M_{rs2}) to the factored bending moment (M_{u2}) at the point considered.

The total factored load and the performance ratio for a

Type 1 Beam are given by:

$$W_{u2} = L_b C_{pb} [1.4(W_{fs} + W_{cdw}) + 1.6W_{cl2}]$$

$$R_{u2} = M_{u2}/M_{rs2}$$

The total factored load and the performance ratio for a Type 2 Beam are given by:

$$W_{u2} = L_b C_{pb} N_{sb} [1.4(W_{fs} + W_{cdw}) + 1.6W_{cl2}]/N_{sp}$$

$$R_{u2} = M_{u2}/M_{rs2} \text{ or } M_{u2}/M_b, \text{ whichever is greater.}$$

2.13.3 Structural Performance Ratio - $R_{u3}(\text{Bending})$

This strength design check is made on the composite section. The load combination at this stage includes: structural steel (W_{fs}), dry composite deck (W_{cdd}), occupancy (W_1), moveable partitions (W_{pa}), and the estimated allowance for mechanical services and ceilings (W_{ms}). The performance ratio measures the resistance of the composite section (M_{rc}) to the factored bending moment (M_{u3}) at the point considered, and has the same meaning for Type 1 and 2 Beams.

The total factored load for a Type 1 Beam and the performance ratio for both types of beam are given by:

$$W_{u3} = L_b C_{pb} [1.4(W_{fs} + W_{cdd} + W_{ms}) + 1.6(W_1 + W_{pa})]$$

$$R_{u3} = M_{u3}/M_{rc}(\text{Bending})$$

2.13.4 Structural Performance Ratio - $R_{u3}(\text{Shear})$

It is assumed in this work that all vertical shear is carried by the web of the steel section. In light plate girders, if this condition is not satisfied at the end of the member where there is only vertical shear, the other design checks are superfluous.

The critical shear strength of an unstiffened web (V_{cr}) is tabulated in Appendices 5 and 6 for a range of web thickness (T_w), and web depth (D_w) in both steel grades. These tables give the user a starting point from which to design a light plate girder. The web in

both the steel and composite sections must still be checked at points along the beam for the interactive effects of shear, edge loading, web bending, and web compression.

The performance ratio measures the resistance of the unstiffened web (V_{cr}) to the factored end shear.

$$R_{u3}(\text{Shear}) = 0.5W_{u3}/V_{cr}$$

2.13.5 Structural Performance Ratio - R_{s1}

This design check is made on the steel section to find the Calculated Camber (D_{ca}). In these design studies it is assumed that the steel section will be cambered for the combined deflection of the steel section (D_{f1}) at the wet deck stage, and that of the composite section (D_{fs}) due to shrinkage strain in the concrete.

The load combination at the wet deck stage includes: structural steel (W_{fs}) and wet composite deck (W_{cdw}). The total unfactored load on a Type 1 Beam is:

$$W_{s1} = L_b C_{pb} [W_{fs} + W_{cdw}]$$

The shrinkage deflection (D_{fs}) is produced by the shrinkage force (P_{sh}) acting at the centroid of the composite deck, as shown in Fig 19.

The performance ratio is the camber deflection divided by a fixed limit of $L_b/250$:

$$R_{s1} = D_{ca}/(L_b/250), \text{ and } D_{ca} = D_{f1} + D_{fs}.$$

A limit of $L_b/250$ is placed on the Calculated Camber to avoid an excessive amount of extra concrete due to "ponding", which would arise in a level screeded floor if the beams were under or over cambered. The true camber is difficult to estimate because of the restraining effects of end connections, the profiled deck, and secondary beams. The one sure way to keep the problem in bounds is to put a reasonable limit on the Calculated Camber. The steel section is usually Shop Cambered for about 65% of the Calculated Camber.

2.13.6: Structural Performance Ratio - R_{s2}

This design check is made to find the Service Deflection (D_{max}) of the composite section. The deflection is the sum of the deflections due to the service loads (D_{f2}), partial shear connection (D_{pc}), and the long term portion of the service loads (D_{f2+}) - the actual creep deflection.

The load combination for the service deflection includes: the occupancy (W_1), moveable partitions (W_{pa}), and mechanical services (W_{ms}). The total unfactored service load on a Type 1 Beam is:

$$W_{s2} = L_b C_{pb} [W_1 + W_{pa} + W_{ms}]$$

The long term portion of the service load in a Type 1 Beam is given in the Draft BS 5950: Part 3.1: [2]

$$W_{s2+} = L_b C_{pb} [0.33(W_1 + W_{pa}) + W_{ms}]$$

The performance ratio is the Service Deflection divided by the fixed limit of $L_b/360$ given in BS 5950: Part 1: [1].

$$R_{s2} = D_{max}/(L_b/360), \text{ and } D_{max} = D_{f2} + D_{f2+} + D_{pc}$$

2.13.7 Structural Performance Ratio - R_{s3}

As mentioned above (2.13), a disadvantage of unproped construction is the necessity to check for premature yield at the bottom flange of the steel section, a condition which may cause a permanent deformation in the member under serviceability loading.

The elastic stress in the bottom flange is the sum of the stress in the steel section (F_{bt1}) from the deck pouring stage loads (W_{s1}), and the stress in the composite section (F_{bt2}) from the service loads (W_{s2}).

In the design studies, the total stress in the bottom flange is limited to $0.9F_y$, and the performance ratio is given by:

$$R_{s3} = (F_{bt1} + F_{bt2})/0.9F_y$$

2.13.8: Structural Performance Ratio - R_{s4}

In this work, the dynamic response of the long span composite floor members has been assessed by the method given in the Canadian "Guide to floor vibrations" [50]. In Canadian practice, the annoyance threshold to floor vibrations is taken as a function of the Peak Acceleration (A_o), the Percentage of Critical Damping (PCD), and the Fundamental Frequency of the member (Hz). These properties are applied in the manner shown in Fig 53 to 56.

In the design studies it has been assumed that the floor assembly has a PCD of 4.5%, which corresponds to a Peak Acceleration (A_{ot}) of 3%G. The performance ratio is given by:

$$R_{s4} = A_o / A_{ot}$$

2.13.9 Hierarchy in the Performance Ratios

The constant objective of the studies was to obtain the minimum mass beam for the given conditions, and where possible have the design governed by R_{u3} or R_{s2} which would govern a propped beam. The lines of floor bracing supplied, the use made of partial shear connection which reduces strength and increases deflection, shear stud distribution, section symmetry, etc., were all made subordinate to this objective.

CHAPTER 3: REVIEW OF SOURCE MATERIAL AND APPROPRIATE DESIGN CRITERIA

3.0 General Remarks on Source Material

As explained above (1.4), during the early design studies at Imperial College, the designers experienced considerable difficulty in the selection of a number of important design parameters within which to design long span composite beams suitable for a modern building. The principal barrier to fixing these parameters was the absence of a modern British code on composite construction in buildings.

To overcome the absence of recent codified guidance on composite beams, and the complete dearth of comparable design studies in the field, the important design criteria have had to be selected from several sources. In this selection, the guiding principal has been to choose design values and procedures which have been or are likely to be codified in British practice, and should therefore have a broad support by practising engineers.

In matters such as web penetrations and composite trusses, where there does not appear to be even a tentative move to codify specific design procedures, the work of researchers and engineers with an established reputation in their field has been used.

3.1 Review of Source Material

The important parameters used in this work, with the sources from which they are derived, are given below, and are compared with those used in the early Imperial College studies and by other authorities and authors. Wherever practicable, the values are taken from design codes which are of recent publication or in a draft version, and a review is now given of the use made of such sources.

3.1.1 BS 5950: Part 1: Code of practice for design in simple and continuous construction: hot rolled sections, 1985 [1]

Since this is a modern document with wide acceptance in both the practice and teaching of structural steel design, it has been a firm objective of both the early and the present studies to apply the provisions of this code to the design of the steel section and, when appropriate, extend them to the composite section.

However, there are three important provisions of BS 5950, which are thought to be inappropriate or inadequate, and have been modified for inclusion in this work. These are:

1) **Section Compactness (Clause 3.5):** The code classifies steel sections as plastic, compact, semi-compact, or slender. However, it also requires that all sections with $D_w/T_w > 63K_s$ be checked for the interactive effects of shear and edge loading on web buckling. Since the plastic classification does not apply to simply supported members, it is found more convenient to adopt the approach taken in the bridge specification, BS 5400: Part 3 [4], and classify sections as compact or non-compact. In this work then:

A compact steel section can sustain plastic stresses with full yield (F_y) in the flange, but may have reduced stresses (F_{yr}) in the web, when $63K_s < D_w/T_w < 98K_s$ (Fig 13:A and 14:A)

A non-compact steel section sustains elastic stresses, which may be reduced in the flange (F_{yf}) and the web (F_{yr}) (Fig 13:B and 14:B)

2) **Interaction Equation for Webs (Clause 4.4):** In BS 5950 the interaction equation for the effects of shear, edge loading, bending, and compression in webs is given in its Appendix H.3. In this work the equation has been used in the form given in BS 5400: Part 3 [4], which is explained by Harding and Dowling [33] and Chatterjee [34]. In the absence of edge loading, the equations of BS 5950 and BS 5400 take the same form and agree with that given by Johnston [36]. In the absence of web compression and the presence of edge loading, the forms are markedly different. Whereas the interaction equation given in BS 5400 is the product of considerable research and professional scrutiny, there appears to be no such authority for the form used in BS 5950.

(3) **Web Openings (Clause 4.15):** The provisions of the code are totally vague and directed to castellated beams. In this work, the design procedures for checking the resistance of unreinforced and reinforced rectangular openings in universal sections are derived from research and industrial practice in Canada.

3.1.2 Draft Eurocode 4: Composite steel and concrete structures, Oct 1984 [6]

This draft code, which has been subjected to lengthy consideration and put forward for public scrutiny in a commentary by Smith and Johnson [23], has a number of provisions which greatly simplify the

design process. The provisions, which have been embodied in principle in these design studies, are:

1) Compactness of Composite Section (Clause 4.2): The composite section can be classified as compact from the position of the Plastic Neutral Axis (PNA) in the composite section. Previously, composite sections were deemed to be compact and designed to plastic stresses only if the steel section itself was compact, or in the case of simply supported beams, the PNA of the composite section lay in the concrete or the compression flange of the steel section. For example, this is the method used in BS 5400: Part 5 [5]. The code gives the limit of web compactness as $3lK_s(D_w/d_1)$.

2) Compactness of Steel Flange in Composite Section (Clause 4.2.2.1(a)): The steel compression flange may be classified as plastic if it is attached to the concrete flange by shear connectors to the provisions of the code. The code's requirements for shear connection are not onerous and would be met in most composite designs. This provision greatly simplifies the design of the composite section by plastic theory. The absence of this rule greatly complicated the early design studies.

The validity of this provision is somewhat borne out by the Imperial College test [32], as reported by Raven and Glover [30]. In their paper, the growth of stress at the bottom and top steel flanges is plotted for the full working load of 500 kN as it is gradually applied to the composite section. The stress range at the bottom flange is given as 100 to 300 N/mm², while the top flange range is only 55 to 70 N/mm².

3) Shrinkage Strain in Concrete (Clause 3.1.3): The values of shrinkage strain in concrete, applicable to the British environment, are given as 200 and 300 micro-strain for normal weight and lightweight concrete respectively. These straightforward values, which are not cloaked in graphs and vague rules for interpreting same, agree with those used by Chien and Ritchie [27], and are used in this work.

3.1.3 Draft BS 5950: Part 3.1: Code of practice for design of composite beams, March 1988 [2]

If this code, even in its present draft form, had been available at the start of this work, the selection of many important design values and procedures for designing the composite section would have been greatly simplified.

Most, but not all, of the design values and procedures used in the studies are now the same as those used in this draft code. In particular, they both use the same values and concepts for:

- i) effective breadth of composite flange (Clause 3.6.4).
- ii) effective thicknesses of composite flange which depend on the span direction of profiled decks (Clause 3.6.2).
- iii) design strength of concrete in plastic design (Clause 3.7.1).
- iv) static strength of stud shear connectors (Clause 4.4.6).
- v) the concept that steel flanges connected to the concrete may be considered compact (Clause 3.8.2).
- vi) the concept that the moment of resistance of both compact and non-compact composite sections may be derived from plastic stress distributions (Clause 3.8.4).

These design values are discussed and compared with those used by other authorities later in this chapter.

There are two important areas in which the design provisions of these studies differ significantly from those of the draft code of practice. Both relate to the method used to calculate the moment of resistance of the composite section.

1) Plastic Moment Capacity of Compact Composite Section (Clause 3.7.1): In the draft code the composite section is considered compact where $D_w/T_w < 80K_s(D_w/2d_1)$. However, the code also requires that the web be checked for the effects of shear buckling in accordance with BS 5950: Part 1 [1], when $D_w/T_w > 63K_s$.

The draft code then imposes a further check on sections where the shear is high. A reduced moment capacity is calculated with an interaction formula which takes account of shear but not edge loading (Clause 4.3.4). This approach seems flawed. Since the BS 5950: Part 1 web buckling check allows for the interactive effects of shear and edge loading on the web bending stresses, it would seem unnecessary to further reduce the section strength for the effects of the same shear.

2) Plastic Moment Capacity of Non-compact Composite Section (Clause 3.8.4): In the draft code, semi-compact and slender composite sections are also based on plastic stresses but are given a reduced moment capacity by removing an area of web above the PNA. This method is similar to one proposed by Johnson and Buckby in their volume on composite bridges [25]. The disadvantages of this approach are that it does not satisfy force balance (ie. longitudinal equilibrium) in the section

and leaves a stress pattern in the web which is difficult to apply to the interaction equation.

In the design studies, as with the draft code, a plastic stress distribution is assumed for all sections. However, the moment of resistance of the composite section is calculated with an effective web thickness (T_{we}), which is found by satisfying the web buckling equation with the actual shear and edge loading in the presence of a reduced bending stress (F_{yr}) (Fig 17). This procedure, which satisfies both the interaction equation and force balance in the composite section, is applied to compact and non-compact sections and is explained below in Chapter 4. The method requires some iteration but takes only seconds with the spreadsheet format.

3.1.4 BS 5400: Part 5: Code of practice for design of composite bridges [5]

In the early Imperial College design studies, serious consideration was given to using the bridge design codes, Parts 3 and 5, for the design studies, since they were both documents founded on recent research, close professional scrutiny, and had had by that time considerable usage in industry. It was eventually decided that the provisions of these codes were probably too refined for most building structures and, perhaps more importantly, lacked a satisfactory method for treating compactness in composite sections.

However, because of the established use of this code, a number of important design variables were selected from it:

- i) nominal static strength of shear stud connectors in solid slabs and the strength reduction for use with lightweight structural concrete (Clause 5.3.2).

- ii) design strength of regular and lightweight concrete in longitudinal shear (Clause 6.3.3.2).

- iii) modular ratio for long term loading (Clause 4.2.3).

3.1.5 BS 5400: Part 3: Code of practice for design of steel bridges [4]

While there are no provisions of this code which have been directly embodied in these studies, there are two important concepts which have been adopted:

1) Effective Web Thickness (T_{we}) and the Moment of Resistance of Non-compact Steel Sections: In the code, the reduced pure bending resistance of a non-compact steel section is calculated with an effective web thickness (T_{we}) (Clause 9.4.2.5) and a reduced compression flange bending stress (Clause 9.4.2.4). The shear impairment of the bending resistance is found from an interaction equation (Clause 9.9.3). While the code procedure is easy to apply, its main shortcomings are the lack of provision for edge loading and the use of compactness limits which are much less than those given in BS 5950: Part 1.

2) Interaction Equation for Webs (Clause 9.11.4.4): For the reasons given above (3.1.1), the interactive effects of bending, compression, shear, and edge loading on the web resistance in steel and composite sections are calculated with an expression in the form given in this bridge code. However, the values used for resistance to buckling are the elastic critical values given in BS 5950: Part 1, which tend to be more conservative for the unstiffened webs and depth-thickness ratios studied here.

3.1.6 CAN3-S16.1-M84: Steel structures for buildings (Limit state design), Canadian Standards Association [7]

In the design studies, several design problems arose for which no useful guidance could be found in British practice, and to fill these voids resort was made to the Canadian Steel Structures for Buildings Code, and the Handbook of Steel Construction published by the Canadian Institute of Steel Construction [17]. The provisions of these documents, which have been adopted in the studies, are:

1) Plastic Moment Capacity of the Composite Truss (Clause 17.4): This code requires that the resistance of the composite truss be provided by the concrete flange and the bottom chord, and that the top chord be neglected in the calculation. The effects of this provision are twofold. Firstly, the Plastic Neutral Axis will always lie in the concrete. Secondly, the option of partial shear connection cannot be considered in the design process.

2) Floor Vibrations (Clause 6.2.3 and Handbook Appendix G): The criteria for assessing human annoyance and perception to floor vibrations in quiet occupancies such as residences, schools and offices, is shown in Fig G3 of the handbook. The method, which is easy to use, has been codified for five years and has wide acceptance in Canada.

3) Deflection of Composite Sections due to Shrinkage Strain in Concrete (Clause 17.3.1.1(c) and Handbook Appendix L): The method is shown in our Fig 19 for prismatic and non-prismatic sections and is explained below (3.7.1).

3.1.7 Unreinforced and Reinforced Web Openings in the Steel and Composite Sections - Prof.Redwood et al [37 to 46]

1) Web Openings in the Steel Section: Design formulae for the ultimate capacity of steel sections at web openings and limitations on their application are given in the Canadian Handbook of Steel Construction, and are based on the recommendations of Redwood and Shrivastava [41]. In the handbook, formulae are given in non-dimensionalized forms which are unduly complicated and obscure the effect of eccentricity at the opening on the perforated section strength. The formulae, which are used in the studies for unreinforced and reinforced web openings, are derived from the work of Redwood [39,44] in Appendices 11 and 12. These design methods are based on exhaustive tests and have been applied in Canadian practice for some twenty years.

2) Web Openings in the Composite Section: Design formulae for the ultimate capacity of composite sections at unreinforced web openings are derived from the papers by Redwood and Poubouras [38,39]. The analytical methods put forward in these papers have been justified by a number of full-scale tests.

The formulae for reinforced web openings have been derived by the writer by extending the simple design approach devised by Redwood for the steel section to the composite section. Formulae for both cases are derived in Appendices 13 and 14.

3.1.8 Proprietary Literature [20 and 21]

The properties and load carrying capabilities of profiled steel decking commonly used for composite floor systems in British practice has been taken from producers' literature [20,21], which state that the designs have been made in accordance with BS 5950: Part 4 [3]. At this time, the maximum span for unpropped decking would appear to be 3000 mm, and is the value used in the studies.

3.2 Materials and Design Values

In this section of the work the important material and design values employed in the studies are discussed. These values are compared with those used by other authorities on composite construction. The manner in which they are entered or calculated by the spreadsheet computer programme is explained, and can be followed by perusal of the Designated Studies in Parts 2 to 8 (Volume 2).

3.2.1 Structural Steel, Reinforcing Fabric and Profiled Steel Deck

There is nothing contentious about the design values chosen for these materials, which are taken from BS 4360 [11], BS 4482 [12] and proprietary literature [20,21]. They are the design values used by practising engineers and are tabulated in Appendix 3: Table 1 for user convenience only.

The design strength for the reinforcing fabric (F_{ry}) and profiled steel deck (F_{dy}) are always directly entered at Tables 2 and 3 of the spreadsheets.

The Structural Steel Grade, 43 or 50, is always input at Table 2. The steel design value (F_y), which is fixed by the thickness of the thicker flange, is generated by the programme in Tables 7 or 8 of the spreadsheets.

The Type of Deck (1,2,3) and other properties of the user selected decking, which are always entered at Table 3 of the spreadsheet, allows the programme to calculate the appropriate composite deck properties using the the depth of the composite deck (T_o) and the formulae given in Appendices 7 and 8.

3.2.2 Concrete

The design values for concrete used in the studies are given in Appendix 3: Table 2 for user convenience, and are always input at Table 2 in the spreadsheet. All the design studies are conducted with Grade 25 lightweight concrete, which is a "Standard Building Property" suggested by the Advisory Committee.

The values for the modulus of elasticity (E_c) are taken from BS 8110: Part 2 [10]. The long term modular ratio (m_2) is taken as twice the short term ratio (m_1) and both values are calculated by

the programme when required. The use of separate modular ratios rather than a unified value, which takes account of short and long term loading, allows the programme to calculate the actual creep deflection.

The design values for longitudinal shear (F_{chs}) and Shrinkage Strain (S_{sh}) are also input directly at Table 2 and, in all the studies, are held at 0.7 N/mm^2 and 300 micro-strain respectively. As with several others, these are somewhat contentious values, and it was thought prudent to apply them as direct entries rather than calculate them from concrete grade and density.

The design strengths of concrete in plastic ($0.45F_{cu}$) and elastic ($0.50F_{cu}$) failure are calculated by the programme from the cube strength (F_{cu}) which is always entered at Table 2 of the spreadsheet.

In this work, the plastic moment capacity of a composite section is calculated with a uniform concrete stress of $0.45F_{cu}$ over the full depth to the Plastic Neutral Axis. This value and the full rectangular stress block are design provisions of the draft versions of BS 5950: Part 3 [2] and Eurocode 4 [6], and are recommended by Smith and Johnson [23]. This is an important value. It can be seen from Table 7, that the early studies and a number of other authorities used a value of $0.4F_{cu}$.

Table 7: Comparison of Concrete Design Values - (factor) F_{cu} (N/mm^2)

:-----:-----:-----:-----:-----:-----:-----:-----:									
:Johnson: CIRIA :CONSTRADO: Draft :CSA 16.1:BS 5950 : Early :Present :									
: Vol 1 : No.99 :			:Eurocode:Canadian:Part 3.1:Studies :Studies :						
: [24] : [26] :		[18]	:	[6]	: [7]	:Draft[2]:	:		:
:-----:-----:-----:-----:-----:-----:-----:-----:									
: 0.40 : 0.40 :		0.40	:	0.45	: 0.41	: 0.45	:	0.40	: 0.45 :
:-----:-----:-----:-----:-----:-----:-----:-----:									

3.2.3 Shear Stud Connectors in Solid Slabs

The static strength (Q_{sd}) and design strength (P_{sd}) of 19 and 16 mm shear studs in a solid slab are given in Appendix 4: Table 3. The 19 mm stud is used wherever the connecting metal thickness is greater than 7 mm. The dimensions of the headed studs and the spacing limitations are shown in Fig 12:C.

The static strength values are taken from BS 5400: Part 5 [5]. The same values are used in the draft BS 5950: Part 3.1 [2]. In keeping with the bridge code, the design strengths of studs in normal grade concrete are found by the "80% design" rule, and a further reduction of 15% is applied to studs in lightweight concrete. The γ_m factor is not applied.

The design strength of a 19 mm x 100 mm shear stud connector in Grade 25 lightweight concrete is:

$$P_{sd} = 0.8(0.85Q_{sd}) = 0.8(0.85 \times 95) = 64.6 \text{ kN.}$$

This design value is slightly lower than that given by the draft BS 5950: Part 3.1, where the reduction for use with lightweight concrete is taken as 10%. The stud design value and other options, which apply to the composite member, are entered in the spreadsheet at the start of the composite design section, which is for example Table 13 of the Part 2 spreadsheet.

It can be seen from Table 8, where a selection of shear stud design values in vogue at the time of the original studies are given, that this is a troublesome value, which is apparently subject to erratic interpretation.

Table 8: Comparison of Design Values (P_{sd}) for a 19 mm Shear Stud in Grade 25 Lightweight Concrete (kN)

:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:
:Johnson:	CIRIA	:CONSTRADO:	Draft	:CSA 16.1:	BS 5950	: Early	: Present	:
: Vol.1	: No.99	:		:Eurocode:	Canadian:	Part 3.1:	Studies	:Studies :
: [24]	: [26]	: [18]	:	[6]	: [7]	:Draft[2]:	:	:
:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:
: 56.2	: 72.0	: 77.3	:	55.3	: 63.1	: 68.4	: 77.3	: 64.6 :
:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:	:-----:

3.2.4 Shear Stud Connectors with Composite Decks

The design strength (P_{sd}) of shear studs, when used with composite decks, may be reduced from the solid slab value. The reduction in strength depends on the direction and the geometry of the steel deck profile, and the number of studs in the cross-section. The reduction is measured by a factor (K_{sd}) which is applied to the solid slab design

strength.

For a Type 1 Beam (Fig 12:C):

$$K_{sd} = (0.85/\sqrt{N_r})(B_r/T_d)[(H_{st}/T_d) - 1] \leq 1.0$$

For a Type 2 Beam (Fig 12:C):

$$\begin{aligned} K_{sd} &= 0.6(B_r/T_d)[(H_{st}/T_d) - 1] \leq 1.0 \\ &= 1.0 \text{ when } B_r/T_d > 1.5 \end{aligned}$$

These formulae are given in the draft versions of Eurocode 4 [6] and BS 5950: Part 3.1 [2]. The meaning of the symbols and the value of B_r is given in Fig 12. In the design studies it was found that the Type 2 Beams were unaffected by this factor. The results for Type 1 Beams are shown in Appendix 4: Table 4, where it can be seen that the CF 46 deck, which is used in the design studies (Type 1 Deck), did not impair the stud design strength.

3.3 Composite Concrete Flange

3.3.1 Effective Breadths (B_e)

In this work, the same effective breadth (B_e) is used to calculate both the plastic and elastic properties of the composite member, and also takes account of the type of floor system. The values used for an internal floor beam are:

$$\text{For a Type 1 Beam: } B_e \leq 0.25L_b \leq C_{pb}$$

$$\text{For a Type 2 Beam: } B_e \leq 0.25L_b \leq 0.8C_{pb}$$

In a Type 2 Beam (Fig 6), the deck spans in the same direction as the composite beam, and the 20% reduction is an allowance for the local stresses in the concrete slab or composite deck, which may be additive. The values were taken from an early draft version of BS 5950: Part 3.1. In the early studies the effective breadth was taken as $0.2L_b$. In the spreadsheet, the effective breadth is calculated from the input of floor geometry and beam type at Table 1.

A selection of effective breadths for Type 1 Beams in common use at the time of the original studies, which are also limited to the spacing of the primary beams (C_{pb}), is given in Table 9.

Table 9: Comparison of Effective Breadths (B_e) - (factor) L_b
in Type 1 Beams

:-----:-----:-----:-----:-----:-----:-----:-----:									
:Johnson:	CIRIA	:CONSTRADO:	Draft	:CSA 16.1:BS 5950	: Early	:Present	:		
: Vol.1	: No.99	:		:Eurocode:Canadian:Part 3.1:Studies	:Studies	:			
: [24]	: [26]	: [18]	:	[6]	: [7]	:Draft[2]:	:		
:-----:-----:-----:-----:-----:-----:-----:-----:									
: 0.20	: 0.20	: 0.20	:	0.25	: 0.25	: 0.25	: 0.20	: 0.25	:
:-----:-----:-----:-----:-----:-----:-----:-----:									
** also governed by $16T_o + B_b$									

It is worth noting that the formulae for effective breadth now applied to the studies give values close to those given in BS 5400: Part 5 [5] for the aspect ratios associated with the long span beams.

3.3.2 Effective Thicknesses (T_c or T_o)

With a solid slab composite flange, the effective thickness is the full thickness of the slab. With a composite deck, it has often been the practice to use the concrete above the profile (T_c) only without regard to the direction of the profile relative to the beam. In all of the studies, the effective thickness is dependent on the beam type as shown in Fig 6 to 9. This is the approach taken by Johnson [24] and the draft versions of Eurocode 4 and BS 5950: Part 3.1.

The draft BS 5950: Part 3.1 (Appendix B) gives simplified formulae for the plastic moment capacity of Type 1 composite beams using profiled deck with an effective thickness of T_c . However, the code also suggests that these formulae can be applied conservatively to Type 2 beams, which have a concrete resistance that may already be reduced by using 0.8 of the effective breadth. The code’s simplified approach would considerably undervalue the resistance of some Type 2 composite beams and is not the approach taken here.

The spreadsheet programme generates the appropriate areas of concrete in compression (A_{cc}) and in the longitudinal shear plane (A_{cs}) in Table 3 for the beam type specified in Table 1. For a Type 1 Beam A_{cc} is calculated with T_c and A_{cs} with T_o , and the calculation is reversed for a Type 2 beam.

It can be seen from Table 10 that the composite deck in Type 2 Beams was often under utilized in earlier design methods.

Table 10: Comparison of Effective Thickness (T_c or T_o)

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
Beam:	Johnson:	CIRIA:	CONSTRADO:	Draft	:CSA 16.1:	BS 5950	: Early	:Present:	
Type:	Vol.1	:No.99:			:Eurocode:	Canadian:	Part 3.1:	Studies:	Studies:
:	:	[24]	:	[26]:	[18]	:	[6]	:	[7]
				:Draft[2]:				:	:
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:	1	:	T _c	:	T _c	:	T _c	:	T _c
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:	2	:	T _o	:	T _c	:	T _o	:	T _c
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									

3.4 Compactness Rules

3.4.1 Compactness in Steel Sections

In this work, as explained above (3.1.1), the steel section is classified as compact or non-compact. The compact section, which will support a plastic stress distribution, is defined by the limitations on width to thickness ratios given in BS 5950: Part 1: (Clause 3.5) and, in keeping with British practice, requires both the web and the flange to be compact. The flange is stressed to the full design strength (F_y), and the web, which is checked for buckling when $D_w/T_w > 63K_s$, may have an impaired plastic stress (F_{yr}). The stress distributions for symmetric and mono-symmetric compact sections are shown in Fig 13:A and 14:A.

The non-compact section, which will support an elastic stress distribution, includes semi-compact and slender sections which may have impaired flange (F_{yf}) and web (F_{yr}) stresses. As with the compact section, the reduced web design strength (F_{yr}) is derived from the web buckling check, when $D_w/T_w > 63K_s$. The reduced flange design strength (F_{yf}) is found from, $F_{yf} = K_f F_y$, where K_f is the stress reduction factor for slender cross-sections given in BS 5950: Part 1 (Clause 3.6.4). When the flange proportions are semi-compact K_f is 1.0 and $F_{yf} = F_y$. The assumed stress distributions for non-compact symmetric and mono-symmetric sections are shown in Fig 13:B and 14:B.

In the spreadsheet, the programme tests the input cross-section dimensions for compactness and uses the code: COMPACT = 1 and NON-COMPACT = 2. The test is made in Tables 7, 8 or 9 (see Parts 2 to 8

in Volume 2). In tapered light plate girders with compact flanges, where the top flange may be compact and the web compactness may vary, the section strength is determined by the compactness at the check locations.

The limiting width to thickness ratios for compact sections and the reduction factors for slender cross-sections are shown, in the forms used in the studies, at Table 11. Since steel sections with asymmetry about the minor axis are not considered, it is found convenient to use full rather than half flange widths as used in BS 5950: Part 1. The symbols are shown in Fig 10.

Table 11: Limits for Compact Sections and Stress Reduction Factors (K_f)

Type of Element	Compact Section	Reduction Factor
Rolled compression flange:	$B_b/T_f < 19K_s$	:
Welded compression flange:	$(B_b - T_w)/T_{tf} < 17K_s$	$:10/[(B_b - T_w)/2T_{tf}K_s] - 3$:
Web - rolled section	$D_w/T_w < 98K_s$	:
Web - welded section	$D_w/T_w < 98K_s (D_w/2d_1)$	:
HSS flange	$(B_b - 3T_w)/T_w < 32K_s$	:
Stem of Tee	$D_b/T_w < 9.5K_s$	$:14/[(D_b/T_w K_s) - 5]$:
Equal leg angle	$D_b/T_w < 9.5K_s$	:

$$K_s = (275/F_y)^{0.5}$$

d_1 = length of web in compression to PNA (Fig 14).

3.4.2 Compactness in Composite Sections

In these studies, as stated above (3.1.3), all unproped composite beams are considered to be compact, and can be designed to sustain the total dead and imposed loads with a plastic moment capacity. This provision is included in the draft BS 5950: Part 3.1, and the principal factors which allow this approach to be taken are:

1) The top flange is compact by virtue of its attachment by shear connectors to the concrete compression flange, and may work at the full design strength (F_y). (Fig 17:A(ii))

2) By stress shedding, the web may work at a reduced design strength (F_{yr}) which satisfies the web buckling check for the interactive effects of shear, edge loading, tension or compression, and

bending, as provided in BS 5950: Part 1 (Fig 17:A(ii)). Where the PNA lies in the concrete or steel flange the web buckling check is made with the applied shear and edge loading in the presence of tension (Fig 15:A and B).

The tapered girder tested at Imperial College [32] had neither a compact flange nor web to the provisions of BS 5950: Part 1, and yet carried the total factored load with ease.

3.5 Shear Stud Distribution

The design procedures allow the shear studs to be distributed with either uniform (Type 1) or triangular (Type 2) density. This option may be selected when the shear stud properties are entered at the beginning of the composite design section (ex. Part 2: Table 13).

The obvious effect of triangular distribution is to increase the strength of the composite section at locations away from the point of maximum bending moment. This effect is shown in Fig 20 for an LPG(C) spanning 18 metres. It can be seen that the moment of resistance of the composite section (M_{rc}) is the same at the point of maximum bending moment for both types of stud distribution, but the triangular distribution considerably enhances the resistance near the end of the beam. The strengthening effect is particularly marked with 50% shear connection. It is particularly useful with composite beams using LPG(T) and LPG(M) members where the resistance of the steel section (M_{rs2}) may be quite small near the end of the taper.

While triangular stud distribution is a marginal benefit to Light Plate Girders, it is an essential requirement in the design of composite trusses. From Fig 16:B, it can be seen that the moment of resistance of a composite truss (M_{rct}) is provided by the concrete flange and the bottom chord of the steel truss. This is a provision of the Canadian Steel Structures Code [7] which is applied in these studies. It can also be seen from Fig 9 that the resistance of the steel truss (M_{rt2}) at the connection is almost zero. The effects of stud distribution on the moment of resistance of the composite section (M_{rct}) is shown in Fig 21. Whereas the triangular distribution provides adequate strength, the uniform distribution allows the resistance to fall off linearly and lie within the applied bending moment envelope.

A less obvious application for triangular distribution to

Light Plate Girders can be seen in Fig 16:A(i) and (ii). Here the PNA lies in the web and, as the resistance of the concrete reduces, d_1 increases. That is to say, the length of web in compression increases and the web becomes more susceptible to buckling.

The direct effect of triangular distribution is to increase the resistance of the concrete (R_c) at points away (x) from the maximum bending moment (L_{max}). If the resistance of the concrete at the point of maximum bending moment is:

$$R_{c(max)} = 0.45F_{cu} B_e A_{cc} \%C_o, \text{ then}$$

For uniform density stud distribution:

$$R_c = R_{c(max)} (x/L_{max}) \text{ (linear)}$$

For triangular density stud distribution:

$$R_c = R_{c(max)} [(2x/L_{max}) - (x/L_{max})^2] \text{ (parabolic)}$$

3.6 Partial Shear Connection ($\%C_o$)

Partial shear connection is the technique of providing less shear connectors than are required to develop the full resistance of the concrete at the point of maximum bending moment. In the studies, it is measured as a percentage ($\%C_o$) of the stud connectors necessary for full connection, and is an option entered in the spreadsheet with the stud design strength, etc., at the start of the composite design section (ex. Part 3: Table 15). It is used with a number of limitations:

1) The percentage connection ($\%C_o$) is never less than 50%, which is a requirement common to most codes.

2) It is not applied to members spanning more than 20 metres. This is specified in the draft versions of Eurocode 4 and BS 5950: Part 1.3.

3) It is not applied to composite trusses. This is a requirement of the Canadian Code [7].

The important advantage is the substantial reduction in the number of shear stud connectors on beams which are heavily governed by deflection, premature yield, or the steel section strength. For example, all the roof beams studied in Part 2 and 3 met the design conditions with 50% connection.

The disadvantages of partial shear connection are:

- 1) It reduces the moment of resistance (M_{rc}) of the composite section as shown in Fig 15 and 16.
- 2) It lowers the PNA in the composite section.
- 3) It increases the deflection (D_{pc}) of the composite section, which can be calculated by the formula given by Johnson [24] for unpropped construction as:

$$D_{pc} = 0.3D_{f2}[(I_t/I_x) - 1](1 - 0.01\%C_o)$$

For example, Study No.29 of Part 3 uses 75% connection which increases the service deflection by 15% (ex. Part 3: Table 25).

3.7 Effects of Shrinkage and Creep

The stresses produced by shrinkage and creep are not considered in the performance of the composite section at the Ultimate Limit State. The deflections of the composite section due to shrinkage and creep are calculated. These are the provisions made for effects of shrinkage and creep in the draft versions of Eurocode 4 [6] and BS 5950: Part 3.1 [2].

3.7.1 Shrinkage Deflection (D_{fs})

It is assumed that the deflection of the composite section due to shrinkage of the concrete (D_{fs}) will be all but complete before the floor is in service and, in the studies, this deflection is considered to be a component of the Calculated Camber (D_{ca}).

The deflection of a prismatic and tapered member due to the shrinkage force (P_{sh}) is shown in Fig 19. For the prismatic member, it can be seen that this force produces a constant bending moment (M_{sh}) in the composite beam. It is assumed that the shrinkage force is produced by the area of concrete within the effective breadth (B_e), and not by all the concrete available to the beam (C_{pb}).

An example of the shrinkage deflection calculation is given in Part 2: Table 19. In this example, the shrinkage deflection is 23% of the Calculated Camber (D_{ca}).

The shrinkage deflection for a prismatic member is calculated from:

$$P_{sh} = S_{sh} E_c B A_{cc}$$

$$M_{sh} = P_{sh} (Y_{es} - Y_{cd}) = S_{sh} E_c B A_{cc} (Y_{es} - Y_{cd})$$

$$D_{fs} = M_{sh} L_b^2 / (8 E_s I_t) \text{ for a constant bending moment.}$$

$$= S_{sh} B A_{cc} (Y_{es} - Y_{cd}) L_b^2 / (8 m_1 I_t)$$

3.7.2 Creep Deflection (D_{f2+})

The actual creep deflection (D_{f2+}) is calculated as a component of the service deflection (D_{max}). The modular ratio for the long term loads (m_2) is taken as twice the ratio for the short term loading (m_1). The portion of the imposed loads, which is taken as long term loading and given above (2.13.6), is the loading recommended in the draft BS 5950: Part 3.1.

The actual creep deflection is calculated from:

$$D_{f2+} = K_{df} W_{s2+} L_b^3 [(1/I_{tc}) - (1/I_t)] / E_s$$

K_{df} is a deflection factor which takes account of the type of loading on the beam. A typical creep deflection calculation is shown in Part 2: Table 20. It can be seen that the creep deflection is only 6% of the Service Deflection (D_{max}).

3.8 Longitudinal Shear Reinforcement (R_r)

It is assumed that the critical sections for failure in longitudinal shear is along the planes adjacent to the steel beam, as shown in Fig 12:C. The length of the shear planes (L_{sp}) is the distance between the design check locations, and the force on the shear planes is the drop in the resistance of the concrete over L_{sp} . The design procedure is arranged to find the percentage of reinforcement (R_r) required, and is found from the formulae:

$$V_{ur} = V_{hs} - V_{uc} - V_{usd}$$

$$\begin{aligned} V_{hs} &= \text{the longitudinal force in the shear plane.} \\ &= 0.5(R_{cx1} - R_{cx2}) \quad \text{for a Type 1 beam} \\ &= 0.5(R_{cx1} - R_{cx2})(B_e - 1.2B_b)/B_e \quad \text{for a Type 2 beam} \end{aligned}$$

$$\begin{aligned} V_{uc} &= \text{longitudinal shear capacity of the slab.} \\ &= F_{chs} A_{cs} L_{sp} \end{aligned}$$

For a Type 1 beam the area of the shear plane is the gross area of the composite deck including the profile, and for a Type 2 beam the area is that of the continuous slab.

$$V_{usd} = \text{longitudinal shear capacity of profiled deck.}$$

The draft BS 5950: Part 3.1 (Clause 4.6.4) has provisions for the profiled steel deck to act as transverse reinforcement. Since the contribution of the deck will depend on its continuity at the beam and the strength of its connection to the beam, the value placed on this resistance should be conservative and based on a knowledge of the deck splicing and method of fixing.

$$\begin{aligned} V_{ur} &= \text{longitudinal capacity of the reinforcement.} \\ &= 0.87(R_{r cs} F_{ry}) L_{sp} \end{aligned}$$

The area of reinforcement required in the shear plane can be found from this relationship. A typical calculation for a Type 1 Beam is shown in Part 5: Table 29. The minimum amount of reinforcement fabric allowed is 0.1%, and this amount is provided should the calculation require less. The calculation for a Type 2 Beam can be seen in Part 2: Table 23.

CHAPTER 4: DESIGN APPROACH ADOPTED FOR STUDIES ON SOLID WEBBED SECTIONS (PARTS 2 TO 7)

4.0 General Remarks

In this part of the work, the important formulae used in the computer design programmes are developed, explained, and cross referenced to the tables of Parts 2 to 7 (Volume 2) in which they are applied. The actions and resistances implied by the formulae are assessed within the regimen of the eight Structural Performance Ratios given earlier (2.13.1 to 8).

The eight Structural Performance Ratios (R_{u1} , etc) imply the load combinations as well as the tests for structural adequacy. The references to tables in Parts 2 to 7 imply both the spreadsheet display format (Designated Studies) and Manual Design Calculations.

Formulae pertaining to unperforated Universal Sections and Light Plate Girders are usually referred to Parts 2 and 5. The computer routine of Part 5, which can perform all the functions of Parts 3 and 4, designs either a roof or floor beam in each study and, consequently, is shorter and more easily followed than Parts 3 and 4. Formulae for unreinforced and reinforced web openings in Universal Sections are referred to in Parts 6 and 7 respectively. The design methods and formulae applied to trusses are discussed in Chapter 5.

Since the Manual Design Procedures of Parts 2 to 8 contain jointly one hundred and thirty pages of numerical work, the formulae of Chapters 4 and 5 are given without calculations.

4.1. Applied Loads, Bending Moments and Shears

In the design procedures, total load on the member (W_{u1} , etc), applied vertical shears (V_{u1} , etc), and bending moments (M_{u1} , etc) are always calculated from the unfactored loads and input geometry of Table 1. The latter includes: Type of Beam (1,2), beam span (L_b), centres of the primary beams (C_{pb}), and number of secondary beams or design check locations for Type 1 Beams (N_{sb}). The total loads for the seven design cases, which are calculated for the load combinations discussed earlier (2.13.1 to 8), are always displayed in Table 6.

The applied bending moments and shears are calculated at distances from the end of the beam (x), which are generated by the pro-

gramme from the number of secondary beams (N_{sb}) or check locations input at Table 1. The number of point loads (N_{pl}) to the left of the point considered (x) is generated by the spreadsheet in cells outwith the display tables.

Taking load case R_{u2} as an example, bending moments and shears are calculated with the following formulae:

For a Type 1 Beam (Part 5: Tables 13 and 14):

$$V_{u2} = 0.5W_{u2}[1 - (2x/L_b)]$$

$$M_{u2} = 0.5W_{u2}x[1 - (x/L_b)]$$

For a Type 2 Beam (Part 4: Tables 12 and 13):

$$V_{u2} = 0.5W_{u2}[1 - (2N_{pl}/N_{sb})]$$

$$M_{u2} = 0.5W_{u2}\{x - 2N_{pl}[x - 0.5C_{sb}(1 + N_{pl})]/N_{sb}\}$$

While these are somewhat mundane formulae, they are used in each of Parts 2 to 7, and demonstrate how the programme works with a minimum of input variables (L_b , N_{sb} and unfactored loading) and generated values (x , N_{pl} and total load on member). The bending moments and shears for load cases R_{u3} , R_{s1} , etc., are found from those for R_{u2} by proportioning the total beam loads, which are always calculated in the spreadsheet at Table 6 (Parts 2 to 7).

4.2 Steel Section - Elastic Flexural Properties

From the dimensions entered at Table 7 (Parts 2 to 7), the computer programme generates the flexural properties which are required in the calculation of section resistance, deflections, and the elastic stresses of premature yield.

For Universal Beams (Parts 2, 6, 7): The input dimensions are taken from the Constrado Handbook [15], and the flexural properties are calculated with the formulae given in Appendix 9. The design strength of the section (F_y) and the test for section compactness are calculated in Part 2: Table 8. It can be seen from Chapter 2 (2.7) that the computer running times are short. The slowest part of the procedure is often entering the section input variables. It is more time efficient to calculate the section properties from a minimum of input than to enter the endless values from prepared tables. This approach is also less prone to error. The process could be further improved by the use of Data Banks, as used in the truss design procedure (Part 8).

For Light Plate Girders (Parts 3, 4, 5): The input dimensions are selected by the user, and the flexural properties are calculated in Part 5: Tables 8, 9, 10 and 23 with the formulae for mono-symmetric sections given in Appendix 10. The input at Table 7 also includes the length (L_t) and degree of taper (DEPTH RATIO). The application of Data Banks to Light Plate Girders would be counter-productive, since it would take away much of the user's ability to refine the cross-sectional dimensions of the fabricated section to the design conditions.

4.3 Composite Section - Elastic Flexural Properties

The programme calculates the second moment of area of the composite section for short (I_t) and long term (I_{tc}) loading. The steps in the calculation for short term loading are:

m_1 = modular ratio for short term loading = E_s/E_c (Table 2).

A_{cf} = transformed area of concrete = $A_{cc}B_e/m_1$ (Tables 1 and 3).

I_{cf} = second moment of area of the transformed concrete slab.
 $= I_{cd}B_e/m_1$ (Tables 1 and 3).

Y_{cd} = distance from top to centroid of concrete slab (Table 3).

Y_{es} = distance from the top to ENA of the composite section.
 $= [A_s(Y_{et} + T_o) + A_{cf}Y_{cd}]/(A_s + A_{cf})$

I_t = second moment of area of the composite section.
 $= A_s(Y_{et} + T_o - Y_{es})^2 + A_{cf}(Y_{es} - Y_{cd})^2 + I_x + I_{cf}$

The long term second moment of area is calculated in the same way with m_2 , where $m_2 = 2m_1$. The spreadsheet calculations for long and short term second moments of area of the composite section are given for Universal Beams in Part 2: Tables 17 and 18, and for Light Plate Girders in Part 5: Table 24.

Since these properties are applied to design checks at the Serviceability Limit States, no provision is made to discount concrete in tension when the ENA lies in the concrete. This is a provision of the draft BS 5950: Part 3.1 (Clause 3.7.2.3), which gives a simplified formula (Appendix B.3.3) for calculating the second moment of area of an

uncracked composite section. However, the simplification discounts the area of concrete in the profile of the steel deck in Type 2 beams, and thus reduces the stiffness of the composite section for a marginal computational convenience. In the design studies, the calculations are made using the formulae given above, which are arranged to take full account of the available concrete in Type 2 beams.

4.4 Design of Steel Section to the Ultimate Limit States

The design theories and formulae used in Parts 2 to 7, which are applied to the four Ultimate Limit State load cases, R_{u1} , R_{u2} , $R_{u3}(\text{Bending})$ and $R_{u3}(\text{Shear})$, are developed and explained. The design of trusses to the Ultimate and Serviceability Limit States is dealt with in Chapter 5.

4.4.1 Steel Section - Lateral Torsional Buckling

Prior to fixing the steel deck or setting of the concrete, the steel section may be susceptible to lateral torsional buckling. The conditions vary for Type 1 and 2 beams and depend on restraint to the beam and the load case. In keeping with the intention stated above (3.1.1), the provisions of BS 5950: Part 1 relating to lateral torsional buckling are used in the studies.

4.4.2 Steel Section - Lateral Restraint

Type 1 Beams (R_{u1}): In floor plans B and C (Fig 1 and 2): Before the steel deck is secured to the floor beams, the beams may be unrestrained over the full span and may not sustain the temporary loading of workmen, erection equipment, and unfixed steel decking. One or more lines of floor bracing may be required to stabilize the beams. The sole purpose of the bracing is to hold the top flange of the steel beams in place until the steel deck is secured by welding or other fasteners. The bracing may be light gauge, single bolted, angles or tubes. The number of lines of bracing is an option which is always entered at Table 7 (Parts 2 to 5). It can be seen from Fig 12:C that the top flange of these beams is fully restrained after deck fixing and stud installation. It will be seen below that load case R_{u1} does not apply to Type 2 beams.

Type 2 Beams (R_{u2}): In floor Plan D (Fig 2): The beams are restrained by the secondary members. However, Fig 12:C shows that the steel deck, which has a profile module that seldom fits the beam spacing, is often split at the beam line and affords the beam limited restraint. It is usual to assume that Type 2 beams are unrestrained between secondary members until the concrete has set. This is the position taken by Chien and Ritchie [27], and a provision of BS 5950: Part 3.1 (Clause 2.3.2). For Type 2 beams the lines of bracing option at Table 7 (Parts 2 to 5) is entered as zero. It follows that these beams need not be checked for R_{u1} , since they must sustain the higher load of R_{u2} in the same condition of restraint.

4.4.3 Steel section - Effective Length (L_e)

The recommendation of BS 5950: Part 1 for effective lengths with destabilizing loads are followed.

Type 1 Beam (ex. Part 5: Table 9):

Without floor bracing,

$$L_e = 1.2L_b$$

With one line of floor bracing,

$$L_e = 0.6L_b$$

With more than one line of floor bracing, $L_e = 1.0L_b / (BRCG + 1)$

Type 2 Beam (ex. Part 2: Table 9):

$$L_e = 1.0L_b / (N_{sb} + 1)$$

4.4.4 Steel section - Resistance to Lateral Torsional Buckling

The resistance of the steel section to lateral torsional buckling (M_b) is calculated to the provisions of BS 5950: Part 1: Appendix B.2.

The steps in the spreadsheet calculation of M_b are, for obvious reasons, in reverse order to that given in Appendix B.2. The symbols used are those established for this work.

$$\lambda = \text{slenderness ratio} = L_e / R_y$$

$$u = \text{buckling parameter} = \{[4S_x^2(1 - (I_y/I_x))]/(A_s^2H_s^2)\}^{0.25}$$

$$J_i = \text{torsional index} = 0.566H_s(A_s/J_t)^{0.5}$$

$$N = T_{tf} / (T_{tf} + T_{bf})$$

ψ = mono-symmetry index = 0 for $N = 0.5$
= $0.8(2N - 1)$ for $N > 0.5$ = $1.0(2N - 1)$ for $N < 0.5$

v = slenderness factor.
= $\{[4N(1 - N) + 0.05(\lambda/J_i)^2 + \psi^2]^{0.5} + \psi\}^{-0.5}$

L_{1t} = equivalent slenderness factor = $n\lambda_{uv}$, where n is always unity with destabilizing loads.

L_{1o} = limiting equivalent slenderness = $0.4(\pi^2 E_s / F_y)^{0.5}$

E_{1t} = Perry Co-efficient

For rolled sections:

= $0.007(L_{1t} - L_{1o})$, but $E_{1t} > 0$

For welded sections:

= $0.014L_{1o}$, but $E_{1t} < 0.014(L_{1t} - L_{1o})$, $> 0.007(L_{1t} - L_{1o})$, > 0

M_p = plastic moment capacity of steel section = $F_y S_x$

M_y = elastic moment capacity = $F_y Z_x$

M_{cr} = elastic critical moment = $\pi^2 M_p E_s / (F_y L_{1t}^2)$

F_{ib} = $0.5[M_p + M_{cr}(1 + E_{1t})]$

M_b = lateral torsional buckling resistance moment.
= $(M_{cr} M_p) / [F_{ib} + (F_{ib}^2 - M_{cr} M_p)^{0.5}]$

The performance ratio R_{u1} is always checked at the point of maximum bending moment. This is obvious for rolled sections, but even with tapered beams the bending moment (M_{u1}) falls off more quickly than the beam resistance (M_b).

The calculations for a Type 2 compact universal section are given in Part 2: Tables 9 and 10, and for a Type 1 non-compact tapered girder in Part 5: Tables 9 and 10.

4.4.5 Steel section - Effects of Shear (Von Mises Criterion)

BS 5950: Part 1: recommends that the moment capacity of compact sections are not impaired by the applied shear when $V_{u2} < 0.6V_y$, and that the effect of shear yield on non-compact sections may be ignored. In these studies, the applied shear on the unpropped steel section is never close to $0.6V_y$, and no consideration is given to this effect on unperforated sections. However, as will be explained below, the effect on composite sections, perforated sections, and truss chords is carefully monitored.

4.4.6 Steel Section - Moment Capacity of Universal Beams

Universal sections need not be checked for web buckling since the web slenderness does not reach the limit of $63K_s$. When the section is fully restrained by the steel deck (Type 1 beam, Fig 12:C), the resistance is given by:

$$M_{rs2} = F_y S_x \text{ for compact} = F_y Z_x \text{ for non-compact sections.}$$

4.4.7 Steel Section - Moment Capacity of Light Plate Girders

Light Plate Girders will almost always have a web slenderness greater than $63K_s$. Their bending resistance, which may be impaired, is calculated to the provisions of BS 5950: Part 1 (Clause 4.4.4.2) which, as discussed earlier (3.1.1), are partially modified in these studies.

Light Plate Girders, which have compact flanges and webs with D_w/T_w in the range $63K_s$ to $98K_s (D_w/2d_1)$, are considered compact.

In BS 5950: Part 1: Clause 4.4.4.2(c), it is recommended that the resistance of the section take account of the interactive effects on the web of shear, edge loading, compression, and bending. In this work, it is assumed that the interaction can be satisfied with a reduced web bending stress (F_{yr}) in the presence of the applied shear and edge loading. The impaired stress distributions, which are assumed in these studies for symmetric and mono-symmetric Light Plate Girders, are shown in Fig 13 and 14.

For the reasons given earlier (3.1.1), the interaction equation is used in the form given by the bridge code BS 5400: Part 3: Clause 9.11.4.4.

$$(M_{w2}/M_{wcr})^2 + (V_{u2}/V_{cr})^2 \pm \{ABS[(P_{e2}/P_{ecr})^2 \pm (P_{w2}/K_{cr}P_{wcr})^2]\}^{0.5} \leq 1.$$

The web resistance moment and forces are given by BS 5950: Part 1: Appendix H.3. These values are the elastic critical buckling values for bending, compression, shear and edge loading. They are more conservative than the values given in BS 5400: Part 3 for values of $D_w/T_w > 100$ in unrestrained panels.

M_{wcr} = buckling resistance moment of the web.

$$= S_{xw}[1630/(D_w/T_w)]^2, \text{ but less than } F_y S_{xw}$$

The critical bending strength is based on a plastic rather than an elastic stress distribution (Fig 13:A).

V_{cr} = critical shear strength of an unstiffened web = $T_w D_w Q_{cr}$

Q_{cr} is found from:

Q_e = elastic critical shear strength of an unstiffened web.

$$= [1000/(D_w/T_w)]^2$$

L_w = equivalent slenderness of the web

$$= (0.6F_y/Q_e)^{0.5}$$

$Q_{cr} = 0.6F_y,$ when $L_w < 0.8$

$$= 0.6F_y[1 - 0.8(L_w - 0.8)], \text{ when } 0.8 < L_w < 1.25$$

$$= Q_e, \text{ when } L_w > 1.25$$

In Grade 50 steel, the elastic critical shear strength (Q_e) governs the shear resistance of the unstiffened web when $D_w/T_w > 86$.

P_{wcr} = critical axial strength of the web.

$$= T_w D_w [850/(D_w/T_w)]^2, \text{ but less than } F_y T_w D_w$$

The critical axial stress is derived from the well known formula: $k(\pi^2 E_s)/[12(1-\nu^2)(D_w/T_w)^2]$ with $k = 4$.

K_{cr} = reduction factor applied to the critical axial strength of web.

$$= 0.6 + 0.4(D_c/D_w)$$

This factor, which is not included in the BS 5950 formula, is applied in this work to reduce the critical axial strength of the web

(P_{wcr}) in mono-symmetric compact steel sections and composite beams, where the axial load is distributed over a length $D_c < D_w$. This effect is shown in Fig 14:A and 17:B, where it can be seen that with a plastic stress distribution the separation of web stresses concentrates the compression stress at the centreline of the web. The critical buckling strength for this condition is clearly less than that assumed in BS 5950: Part 1 and the reduction factor (K_{cr}), which is taken from the work of Roberts [35] as shown in Fig 18:A, is introduced to take account of this reduced resistance.

$$P_{ecr} = \text{critical compressive strength for edge loading.}$$

$$= 2.75E_s T_w / (D_w / T_w)^2, \text{ but less than } F_y T_w$$

The calculation of the web resistances in a tapered girder, which are applied to both the steel and composite sections, are given in Part 5: Table 11.

The actions on the web due to bending and compression are calculated from the web stress which may be impaired (F_{yr}). The actions due to shear and edge loading are the applied values of load case R_{u2} (2.13.2).

$$M_{w2} = \text{bending moment in the web for } R_{u2}$$

$$= F_{yr} T_w d_1 (D_w - d_1), \quad \text{Compact section (Fig 13:A and 14:A).}$$

$$= 0.5 F_{yr} (D_w / d_1) Z_{xw}, \quad \text{Non-compact section (Fig 13:B and 14:B).}$$

$$P_{w2} = \text{axial load in web for } R_{u2}$$

$$= F_{yr} T_w (D_w - 2d_1), \quad \text{Compact section (Fig 13:A and 14:A).}$$

$$= F_{yr} T_w D_w [1 - 0.5(D_w / d_1)], \quad \text{Non-compact section (Fig 13:B and 14:B).}$$

The formulae for M_{w2} and P_{w2} are given in a form which can be applied to both symmetric and mono-symmetric sections.

$$V_{u2} = \text{factored vertical shear for } R_{u2} \quad (4.1)$$

$$P_{e2} = \text{factored edge loading for } R_{u2}$$

$$= W_{u2} / L_b, \quad \text{for Type 1 beams (Fig 18:B).}$$

$$= W_{u2} / (1.25 D_w N_{sb}), \quad \text{for Type 2 beams (Fig 18:B).}$$

For a Type 2 beam, the edge loading is imparted by the secondary beams, which are connected in the web and offer some restraint to the web. For both these reasons the load is distributed over a length

25% greater than that specified in BS 5950: Part 1.

The calculation of the actions on the web of a tapered girder can be seen in Part 5: Table 13, where the reduced design strength of the non-compact top flange (F_{yf}) is also calculated.

$$\begin{aligned} F_{yf} &= F_y, & \text{for a compact section (Fig 13:A and 14:A).} \\ &= K_f F_y, & \text{for a non-compact section (Fig 13:B and 14:B).} \end{aligned}$$

$$\begin{aligned} K_f &= \text{stress reduction factor for slender outstands (Table 11).} \\ &= 10 / \{ [(B_b - T_w) / 2T_{tf} K_s] - 3 \}, \text{ but } \leq 1.0 \end{aligned}$$

It can be seen from Part 5: Tables 12 and 13 that the value of F_{yr} , which satisfies the interaction equation, is found by iteration. The iterative process begins in Table 12 where the actual web thickness T_w is multiplied by a REDUCTION FACTOR, which is user input, to give an effective web thickness T_{we} . Reduced section moduli are then calculated for a steel section with a reduced web thickness T_{we} .

In Table 13, F_{yr} is found by the programme by applying the same REDUCTION FACTOR of Table 12 to F_{yf} . M_{w2} and P_{w2} are calculated with F_{yr} and applied to the interaction equation which is tested in the end column of Table 13. The iteration is ended by the user when the interaction equation is just satisfied.

The resistance of the girder (M_{rs2}) can now be found directly with the reduced elastic (Z_{xe}) or plastic (S_{xe}) modulus calculated in Table 12.

$$\begin{aligned} M_{rs2} &= F_y S_{xe}, & \text{for a Compact section.} \\ &= F_{yf} Z_{xe}, & \text{for a Non-compact section.} \end{aligned}$$

The manual iteration takes less than a minute. The girder resistance calculated in this way satisfies both the interaction equation and force balance in the cross-section. It is worth noting that force balance is not satisfied in asymmetric sections, if the provisions of BS 5400: Part 1: (Clause 4.4.4.2(c)) are applied literally.

In steel sections with Type 3 Symmetry (Fig 11) and at almost all cross-sections of the composite section (Fig 15 to 17) the axial force in the web will be tensile, which has a very beneficial effect on web buckling. In the interaction formula, the compression term

$(P_{w2}/K_{cr}P_{wcr})$ is taken as negative but, as recommended by Harding [52], is restricted to a maximum value of -1.0. While not expressly stated, BS 5400: Part 3 also restricts the strengthening effect of tension in the web to -1.0. However, there is no limit for this condition in BS 5950: Part 1; an omission which could theoretically allow the applied shear to be two or three times the critical shear strength in composite sections where there is no web bending (Fig 15:A(i)).

4.4.8 Steel Section - Moment Capacity of Symmetric Light Plate Girders

It can be seen from Fig 13 that there is no axial force in the web of symmetric Light Plate Girders. Although this is almost never the case with composite sections, it greatly simplifies the application of the interaction equation to the steel section, which is now soluble without iteration in the following simplified form:

$$(M_{w2}/M_{wcr})^2 + (V_{u2}/V_{cr})^2 + (P_{e2}/P_{ecr}) \leq 1.0$$

The moment (M_{w2}) in the web can be found directly from:

$$M_{w2} = M_{wcr} [1 - (V_{u2}/V_{cr})^2 - (P_{e2}/P_{ecr})]^{0.5}$$

The moment of resistance of compact and non-compact steel sections is then given by:

$$M_{rs2} = M_{w2} + F_{yf} A_f H_s, \text{ and } F_{yf} = F_y \text{ for a compact flange.}$$

4.4.9 Steel Section - Performance Ratios R_{u1} and R_{u2}

Universal Beams: The performance ratios are calculated for a Type 2 beam in the Part 2: Designated Study: Table 11, where the following tests are made for the governing ratios:

For a Type 1 Beam:

$$M_{rs1} = M_b$$

$$R_{u1} = M_{u1}/M_{rs1}$$

$$M_{rs2} = M_p \text{ for a compact section.}$$

$$= M_y \text{ for a non-compact section.}$$

$$R_{u2} = M_{u2}/M_{rs2}$$

For a Type 2 Beam:

$$R_{u1} = 0 \text{ (2.13.1)}$$

$$M_{rs2} = M_b, \quad \text{if section is Compact.}$$
$$= \text{lesser of } M_b \text{ and } M_y, \text{ if section is Non-compact.}$$

$$R_{u2} = M_{u2}/M_{rs2}$$

Light Plate Girders: The performance ratios are calculated for a Type 1 beam in the Part 5: Designated Study: Table 14, where the following tests are made for the governing ratios at four locations along the girder:

For a Type 1 Beam:

$$M_{rs1} = M_b \quad \text{at the point of maximum bending moment.}$$

$$R_{u1} = M_{u1}/M_{rs1} \text{ at the point of maximum bending moment.}$$

$$M_{rs2} = \text{moment of resistance as calculated above (4.4.7).}$$

$$R_{u2} = M_{u2}/M_{rs2}$$

For a Type 2 Beam:

$$R_{u1} = 0 \text{ (2.13.1)}$$

$$M_{rs2} = \text{the lesser of } M_b \text{ and } M_{rs2} \text{ as calculated above (4.4.7)}$$

$$R_{u2} = M_{u2}/M_{rs2} \text{ along the beam.}$$

For example, R_{u1} and the maximum value of R_{u2} are recorded in the Part 5: Summary of Structural Performance: Table 32.

4.5 Perforated Steel Section (Parts 6 and 7)

The formulae used to calculate the moment capacity of the perforated steel section at unreinforced and reinforced web openings are derived from the work of Redwood and collaborators [37 to 45], and the provisions included in the CISC Handbook [46].

For Parts 6 and 7, the properties of the web opening are entered at Table 8, and include: the location from the end of the beam (X_o), length of opening (L_o), height of opening (H_o), eccentricity of the opening (e), and the area of reinforcement (A_r) above and below the opening. The details at unreinforced and reinforced opening are shown in Fig 23.

The design theory applies to rectangular openings in compact sections and is valid within the bounds given by Redwood and Shrivastava [41] and adopted by the CISC [46]:

$$L_o/H_o + 6(H_o/D_b) \leq 5.6$$

$$L_o/H_o \leq 2.2$$

$$V_{u2} \leq 0.45V_y$$

$$D_w/T_w \leq 83K_s$$

For both the reinforced and unreinforced perforated steel sections, the interaction of shear and bending at the opening is represented diagrammatically by Fig 22, and algebraically by the following elliptic equation:[37,39]

$$[(M_{rso} - M_1)/(M_o - M_1)]^2 + (V_{u2}/V_1)^2 \leq 1.0$$

The moment capacity of the steel section at the web opening (M_{rso}) is then given by:

$$M_{rso} = M_1 + (M_o - M_1)[1 - (V_{u2}/V_1)^2]^{0.5}$$

It is assumed that the mode of failure at the unreinforced opening is represented by Fig 24:A. V_1 is the ultimate shear force at the opening and M_1 is the maximum bending moment at the centreline of the opening which can be sustained in the presence of V_1 . M_o is the pure bending resistance at the opening with zero shear. The assumed stress distribution for M_o , with the PNA above the opening, is shown in Fig 24:B [38].

The assumed modes of failure for a reinforced opening are shown in Fig 25:A and B, where V_1 , M_1 , and M_o have the meaning given for the unreinforced opening. However, it is assumed in this case that the normal stress in the web is zero, and that all bending resistance comes from the steel section flanges and the reinforcement at the opening [44].

4.5.1 Perforated Steel Section - Moment Capacity at Unreinforced Openings (Part 6)

The formulae used in this part of the work are derived from the work of Redwood and Poubouras [38,39] and are developed for completeness in Appendix 11. The formulae are not given in the non-dimen-

sionalized forms usually preferred by Redwood [39] and adopted by CISC [46], but in simpler forms which, in the writer's opinion, do not obscure the method and clarify the effect of eccentricity (e) at the web opening.

The flexural properties at the unreinforced openings are generated by the Part 6 spreadsheet at Tables 10 and 11. The formulae applied in the spreadsheet and arithmetic calculations are given in the Part 6: Manual Design Procedure.

V_1 , M_1 , M_o , and M_{rso} are calculated in the Part 6 spreadsheet Tables 12 to 14, where:

$$\begin{aligned} V_1 &= V_{1t} + V_{1b}, \text{ portions of } V_1 \text{ carried at top and bottom tee.} \\ &= V_{yt} [K_t / (1 + K_t)]^{0.5} + V_{yb} [K_b / (1 + K_b)]^{0.5} \end{aligned}$$

$$V_{yt} = 0.6 F_y T_w S_t, \text{ and } K_t = 3 (S_t / L_o)^2 \text{ (Fig 23)}$$

$$V_{yb} = 0.6 F_y T_w S_b, \text{ and } K_b = 3 (S_b / L_o)^2 \text{ (Fig 23)}$$

$$\begin{aligned} M_1 &= R_f D_b [1 - (R_{wbr} / R_f)] \text{ if } e \text{ +ve (ie. up).} \\ &= R_f D_b [1 - (R_{wtr} / R_f)] \text{ if } e \text{ -ve (ie. down).} \end{aligned}$$

$$R_f = \text{resistance of the flange} = F_y T_f B_b$$

$$\begin{aligned} R_{wtr} &= \text{resistance of web of top tee impaired by shear.} \\ &= (3V_{yt}^2 - 3V_{1t}^2)^{0.5} \end{aligned}$$

$$\begin{aligned} R_{wbr} &= \text{resistance of web of bottom tee impaired by shear.} \\ &= (3V_{yb}^2 - 3V_{1b}^2)^{0.5} \end{aligned}$$

$$\begin{aligned} M_o &= \text{moment of resistance under pure bending (Fig 24:B)} \\ &= F_y S_{xo} \end{aligned}$$

The plastic modulus (S_{xo}) of the perforated section is calculated in Part 6:Table 10 from the dimensions of the perforated steel section. Redwood and CISC [39,46] give a unified formula for (M_o/M_p). However, since the design procedures also require the elastic properties at the web opening (I_{xo} and Z_{xo}), it is found more convenient to calculate the flexural properties of the perforated section under pure bending from first principles with due account taken of the position of the Plastic Neutral Axis.

$$M_{rso} = M_1 + (M_o - M_1)[1 - (V_{u2}/V_1)^2]^{0.5}$$

M_{u2} = factored bending moment at centreline of the web opening (X_o)

$$R_{u2} = M_{u2}/M_{rso} \text{ (Part 6: Table 14).}$$

4.5.2 Perforated Steel Section - Moment Capacity at Reinforced Openings (Part 7)

The formulae used in this part of the work are derived from the work of Redwood [44], and are developed for completeness in Appendix 12. The formulae are not given in the non-dimensionalized forms usually used by Redwood [39] and adopted by CISC [46], but in the simple expressions originally derived by Redwood [44].

The flexural properties at the reinforced opening are calculated in Part 7: Table 10 and 11, and the formulae and detail calculations are given in the Manual Design Procedure.

V_1 , M_1 , M_o , and M_{rso} are calculated in the Part 7 spreadsheet Tables 12 to 14, where:

$$V_1 = 2R_r(D_b - H_o)/L_o < V_{yt} + V_{yb}$$

R_r = resistance of reinforcing flat bars above and below web opening.

$$= F_y A_r$$

$$M_1 = D_b(R_f - R_r)$$

M_o = moment of resistance under pure bending (Fig 25:B)

$$= F_y S_{xr}$$

As with the unreinforced opening, the plastic modulus (S_{xo}) of the reinforced section and other flexural properties for pure bending are calculated from first principles in Part 7: Table 10. The formulae for deriving S_{xo} take account of the four possible positions of the Plastic Neutral Axis.

$$M_{rso} = M_1 + (M_o - M_1)[1 - (V_{u2}/V_1)^2]^{0.5}$$

M_{u2} = factored bending moment at centreline of the web opening (X_o)

$$R_{u2} = M_{u2}/M_{rso} \text{ (Part 7: Table 14).}$$

4.6 Design of Composite Section to the Ultimate Limit States

In this work, the resistance of the composite section is always calculated as the plastic moment capacity, even though the beam is unpropped and the steel section may be slender. However, impairment of the moment of resistance of the composite section by shear yielding (Von Mises Criterion) and web buckling are both considered in the design studies on solid webbed members. A check of premature yield is also made to test the ability of the composite section to sustain without permanent deformation the additive stresses of staged construction which occur in unpropped beams.

4.6.1 Composite Sections - Effects of Shear (Von Mises Criterion) and $R_{u3}(\text{Shear})$

In Parts 2 to 5 of this work, the moment of resistance of the composite section is checked for strength impairment by shear yield when $V_{u3} > 0.3V_y$.

This limit, which is a commonly used value, is the one recommended by Johnson [25] and CONSTRADO [18], and is only slightly less than that given in the draft Eurocode 4 [6] as $0.33V_y$. The draft BS 5950: Part 3.1 gives a limit of $0.5V_y$.

In all the design procedures, it is assumed that all shear is carried by the web of the steel section, and the contribution of the web to the resistance of the composite section is reduced by using an effective web thickness (T_{we}), where:

$$T_{we} = T_w [1 - (V_{u3}/V_y)^2]^{0.5}$$

The calculations are made in Part 2: Table 12 and Part 5: Table 15 to find the effective area of the steel section (A_{se}) and distance to the neutral axis (Y_{te}). The latter would change in a mono-symmetric girder. The moment of resistance of the composite section is calculated with the effective properties, T_{we} , A_{se} , and Y_{te} .

The ability of the composite section to sustain the maximum end shear is also tested at this point (2.13.4). The tests are:

$$R_{u3}(\text{Shear}) = 0.5W_{u3}/V_y \quad \text{for a Universal Section.}$$

$$R_{u3}(\text{Shear}) = 0.5W_{u3}/V_{cr} \quad \text{for a Light Plate Girder.}$$

4.6.2 Composite Section - Moment Capacity with Shear Yield (Von Mises)

The plastic resistance blocks of the steel section and the concrete flange of the composite section, at the point of maximum bending moment, are shown in Fig 15:A(i),B(i) and 16:A(i). It can be seen that the moment of resistance of the composite section depends on the position of the PNA and, from the accompanying figures (ii), that the effect of partial shear connection ($\%C_o$) is to drop the PNA into, or further into, the steel section.

The stress distributions, which are shown for 50% partial shear connection in Fig 15(ii) and 16(ii), may be considered in two ways:

Firstly, the effect at the point of maximum bending when the connectors supplied are only half the number required to fully develop the resistance of the concrete flange.

Secondly, the effect at the quarter point when 100% connection is provided at the point of maximum bending moment.

In this work, partial shear connection ($\%C_o$) is defined as the former. However, the important point to be taken is that partial shear connection ($\%C_o$) at the max. bending moment, or at an intermediate point on the beam, causes the PNA to lie in the steel.

It can be seen from Part 2: Tables 14 to 16, and Part 5: Tables 17, 18 and 22, that the moment of resistance of the composite section (M_{rc}) is calculated at several locations (x), as well as at the point of maximum bending moment (L_{max}). These multiple design checks monitor the effects of shear yield and web buckling on the resistance of the composite section as it falls off towards the support. For user convenience, the section resistances are calculated from the beam end towards the centre, although a check must first be made to find the position of the PNA at the point of maximum bending moment.

The available resistances of the steel section ($R_{s(max)}$) and the concrete flange ($R_{c(max)}$) at the point of maximum bending moment are calculated in Part 2: Table 13, and Part 5: Table 16, from:

$$R_{s(max)} = F_y A_{se}$$

$$R_{c(max)} = 0.45 F_{cu} B_e A_{cc} \%C_o$$

At the point of maximum bending moment, the actual resistance of the concrete (R_c) and the position of the Plastic Neutral Axis are found from the following tests:

Case 1: PNA lies in the concrete (ex. Part 5: Table 17).

$$R_{c(max)} > R_{s(max)} \text{ and } R_c = R_{s(max)}$$

Case 2 or 3: PNA lies in the steel flange or web (ex. Part 2: Table 14)

$$R_{c(max)} < R_{s(max)} \text{ and } R_c = R_{c(max)}$$

At intermediate points on the beam (x), the resistance of the concrete ($R_{c(x)}$) is dependent on the type of stud distribution and is found from: (3.5)

$$R_{c(x)} = R_c (x/L_{max}), \quad \text{with Type 1 stud distribution.}$$

$$R_{c(x)} = R_c [(2x/L_{max}) - (x/L_{max})^2], \text{ with Type 2 stud distribution.}$$

The available resistance of the steel section ($R_{s(max)}$) may vary along the beam, slightly from the effect of shear yield, and substantially from the section geometry of tapered girders (ex. Part 4: Table 14 and 16). The symbols given in Fig 15 and 16 and applied in the design formulae, which have not been explained above, are now defined:

Y_{cd} = distance from top to centroid of concrete slab.

Y_{cc} = distance from top to centroid of concrete stress block.

R_{sc} = resistance of steel stress block in compression.

A_{sc} = area of steel section in compression.

R_{tf} = resistance of top flange of steel section.

R_s = resistance of steel stress block in tension.

T_{fc} = thickness of steel flange on compression.

Y_{te} = distance from top to ENA of steel section.

d_1 = length of web in compression.

d_2 = distance from top of steel beam to centroid of compression steel stress block.

d_3 = distance from underside of steel beam to centroid of tension steel stress block.

Y_1 = lever arm to compression steel stress block.

Y_2 = lever arm to concrete stress block.

Taking account of the position of the PNA, the moment of

resistance of the composite section (M_{rc}) is calculated from:

Case 1: PNA in the concrete (Fig 15:A)

$$R_c = R_{s(max)}$$

$$Y_2 = T_o + Y_{te} - Y_{cc}$$

$M_{rc} = R_c Y_2$, Case 1 can only occur at the maximum bending moment.

Case 2: PNA in top flange of steel section (Fig 15:B)

$$R_c < R_{s(max)}$$

$$R_{sc} = 0.5(R_{s(max)} - R_c) \text{ and } R_{sc} < R_{tf}$$

$$A_{sc} = R_{sc}/F_y \text{ and } T_{fc} = A_{sc}/B_b$$

$$Y_1 = (A_{se} Y_{te} - 0.5 A_{sc} T_{fc}) / (A_{se} - A_{sc}) - 0.5 T_{fc}.$$

$$Y_2 = Y_1 + 0.5 T_{fc} + T_o - Y_{cc}$$

$M_{rc} = R_c Y_2 + R_{sc} Y_1$, Case 2 can occur at any point on the beam.

Case 3: PNA in web of steel section (Fig 16:A)

$$R_c < R_{s(max)}$$

$$R_{sc} = 0.5(R_{s(max)} - R_c) \text{ and } R_{sc} > R_{tf}$$

$$A_{sc} = R_{sc}/F_y$$

$$d_1 = (A_{sc} - T_{tf} B_b) / T_{we}$$

$$d_2 = [0.5 B_b T_{tf}^2 + T_{we} d_1 (T_{tf} + 0.5 d_1)] / A_{sc}$$

$$d_3 = [A_{se} (D_b - Y_{te}) - A_{sc} (D_b - d_2)] / (A_{se} - A_{sc})$$

$$Y_1 = D_b - d_2 - d_3$$

$$Y_2 = D_b + T_o - Y_{cc} - d_3$$

$M_{rc} = R_c Y_2 + R_{sc} Y_1$, Case 3 can occur at any point on the beam.

In Part 2, calculations using these formulae are made in Tables 13 to 15. The moment of resistance and performance ratio of the composite section at each check location are displayed in Table 16, and the maximum value of the performance ratio is reported in Table 26. The performance ratio is calculated as:

$$R_{u3}(\text{Bending}) = M_{u3}/M_{rc}$$

In Part 5, calculations using these formulae are made in Tables 16 to 18, The moment of resistance and performance ratio of the composite section at each check location are displayed under Von Mises in Table 22.

The foregoing design procedure, for finding the plastic moment capacity of a composite section (M_{rc}) with the three cases of Plastic Neutral Axis location, is commonly used. It is for example used in the draft BS 5950: Part 3.1: Appendix B.2, where a set of unified formulae give the plastic moment capacity for the three PNA cases for compact and non-compact sections. However, the code's formulae are derived from simplified assumptions, such as steel section symmetry, available concrete in profiled decks, and section compactness.

In these design studies, the procedures allow the user to manipulate steel section asymmetry, partial shear connection and stud distribution, fully utilize the available concrete, and view each step of the calculation.

4.6.3 Composite Section - Moment Capacity with Light Plate Girders (Web Buckling)

The effect of web impairment by buckling on the stress distribution of the composite section is shown in Fig 17. It is assumed that the web can shed bending stresses and work at a reduced stress (F_{yr}) in the presence of the unreduced shear and edge loading.

It can also be seen from Fig 17:B that the axial force in the web is tensile. The interaction equation given above (4.4.7) may now be used in the form:

$$(M_{w2}/M_{wcr})^2 + (V_{u2}/V_{cr})^2 - \{ABS[(P_{e2}/P_{ecr})^2 - (P_{w2}/K_{cr}P_{wcr})^2]\}^{0.5} \leq 1$$

The formulae used to calculate the critical buckling web

resistance moments and forces, M_{wcr} , V_{cr} , P_{ecr} and P_{wcr} , are the same as those used for the steel section (4.4.7), and are applied in Part 5: Table 11. K_{cr} has the same meaning but is calculated for the the composite section in Part 5: Table 21. The actions on the web are given by:

$$M_{w3} = \text{bending moment in the web for } R_{u3} = F_{yr} T_w d_1 (D_w - d_1)$$

$$V_{u3} = \text{factored vertical shear for } R_{u3} \quad (4.1)$$

$$P_{w3} = \text{axial load in web for } R_{u3} = -F_{yr} T_w (D_w - 2d_1) \text{ (tension)}$$

$$P_{e3} = \text{factored edge loading for } R_{u3}.$$

It can be seen from Part 5: Tables 19 to 21 that the value of F_{yr} , which satisfies the interaction equation, is again found by iteration. In Table 19, the user input REDUCTION FACTOR is applied to the design strength (F_y , Table 7) to give F_{yr} . A reduced resistance of the steel section is then calculated from:

$$R_{s(max)} = F_y A_s - (F_y - F_{yr}) T_w D_w$$

The iteration starts with the REDUCTION FACTORS at unity. When the PNA lies in the concrete or the steel flange (PNA case 1 and 2, Table 19) the iteration process is not required. The properties of the composite section are then calculated for those cases in the manner given above (4.6.2), but with the full web thickness (T_w).

Where the PNA lies in the web (Case 3), the iteration proceeds until the interaction formula is satisfied in the end column of Table 21. The plastic moment capacity of a composite section using Light Plate girders is then calculated from:

Case 3: PNA in web of Light Plate Girder (Fig 17:A(ii))

$$R_c < R_{s(max)} \text{ (reduced value)}$$

$$R_{sc} = 0.5(R_{s(max)} - R_c) \text{ and } R_{sc} > R_{tf}$$

$$A_{sc} = R_{tf}/F_y + (R_{sc} - R_{tf})/F_{yr}$$

$$Y_{te} = [F_y A_s Y_{et} - (F_y - F_{yr}) T_w D_w (T_{tf} + 0.5 D_w)] / R_{s(max)}$$

$$d_1 = (A_{sc} - T_{tf} B_b) / T_w$$

$$d_2 = [0.5 R_{tf} T_{tf} + (R_{sc} - R_{tf})(T_{tf} + 0.5 d_1)] / R_{sc}$$

$$d_3 = [R_{s(max)}(D_b - Y_{te}) - R_{sc}(D_b - d_2)] / (R_{s(max)} - R_{sc})$$

$$Y_1 = D_b - d_2 - d_1$$

$$Y_2 = D_b + T_o - Y_{cc} - d_3$$

$$M_{rc} = R_c Y_2 + R_{sc} Y_1$$

When the PNA does not lie in the web (Case 1 and 2), the web strength is checked with the applied shear and edge loading in the presence of pure tension. The interaction equation now takes the form:

$$(V_{u3}/V_{cr})^2 - [1 - (P_{e3}/P_{ecr})^2]^{0.5} \leq 1$$

Although it can be seen from Part 5: Table 19 that the REDUCTION FACTORS for the design checks at 6000 and 7500 mm are 1.0, the interaction equation is still checked with $M_{w3} = 0$ in Table 21. Webs which fail this check are considered inadequate.

As with the steel section, the iteration takes less than a minute, and the resistance of the composite section satisfies both the interaction equation and force balance in the cross-section.

The moments of resistance of the composite section and the performance ratios at the design check locations are displayed in Table 22 under Web Buckling. The Performance ratio is calculated as:

$$R_{u3}(\text{Bending}) = M_{u3}/M_{rc},$$

The greater maximum value of R_{u3} (Von Mises) and R_{u3} (Web Buckling) from Table 22 is recorded in Part 5: Table 32.

4.7 Perforated Composite Section (Parts 6 and 7)

The formulae used to calculate the moment capacity of the perforated composite section at unreinforced web openings are derived from the work of Redwood, Wong, and Poumbouras [38, 39 and 40].

The formulae used to calculate the moment capacity of the perforated composite section at reinforced web openings are derived by extending the simplified approach used for the steel section to the composite section.

The properties of the web opening are entered at Parts 6 and 7: Table 8, and include: the location from the end of the beam (X_o), length of opening (L_o), height of opening (H_o), eccentricity of the opening (e), the area of reinforcement (A_r) above and below the opening, and the number of shear studs in the length of the web opening (N_h). The details at the opening are shown in Fig 23. The design theory applies to rectangular openings in compact sections and is valid for the boundary conditions given for the steel section (4.5).

For both the reinforced and unreinforced composite section, the interaction of shear and bending at the opening is represented diagrammatically by Fig 22, and algebraically by the following elliptic equation:[37,39]

$$[(M_{rco} - M_1)/(M_o - M_1)]^2 + (V_{u3}/V_1)^2 \leq 1$$

From this interaction expression, the moment capacity of the composite section at the web opening (M_{rco}) is given by:

$$M_{rco} = M_1 + (M_o - M_1)[1 - (V_{u3}/V_1)^2]^{0.5}$$

It is assumed that the mode of failure at the unreinforced opening is represented by Fig 24:A. V_1 and M_1 have the meaning given above (4.5.1) for the steel section. M_o is the resistance at the opening when the shear is zero. It is calculated from the plastic stress distribution shown in Fig 24:B.

The assumed mode of failure for a reinforced opening is shown in Fig 25:A and B. V_1 , M_1 , and M_o have the meaning given for the unreinforced opening. However, it is assumed in this case that the normal stress in the web is zero and that all bending resistance comes from the flanges (R_f), the reinforcement at the opening (R_r), and

the available concrete resistance (R_{ch}). R_{ch} is the resistance of the concrete at the high moment end of the opening, and is limited to the resistance of the shear studs (N_h) at the opening. It is assumed that the concrete at the low moment end is cracked and the resistance of the concrete is zero.

4.7.1 Perforated Composite Section - Moment Capacity at Unreinforced Web Openings (Part 6)

The formulae used in this part of the work are derived for completeness in Appendix 13. The moment of resistance of the composite section with zero shear (M_o) at the centreline of the unreinforced opening is calculated from first principles in Part 6: Tables 16 and 17, and the detail calculations are given in the Manual Design Procedure. V_1 , M_1 , and M_{rco} are calculated in Tables 18 and 19.

$$V_1 = V_{1b} + V_{yt} [A_1 B_1 + (3A_1^2 - 3B_1^2 + 9)^{0.5}] / (3 + A_1^2)$$

$$V_{1b} = V_{yb} [K_b / (1 + K_b)]^{0.5}, \text{ as for steel section (4.5.1)}$$

$$V_{yb} = 0.6 F_y T_w S_b, \text{ and } K_b = 3(S_b / L_o)^2$$

$$A_1 = L_o / S_t$$

$$B_1 = R_{ch} (T_o - 0.5 T_{co}) / (S_t V_{yt})$$

$$R_{ch} = N_h P_{sd}$$

$$T_{co} = T_c R_{ch} / R_{c(min)}$$

$$M_1 = R_f D_b [1 - (R_{wbr} / R_f)] + 0.5 R_{ch} (T_o - 0.5 T_{co})$$

$$R_f = \text{resistance of the flange} = F_y T_f B_b$$

$$\begin{aligned} R_{wbr} &= \text{resistance of web of bottom tee impaired by shear.} \\ &= (3V_{yb}^2 - 3V_{1b}^2)^{0.5} \end{aligned}$$

$$M_o = \text{moment of resistance of composite section under pure bending}$$

M_o is the pure bending plastic moment capacity of a composite section with an unreinforced web opening, and is calculated from first principles with due regard to the position of the Plastic Neutral Axis in Part 6: Tables 16 and 17. The plastic stress distribution of Fig 24:B is drawn for the PNA above the web opening. However, the PNA may lie below the web opening where the eccentricity of the web opening is positive (ie. up) and the available resistance of the concrete is small (ie. near end of beam).

$$M_{rco} = M_1 + (M_o - M_1)[1 - (V_{u3}/V_1)^2]^{0.5}$$

M_{u3} = factored bending moment at centreline of the web opening (X_o)

$$R_{u3}(\text{Bending}) = M_{u3}/M_{rco}$$

The performance ratios are calculated for each opening location in Part 6: Table 19.

Except for M_1 , the formulae are essentially those given by Redwood [39], who gives the formula for M_1 in a non-dimensionalized form which rather clouds an otherwise simple result. It can be seen from above that in the perforated composite section M_1 is simply the resistance of the steel section (4.5.1) plus the contribution from the concrete.

V_1 , M_o , and M_{rco} are calculated in the form given by Redwood [39].

4.7.2 Perforated Composite Section - Moment Capacity at Reinforced Web Openings (Part 7)

The formulae used in this part of the work are derived for completeness in Appendix 14. The moment of resistance of the composite section with zero shear (M_o) at the centreline of the unreinforced opening is calculated in Part 7: Tables 16 and 17, and the detail calculations are given in the Manual Design Procedure. V_1 , M_1 , and M_{rco} are calculated in Tables 18 and 19, where:

$$V_1 = 2R_r(D_b - H_o)/L_o + R_{ch}(T_o - 0.5T_{co} - S_b)/L_o$$

$$< V_{yo} = V_{yt} + V_{yb}$$

R_r = resistance of reinforcing flat bars above and below web opening.
 $= F_y A_r$

$$M_1 = D_b(R_f - R_r) + 0.5R_{ch}(T_o - 0.5T_{co}) + R_{ch}(D_b - 0.5S_b)$$

M_o = moment of resistance of composite section under pure bending.

M_o is the plastic moment capacity with pure bending of a composite section with a reinforced web opening, and is calculated from first principles with due regard to the position of the Plastic Neutral Axis in Part 7: Tables 16 and 17. The plastic stress distribution shown in Fig 25:B is for one of four possible locations of the PNA in the web.

$$M_{rco} = M_1 + (M_o - M_1)[1 - (V_{u3}/V_1)^2]^{0.5}$$

M_{u3} = factored bending moment at centreline of the web opening (X_o)

$$R_{u3}(\text{Bending}) = M_{u3}/M_{rco}$$

The performance ratios are calculated for each opening location in Part 7: Table 19.

4.8 Design to the Serviceability Limit States

The formulae and performance limits applied to the four Serviceability Limit State load cases, R_{s1} , R_{s2} , R_{s3} , and R_{s4} of Parts 2 to 7, are explained.

4.8.1 Deflection of Steel Section (R_{s1})

This deflection calculation is made to find the Calculated Camber (D_{ca}), which is limited to a value of $L_b/250$ for the reasons given earlier (2.13.5). The primary use of the Calculated Camber is to fix the Shop Camber, which is usually taken at about 65% of the theoretical value.

The calculated camber includes the deflection of the unpropped steel section before composite action is obtained, and the shrinkage deflection of the composite section.

$$D_{ca} = D_{f1} + D_{fs}$$

D_{f1} is the deflection due to load case R_{s1} (2.13.5) and D_{fs} is the shrinkage deflection (3.7.1). For prismatic sections (Part 2: Tables 17 and 19), the deflection is calculated from:

$$D_{ca} = K_f W_{s1} L_b^3 / (E_s I_x) + S_{sh} B_e A_{cc} (Y_{es} - Y_{cd}) L_b^2 / (8 m_1 I_t)$$

$$\begin{aligned} K_f &= \text{deflection factor} = 0.013 && \text{Type 1 Beam.} \\ &= (N_{sp} - N_{sp}^{-1}) [3 - 0.5(1 - N_{sp}^{-2})] / 192 N_{sb} && \text{Type 2 Beam - } N_{sp} \text{ odd.} \\ &= N_{sp} [3 - 0.5(1 + 4 N_{sp}^{-2})] / 192 N_{sb} && \text{Type 2 Beam - } N_{sp} \text{ even.} \end{aligned}$$

The deflection factors (K_f) are the well known solutions given in the Steel Designers' Manual [16], and are easily calculated from the input at Table 1 (Parts 2 to 7).

For tapered girders the deflection is calculated by the principle of virtual work using numerical integration. In Part 5 the flexural properties of the steel and composite sections, which are used in the deflection calculations, are given in Tables 23 and 24 at an ordinate length (h) of 1500 mm. The calculated camber is found from:

$$D_{ca} = D_{f1} + D_{fs}$$

$$D_{ca} = 2 \int_0^{0.5L} [(M_{s1} M_i) / (E_s I_x)] dx + 2 \int_0^{0.5L} [(M_{sh} M_i) / (E_s I_t)] dx$$

The calculation is actually made by numerical integration using Simpson's Rule (Part 5: Tables 23 to 25):

$$D_{ca} = 0.333h (\Sigma [(MULT) M_{s1} M_i / I_x] + \Sigma [(MULT) M_{sh} M_i / I_t]) / E_s$$

h = ordinate interval used in numerical integration.

MULT = ordinate multiplier used in Simpson's Rule with symmetry.
= 8 (odd), 4 (even), 4 (odd at mid-span), 2 (even at mid-span).

M_i = virtual moment due to unit load at mid-span = $0.5x$

M_{s1} = unfactored bending moment for load case R_{s1} .

M_{sh} = moment on composite section due to shrinkage strain.
= $P_{sh} (Y_{es} - Y_{cd})$ (3.7.1)

The performance ratio is calculated in Part 2: Table 19 and Part 5: Table 25, as:

$$R_{s1} = D_{ca}/(L_b/250)$$

While some consideration was given to finding the deflections of tapered sections by approximate continuous functions, which would take account of the degree and length of taper, and shorten the arithmetic, it was decided that the use of virtual work with numerical integration would give the better results, since it required no additional input and the heavy arithmetic is absorbed by the machine.

4.8.2 Deflection of Composite Section (R_{s2})

As stated above (2.13.6), this calculation is made to find the Service Deflection (D_{max}), which is limited to a value of $L_b/360$.

D_{max} includes the deflections due to service loads on the composite beam (D_{f2}), the long term portion of the service loads (D_{f2+}), and the effect of partial shear connection (D_{pc}).

$$D_{max} = D_{f2} + D_{f2+} + D_{pc}$$

For prismatic sections (Part 2: Tables 17, 18 and 20), the separate deflections are calculated from:

$$D_{f2} = K_f W_{s2} L_b^3 / (E_s I_t)$$

$$D_{f2+} = K_f W_{s2+} L_b^3 [(1/I_{tc}) - (1/I_t)] / E_s \quad (3.7.2)$$

$$D_{pc} = 0.3 D_{f2} [(I_t/I_x) - 1] (1 - 0.01\% C_o) \quad (3.6)$$

K_f = deflection factor given in 4.8.1.

For tapered girders, the deflection is calculated by the principle of virtual work using numerical integration, as described above for the steel section. In Part 5, the flexural properties of the composite section, which are used in the deflection calculations, are calculated in Tables 24 at an ordinate length (h) of 1500 mm.

The Service Deflection is found from:

$$D_{\max} = D_{f2} + D_{f2+} + D_{pc}$$

$$D_{f2} = 2 \int_0^{0.5L} [(M_{s2} M_i) / (E_s I_t)] dx$$

$$D_{f2+} = 2 \int_0^{0.5L} [(M_{s2+} M_i) / (E_s I_{tc})] dx - D_{f2} (W_{s2+} / W_{s2})$$

The definite integrals are solved with numerical integration using Simpson's Rule (Part 5: Tables 24 and 26).

The deflection due to partial shear connection is calculated with a formula in the form given in BS 5950: Part 3.1. Since the second moment of area of both the composite and steel section varies, it is found more convenient to work with deflections rather than stiffnesses. The deflection of the steel section is that due to the load carried by the composite beam.

$$W_{s1} = \text{unfactored loading for load case } R_{s1}$$

$$W_{s2} = \text{unfactored loading for load case } R_{s2}$$

$$D_{pc} = 0.3[D_{f1}(W_{s2}/W_{s1}) - D_{f2}](1 - 0.01\%Co)$$

The performance ratios are calculated in Part 2: Table 20 and Part 5: Table 26, as:

$$R_{s2} = D_{\max} / (L_b / 360)$$

The calculation of the service deflection ($D_{\max}$) is arranged to show the contributions made by long term loading (D_{f2+}) and partial shear connection (D_{pc}). It will be seen later from the design studies, that while the creep deflection for unpropped beams is small and can probably be ignored, that from partial shear connection may be considerable.

4.8.3 Premature Yield (R_{s3})

This check is made on unpropped beams to ensure that the bottom flange remains elastic under serviceability conditions. In the design studies, the bottom flange stress limit under service conditions has been fixed at $0.9F_y$. This is the value used by a number of authorities, Johnson [24], CONSTRADO [18], CSA [24], and Chien and Ritchie [27]. It is also an allowance for the elastic stresses due to shrinkage and creep which are not considered in the analysis.

The total stress in the bottom flange is the sum of the stress carried by the steel section from the deck pouring stage (F_{bt1}), and that carried by the composite section under service loading (F_{bt2}).

$$F_{bt1} = M_{s1}/Z_{xb}$$

$$F_{bt2} = M_{s2}/Z_{xt}$$

The performance ratio is calculated from:

$$R_{s3} = (F_{bt1} + F_{bt2})/0.9F_y$$

The calculations are given for a Universal Beam in Part 2: Table 21, and a tapered girder in Part 5: Table 27, where this test is made at the check locations along the beam. The need for these additional checks can be clearly seen in the results for an eighteen metre tapered girder, which are given in Part 4: Designated Design Study No.28: Table 26. The maximum bottom flange stress occurs at a point substantially removed from the centreline of the member. Since it is not feasible to check all cross-sections, the maximum stress may be slightly greater and this is a further reason to use $0.9F_y$.

4.8.4 Floor Vibrations (R_{s4})

The long span, column free, lightweight composite floor systems used in the design studies have less inherent damping than conventional framing systems, and consequently are more susceptible to floor vibrations.

It is assumed that the floor systems are not subject to the continuous vibrations produced by machinery, street traffic, and rhyth-

mic human activity which is found in special occupancies such as concert halls, drill halls, gymnasias, etc., but are subject to the transient or walking vibrations, which are caused by footstep impact in quiet occupancies such as schools, offices, hotels rooms, and residential accommodation.

In this work, the acceptance criteria for walking vibrations are those given by the Canadian Institute of Steel Construction [17], and shown in Fig 53 to 56. The method is based on the "Heel Drop" impact test and the key factors used in the calculation are the fundamental frequency of the floor system (F_s), the peak acceleration (A_o), and the percentage of critical damping (PCD).

A value of 4.5 PCD has been used in all the studies. This is a conservative value which, according to the CISC method, lies half way between the damping of a bare floor and a floor in service but without partitions. It can be seen from Fig 54 that a PCD of 4.5 gives the threshold peak acceleration (A_{ot}) as 3%G.

The calculations can be seen in Part 2: Tables 24 and 25. The second moment of area of the composite beam (I_t) is calculated in Table 24 using an effective breadth (B_{dy}) which is equal to the beam spacing (C_{pb}). It is assumed that all the concrete available to the beam will participate in the dynamic response.

$$\begin{aligned} F_1 &= \text{fundamental frequency of the primary floor beam.} \\ &= 156000 [E_s I_t / (W_{s4} C_{pb} L_b^4)]^{0.5} \end{aligned}$$

$$\begin{aligned} F_2 &= \text{fundamental frequency of the secondary floor beam.} \\ &= 9.0 \text{ Hz and is taken from Part 2: Design Study No.1} \end{aligned}$$

$$F_s = [F_1^{-2} + F_2^{-2}]^{-0.5}$$

$$\begin{aligned} A_{em} &= \text{equivalent area of the generalized mass} \\ &= 40 T_o L_b \text{ Type 1 Beam} = C_{pb} L_b N_{sb} / N_{sp} \text{ Type 2 Beam.} \end{aligned}$$

$$\begin{aligned} A_o &= \text{peak acceleration.} \\ &= 60 F_s / [(W_{s4} + 0.5 W_{pa} + 0.25 W_1) A_{em}] \end{aligned}$$

$$R_{s4} = A_o / A_{ot}, \text{ (ex. Part 2: Tables 25 and 26).}$$

4.8.5 Deflection of Steel Section due to Web Openings (D_{co})

The deflection at mid-span (D_{co}), due to an unreinforced web opening in the steel section, is calculated in Part 6: Table 22. This deflection increases the Calculated Camber (D_{ca}) at the centre-line of the beam and is caused by two effects at the opening:

Firstly, the distortion at the web opening (D_{vo}) due to Vierendeel shear.

Secondly, the reduced stiffness of the steel section at the opening.

The flexural properties of the perforated steel section (I_{xo}), and the top and bottom tees at the web opening (I_{tt} , I_{bt}) are calculated in Part 6: Tables 10 and 11. The effect of the Vierendeel deflection at the centreline of the beam is shown in Fig 19:B.

For load case R_{s1} , the vertical shear at the opening is V_{s1} , which is carried by the top (V_{st1}) and bottom (V_{sb1}) tees in proportion to their stiffness in bending and shear.

$$V_{s1} = V_{st1} + V_{sb1}$$

Allowing for the stiffness in bending and shear of the tees, the portion of V_{s1} carried by the bottom tee is:

$$V_{sb1} = V_{s1} / \{1 + [(L_o^2/12I_{bt}) + (3.12/T_w S_b)] / [(L_o^2/12I_{tt}) + (3.12/T_w S_t)]\}$$

$$\begin{aligned} D_{vo} &= \text{deflection at web opening due to Vierendeel action (Fig 19:B)} \\ &= V_{sb1} L_o [(L_o^2/12I_{bt}) + (3.12/T_w S_b)] \end{aligned}$$

$$D_{co} = 0.5D_{vo} + W_{s1} L_o [X_o^2 (1 - (X_o/L_b))] [(1/I_{xo}) - (1/I_x)] [K_{df}/0.013]$$

The large term is the deflection at the centreline of the beam due to the reduced stiffness (I_{xo}) at the web opening (X_o) for a Type 1 Beam. The $[K_{df}/0.013]$ ratio is a multiplier which allows for the point loading in Type 2 Beams.

Since this procedure allows for web openings at four different locations (X_o), or four variations of web opening details at one location, the deflections (D_{vo}) and (D_{co}) are reported separately in millimetres, and are not included in the Calculated Camber (D_{ca}) of the unperforated beams designed in Part 2. It will be seen from the

studies of Parts 6 and 7 (Chapter 6) that these additional deflections are tiny and could safely be ignored.

The deflections for a reinforced web opening are calculated in Part 7: Table 22 and use the flexural properties of the reinforced steel section (I_{xo}) and top and bottom tees (I_{tt} , I_{bt}) calculated in Tables 10 and 11. The calculations for D_{vo} and D_{co} are otherwise the same as those applied to the unreinforced opening.

4.8.6 Deflection of Composite Section due to Web Openings (D_{f2o})

The increase in the Service Deflection (D_{max}) due to an unreinforced or reinforced web opening in the composite section (D_{f2o}) is calculated in Part 6 or 7: Table 24.

The flexural properties of the perforated composite section (I_{to}), and the top and bottom tees at the web opening (I_{ct} , I_{bt}), are calculated in Parts 6 and 7: Tables 20, 21, and 11.

For load case R_{s2} , the vertical shear at the opening is V_{s2} which is carried by the composite top tee (V_{st2}) and bottom steel tee (V_{sb2}) in proportion to their stiffness in bending and shear. It is assumed that all the vertical shear is carried by the stem of the tees.

The calculations are otherwise the same as those for the steel section and the results are given in Parts 6 and 7: Table 24 and summarized in Table 26.

5.0 General Remarks on Design of Trusses

As stated above (2.8.7), the computer procedure of Part 8 is structured to design Warren Trusses with hollow structural chords and unbattened double angle diagonals. The web framing configuration and cross-section of the truss are shown in Fig 26:A and 27:A. Symbols used in the design formulae are given in the key diagram, Fig 28.

Although a Warren Truss has many more components than a Light Plate Girder, it can be seen from Table 5 (2.7) that the elapsed time for a computer run and a total design are about the same. The reason for this is the clear separation of the structural functions of the top chord, bottom chord, and web diagonals. Unlike the solid webs of plate girders, which are an integral part of the bending resistance to several load cases in both the steel and composite member, the truss diagonals carry shear loads only and need only be designed for the worst case.

However, to justify the "gussetless" truss construction sought in these studies, it is necessary to make careful strength checks at each panel point for the interactive effects of shear transferred by the diagonals (V_{un}), normal force in the chord (P_{un}), and bending moments due to panel point eccentricity (M_{ecn}) and local chord loading (M_{lb}). The forces and bending moments acting at panel points are shown in Fig 28.

The references to tables in Part 8 imply both the Designated Study and the Manual Design Calculations.

5.1 Applied Loads, Bending Moments and Shears

As described earlier (2.13.1 to 8), the total loads on the members are always calculated from user input at Tables 1 to 5, and are displayed in Table 6. The shears (V_{un}), bending moments (M_{un}), and normal force in the chords (P_{un}) are calculated from the total load carried by the member, and the floor and truss geometry.

For example, the shears, bending moments, and normal force at the top chord panel points for load case R_{u2} are given by:

For Type 1 Truss:

$$V_{u2} = 0.5W_{u2}[1 - K_p(1 + 2N_{p1})]$$

$$M_{u2} = 0.5W_{u2}(P_p L_p(1 - K_p) - 2K_p N_{p1}[P_p L_p - 0.5L_p(1 + N_{p1})])$$

$$P_{u2} = M_{u2}/D_{cc}$$

For Type 2 Truss:

$$V_{u2} = 0.5W_{u2}[1 - 2K_p N_{p1}]$$

$$M_{u2} = 0.5W_{u2}(P_p L_p - 2K_p N_{p1}[P_p L_p - 0.5L_p(1 + N_{p1})])$$

$$P_{u2} = M_{u2}/D_{cc}$$

The symbols used in these formulae for Type 1 and 2 Trusses

are:

N_p = number of panels at top chord of truss.

This value is user input at Part 8: Table 1.

P_p = panel point number.

L_p = length of panel = L_b/N_p

K_p = loaded point factor.

= $(W_{un})/(N_p+1)$ for Type 1 Truss

= $(W_{un})/(N_p-1)$ for Type 2 Truss

D_{cc} = distance between centroids of chords of steel truss.

L_p , K_p , D_{cc} , are calculated by the spreadsheet at Table 8. The panel point numbers (P_p) are generated by the programme in Tables 12, 14, and 15, to accord with the truss geometry and the truss component being checked. The above formulae are given for the top chord panel points of the steel truss, but are slightly modified for design checks on the panel lengths, bottom chord panel points, and the composite section.

5.2 Design of Trusses to the Ultimate Limit States

To ensure adequate chord strength, at and between panel points, the number of design checks required is greater than that previously applied to the solid webbed members (Chapter 4). It can be seen from Part 8: Tables 25: Summary of Structural Performance, that the design procedure makes seven tests of structural performance at the Ultimate Limit States.

R_{u1} - Top Chord - Type 1 Truss: (Tables 9 and 11).

This ratio measures the resistance of the steel top chord

in out-of-plane strut buckling (P_{c1}) to the maximum factored normal force in the top chord (P_{u1}). For the reasons given earlier (4.4.2), this design check does not apply to the Type 2 truss.

R_{u2} - Top Chord - Type 1 Truss: (Tables 10 and 11).

This ratio measures the resistance of the steel top chord in in-plane strut buckling between panel points (P_{c2}) and bending (M_{rs2}) to the maximum factored normal force in the top chord (P_{u2}) and local bending moments (M_{lb}). The resistance is measured by an interaction equation, and it is assumed that the top chord is restrained against out-of-plane buckling by the fixed steel decking.

R_{u2} - Top Chord - Type 2 Truss: (Tables 10 and 11).

This ratio measures the lesser resistance of the steel top chord in in-plane or out-of-plane strut buckling between the secondary members (P_{c2}) to the maximum factored normal force in the top chord panel (P_{u2}). It is assumed that the chord can buckle in either plane between the secondary members which are connected at the panel points.

$R_{u2}(\text{Shear})$ - Top Chord - Type 1 and 2 Trusses: (Table 12).

This ratio measures the resistance of the steel section at each panel point, when its squash load compression strength (P_y) and bending strength (M_p or M_y) are impaired by the panel point shear (V_{u2}), to the factored normal force (P_{u2}) and local bending moments (M_{lb} , M_{ec2}). The resistance is measured at each top chord panel point by an interaction equation, and it is assumed that the top chord is fully restrained against buckling in either plane by the fixed steel decking, secondary beams, and truss diagonals.

R_{u3} - Composite Truss - Type 1 and 2 Trusses: (Tables 13 and 14).

This ratio measures the resistance of the composite truss (M_{rct}) to the maximum factored bending moment (M_{u3}) at the mid-span and at locations in line with each bottom chord panel point.

$R_{u3}(\text{Shear})$ - Top Chord - Type 1 and 2 Trusses: (Table 15).

This ratio measures the resistance of the steel chord (V_y) to the maximum factored shear (V_{u3}) at top chord panel points. It is assumed that all other actions are absorbed by the concrete flange. The user has the option to use gussets, if he does not wish the

top chord to fail this check.

$R_{u3}(\text{Shear})$ - Bottom Chord - Type 1 and 2 Trusses: (Table 12).

This ratio measures the resistance of the steel section at the bottom chord panel points, when its tensile strength (P_y) and bending strength (M_p or M_y) are impaired by the panel point shear (V_{u3}), to the factored tensile normal force (P_{u3}) and local bending moments (M_{ec3}). The resistance is measured at each bottom chord panel point by an interaction equation. The user again has the option to use gussets if the panel point fails this check.

$R_{u3}(\text{Shear})$ - Web Diagonals - Type 1 and 2 Trusses: (Tables 16 and 17).

This ratio measures the resistance of the double angle ties (P_{t3}) and struts (P_{c3}) to the maximum factored axial loads (P_{u3}), which are derived from the panel shears (V_{u3}).

5.3 Steel Truss

The top chord of the steel truss is designed to resist the non-composite load combinations only, and is discounted from the resistance of the composite section.

5.3.1 Steel Truss - Chords - Flexural Properties

The flexural properties of the chords, which are called up from Data Banks with user entry Part Numbers, are displayed in the Part 8 spreadsheet at Table 7. The 'x' and 'y' axes of the top and bottom chord sections as given in Table 7 are parallel to the 'X' and 'Y' axes of the truss shown in Fig 9. To increase the lateral stiffness during deck erection (R_{u1}), the spreadsheet allows the user to ROTATE the major axis (x - x) of Rectangular Hollow Structural Sections, as given in Data Bank: Table B, into the minor axis (Y - Y) of the truss.

5.3.2 Steel Truss - Top Chord - Strut Buckling

Prior to fixing the steel decking (R_{u1}) and the setting of concrete (R_{u2}), the top chord is susceptible to strut buckling. As previously described for solid webbed members (4.4.2), the conditions vary for Type 1 and 2 trusses, and depend on the restraint offered to

the top chord and the load case (R_{u1} or R_{u2}).

5.3.3 Steel Truss - Top Chord - Lateral Restraint

Type 1 Truss: As explained earlier (4.4.2), while the steel decking is being placed (R_{u1}), one or more lines of bracing may be required to restrain the top chord of the truss. During concrete placement (R_{u2}), it is assumed that the steel decking is fixed and fully restrains the top chord from out-of-plane buckling (Y-Y axis), but not from in-plane buckling between panel points (X-X) axis.

Type 2 Truss: For load case R_{u2} , it is assumed that the top chord can buckle in either plane (X or Y) in the panel length between the restraints offered by the secondary beams and the truss diagonals at the panel points (Fig 9).

5.3.4 Steel Truss - Top Chord - Slenderness (λ)

Load Case R_{u1} - Type 1 Truss (Part 8: Table 9)

$$\lambda = L_b / [(BRCG + 1)R_y]$$

Load Case R_{u2} - Type 1 Truss (Part 8: Table 10)

$$\lambda = (L_p - E_{tc}) / R_x$$

Load Case R_{u2} - Type 2 Truss (Part 8: Table 10)

$$\lambda = \text{lesser of } (L_p - E_{tc}) / R_x \text{ or } L_p / R_y$$

As shown in Fig 28, L_p is the panel length and E_{tc} is the eccentricity at the top chord panel points.

5.3.5 Steel Truss - Resistance of Top Chord to Out-of Plane Strut Buckling - (R_{u1})

The resistance of the top chord of the steel truss to out-of-plane strut buckling is calculated to the provisions of BS 5950: Part 1: Appendix C.1. The calculations are made in Part 8: Tables 9 and 11 using the symbols developed for this work.

Within each braced length, the chord axial load will reduce from a maximum value. In a refined design, a load less than the maximum value could be used to assess the chord resistance. This factor has not been considered in the design procedure, as an allowance for the small local bending moments which occur with this load condition.

λ = slenderness ratio = $L_b / [(BRCG + 1)R_y]$ (Table 9)

P_y = ultimate axial strength of steel section (Table 9)
 $= F_y A_s$ (Tables 7 and 8)

P_e = Euler strength = $\pi^2 E_s A_s / (\lambda)^2$ (Table 9)

L_{10} = limiting slenderness = $0.2(\pi^2 E_s A_s / P_y)^{0.5}$ (Table 9)

E_{1t} = Perry co-efficient (Table 9)
 $= 0.002(\lambda - L_{10})$ for hollow structural sections.

F_{ib} = $0.5[P_y + (E_{1t} + 1)P_e]$ (Table 9)

P_{c1} = compressive resistance of steel section for load case R_{u1}
 $= P_e P_y / [F_{ib} + (F_{ib}^2 - P_e P_y)^{0.5}]$ (Table 9)

The performance ratio (R_{u1}) is calculated in Tables 11 and 25 for the maximum axial load in the chord (P_{u1}), as:

$$R_{u1} = P_{u1} / P_{c1}$$

5.3.6 Steel Truss - Resistance of Top Chord to Local Strut Buckling - (R_{u2})

The calculations, which are made in Part 8: Tables 10 and 11, allow for the effects of local panel bending moments (M_{1b}) in the Type 1 truss. For load case R_{u2} , the compressive resistance of the steel section (P_{c2}) is calculated in Table 10 with the formulae given above (5.3.5), but now taking account of the slenderness appropriate to Type 1 and 2 trusses, as given in 5.3.4.

The chord must satisfy an interaction equation which is also the performance ratio:

$$R_{u2} = (P_{u2} / P_{c2}) + (M_{1b} / M_{rs2}) \leq 1$$

The symbols in this relationship mean the following:

P_{u2} = factored axial load for load case R_{u2} (5.1).
 $= M_{u2} / D_{cc}$

P_{c2} = compressive resistance of steel section for load case R_{u2}

M_{1b} = factored local bending moment (Table 11 and Fig 28).

= $0.042K_p W_{u2}(L_p - E_{tc})^2/L_p$, for a Type 1 truss.

= 0 for a Type 2 truss.

For a Type 2 truss, it is assumed that all loading is supported at the panel points.

M_{rs2} = moment of resistance of steel section for load case R_{u2} .

= $F_y S_x$ for compact section = $F_y Z_x$ for non-compact section.

The performance ratio, R_{u2} , is calculated in Table 11 for the bending moment value at the centreline of the top chord panel nearest to the mid-span of the truss, and this result is summarized with other ratios in Table 25.

5.3.7 Steel Truss - Impaired Resistance at Top Chord Panel Points -

$R_{u2}(\text{Shear})$

These calculations are made in Part 8: Table 12. It is assumed that the top chord is restrained at the panel points and that strut buckling need not be considered (5.2). This design check is made to demonstrate that the "gussetless" truss has adequate strength at the top chord panel points to resist the interactive effects of compression, shear, and bending moments due to local loading and eccentricity.

The impaired steel section at the top chord panel point must satisfy an interaction equation, which is also the performance ratio:

$$R_{u2}(\text{Shear}) = (P_{u2}/P_{yr2}) + [(M_{1b} + M_{ec2})/M_{rs2}] \leq 1$$

P_{yr2} = ultimate axial strength of the steel section impaired by shear interaction (Von Mises criterion).

$$= P_y \{1 - (A_v/A_s)[1 - (1 - (V_{u2}/V_y)^2)^{0.5}]\}$$

The strength impairment is applied when $V_{u2} > 0.3V_y$

A_v = shear area of steel section = $A_s D_b / (D_b + B_b)$

V_y = ultimate vertical shear capacity of steel section = $0.6F_y A_v$

$M_{lb} = 0.084K_p W_{u2}(L_p - E_{tc})^2/L_p$, for a Type 1 truss.
 = 0 for a Type 2 truss.

M_{ec2} = factored bending moment due to panel point eccentricity for
 load case R_{u2}
 = $0.25E_{tc}(2V_{u2} - K_p W_{u2})$

$M_{rs2} = F_y Z_x$ for non-compact section - $V_{u2} < V_y$
 = $F_y S_x$ for compact section - $V_{u2} < 0.6V_y$
 = $F_y(S_x - S_v[(2.5V_{u2}/V_y) - 1.5])$ - $V_{u2} > 0.6V_y$

S_v = plastic modulus of shear area = $0.25A_v D_b$

The rules, applied above for shear impairment of the bending resistance of compact and non-compact steel sections, are in accordance with BS 5950: Part 1: Clause 4.2.6. Guidance for shear impairment of the compressive resistance of a steel section is not given in the code. Since the webs of a hollow structural section make a much greater contribution to the axial strength than to the flexural strength, it is deemed advisable to limit the shear level of non-impairment to $0.3V_y$.

5.4 Composite Truss

As shown in Fig 16:B, the plastic moment capacity of the composite truss is derived from the resistance of the concrete (R_c) and the lever arm to the centreline of the bottom chord (Y_2). The top chord of the steel truss does not participate in the bending resistance of the composite section, and consequently the Plastic Neutral Axis must always lie in the concrete. This is a provision of CSA S16.1 [7], and is the approach taken by Chien and Ritchie in their textbook on composite construction [27].

However, in keeping with the assumption that all shear is carried by the steel section, the shear capacity at the top chord panel points (V_y) must be greater than the shear transferred between the diagonals (V_{u3}).

For reasons given earlier (3.5), and demonstrated in Fig 21, Triangular Stud Distribution is always used in the design studies on trusses. Since the Plastic Neutral Axis always lies in the concrete, partial shear connection is an option which cannot be applied to trusses. The moment of resistance of the composite truss (M_{rct}) is calculated at the point of maximum bending moment and at locations along the member which are in line with the bottom chord panel points. At the "gussetless" bottom chord panel points the steel section must sustain the resistance of the concrete (R_c) in the presence of shear (V_{u3}) and eccentric bending moments (M_{ec3}), whereas in the panel lengths the chord need only sustain R_c .

The calculations for the resistance of the composite truss, and the strength checks at the top and bottom chord panel points are made in Part 8: Tables 14 and 15 respectively.

5.4.1 Composite Truss - Plastic Moment Capacity - (R_{u3})

The resistance of the composite truss is calculated in Part 8: Tables 13 and 14 from:

$$\begin{aligned} R_{c(max)} &= \text{available resistance of concrete with } 100\%C_o \text{ (Table 13).} \\ &= 0.45F_{cu}B_eA_{cc} \end{aligned}$$

$$\begin{aligned} R_{c(min)} &= \text{available resistance of concrete above deck (Table 13).} \\ &= 0.45F_{cu}B_eT_c \end{aligned}$$

$$R_{bc} = \text{resistance of truss bottom chord (Table 13)} = F_yA_s$$

$$R_c = \text{resistance of concrete stress block (Table 13)} = 0.95R_{bc}$$

The resistance of the concrete at the point of maximum bending moment is reduced by 5% to allow for the effects of shear and bending on the resistance of the bottom chord at the panel point nearest to mid-span, where the strength of the steel section will fall off more quickly than the concrete with Type 2 stud distribution.

$$\begin{aligned} Y_{cs} &= \text{distance to top from centroid of continuous slab (Table 13).} \\ &= 0.5T_c \end{aligned}$$

Y_{cc} = distance from top to centroid of concrete stress block (Table 14).
 $= Y_{cs} R_c / R_{c(min)}$, when $R_c < R_{c(min)}$.
 $= Y_{cs} + (Y_{cd} - Y_{cs})(R_c - R_{c(min)}) / (R_{c(max)} - R_{c(min)})$,
when $R_c > R_{c(min)}$.

Y_2 = lever arm to concrete stress block (Table 14 and Fig 16:B).
 $= D_t + T_o - Y_{cc} - 0.5D_b$

M_{rct} = resistance of composite truss.
 $= R_c Y_2$, at the point of maximum bending moment.

At bottom chord panel point locations away from the mid-span the resistance is calculated for Type 2 stud distribution as:

$$R_c = 0.95R_{bc} \{ 4(P_p - 0.5)/N_p - [2(P_p - 0.5)/N_p]^2 \}$$

P_p = panel point number (at bottom chord Fig 28).

Y_{cc} , Y_2 and M_{rct} are re-calculated with R_c at P_p .

5.4.2 Composite Truss - Shear Resistance at Top Chord Panel Points - $R_{u3}(\text{Shear})$

These calculations are made in Table 15 to ensure that un-gusseted top chord panel points can sustain the shear transferred between the diagonals. The performance is summarized in Table 25 under Top Chord and calculated from:

$$R_{u3}(\text{Shear}) = V_{u3} / V_y$$

5.4.3 Composite Truss - Impaired Resistance at Bottom Chord Panel Points - $R_{u3}(\text{Shear})$

This design check is made at each panel point to ensure that the bottom chord can sustain the normal force (P_{u3}) in the presence of shear (V_{u3}), and the eccentric bending moment (M_{ec3}).

The impaired steel section at the bottom chord panel point must satisfy an interaction equation, which is also the performance

ratio:

$$R_{u3}(\text{Shear}) = (P_{u3}/P_{yr3}) + (M_{ec3}/M_{rs3}) \leq 1$$

P_{u3} = factored axial load for load case $R_{u3} = R_c$ at P_p

P_{yr3} = ultimate axial strength of the steel section impaired by shear interaction for load case R_{u3} (Von Mises criterion).
 $= P_y \{1 - (A_v/A_s)[1 - (1 - (V_{u3}/V_y)^2)^{0.5}]\}$

The strength impairment is applied when $V_{u3} > 0.3V_y$, and A_v , P_y , and V_y , have the meanings given earlier for the top chord panel points (5.3.7).

M_{ec3} = factored bending moment due to panel point eccentricity for load case R_{u3}
 $= 0.5E_{bc}V_{u3}$, for internal panel point.
 $= E_{bc}V_{u3}$, for end panel point (P_{p1} in Fig 28).

$M_{rs3} = F_y Z_x$ for non-compact section - $V_{u3} < V_y$
 $= F_y S_x$ for compact section - $V_{u3} < 0.6V_y$
 $= F_y (S_x - S_v[(2.5V_{u3}/V_y) - 1.5])$ - $V_{u3} > 0.6V_y$

5.4.4 Composite Truss - Resistance of Diagonal Ties - $R_{u3}(\text{Tie})$

The capacity of the diagonal ties are checked in Table 16, where the appropriate properties for the equal leg angles used in the design studies are called up from Data Banks by user entry Part Numbers. The member strength is based on the provisions of BS 5950: Part 1: (Clause 4.6.3.1).

P_{u3} = factored axial load for load case R_{u3} (on two angles).
 $= V_{u3}L_d/D_{cc}$

P_{t3} = tensile resistance of the steel section for load case R_{u3} .
 $= 2F_y(0.75A_s)$, for double equal leg angles.

$$R_{u3}(\text{Tie}) = P_{u3}/P_{t3}$$

A check is also made to ensure that the slenderness ratio of the angles selected by the user do not exceed 350.

5.4.5 Composite Truss - Resistance of Diagonal Struts - $R_{u3}(\text{Strut})$

The compressive capacity of the diagonal struts are checked in Table 17, where the appropriate properties for the equal leg angles used in the design studies are called up from Data Banks by user entry Part Numbers. The member strength is calculated by the provisions of BS 5950: Part 1: Appendix C.1.

The slenderness ratio of the equal leg angles is taken as $0.85L_d/(\lambda)$, and is limited to 180.

The axial loads in the struts and ties for a given panel are equal. The performance ratio is calculated from:

$$R_{u3}(\text{Strut}) = P_{u3}/P_{c3}$$

P_{c3} = compressive resistance of the steel section for load case R_{u3} .

P_{c3} is the resistance of two angles and is calculated by the steps given earlier for chord resistance to buckling (5.3.5).

5.5 Design of Trusses to the Serviceability Limit States

The serviceability load cases applied to the trusses designed in these studies are:

(i) R_{s1} to find the Calculated Camber (D_{ca}), which is the sum of the steel truss deflection (D_{f1}) and the deflection of the composite truss due to shrinkage strain in the concrete (D_{fs}).

(ii) R_{s2} to find the Service Deflection (D_{max}), which is the sum of the deflections of the composite truss due to short (D_{f2}) and long (D_{f2+}) term loading. Since all trusses are designed with 100% shear stud connection, the deflection due to partial shear connection (D_{pc}) need not be considered.

(iii) R_{s4} to find the dynamic response of the composite member.

The test for Premature Yield (R_{s3}), which is applied to the design of all the solid webbed members of Parts 2 to 7, is not required in this design procedure. Since the plastic moment capacity of the composite truss is based on a maximum bottom chord stress of $0.95F_y$ and must resist loading factored to 1.4 (DL) and 1.6 (LL), it is totally unlikely that the bottom chord stress could reach $0.9F_y$ with unfactored loading. This can be demonstrated by the Part 8: Designated Study: Tables 19 and 20, where the total bottom stress is given by:

$$(P_{s1} + P_{s2})/A_s = (169 + 356)10^3/2340 = 224 \text{ N/mm}^2$$

This value is safely less than the assumed limit for Premature Yield of $0.9F_y$ (320 N/mm^2).

5.5.1 Deflection of Steel Truss (R_{s1})

The calculations are made in Part 8: Table 19 to find the Calculated Camber (D_{ca}), where:

$$D_{ca} = D_{f1} + D_{fs}$$

The deflections are calculated by the principle of virtual work, and are found from the summation of the product of virtual loads and member strains for each truss component.

$$\begin{aligned} D_{f1} &= \text{deflection due to load case } R_{s1} \\ &= \Sigma[(nP_i P_{s1} L)/(E_s A_s)] \end{aligned}$$

n = number of pieces.

P_i = virtual axial force due to unit load at mid-span.

P_{s1} = unfactored axial force due to load case R_{s1}

A_s = area of top and bottom chords and average area of diagonals in each panel.

L = L_p for chords = L_d for diagonals.

D_{fs} = shrinkage deflection.

$$= \Sigma[(nP_i P_{sc} L_p)/(E_s A_{ctc})] + \Sigma[(nP_i P_{sc} L_p)/(E_s A_s)]$$

The shrinkage deflection is produced by compression in the composite top chord (A_{ctc}) and tension in the steel bottom chord (A_s). It can be seen from Part 8: Table 19 that the web diagonals do not contribute to the shrinkage deflection.

P_{sc} = force in chords due to shrinkage strain.

A_{ctc} = transformed area of the composite truss.

= $A_s + A_{cf}$, calculated in Part 8: Table 18.

The performance ratio (R_{s1}) is calculated in Part 8: Table 21, and is given by:

$$R_{s1} = (D_{f1} + D_{fs}) / (L_b / 250)$$

5.5.2 Deflection of Composite Truss (R_{s2})

The Service Deflection (D_{max}) is calculated in Table 20, using the transformed area of the composite top chord (A_{ctc}) for short and long term loading which are derived in Table 18. Since the effect of partial shear connection need not be considered, the Service Deflection in trusses is simply the sum of the deflections due to short (D_{f2}) and long (D_{f2+}) term loading on the composite section.

$$D_{max} = D_{f2} + D_{f2+}$$

$$D_{f2} = \Sigma[(nP_i P_{s2} L_p) / (A_{ctc}) + (nP_i P_{s2} L_p) / (A_s) + (nP_i P_{s2} L_d) / (A_s)] / E_s$$

The three summations are the contributions of the composite top chord, bottom chord, and web diagonals.

$$D_{f2+} = \Sigma[(nP_i P_{s2+} L_p) / (A_{ctc}) + (nP_i P_{s2+} L_p) / (A_s)] / E_s - D_{f2} W_{s2+} / W_{s2}$$

P_{s2} = unfactored axial force due to load case R_{s2}

P_{s2+} = long term portion of P_{s2}

The long term deflection is due to the reduced stiffness of the top chord and the slight reduction in lever arm to the bottom chord. The web diagonals are unaffected by either of these effects, and do not contribute to the long term deflection (Table 20).

5.5.3 Floor Vibrations (R_{s4})

The dynamic response (A_o) is calculated in Part 8: Table 23. An average value of the second moment of area of the composite truss is derived from:

$$I_{t(ave)} = K_{df} W_{s2} L_b^3 / (E_s D_{f2})$$

The remainder of the calculations are similar to those used for the solid webbed members (4.8.4), and the performance ratio is again given as:

$$R_{s4} = A_o / A_{ot}, \text{ with } A_{ot} = 3\%G.$$

CHAPTER 6: DISCUSSION OF RESULTS AND THEIR APPLICATION

6.0 General Remarks

A catalogue of the three hundred design studies completed in this work is given in Volume 2: Part 1: Design Studies on Micro-fiche.

In Parts 2 to 8 (Volume 2), the results obtained from the design studies are tabulated by: study number, roof or floor beam, beam span (L_b), and Type 1 or 2 beam. For example, this format can be seen in Part 2: Results of Design Studies: Tables 1 to 10. The tabulated results include: member dimensions and mass (kg/m), the structural performance values for R_{u1} to R_{s4} , and some important physical properties. The latter include: type of beam symmetry, type of stud distribution, number of lines of floor bracing (Type 1 beams only), number of shear studs, size of shear studs, percentage shear connection ($\%C_o$), and percentage of reinforcing fabric (R_r).

While it will be seen that the design studies demonstrate the mass saving efficiency and other merits of the several member types considered, it must be borne in mind that the results are strictly valid only for the "Standard Building Properties" (2.3), which have been adopted for these studies. A radical change to the input and some of the design limits, such as those chosen for Calculated Camber ($L_b/250$) and Premature Yield ($0.9F_y$), could alter the relative advantage suggested by the results for some of the studies.

With the foregoing in mind, the design procedures themselves, which allow the user to adjust to his particular conditions almost all input and design constraints, are perhaps the more useful outcome of the studies. It therefore follows in this review that the results will not only be used to demonstrate the relative advantages of the members studied, but also the manner in which the design procedures may be improved.

6.0.1 Cross-references to Tables in Parts 2 to 8 (Volume 2)

In the discussion of the results and implications for the spreadsheet routines arising from the design studies, a number of cross-references are made to tables in several sections of Parts 2 to 8 of Volume 2. To clarify which tables are being referred to in Parts 2 to 8

the following terminology is used:

(a) Results: implies the Results of Design Studies.

(b) Study No.23: a specified design study implies the Designated Study.

(c) Micro-fiche: implies the design studies recorded on micro-fiche at the end of Volume 2.

6.1 Discussion of Design Studies - Parts 2 to 8

6.1.1 A General Refinement of the Spreadsheet Routine

Whereas in Parts 2 to 4 the design routines allow the user to study a roof and floor beam in one study, the routines of Parts 5 to 8 restrict each study to a roof or floor beam. Since this simplification greatly shortens the computer running time and allows the user to focus attention more closely on each design, it should probably be applied to all the computer design procedures. More importantly, the common input of the spreadsheet Tables 1 to 4 (Parts 2 to 8), which includes floor geometry, steel grades, and composite deck construction, may not always be appropriate to both roof and floor beams.

An example of the disadvantages of common input can be seen in Part 2: Results: Tables 3 and 4, where the performance ratios for Type 1 and 2 roof beams are given. The ratios show that the designs are almost totally dominated by steel section deflection (R_{s1}), and would suggest that the use of the higher priced Grade 50 rather than Grade 43 steel is probably an unnecessary refinement.

While it may be thought unduly limiting to restrict a computer programme to the design of one member, it must be remembered that a primary floor beam may repeat itself several hundred times in the same structure and, as such, deserves careful and precise consideration.

6.1.2 Part 2: Universal Rolled Sections (Fig 5)

Overview of Studies in Part 2: The catalogue of studies in Part 1: Table 1 shows that a total of seventy design studies were conducted. These studies involved Universal Beams and Columns, Type 1 and 2 roof and floor systems, and spans ranging from 6 to 24 metres. At each span, the studies were made to find the minimum mass, as well as the minimum depth beam, which would satisfy the design constraints.

The findings are tabulated in Results: Tables 1 to 10, and are also illustrated in Fig 29 to 32, 45, 46 and 53. The Designated Study is No.23, in which designs are made on an 18 metre span to find the minimum mass Type 2 roof and floor beams.

Structural Efficiency of Member: Fig 29 to 32 show that the designer has little control over floor zone depth with Universal Sections. Often a shallower section, which will satisfy the conditions, is not available, and when there is a suitable section the modest saving in beam depth is usually penalized by a substantial increase in beam mass.

For example, the generalizations made in the previous paragraph are demonstrated for Type 1 roof beams in Fig 29 and Table 1 of the Results, where it can be seen that there are no minimum depth solutions at 15 and 18 metre spans, and that the minimum depth solution at 21 metres entails an increase in beam mass of 62% for a reduction in beam depth of 16%.

Tables 7 and 10 of the Results show that for spans less than 20 metres the design constraints were met by all roof beams with $50\%C_o$ (the allowable minimum for the studies), and a number of floor beams with less than $100\%C_o$. Since the shear stud count is an important cost factor in composite construction, partial shear connection should be treated as a serious design option.

The consistent application of the minimum allowable level of shear connection to lightly loaded rolled sections (ie. roof beams) is explained by Fig 45 and the Results (Tables 3 and 4), both of which show that these rolled sections are totally governed by steel section deflection (R_{s1}). In particular, the tabulated values show that the margin of the governing ratios (R_{s1}) over those for composite section strength (R_{u3}) and stiffness (R_{s2}), which are both adversely affected by partial shear connection, is such that the minimum value of shear connection may be used.

On the other hand, Fig 46 and Results (Tables 5 and 6) explain why less use can be made of partial shear connection in the more heavily loaded rolled sections (ie. floor beams). It can be seen that floor beams are marginally governed by composite section deflection (R_{s2}). Since this performance ratio is governing and is also adversely affected by partial shear connection (3.6), there is less scope for reducing the number of shear studs.

As explained earlier (2.13.5 and 7), these design studies impose limits on the Calculated Camber (R_{s1}) and Premature Yield

(R_{s3}) of $L_b/250$ and $0.9F_y$ respectively. These limits may be considered somewhat contentious, and are not, for instance, provisions of the draft BS 5950: Part 3.1, where there is no limit on R_{s1} and the limit for Premature Yield is F_y . In Fig 46, the all-black bar charts are superimposed on those of the design studies to represent the draft code's provisions for R_{s1} and R_{s3} . It can be seen that the changes to these design limits has little effect on the governing behaviour of these rolled sections.

Implications for Design Process: It can be seen from Study No.23: Tables 12 to 16, that design provisions are made to check the effect of shear yield on the bending resistance of the composite section at intervals along the beam, and that triangular stud distribution is available to enhance the resistance at points away from the maximum bending moment. The latter option entails a stud count and check of reinforcing steel (Tables 22 and 23 of spreadsheet) at intervals along the beam. The seventy design studies indicate that these refinements are not required for long span composite beams using unperforated rolled sections.

Strength impairment due to shear yield is applied when $V_{u3} > 0.3V_y$. The values given in Tables 3 to 6 of the Results show that the performance ratio R_{u3} , which measures the effect of the maximum shear, exceeded 0.3 by a very small margin in only four out of seventy beams. It follows from this that the reduction of the moment of resistance of the composite section by shear yield was never a governing factor.

Results: Tables 9 and 10 show that all the design studies used uniform stud distribution. The results for seventy beams recorded on Micro-fiche: Table 16 show that $R_{u3}(\text{Bending})$, which measures the strength of the composite section at several check locations, was always a maximum at the point of maximum bending moment and, consequently, the member resistance with uniform stud spacing never lay inside the bending moment envelope.

From the foregoing, it can be concluded that design procedures for unperforated Universal Sections need not allow for intermediate checks along the member. It is sufficient to check the resistance at the point of maximum bending moment, and distribute shear studs and reinforcing fabric uniformly from this point. Implementation of these findings would allow Tables 14,15,16,22 and 23 of the Part 2 spreadsheet to be reduced to one line calculations and, when coupled

with the roof or floor beam option, would shorten the procedure by half.

However, as will be seen later, these simplifications should not be applied to perforated Universal Sections, where the shear impaired section resistance at the web opening will often govern, and non-uniform shear stud distribution can enhance the strength at points along the composite member.

While the primary purpose of the design studies on unperforated Universal Sections is to give a basis for comparison with the alternative steel sections, there are many situations where rolled sections will be the first, and the correct choice. The Part 2 design procedure will quickly size members for such cases.

6.1.3 Part 3: Light Plate Girders of Constant Depth

Overview of Studies in Part 3: The catalogue of studies in Part 1: Table 1 shows that a total of seventy eight design studies were conducted with prismatic Light Plate Girders. These studies involved Type 1 and 2 roof and floor beams with spans ranging from 9 to 24 metres. At each span length, studies were made to find the minimum mass built-up sections which would satisfy the design constraints at span-depth ratios of 15, 20 and 25. While most of the studies were made on symmetric sections, six studies were conducted on Type 2 floor beams with asymmetric steel sections (Fig 11: Type 2 Symmetry).

The study findings are given in Tables 1 to 11 of the Results, and also drawn in Fig 33 to 36, 47, 48 and 54. The Designated Study is No.29, in which the minimum mass Type 2 roof and floor beams with a span-depth ratio of 20 are found for a span of 18 metres. Additional studies were made on the floor beam of this Designated Study to demonstrate the effects of partial shear connection and stud distribution on the resistance of the composite section. The results are shown in Fig 20, and the additional studies are recorded on the Part 3: Microfiche as 19 A, 19 B, 19 C and 19 D.

Structural Efficiency of Member: The mass efficiency of these prismatic built-up sections is demonstrated in Fig 33 to 36, where solutions at the three span-depth ratios used in the studies are compared with those for the corresponding rolled sections. In general terms, the results would indicate that the fabricated members appear to be more efficient at a span-depth range of 15 to 20, and the heavily loaded Type 2 floor and roof beams are quite inefficient at 25.

However, the important points to be taken from the results are that there are no gaps in the solutions, as was found with the rolled sections (6.1.2), and the designer can always produce a section which is a reasonable compromise between mass efficiency and beam depth.

Looking in more detail at Fig 33 and 35, where the results for the more lightly loaded Type 1 and 2 roof beams are given, it can be seen that these sections are marginally more efficient at a span-depth ratio of 20 than at 15. Since the deeper member would usually be expected to be more efficient, this is a somewhat surprising result. The reasons for this anomaly are twofold.

Firstly, because of the light loading, the deeper section requires very thin flanges which are under utilized in the unproped steel section by working at stress levels which may be far below yield. The effect of this is demonstrated in Tables 3 and 4 of the Results, where it can be seen that all members with a span-depth ratio of 15 are governed by the resistance of the unproped steel section (R_{u2}).

Secondly, the shear resistance of the web becomes less efficient as the web depth grows and often causes the deeper section to have a thicker web than the shallower section for the same level of applied shear. This is demonstrated in Appendices 5 and 6, and Table 11 of the Results.

The results for the more heavily loaded Type 1 and 2 floor beams are given in Fig 34 and 36, which indicate that a span-depth ratio of 20 gives a marginal mass benefit over that at 15 in the Type 1 beam, and, although to a lesser extent, the reverse is true in the Type 2. In Table 11 of the Results, where the cross-section dimensions of the floor beams are listed, it can be seen that the beams with a span depth-ratio of 15 have, with one exception, thicker webs than those at 20. The heavier webs are not offset by savings in the flanges, which tend to be close to minimum practical sizes.

The studies indicated that only the more heavily loaded Type 2 floor beams, with a span depth ratio of 20, showed even a modest benefit from asymmetry. This is an obvious finding for all Type 1 and 2 roof beams which are heavily governed by either steel section resistance (R_{u2}) or stiffness (R_{s1}), neither of which is improved by asymmetry (Fig 47). In floor beams with a span-depth ratio of 15, the flange proportions are such that the material re-distribution required by asymmetry would produce impractical sizes. The very poor mass efficiency of Type 2 floor beams at span depth-ratios of 25 is unlikely to be

transformed by a modest re-arrangement of flange areas (Fig 36).

Part 3: Results: Tables 1 and 2 show, that for spans less than 20 metres, full advantage can be taken of partial shear connection ($50\%C_0$) in the design of all roof members and floor members at a span-depth ratio of 15, and some reduction may also be allowed in the design of the other floor members.

It is important in these studies to understand the design approach applied to the resistance of unproped steel sections. Looking initially in some detail at Study No.29, Tables 12 and 13 show that the resistance of the steel section for Type 1 and 2 beams ($M_{rs2(i)}$) is calculated at mid-span and intervals along the beam. This is the resistance of a steel section which may have impaired flange and web strength. For the Type 2 beam of this study, the resistance of the steel section to lateral torsional buckling at the point of maximum bending moment is also calculated ($M_{rs2(ii)} = M_b$), and compared with the impaired resistance. Table 13 shows that the roof beam is governed by lateral torsional buckling (M_b) and the floor member by impaired strength ($M_{rs2(i)}$).

The analysis for the single case is borne out by an examination of Part 3: Micro-fiche, where Tables 12 and 13 of the spreadsheets for seventy eight prismatic girders show that the governing ratio for the steel, R_{u2} , may occur at intermediate points, and that the strength of Type 2 beams is usually governed by section impairment rather than lateral torsional buckling.

Implications for the Design Process: It can be seen from Study No.29: Tables 14 to 21, that the moment of resistance of the composite section (M_{rc}) is checked for the effects of both shear yield (Von Mises criterion) and web buckling at mid-span and intermediate points. The performance ratios ($R_{u3(Bending)}$) for both checks at each location are displayed in Table 21. The resistance of the web to the maximum shear at the end of the beam is calculated in Table 14 ($R_{u3(Shear)}$). An analysis of these performance ratios for all the studies of Part 3, which can be seen on Micro-fiche at Table 21, offers two important results:

(i) Even with a prismatic steel section, the governing ratio for bending resistance of the composite section is not always at the mid-span. For this reason it is important to make design checks at intermediate points on the beam.

(ii) The governing ratio for prismatic sections is always

due to web buckling and never due to shear yield. Although, in a few instances, the ratio for shear yield may be higher near the end of the beam, it is never the governing ratio. For this reason, it would be reasonable to discount the design checks for impairment by shear yield of composite sections using prismatic girders, and delete Tables 14, 16 and 17 from the spreadsheet tables. However, since it will be shown later that shear yield may govern some of the designs using tapered members, this expedient cannot be taken as generally applicable to Light Plate Girders.

Part 3: Results: Table 2,6 and 10, and Fig 36, include results for studies on six Type 2 floor beams with mono-symmetric steel sections at the economic span-depth ratio of 20. From Fig 36, it is obvious that the mass advantage gained from mono-symmetry is marginal, and it is doubtful that the small gain in mass advantage warrants the increase in design complexity produced by non-symmetry.

It can be seen from spreadsheet Tables 18 to 21 of the Designated Study, that the accurate and force balanced moment of resistance of the composite section is calculated with a manually input iteration which, although not too time consuming, disrupts the flow of the procedure.

A further disadvantage of non-symmetry is found in the composite beam section when the steel section has Type 2 symmetry and the area of the bottom flange is substantially greater than that of the top flange. Near the end of such members, the contribution of the concrete flange to the resistance of the composite section will be small, and the preponderance of the bottom flange area may cause the Plastic Neutral Axis to fall below the mid-depth of the web. This effect will produce a net compression in the web which, in the presence of the high shear and edge loading, will greatly reduce the contribution of the web to the strength of the composite section. This condition cannot occur with a symmetric section.

As discussed above (4.4.8), the resistance of symmetric steel sections can be calculated without the iteration used in Table 12 of the spreadsheet. By the use of two simplifying design assumptions, the resistance of composite beams with symmetric steel sections may also be found without iteration.

(i) The compression ratio ($Pw3/KcrPwcr$) be taken as -1.0, The studies show that this ratio is almost always equal or very close to this limiting value.

(ii) The location of the Plastic Neutral Axis does not move due to web impairment. This is a conservative assumption, since web impairment in a symmetric section moves the axis upwards and increases the resistance of the composite section (Fig 17).

When the Plastic Neutral Axis lies in the web, the interaction expression for web buckling can then be written as:

$$(M_{w3}/M_{wcr})^2 + (V_{u3}/V_{cr})^2 - [1 - (P_{e3}/P_{ecr})^2]^{0.5} \leq 1$$

The bending moment in the web can then be found directly from:

$$M_{w3} = M_{wcr} \{1 - (V_{u3}/V_{cr})^2 + [1 - (P_{e3}/P_{ecr})^2]^{0.5}\}^{0.5}$$

The resistance of the composite section about the mid-depth of the steel section is then given by:

$$M_{rc} = R_c (0.5D_b + T_o - Y_{cc}) + F_y A_f H_s + M_{w3}$$

In summary, the spreadsheet design procedure for prismatic Light Plate Girders could be simplified by:

- (1) use of the roof or floor beam option.
- (2) use of symmetric steel sections.
- (3) discount shear yield checks in the composite section.

6.1.4 Part 4: Plate Girders with a Constant Taper (Fig 7) and the Full Scale Test at Imperial College

Overview of Studies in Part 4: The catalogue of studies in Part 1: Table 2 shows that a total of eighty one design studies were conducted with Tapered Light Plate Girders. As with the constant depth girders described above, these studies involved Type 1 and 2 roof and floor beams, span-depth ratios of 15, 19, 20 and 25, and span lengths in the range of 9 to 24 metres. All girders had an end depth equal to half the mid-span depth (DEPTH RATIO = 0.5). Except for No.43 to 45, the studies were again made to find the minimum mass tapered section which would satisfy the design constraints. Studies No.43 to 45 were made to test the performance of an asymmetric section, which had the same geometry and component sizes as those used in the Imperial College full

scale test [30,32]. While most of the studies were made on symmetric sections, six studies, excluding No.43 to 45, were conducted on Type 2 floor beams with asymmetric steel sections at a span-depth ratio of 20 (Fig 11: Type 2 Symmetry).

The study findings are presented in Tables 1 to 11 of the Results, and also drawn in Fig 37 to 40, 49, 50 and 55. The Designated Study is No.28, in which the minimum mass Type 2 roof and floor beams are found for a span-depth ratio of 15 and a span of 18 metres.

Structural Efficiency of Member: The mass efficiency of the fully tapered sections is demonstrated in Fig 37 to 40, where solutions at the three span-depth ratios used in the studies are compared with those for the corresponding rolled sections. In general terms, the results for all four types of beam would indicate that these fabricated members appear to be most efficient at a span-depth range of 15, and are moderately to grossly inefficient at 25. This behaviour is substantially different from that demonstrated by the prismatic girders in Fig 33 to 36.

At a span-depth ratio of 15, the improved performance of the tapered girder over the prismatic girder can best be demonstrated by comparing their cross-sectional properties, which are listed in Table 11 of the Results for Parts 3 and 4. Comparison will show that, for 70% of the studies at a span-depth ratio of 15, the tapered members have a web thickness which is one or more millimetres thinner than that of the corresponding prismatic members. This reduction in web thickness is a product of the web geometry of the tapered member. The critical shear strength (V_{cr}) of the web increases as the web depth reduces in the areas of high applied shear. The effect of web depth on shear resistance is demonstrated in Appendices 6 and 7.

However, the mass efficiency of these built-up sections will only be fully realized, if material procurement and web cutting procedures ensure negligible plate wastage. In the design studies, zero wastage is assumed, and the unit mass (kg/m) is calculated with the net web provided in the finished girder.

From the foregoing, it follows that some of the material saved by both the web shape and enhanced shear resistance can now be more usefully disposed to the flanges. This re-distribution of material will increase both the bending resistance and stiffness of the steel and composite sections. It can be seen from Fig 49 and 50, and Tables 3 to 6 of the Results that, with a few exceptions, the tapered beams are

heavily governed by the deflection of either the steel or composite sections. These conditions are best satisfied by deeper sections which now have more efficient webs and more robust flanges.

A comparison of the results in Fig 36 with those in Fig 40 shows that asymmetry produces a slightly greater mass reduction in the tapered girders than it does in the prismatic members. However, the studies again indicated that only the heavily loaded Type 2 floor beams with a span-depth ratio of 20 could be much improved by asymmetry. It can be seen from Fig 50 that the design of floor beams using tapered girders is dominated by deflection of the composite section (R_{s2}). Type 2 asymmetry in the steel section increases the stiffness of the composite member, and makes it less vulnerable to this deflection. This enhancement of composite section stiffness results in the reductions in beam mass shown in Fig 40 at a span-depth ratio of 20. A similar improvement could not be realized at a span-depth ratio of 15, since it can be seen from Table 11 of the Results that symmetric beams with this ratio already have light flanges, which would become impractically small if subjected to the further reduction required by asymmetry.

For spans less than 20 metres, Part 4: Results: Tables 1 and 2 show that full advantage can be taken of partial shear connection ($50\%C_o$) in the design of all roof members, but only a modest reduction could be allowed in the design of floor members. These effects are demonstrated in Fig 49 and 50. The former shows that roof beams are governed by either steel section deflection (R_{s1}) or strength (R_{u2}), and since neither of these are affected by partial shear connection the minimum shear connection can be justified. The latter shows that floor beams are mostly governed by composite section deflection (R_{s2}). Since this deflection is increased by partial shear connection, there is little scope for reducing the stud count.

As mentioned earlier (2.8.3), a full scale test was conducted on a tapered girder at Imperial College [30,32]. The test was conducted on a span of 15300 mm with a built-up section which had an asymmetric cross-section, span-depth ratio of 19, and a taper of 0.5. The cross-section dimensions of the girder are given in Results: Table 11: Studies 43 to 45. The unit mass of the tested girder (109 kg/m), which is recorded on Fig 40, is somewhat greater than expected at a span-depth ratio of 19.

The test rig was devised to model the floor arrangement used in these studies for Type 2 beams. While the framing arrangement,

thickness of the composite deck (T_o), and some of the nominal material values were the same as those used in these studies, there were a number of significant differences. These included: the steel decking (Type 2 instead of 1), concrete cube strength (30 instead of 25), and the shear stud count (67 studs). The shear stud resistance provided for the test beam was somewhat less than a 50% partial shear connection as calculated in these studies. The low stud count was probably derived from design values similar to those used in the early Imperial College studies (3.2 and 3.3). The pertinent values are: Effective Breadth ($B_e = 0.2L_b$), Effective Thickness (T_c), Concrete Design Value ($0.40F_{cu}$), and Stud Design Value (77.3 kN). By the earlier thinking, the application of these values to the test rig girder would give 100% connection.

In their report on the test, Hargreaves and Hobbs [32] stated that the limit for the test rig was reached at a load factor of 1.98 (988 kN), which was well above the design ultimate load factor of 1.6 (800 kN) when calculated with nominal values. In the Part 4: Design Studies No.43 to 45, the averaged load factor is 1.54.

Recognising that the test rig was constructed to model a tapered girder in a Type 2 floor arrangement, Studies No.43 to 45 were run on the spreadsheet to assess the performance of the same girder within the design theory and constraints used in these studies. The results are given in Part 4: Results: Tables 1 to 11.

i) Study No.43: The test girder is analysed at $100\%C_o$ within the framework of the "Standard Building Properties" and design criteria developed for this work. Results: Table 6 show that, while the section is governed by steel section deflection (0.907), there is a reserve of strength and stiffness. At $100\%C_o$, a minimum mass study would allow some refinement of the flange proportions and reduce the unit mass by about 5% to 104 kg/m, which, at a span-depth ratio of 19, is more in keeping with the mass efficiency indicated by Fig 40. The performance ratios for the ultimate shear and bending resistances (0.836, 0.830) suggest that the section could sustain a design load factor of 1.84.

ii) Study No.44: The test girder is analysed at $100\%C_o$ with the Super-Holorib steel decking (Type 2) and Grade 30 lightweight concrete used in the Imperial College test. The other "Standard Building Properties" and design criteria developed for this work are applied. Results: Table 6 show that the section is now heavily governed by steel section deflection (0.966), which leaves little scope for member refine-

ment and mass reduction. The increased deflection is entirely due to the increased load of wet concrete arising from the profile geometry of the Holorib steel decking (Fig 12). The total deflection was calculated at 59.1 mm, which is the sum of the contributions from steel section deflection under wet concrete (46.6 mm) and concrete shrinkage (12.5 mm). The test report [32] gave the total free deflection for this condition as 17.4 mm. There are three reasons why this value may be a gross underestimate of the deflection found in actual floor arrangements.

a) The girder used in the test rig was only required to support a 3.0 m strip of wet composite deck, which is half the 6.0 m spacing of the floor being modelled. This effect would double the measured 17.4 mm.

b) In the test rig, the deck profile was centred on the top flange where through-stud-welding securely fixed it to the tapered beam. This arrangement caused the steel deck to act compositely with the beam, reversed the asymmetry, and enhanced the stiffness of the tapered beam. The effect, which is mentioned by Raven and Glover [30], shows in the unexpected elastic stress distribution in the steel section at the wet deck stage. In most floor arrangements, it is unlikely that deck and floor plan modules will allow the convenient centering assumed in the test. It is more likely that the deck will require to be broken as shown in Fig 12:C. The deck fixing, if any, required for a broken deck is much less likely to engage the steel decking in composite action.

c) No attempt was made in the test [32] to monitor the deflection of the composite section due to shrinkage strain, which would normally be complete in about forty days [27]. The working load cycle was applied to 14 day concrete. However, after the working load cycle, the test demonstrated a residual deflection of 7.7 mm, which may have been due, in part, to shrinkage and should properly be considered part of the Calculated Camber.

iii) Study No.45: The test girder is analysed at 50% C_o with the Super-Holorib steel decking (Type 2) and Grade 30 lightweight concrete used in the Imperial College test. The other "Standard Building Properties" and design criteria developed for this work are applied. Results: Table 6 show that the section fails the performance ratio test for composite section deflection (1.048). The total deflection was calculated at 44.6 mm, which is the sum of the contributions from the elastic deflection of the composite section (33.4 mm), creep deflection (2.3 mm), and 50% partial shear connection (8.8 mm). The calculated

Service Deflection load (W_{s2}) is 390 kN, which on the test report plot of working load displacements gives a deflection of 33 mm. This close agreement with the calculated elastic deflection (33.4 mm) brings into question the magnitude of the calculated deflection for partial shear connection (8.8 mm). When the test girder was subjected to seven days of sustained loading, the deflection increased by 3.7 mm. Most of the increase occurred on the first day, which might suggest some slip due to the small number of shear studs installed on the test girder.

Notwithstanding the above analyses, the overriding consideration is implied by Fig 40, where it can be seen that the mass efficiency of the tested girder is far from optimum and only marginally better than a rolled section. In spite of these drawbacks, the test was commissioned by an experienced fabricator who recognised an industrial need for a tapered section which would freely accommodate building services in a reasonable floor zone.

Implications for the Design Process: In Study No.28: Table 11, the web buckling resistance properties (M_{wcr} , etc) are calculated at mid-span and intervals along the span. These values are used to calculate the reduced resistance of the web (F_{yr}) in both the steel (Tables 12 and 13), and composite sections (Tables 18 to 21).

It can be seen from Study No.28 that the resistance of the steel section (M_{rs2}) is calculated in Tables 12 and 13. For Type 2 tapered girders, the resistance to lateral torsional buckling (M_b) is calculated at mid-span, and the restrained resistance with possible flange and web impairment ($M_{rs2(i)}$) is calculated at mid-span and intervals along the beam. The performance ratio, R_{u2} , in Table 13 shows that the roof beam (G_{r1}) is governed by lateral torsional buckling at mid-span (9000 mm), and the floor beam (G_{f1}) by impaired strength at a location away from mid-span (6000 mm).

An analysis of Part 4: Micro-fiche: Table 13 for eighty one tapered girders shows:

(i) all Type 1 roof and floor beams are governed by impaired resistance away from the mid-span.

(ii) most Type 2 roof and floor beams are governed by impaired resistance away from mid-span, but a number are governed by lateral torsional buckling at mid-span.

It is concluded, that the design check of impaired resistance at intermediate locations is necessary for both Type 1 and 2 members, and dual checks of section resistance are necessary for Type 2.

As with the prismatic plate girders (6.1.3), it can be seen from Study No.28, that the moment of resistance of the composite section (M_{rc}) is calculated for the effects of both shear yield (Tables 14 to 17) and interactive web buckling (Tables 18 to 20). The results for both resistances at several design check locations along the beam are displayed in Table 21.

Unlike the constant depth girder, where shear yield never governed the resistance, perusal of Table 21 in the Part 4: Micro-fiche shows that web buckling always governs in Type 1 members, but only 50% of Type 2 members. In the more heavily loaded members, the web shear is often greater than $0.3V_y$ and, as the web is shallower away from the mid-span, it becomes less prone to web buckling but more so to shear yield.

In all of the Part 4 design studies (Micro-fiche: Table 21), the governing performance ratio ($R_{u3(Bending)}$) for the resistance of the composite section was always found at a point well removed from mid-span. For example, this effect can be seen in Study No.28: Table 21, where the roof beam (G_{r1}) is governed at 3000 mm and the floor beam (G_{f1}) at 6000 mm. Similar behaviour can be seen in the check for Premature Yield at Table 26.

Unlike the prismatic girder (Part 3), it is clear from the above that the tapered girder will show some benefit from asymmetry at a span-depth ratio of 20, and requires to be checked for shear yield in the composite section. It would seem that the roof or floor beam option is the only substantial change which could be usefully applied to the spreadsheet procedure of Part 4.

6.1.5 Part 5: Light Plate Girders with a Modified Taper (Fig 8)

Overview of Studies in Part 5: The Catalogue of Studies in Part 1: Table 2 shows that forty two design studies were conducted with these partially tapered Light Plate Girders. Unlike the prismatic and fully tapered girders, the studies were restricted to Type 1 and 2 floor beams with span-depth ratios of 15, 20 and 25, and spans ranging from 9 to 24 metres. All girders had an end depth equal to half the mid-span depth (DEPTH RATIO = 0.5), and a taper length (L_t) of 3000 mm. While most of the studies were made on symmetric sections, six studies were conducted on Type 2 floor beams with asymmetric steel sections (Type 2 Symmetry) at a span-depth ratio of 20. All studies were made to find the

minimum mass modified tapered section which would satisfy the design conditions.

The Results are tabulated in Tables 1 to 6, and are also illustrated in Fig 41, 42, 51, and 56. The Designated Study is No.8, in which the minimum mass Type 1 floor beam is found for a span-depth ratio of 20 and a span of 15 metres.

Structural Efficiency of Member: It can be seen from Fig 41 and 42, that Type 1 and 2 floor beams using modified tapered girders are more efficient in the span-depth range 15 to 20, and that the Type 2 members are clearly quite inefficient at 25.

As stated above, all Part 5 design studies were conducted with a degree of taper of 0.5, and taper length of 3000 mm. The modified tapered girder's performance is greatly affected by both these dimensions, and its structural behaviour will always lie somewhere between those of the prismatic and fully tapered members. It can be seen from Fig 50 and 51 that the modified tapered girder is not quite so dominated by deflection as the fully tapered member. This is simply a reflection of the increased stiffness which results from having the full girder depth over the middle portion of the beam.

Fig 40 and 42 show that the modified girder is more mass efficient than the fully tapered girder at a span-depth ratio of 20. This is more clearly demonstrated in Tables 18 and 19, which show that symmetric modified girders offer a mass improvement of 8 to 10% over the equivalent fully tapered member. For asymmetric sections, Table 19 shows that the improvement ranges from 10 to 18%. At 15 m span, compare the Imperial College test girder at 109 kg/m with the study result of 87 kg/m for a modified girder. The modified girder clearly offers a benefit in mass efficiency where floor zone is a priority and services can be constrained to run close to perimeter columns.

For spans less than 20 metres, Part 5: Results: Table 1 shows that floor beam designs using modified members can take little advantage of partial shear connection. This is of course a consequence of the susceptibility that these members have for composite section deflection. The performance ratios in Tables 2 and 3 of the Results show that R_{s2} does not always govern but has a high enough value to preclude stud reduction.

It can be seen from Fig 42 that asymmetry produces a greater mass reduction in the modified tapered girders than it does in the prismatic members (Fig 36). However, the studies again indicated

that only the more heavily loaded floor beams (Type 2) could be much improved. This is demonstrated in Table 6 of the Results, where it can be seen that the flange proportions for studies on symmetric sections at a span-depth ratio of 15 are quite light, and would be impractical if further reduced by asymmetry.

Implications for the Design Process: The Part 5 spreadsheet procedure allows the degree and length of taper to be varied, uses the roof or floor beam option, and checks the resistance of the composite section for the effects of shear yield and interactive web buckling. Although it is unnecessarily involved for the constant depth member, the procedure can be applied to the three types of plate girder.

6.1.6 Part 6: Universal Sections with Unreinforced Web Openings (Fig 6)

Overview of Studies in Part 6: The Results given in Tables 1 and 2 show that a total of twelve design studies were made to find the minimum mass Type 1 and 2 floor beams with unreinforced web openings. Tables 3 and 4 of the Results show that a total of forty one web opening locations were investigated. The web openings were located (X_o) at 1.5, 4.5, 6.0, 7.5, 9.0 and 10.5 metres to accord with the span of the beam and the floor geometry. In Type 2 beams, the web openings were located between the secondary beams.

All studies were conducted with the height (H_o) and length (L_o) of the web openings equal to $0.5D_b$ and $2H_o$ respectively. The web opening eccentricity (e) and number of studs at the opening (N_h) were varied to the best advantage of the member being studied. At each span length, the starting point for most of the studies was an attempt to justify the minimum mass unperforated Universal Section designed in Part 2 for the same span, floor geometry, and loading, but with web openings added at several locations. The exceptions to this approach are Studies No.11 and 12, in which the effects of eccentricity at the opening and stud distribution are monitored at one location.

The Part 6 design procedure is confined to investigating the effects at the openings. It is assumed that the members designed in Part 2 will still satisfy all other design conditions. A comparison of the unperforated and perforated steel sections produced by the design studies of Parts 2 and 6 for Type 1 and 2 floor beams is given in Tables 12 and 13.

less critical locations of the openings.

Table 13: Comparison of Unperforated and Perforated Floor Beams with Unreinforced Web Openings in Type 2 Beams - Parts 2 and 6.

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:											
:Beam:	Span:	Unperforated :				Perforated Beam				:	:
:Type:	Lb :	:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:	: mm :	Beam Size:	Mass:	Beam Size	:Mass:	X_o - metres				:	:
:	:	: Part 2	:kg/m:	Part 6	:kg/m:	1.5:	4.5:	6.0:	7.5:	9.0:	10.5:
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:											
:	2 :	12000:UBx533x201:	101:	UB(U)x610x229:	101:	X :	X :	:	:	:	:
:	2 :	15000:UBx686x254:	125:	UB(U)x762x267:	147:	:	X :	:	X :	:	:
:	2 :	18000:UBx762x267:	173:	UB(U)x838x292:	176:	X :	X :	:	X :	:	:
:	2 :	21000:UBx914x305:	201:	UB(U)x914x305:	253:	X :	X :	:	X :	:	X :
:	2 :	24000:UBx914x419:	343:	UB(U)x914x419:	343:	X :	X :	:	X :	:	X :
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:											

Implications for the Design Process: In Tables 5 and 6 of the Results, the increase in the Calculated Camber (D_{co}) and Service Deflection (D_{f2o}) due to each web opening is reported in millimetres. It can be seen that the increase for a given opening is tiny and could probably be ignored.

Table 14: Effects of Eccentricity (e) and Stud Distribution (1,2) on the Resistance of the Steel and Composite Sections - Type 1 Beams

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:												
:Study: e		: Steel Section :			Composite Section							:
: No. :		:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:										
:	: mm :	M_{rso} - kNm	:	Stud Type:	M_{rco} - kNm:	Stud Type:	M_{rco} - kNm :					
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:												
:11,12:+100:		1097	:	1	:	1471	:	2	:	1514	:	
:11,12:+ 50:		1159	:	1	:	1374	:	2	:	1415	:	
: 3 :	0:	1225	:	1	:	1309	:	-	:	-	:	
:11,12:- 50:		1159	:	1	:	1284	:	2	:	1325	:	
:11,12:-100:		1097	:	1	:	1265	:	2	:	1308	:	
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:												

It can be seen from Table 1 of the Results that studies 11 and 12 are not included for comparison in Table 12. These studies were made on the 18 metre member of Study No.3 to demonstrate the effects of eccentricity (e) and stud distribution on the resistance of the steel

(M_{rso}) and composite (M_{rco}) sections with a web opening at 4500mm. The effects are recorded in Table 14 for values taken from Tables 14 and 19 of Part 6: Micro-fiche: Studies 3, 11 and 12.

These results show that positive or negative eccentricity significantly weakens the steel section. On the other hand, both positive eccentricity and Type 2 stud distribution strengthen the composite section. These are of course obvious effects, since the former causes metal to be removed closer to the Plastic Neutral Axis, and the latter simply enhances the concrete resistance at the opening.

The Results in Table 5 show that the resistance of the steel section (R_{u2}) never comes close to governing any of the studies. Positive eccentricity (e up) and Type 2 stud distribution should be considered useful tools for improving the resistance at web openings. Although the results for this example show that positive eccentricity has a greater strengthening effect than triangular stud distribution, the design theory requires that the assumed number of studs (N_h) be located at the opening. In many instances, unless additional studs are added, this number would not be provided by uniform stud distribution at web openings near the supports.

6.1.7 Part 7: Universal Sections with Reinforced Web Openings (Fig 6)

Overview of Studies in Part 7: The Results in Tables 1 and 2 show that a total of twelve design studies were made on Type 1 and 2 floor beams with reinforced web openings. Tables 3 and 4 show that a total of forty one web opening locations were investigated. The web openings were located (X_o) at 1.5, 4.5, 6.0, 7.5, 9.0 and 10.5 metres to accord with the span of the beam and the floor geometry. In Type 2 beams, the web openings were located between the secondary beams.

All studies were conducted with the height (H_o) and length (L_o) of the web openings equal to $0.5D_b$ and $2H_o$ respectively. The web opening eccentricity (e), number of studs at the opening (N_h), and area of reinforcing flat bars (A_r) were varied to the best advantage of the member being studied. As with Part 6, the starting point for each study was to try and justify the minimum mass unperforated Universal Section designed in Part 2 for the same span, loading, and floor geometry, but with reinforced web openings added at several locations.

As for Part 6, the Part 7 design procedure is confined to effects at the openings, and it is again assumed that the members designed in Part 2 will still satisfy all other design conditions. A comparison of the unperforated and perforated steel sections produced by the design studies of Parts 2 and 7 for Type 1 and 2 floor beams is given in Tables 15 and 16.

Structural Efficiency of Member: The values in Table 15 are taken from the Results in Part 2: Table 2 and Part 7: Table 1. It can be seen from these results that in all cases the same section has been justified with reinforced perforations. Results in Part 7: Table 5 show that Type 1 floor beams have relatively low values of the Performance Ratios $R_{u3}(\text{Bending})$ and R_{s3} . This indicates that the overall design is governed by the unperforated section which is designed in Part 2.

Table 15: Comparison of Unperforated and Perforated Floor Beams with Reinforced Web Openings in Type 1 Beams - Parts 2 and 7

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:														
:Beam:	Span:	Unperforated :		Perforated Beam								:		
:Type:	Lb	:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:												
:	:	mm	:	Beam Size:	Mass:	Beam Size	:	Mass:	X _o - metres			:		
:	:	:	:	Part 2	:kg/m:	Part 7	:	:kg/m:	1.5:	4.5:	6.0:	7.5:	9.0:	10.5:
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:														
:	1	:12000:	UBx457x191:	67:	UB(R)x457x191:	67:	X	:	X	:	X	:	:	:
:	1	:15000:	UBx533x210:	92:	UB(R)x533x210:	92:	X	:	X	:	:	X	:	:
:	1	:18000:	UBx686x254:	125:	UB(R)x686x254:	125:	X	:	X	:	:	X	:	X
:	1	:21000:	UBx762x267:	173:	UB(R)x762x267:	173:	X	:	X	:	:	X	:	X
:	1	:24000:	UBx914x305:	201:	UB(R)x914x305:	201:	X	:	X	:	:	X	:	X
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:														

The values in Table 16 are taken from the Results in Table 2 of Parts 2 and 7, and show that a modest amount of reinforcement (0.3 to $0.5A_f$), and shear studs ($N_h = 6$) at the openings, will justify the sections designed in Part 2. However, it can also be seen from the Results in Table 6 of Part 7 that the perforated section is usually governed by either the resistance of the composite section (R_{u3}) or Premature Yield (R_{s3}), and that the values are quite high in some instances.

Table 16: Comparison of Unperforated and Perforated Floor Beams with Reinforced Web Openings in Type 2 Beams - Parts 2 and 7

:	-	:	:	:	:	:	:	:	:
:Beam:	Span:	Unperforated :			Perforated Beam				:
Type:	Lb	:	:	:	:	:	:	:	:
:	:	mm : Beam Size:	Mass:	Beam Size	: Mass:	X _O - metres	:	:	:
:	:	Part 2	: kg/m:	Part 7	: kg/m:	1.5:4.5:6.0:7.5:9.0:10.5:	:	:	:
:	-	:	:	:	:	:	:	:	:
:	2	:12000:UBx533x201:	101:UB(R)x533x201:	101:	X : X :	:	:	:	:
:	2	:15000:UBx686x254:	125:UB(R)x686x254:	125:	: X :	:	X :	:	:
:	2	:18000:UBx762x267:	173:UB(R)x762x267:	173:	X : X :	:	X :	:	:
:	2	:21000:UBx914x305:	201:UB(R)x914x305:	201:	X : X :	:	X :	:	X :
:	2	:24000:UBx914x419:	343:UB(R)x914x419:	343:	X : X :	:	X :	:	X :
:	-	:	:	:	:	:	:	:	:

Implications for the Design Process: The Results in Tables 5 and 6 show that the increase in Calculated Camber (D_{co}) and Service Deflection (D_{f2o}) caused by each web opening is given in millimetres. As was found for the unreinforced openings of Part 6, the increase for a given opening is tiny and can be safely ignored.

From the foregoing, it is reasonable to assume that most steel sections, which have been designed without perforations, can be reinforced and supplied with enough local shear studs to overcome the strength loss due to web openings.

It can be seen from the Results in Table 1 that Studies 11 and 12 are not included in the comparison given in Table 15. These studies were made on the 18 metre member of Part 7: Study No.3 to demonstrate the effects of eccentricity (e) and stud distribution on the resistance of the steel (M_{rso}) and composite (M_{rco}) sections with a web opening at 4500mm. The effects are recorded in Table 17, and the values are taken from Tables 14 and 19 of Micro-fiche: Studies 3, 11, and 12.

While these results are similar, they are less pronounced than those given in Table 14 for the unreinforced openings, but again show that positive or negative eccentricity significantly weakens the steel section. However, both positive eccentricity and Type 2 stud distribution again increase the resistance of the composite section at the web opening.

Table 17: Effects of Eccentricity (e) and Stud Distribution (1,2) on the Resistance of the Steel and Composite Sections - Type 1 Beams

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:Study:	e	:	Steel Section :			Composite Section			
:	No.	:	:	:	:	:	:	:	:
:	:	mm :	M _{rso} - kNm	:	Stud Type:	M _{rco} - kNm:	Stud Type:	M _{rco} - kNm	:
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:	11,12:+100:	:	1284	:	1	:	1641	:	2 : 1706
:	11,12:+ 50:	:	1321	:	1	:	1608	:	2 : 1689
:	3 : 0:	:	1378	:	1	:	1587	:	- : -
:	11,12:- 50:	:	1321	:	1	:	1570	:	2 : 1658
:	11,12:-100:	:	1284	:	1	:	1552	:	2 : 1641
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									

The Results in Part 7: Table 5 show that the measure of resistance of the steel section (R_{u2}) never comes close to governing any of the studies. The use of positive eccentricity (e up) and Type 2 stud distribution should both be considered useful tools for improving the resistance at reinforced web openings in the composite section. Again, triangular stud distribution will ensure that the number of studs assumed by the design are provided at openings near the end of the beam.

6.1.8 Part 8: Warren Trusses with Hollow Structural Chords (Fig 9)

Overview of Studies in Part 8: The Results in Table 1 show that a total of twenty design studies were made with Type 1 and 2 floor trusses, which use a Warren configuration with hollow structural chords and unbattened equal leg angle diagonals, as shown in Fig 26:A and 27:A. The results for trusses with span-depth ratios of 10 and 15, and a span range of 12 to 24 metres, are given in Part 8: Results: Tables 1 to 4 and illustrated in Fig 43, 44 and 52.

The truss design routine assembles member properties from Data Banks, which are attached for completeness to Part 8: Designated Study No.5, in which the minimum mass Type 1 truss is found at a span-depth ratio of 10 and a span of 18 metres.

Structural Efficiency of Member: It can be seen from Fig 43 and 44 that, whereas the lightly loaded Type 1 truss is efficient in the span-depth range of 10 to 15, the heavily loaded Type 2 truss is clearly more efficient at 10.

An important aspect of the truss designs is shown in Fig 52

and Part 8: Results: Table 2, where it can be seen that these trusses are never governed by the serviceability load cases (R_{s1} , R_{s2} and R_{s4}). For the reasons given earlier (5.5), trusses do not require to be tested for Premature Yield (R_{s3}).

Implications for the Design Process: While the interaction of shear, normal force, and local bending moments at the truss panel points gives rise to some additional design checks at the Ultimate Limit States, the Part 8 spreadsheet routine is not lengthy and easy to follow. In Part 8 the Designated Study is No.5, where it can be seen that the spreadsheet provides a list of truss components suitable for cost estimation and material requisition in Table 22.

In the design procedure, the chord resistance at the panel points is checked by an interaction equation, similar to that given in BS 5950: Part 1, but which allows for shear impairment of both axial and bending resistances. Since the Hollow Structural chords used in the design studies are always compact, a more direct and probably more favourable approach would be to find the impaired axial resistance from a plastic stress distribution, which takes account of shear impairment of the web and reverse stress due to bending in the flanges. While this is the method applied to the more complicated problem of finding the ultimate resistance in bending and shear at web openings, its use at truss panel points would be simplified by knowing the shear and bending moment values at the outset.

6.2 Relative Merits of Members Studied in Parts 2 to 5 and 8

Type 1 Floor Beams: The mass per metre of the steel sections studied in Parts 2 to 5 and 8 are given in Table 18 for spans ranging from 9 to 24 metres. The results are given for Light Plate Girders at span-depth ratios of 15, 20 and 25, and for Warren Trusses at 10 and 15. Comparison of the results for Light Plate Girders and Trusses with those for the minimum mass Universal Beams show that:

(i) The mass efficiency of LPG(C) and LPG(M) steel sections is marginally better at a span-depth ratio of 20 than 15, and can effect savings in steel mass of 40 to 50% for spans of 12 metres and greater.

(ii) The mass efficiency of the LPG(T) steel section is best at a span-depth ratio of 15, highly inefficient at 25, and at 15 can effect savings in steel mass of about 50%.

section at 20, and can effect savings in steel mass of about 40%.

(iii) The mass efficiency of TRUSS(H) is markedly better at a span-depth ratio of 10 than 15, and can effect savings of 50 to 65%.

Table 19: Mass Comparison of Type 2 Floor Beams (kg/m)

Part	Table	Serial	Beam	Span/	Symm:	Beam Span (L_b) - metres					
No.	No.	Type	Type	Depth	Type	9	12	15	18	21	24
2	2	UB	2	Min.Mass	-	60	101	125	173	201	343
				Span-Depth		22	22	22	24	23	26
3	2	LPG(C)	2	15	1	47	71	91	119	144	190
			2	20	1	50	78	99	128	154	186
			2	25	1	59	97	131	179	203	257
			2	20	2	47	73	91	121	146	176
4	2	LPG(T)	2	15	1	47	62	81	101	121	146
			2	20	1	61	83	112	142	179	218
			2	25	1	86	136	186	239	299	374
			2	20	2	55	77	103	129	160	197
5	1	LPG(M)	2	15	1	43	65	81	107	131	161
			2	20	1	47	74	95	125	149	180
			2	25	1	70	100	135	177	209	257
			2	20	2	45	69	87	116	136	170
8	1	TRUSS(H)	2	10	-		52	72	81	102	124
			2	15	-		69	93	99	134	139

It is important to interpret Tables 18 and 19 with care and engineering discretion. While the comparison of members at span-depth ratios of 10 to those at 20 may seem inappropriate, it is not the intention of this work to offer hard and fast solutions, but rather to offer optional solutions to the floor steel design problem.

The mass of Universal Sections generally increases with section depth and offers the designer little scope in the management of floor zone and service ducts. For any given project, the economic rolled sections may not be deep enough to accommodate the size of ducts required, and multiple ducts or an underhung system may be required. The alternative steel sections allow the designer to balance the requirements of floor zone and duct size with efficient steel usage.

While some Light Plate Girders are marginally more mass

While it can be seen, that Plan D using Type 2 members has a modest mass advantage over Plan B using Type 1, Plan D has other important advantages.

(i) The number of primary members in Plan D is half that in Plan B. The fabrication costs of Plate Girders and Trusses are obviously higher than that for Universal Sections. A floor system, which halves the number of special primary beams required, clearly reduces this disadvantage.

(ii) The spacing of the columns is not fixed by the spacing of the primary beams.

(iii) Plan D does not require the erection bracing, which is made obsolete by deck fixing.

(iv) The number of web penetrations for air ducts and other services in Universal Sections and prismatic Light Plate Girders is halved.

CHAPTER 7: CONCLUSIONS AND FUTURE WORK

7.0 Optimum Design of Long Span Composite Floor Beams

If there was no move to question the conventional British practice of framing commercial buildings with internal columns and 6 to 9 metre bays, the usefulness of these studies would be very limited. However, there is now a real demand for floor space, uninterrupted by internal columns, which allows the use of open plan offices and yet is flexible to the requirements of a changing tenancy. To satisfy these demands with optimum solutions the structural engineer must have options beyond the conventional Universal Section. Such options will allow him to produce a competitive design within the broader constraints of span length, floor zone, service requirements, and the ability of fabricators to produce the novel sections.

In North America, the move to longer spans has been met with the use of: trusses, Stub-girder with Gerber beam systems, a schedule of readily available built-up symmetric and asymmetric Welded Wide Flanges, Staggered Truss design, and unreinforced perforations in Universal Sections.

In the U.K, while the search for an optimum design can in some measure be met by the design processes and structural members studied in Parts 2 to 8 of this work, as well as the other members discussed later in this chapter (7.2), there are of course other options and possibilities. The flexible Arbed production system and the current consideration being given to the rolling of asymmetric sections are two. The production of a composite profiled deck with a proven span capability, which is greater than those presently available, would greatly enhance the performance of Type 1 Beams and Trusses.

While it must be borne in mind that the member sizes obtained in this work are strictly valid only for the "Standard Building Properties" and some design limits peculiar to the studies, the results in Tables 18, 19, and 20 will give initial guidance on the selection of probable member types for a given floor arrangement. The design procedures embodied in the work will then allow the designer to optimise member selection and cross-section refinement to the actual design variables, as well as the wider project constraints of span, floor zone, and mechanical services.

7.1 Use of Spreadsheets in the Design Process

Initially, the spreadsheet was perceived as a simple tool which would assume the tedium associated with the early studies by guaranteeing arithmetic accuracy without manual checking, and shortening the elapsed times for the design studies. It was immediately apparent that the problem modelling abilities of the spreadsheet would contribute much more. The ease, with which the "slash" commands, built-in functions, and some simple Boolean algebra can be assimilated, allows the user to devise the analytical model best suited to his purpose. The system, which is user friendly, can be quickly mastered and is an ideal implement for engineers with a limited background in computing.

7.1.1 Advantages of Spreadsheets

A major advantage of computing with spreadsheets is complete accessibility to the workings of the programme. Many of the analytical and design programmes available to engineers are "protected" for proprietary reasons or, when "open", use a format which is couched in complicated computer language and logic. Consequently, the conscientious engineer must either take much on trust or spend a great deal of time unravelling the logic and design assumptions built into the software. On the other hand, the spreadsheet method allows the engineer to press the "slash" command edit key and immediately view the formula used to derive the result in any given cell.

It may be thought a minor disadvantage of the spreadsheet routines developed for these studies that they require the "hands on" participation of the user. However, this close involvement allows the user to refine the design to his satisfaction by modifying section properties or other design options, limit recalculation to the cells affected by adjusted input, and print all or any part of the spreadsheet. More importantly, it requires the close and ongoing attention of the designer. An important aspect of professional responsibility which is often diminished by computer aided design.

It can be seen from Table 5 (2.7) that the running times of the spreadsheet routines are short and the elapsed time to complete a refined design is reasonable. This is a major advantage at the conceptual design stage when material values, type of steel section, and even the floor geometry may be tentative.

The feature of the spreadsheet, which allows the user to recalculate any cell range, makes it easy to apply iteration to problems which are not amenable to direct solution. The "hands-on" requirement ensures that the iteration is satisfied to the user's best judgement.

The ease, with which spreadsheet programming handles data, is perhaps its strongest attribute. This is always a problem area in conventional programming where the management of input and output usually entails lengthy and numerous print statements. The adjustable and extensive cell formulation of spreadsheets makes it easy to establish a system of tables which will model the design problem. The user may enter input data directly to these tables and, with the built-in print statements available to the system, may extract at will all or any part of the input and generated data.

7.1.2 Potential Applications for Spreadsheets

In this work, while the technique of spreadsheet programming has been restricted to the design of long span composite beams using a variety of steel sections, there are many other analytical and design problems in structural engineering to which the system could be usefully applied. A few of the more obvious applications for the spreadsheet format could include the analysis and design of:

(1) Statically determinate non-composite beams and trusses

This is an option which could be readily incorporated into the spreadsheet routines developed for this work. It would have an immediate relevance for roof framing in multi-storey buildings, where a steel deck with insulation board and a tar-and-gravel finish is often preferred to a composite concrete deck.

(2) Composite floor slabs using profiled steel decking.

Again, this is an adjunct which could readily be applied to the spreadsheets of Parts 2 to 8.

(3) Elastic and plastic analysis of portal frames. It would be particularly useful in "Butler" type portal frames, where an extensive elastic analysis is usually required because of the thin webs and variable geometry of the frame members.

(4) Bolted and welded connections. While design values for commonly used welded and bolted connections are often tabulated, there are connections which are non-repetitive and too complex to lend themselves to this practice. End plate connections in which the bolts may

have to sustain tension, shear, and forces due to prying action are cases in point. A balanced design, which takes account of bolt gauge, edge distance, bolt diameter and grade, plate thickness and steel grade, and satisfies prying action, can consume a great deal of design time. A spreadsheet routine could be easily derived to tackle this problem.

In an educational role, the complete lack of mystery due to full user accessibility, and the ease with which the logic and methodology can be grasped should make the spreadsheet a valuable teaching tool and an ideal introduction to computing. Many problems arising from the introductory topics in structural analysis and strength of materials could be solved by this method. At the under-graduate level, problem solving is usually more concerned with the understanding and application of the principles involved than the arithmetic accuracy of the solution. This is not the case in professional practice where accuracy is equally important. The solutions to problems solved by spreadsheet modelling will always be arithmetically accurate.

7.1.3 Guidance on Spreadsheet Modelling

Commercial spreadsheets are always supplied with instruction manuals which give comprehensive guidance on the use and scope of the product. There is no intention here to instruct the reader in the mechanics of spreadsheet usage. However, there are a few points of general guidance arising from the design studies which may be of value in most spreadsheet design models:

(i) There should be a clear separation between the input data and the generated results. In the design studies some tables were used for both input data and calculation of results and, in particular, some of the input variables and options associated with the composite member were entered at the start of the composite design section. This approach, which was taken to shorten the running times of the programme, was a mistake. The location of the input has little effect on the running times. However, it is a great benefit to the user to be able to view all the input variables and selected design options on one page. This not only makes for easier adjustment of the input variables and design options but, more importantly, gives the user a clear picture of the structure he is trying to design.

(ii) The sequence of the calculations should fit the flow of the design process. For any given cell, the calculation should use

input or generated results which have been previously entered or calculated.

(iii) Where possible, Data Banks should be built into the spreadsheet computer routine. This approach is taken for the design of Warren trusses in Part 8, where the flexural properties of the hollow structural sections and equal leg angles required by the design procedure are listed by Part Numbers in Data Bank tables. The user enters the Part Numbers at the input stage of the design routine and, when required by the design process, the cell formulae call up the appropriate section property from the Data Banks. Since the user has only to enter a Part Number, the input process is shortened, the likelihood of entering erroneous data is reduced, and the cell formulae, which would otherwise be required to generate the section properties, are reduced to simple call-up statements.

In design practice, engineers make great use of tabulated values for materials and member properties and are comfortable with them. The Data Bank method would lend itself to design procedures which involve structural components for which tabulated properties are normally available. This could include: universal beams, universal tees, angles and other structural elements, castellated beams, composite steel decks, bolt and weld design values, etc.

However, the application of Data Banks to the design of Light Plate Girders would be counter-productive, since this would restrict the designer to some schedule of girders and remove his ability to refine the girder cross-section so that it will just satisfy the design constraints.

(iv) To obtain a concise presentation, the print-out tables of the spreadsheet should be restricted to all input data and only the important generated results. The extensive cell array available to the spreadsheet allows the user to make intermediate calculations outwith the print-out tables.

7.2 Future Studies on Composite Beams with Other Steel Sections

In the early Imperial College studies, consideration was given to forms of steel section which have not been applied in this work. While it is thought that the sections investigated in these studies are probably more amenable to systematic design and have some immediate relevance to the U.K construction practice, there are of

course other steel forms which should be considered in the design of long span composite beams.

7.2.1 Castellated Beams

While, to a fabricator's eye, the high fabrication costs should militate against them, Castellated beams have been a firm favourite in British construction practice for many years. The cross-sectional properties for Castellated Beams using Universal Beams, Universal Columns, and Joists are usually included in British steel design handbooks such as the Steel Designer's Manual [16] and Constrado [15]. The steel construction code, BS 5950: Part 1: Clause 4.15.3 [1], gives some rudimentary guidance on their design.

The conventional castellated beam with its equal size and constant pitch openings has two major disadvantages:

(i) the area provided by the castellations can only accommodate small circular ducts and, unless multiple ducts are used, will only service a limited floor space.

(ii) the regular proportions and pitch of the castellations makes the member vulnerable to Vierendeel action in high shear locations and, because of this, the openings at the ends of simply supported beams may require to be filled or locally stiffened. Even with end reinforcement, it is unlikely that these members could sustain the heavy point loads and shears associated with Type 2 Beams (Plan D). On the other hand, they could probably support the smaller distributed load of a Type 1 Beam (Plans B,C and D).

However, there are structures in which air ducting is not required or the limited opening size in castellated beams would be adequate and, in such applications, the castellated beam would be a valuable option in the design of long span composite beams. The available tabulated values of this member's properties could easily be transcribed to the Data Banks of a spreadsheet design procedure.

Whereas the production of Light Plate Girders would probably be restricted to a few highly mechanized plants, Castellated Beams could be readily supplied by most fabricators because of their long standing use in British practice. As was argued for trusses (2.8.7), to ensure the economic viability of new composite designs, it is important to use steel sections which can be supplied readily and competitively by the industry.

7.2.2 Modified Castellated Beams

In a recent paper [31], Dr.Owens has put forward a modified cutting pattern for Castellated Beams which would greatly reduce the disadvantages described for conventional cutting (7.2.1). The proposed cutting pattern produces a member which has narrow and widely spaced castellations at the end of the beam and elongated openings near to mid-span. This modified geometry allows the beam to sustain higher end shear and yet accommodate larger air ducts in the middle third of the span. While it is again unlikely that these modified members could carry the severe loading of Type 2 Beams without a substantial mass increase, they would certainly bring an enhanced performance to Type 1 Beams.

7.2.3 Stub-girder Construction

The combination of composite Stub-girders and Gerber beams has been used extensively as structural floor framing in Canada and the United States. The Gerber system consists of floor beams which are centred in the space between the stubs and are cantilevered beyond the girders to support shallower suspended beams. Both the Stub-girders and Gerber beams are designed to act compositely. While the major advantage of the system is the provision of unrestricted passage to mechanical services, there are several drawbacks:

(i) The system, as practised in North America, requires to be propped during construction.

(ii) Since the stubs must be the same depth as the Gerber beams, the whole assembly requires a deep floor zone.

(iii) The system does not appear to be economical at spans greater than thirteen metres (Chien and Rithchie [27]).

(iv) The Gerber beams are centred on the space between the stubs; the weak point of the arrangement. This is akin to locating secondary floor beams at the centreline of perforations in Universal Sections, an undesirable arrangement not provided for in these studies..

While a Stub-girder system has not yet been constructed in Britain, recent studies at Cambridge University, which involved full scale testing, have suggested a modified version that may be more attractive to our industry. A tee section top chord has been introduced which makes the steel assembly much stiffer and should obviate the need for propped construction. The new approach allows the floor beams to be

connected to the stubs rather than the girders. This lessens the local bending effects and also prevents the floor beam depth from dictating the depth of the whole assembly.

7.2.4 Perforated Light Plate Girders

For Type 2 Floor Beams, it can be seen from Fig 36 and Table 19 (6.2) that prismatic Light Plate Girders offer a substantial mass advantage over unperforated Universal Sections. The results in Table 13 (6.1.6) indicate that when unreinforced perforations are added to Type 2 Floor Beams the beam mass and depth must be increased to sustain the same design constraints. In this circumstance, the designer may choose to reinforce the opening and retain the same beam size as indicated by the results in Table 16 (6.1.7). Since the cost of cutting and reinforcing an opening in a Universal Section must be about the same as that for a Light Plate Girder, the economies of mass saving will still lie with the fabricated member.

A considerable effort was made to include in these studies a spreadsheet design routine for prismatic Light Plate Girders with reinforced web openings of the form shown in Fig 23:B. However, a satisfactory method for justifying the thin webs along the vertical edge of the opening was not established and the work was set aside. It is understood that the problem of perforations in thin webs is now being investigated by the Steel Construction Institute. If this problem is resolved, a spreadsheet routine could be devised and the full potential of these prismatic built-up girders would then be realized. As with perforated Universal Beams (Parts 6 and 7), a procedure for perforated prismatic Light Plate Girders would only require to check the effects at the openings, since the procedure of Part 3 already checks all other design constraints on the unperforated member.

An alternative approach to the problem of web instability at openings in Light Plate Girders is shown in Fig 57. It can be seen from this figure that the reinforcing flat bars, which are usually disposed horizontally along the opening, are triangulated. This arrangement, which would circumvent the problem of web instability, would carry the web shear across the opening without local bending. The resistance at the opening would be that of a composite truss with the concrete slab and lower steel tee acting as chords.

7.2.5 Trusses with Tee Chords

This is an obvious choice of structural assembly for use with long span composite beams and, as shown in Fig 27:B, is rated in this work as next best to the truss form applied in the Part 8 design studies (Fig 27:A). It has been widely used for many years in both British and North American structural practice. It is difficult to conceive that there is a fabricator who has not assembled this form of truss. However, a comparison with the Hollow Structural chords used in the studies would indicate that the Tee chords are at some disadvantage:

(i) The Tee chords must be stripped from Universal Sections and straightened before assembly, while the H.S.S chords need only be cut to length.

(ii) When the steel deck is being placed, the resistance of a Tee top chord to out-of-plane strut buckling is much less than that of an H.S.S, particularly, when a rotated rectangular section is used. This weakness could involve additional lines of erection bracing.

(iii) The Tee stem would normally be classified as slender and required to work at a reduced stress, while nearly all H.S.S. are compact sections.

(iv) The resistance of the Tee to local and eccentric bending moments in the presence of the high shear transfer between diagonals at unguessed panel points is less than the H.S.S. for comparable weights.

(v) To counteract the low resistance of Tees to both in-plane buckling and local bending moments between panel points in Type 1 Trusses, mid-panel verticals would probably be required. This arrangement is shown in the Modified Warren Truss of Fig 26:B, and would involve more truss components and some restriction on the alignment of services.

In addition to the familiarity which fabricators have with the assembly of this form of truss, the Tee chord has two important advantages:

(i) For a given overall truss depth (D_t), the lever arm of a steel truss (D_{cc}) with Tee chords will be greater than one with H.S.S chords.

(ii) If gussets are required, they can be added by simply extending the Tee stem with a butt-welded flat bar.

Trusses with Tee chords would be amenable to the spread-sheet design process and, as with the H.S.S trusses of Part 8, their components are regular structural sections with tabulated cross-sectional properties which would facilitate the use of Data Banks in the design procedure.

7.2.6 Trusses with Vierendeel Panel at Mid-Span

Although the area available for the passage of air ducts through Warren trusses will normally be sufficient for the service requirements of most buildings, there will be occasions when the duct proportions or the need to minimize the floor zone will require a larger opening. This condition could be satisfied to a degree by the truss configuration shown in Fig 26:D. Here, the web diagonals at the mid-span of the truss are omitted and a Vierendeel panel is formed. This unobstructed panel would be sufficient to accommodate a larger duct size. The remainder of the web would use a triangulated bracing system.

The central panel would require to resist the effects of Vierendeel shear from some portion of an unbalanced occupancy load. The Hollow Structural chords, which were used in the Part 8 studies, have good bending and shear resistance properties and would be well suited to act as chords, as well as end posts for the Vierendeel panel. The major disadvantage of this form of truss assembly, which is also put forward by Dr. Owens [31], is the constraint put on the alignment of air ducts.

The analysis of the resistance of the composite assembly could be made quite systematic by a two step calculation: Firstly, since with full and balanced occupancy loading there will be no shear across the central panel, most of the truss could be designed by the methods of Part 8. Secondly, with an unbalanced portion of the occupancy loading, the resistance of the truss at the Vierendeel panel could be checked by methods which are similar to those applied to unreinforced perforations in Part 6.

7.3 Uncertainties in Analytical Methods

7.3.1 Compactness of Steel Flange in Composite Section

In these studies, the calculation of the plastic capacity of the composite section is greatly simplified by the assumption that

the connected flange of Universal Beams and Light Plate Girders is compact. This is a provision of the draft versions of both Eurocode 4 [6] and BS 5950: Part 3.1 [2], and is only qualified by a modest requirement for the connection of the flange to the solid slab or composite concrete deck. While it is easy to accept the proposition when the steel flange is connected to a solid slab, it is more difficult to do so when a trapezoidal steel deck is used and firm contact is intermittent, as shown in Fig 5.

As mentioned earlier (3.1.2), this provision is in some measure supported by the Imperial College full scale test [30]. However, there does not appear to be a definitive work directly concerned with this problem which would vindicate the assumption of compactness.

7.3.2 Effective Breadth in Type 2 Beams

In these studies, the recommendations of BS 5950: Part 3.1, which pertain to the effective thickness (T_o) and breadth (B_e) of the concrete flange in Type 2 Beams using composite steel deck, have been followed. The code now allows the concrete flange to include the deck profile (T_o) and not, as previously, only the area above the profile (T_c). However, the effective breadth (B_e) is held to the lesser of $0.25L_b$ or $0.8C_{pb}$. This is tantamount to giving with one hand and taking back with the other. The $0.8C_{pb}$ restriction, which is apparently imposed to take account of local bending in the slab, would in some circumstances substantially reduce the concrete available to long span composite beams.

All the studies were conducted with a 1.2 mm thick CF46 profiled deck and a total slab thickness (T_o) of 120 mm. Perusal of the proprietary literature for this product shows that the span limitations are almost totally governed by the ability of the unpropped steel decking to sustain the wet concrete and construction loads. It can also be seen that a 120 mm composite deck can sustain an imposed load equal to the tabulated limit of 10 kN/m^2 . A thicker 160 mm composite deck would actually support close to 20 kN/m^2 on a slightly reduced span.

Many designs of long span beams would profit from the use of thicker decks, and floor geometry might require the use of deck spans less than their maximum. The limit of $0.8C_{pb}$ on the effective breadth takes no account of deck span or thickness.

The code also applies the reduced effective breadth to the

Serviceability Limit States; a provision for which there would appear to be little justification. Many of the design studies are governed by the deflection of the composite section and premature yield. Both of these conditions would be improved by increasing the concrete available to the composite flange.

7.3.3 Edge Loading on Type 2 Beams

Perusal of Tables 11 and 20 of the Part 3 studies on Micro-fiche for prismatic Light Plate Girders at a span-depth ratio of 20 shows that edge buckling ratios vary substantially for Type 1 and 2 floor beams.

From Studies 2,5,8,11,14 and 17 on Type 1 floor beams the range of edge buckling ratios is:

$$(P_{e3}/P_{ecr}) = 0.093 \text{ to } 0.174$$

From Studies 20,23,26,29,32 and 35 on Type 2 floor beams the range of edge buckling ratios is:

$$(P_{e3}/P_{ecr}) = 0.354 \text{ to } 1.016$$

From these results, it can be seen that edge buckling is not an important factor in the design of Type 1 Beams, whereas it is a significant factor in the justification of web strength in Type 2 Beams. It must also be remembered that the edge buckling ratio, which is often greater than that for shear in Type 2 Beams, is applied in the interaction formula as a linear term while the shear term is squared.

In these studies, the resistance of the web to edge loading (P_{ecr}) is taken as the elastic critical buckling strength as given in BS 5950: Part 1: Clause 4.5.2.2 [1] and applied in the interactive web buckling relationship of the code's Appendix H.3. There are several reasons to question the validity of applying this conservative provision to Type 2 Beams:

(i) In Type 2 Beams, secondary members are connected in the web and not on top of the beam as is assumed by the elastic buckling formula. It can be reasoned intuitively that the critical buckling resistance will increase as the point of application of the applied load drops on the beam.

(ii) The connection of the secondary floor members in the web should add a degree of restraint to the web.

(iii) With composite construction, there is always some uncertainty as to whether the load from secondary beams is transferred

to the primary beams by the connection or the concrete slab.

A resolution of these points would perhaps allow a less severe critical buckling formula to be applied to Type 2 Beams.

7.3.4 Interaction Equation for Thin Webs

As discussed earlier (4.4.7 and 4.6.3), the contribution of non-compact webs to the resistance of steel and composite sections is fixed by an interaction equation, which allows for the effects of bending, compression, shear and edge loading. The formula is fairly complex and for asymmetric girders is only fully satisfied by an iterative process. While the iteration presents no difficulty with the spreadsheet computer routine, a more direct method would probably be better understood and more easily applied by practising engineers, particularly, if they lack computer competence.

As discussed earlier for other reasons (3.1.5), the method given in BS 5400: Part 3: Clause 9.4.2.5 for determining the resistance of non-compact sections with unstiffened webs may offer a more direct approach. The bridge design procedure uses the concept of an effective web thickness (T_{we}), which is calculated from a simple formula based on the Neutral Axis depth of a section with an unreduced web. The pure bending resistance (M_D) of the section with an effective web thickness (T_{we}) is then calculated. The shear impaired section resistance is then found from a simple interaction equation. To align the bridge design process with the provisions of BS 5950: Part 1 several differences must be considered:

(i) In the calculation of the pure bending resistance of the section, the formula for finding the effective web thickness would require to account for the grossly different compactness rules of the structures code.

(ii) The interaction formula for calculating the impaired section resistance should include the effect of edge loading, shear, and the strengthening effect of axial tension in the composite section.

7.3.5 Chord Resistance at Gussetless Panel Points

There has as yet been little enthusiasm in the British construction community for composite trusses. However, as engineers search for optimum design solutions to the increasing demand for long

span floor systems, the economic viability of "gussetless" trusses with eccentric panel point connections should become apparent.

While much of the truss design is straightforward and conventional, the highly stressed gusset free panel points require careful attention. In the procedure of Part 8, the chord resistance at the panel point is checked for the effects of axial force, bending due to eccentricity of the diagonals and local loads, and shear transfer between the diagonals. The axial and bending resistances are considered to be shear impaired when the applied shear is greater than $0.3V_y$ and $0.6V_y$ respectively. The latter limit is a general provision of the steel structures code, BS 5950: Part 1, which gives no guidance on shear impairment of axially loaded members.

This is a subtle area and the approach taken in the design studies tends to be lengthy and perhaps overly conservative. A detailed study of these connections on both H.S.S and Tee chords might produce a more direct and less conservative approach.

7.3.6 Deflection Limitations

In the studies, the Service Deflection (D_{max}) limit is fixed at $L_b/360$, which is the deflection limit given in BS 5950: Part 1: Table 5 for beams carrying materials with a brittle finish. The limit for beams with non-brittle finishes is $L_b/200$. However, even though the materials of the suspended ceilings of modern buildings are far from brittle, and few have plaster finishes, the limit on the service deflection is almost always wisely applied as $L_b/360$.

For the long span members considered in this work, the more generous provision of the code ($L_b/200$) would allow Service Deflections up to 120 mm. While the average layman (office occupant) knows that things can break, elastic deformation is quite beyond his ken, and walking uphill to his desk on a supposedly level floor is likely to be a daunting experience. The problem is not of plaster finishes but of human perception. Should the less wary engineer who rigorously applies the letter of the code, and there are many of them, really have the option to use $L_b/200$ in an office floor?

While there is no limit in the structures code for the dead load deflection, in these studies the Calculated Camber (D_{ca}), which measures the deflection of the unpropped steel section, is limited to $L_b/250$. While the out and out structural failure of a floor beam is

rare and quite outwith the writer's experience, contractual postmortems over structural tolerances in general, and non-level floors in particular, are legion: the general contractor's price could never allow for the additional concrete; the engineer's design did not allow for more or less concrete; the mechanical services and finishes contractors expected to work with level surfaces, the tenant wonders why the floor has a hump. Longer spans in composite construction will exacerbate these difficulties. Since the one sure way to keep the problem in bounds is to put a reasonable limit on the dead load deflection, should this limit not be codified ?

The report on the Imperial College test [32] gave the total deflection of the composite section as 50.2 mm ($L_b/300$), when subjected to one day of sustained working load. The measurement of any further deflection due to long term loads (creep), which are likely to produce some increase, was excluded by the limited duration of the test. Raven and Glover [30] gave an account of the close relationship between the predicted and actual elastic deflections of the girder. However, they did not comment on the magnitude of either value or, perhaps more importantly, offer an opinion on a reasonable limit for the Service Deflection.

7.3.7 Limit for Premature Yield

For reasons offered earlier (4.8.3), the limit on Premature Yield used in the studies is $0.9F_y$. The draft BS 5950: Part 1.3 gives a limit of F_y . The governing Performance Ratios for Type 1 and 2 floor beams using Universal Sections and the three types of Light Plate Girder are shown Fig 46,48,50 and 51. Superimposed on these figures are the governing ratios when the limit placed on the Calculated Camber (R_{s1}) is discounted and the limit on Premature Yield is taken as F_y (R_{s3}). The effect of changing these limits is to make unpropped composite beams behave like propped ones, since the designs are then mostly governed by either composite section strength (R_{u3}) or deflection (R_{s2}).

It can be seen from the figures that, whereas the Universal Section (Fig 48) is little affected, the governing ratios for Light Plate girders are substantially changed. The implications of this are that the provisions of BS 5950: Part 3.1 favour the novel sections more than the conventional Universal Sections.

What is the justification for the higher limit of Premature Yield adopted by BS 5950: Part 3.1 ?.

7.4 Concluding Remarks on Member Performance and Applications

7.4.1 Universal Rolled Sections without Perforations (Part 2)

It is obvious from the comparisons of mass efficiency made in these studies (ex. Fig 36) that Universal sections are a poor choice for spans over 12 metres, and dilligent engineers will look to other steel sections for better solutions. There will of course be many instances where shorter spans, insufficient repetition of pieces to justify production, and the lack of fabrication skills will make the other choices less attractive.

At Charing Cross in Glasgow, there is presently being erected an office complex with an irregular Type 2 floor plan. In the more regular long span areas the primary beams are composite tapered girders, while all secondary beams and primary beams in the more irregular areas are rolled sections.

The mixture of rolled and fabricated floor beams identified in the Glasgow building will be typical of many structures. The rolled section will continue to be the first and often the correct choice for many tasks in composite construction. The design procedure established in Part 2 will allow quick and efficient section selection with due regard to the degree of connection required. More importantly, it will ensure that the section selected meets all reasonable constraints at the Ultimate and Serviceability Limit States.

7.4.2 Universal Rolled Sections with Perforations (Parts 2, 6 and 7)

As argued above, there will be structures in which rolled sections will be primary beams and, as such, will require to accommodate the passage of ducts and other services. The Part 2 procedure will allow initial selection of the unperforated section. The procedures of Parts 6 and 7 will measure the effects at web penetrations with due regard to the proportions, locations, and reinforcement (if any) at the openings. From Part 6, the designer will find that he has often the option to design low cost unreinforced openings for a modest or zero mass increase (Tables 12 and 13). From Part 7, on the other hand, he will find that

the section initially selected in Part 2 can almost always be justified with reinforced openings (Tables 14 and 15).

7.4.3 Light Plate Girders (Parts 3, 4 and 5)

Provided the highest priority is not placed on a minimum floor zone and air ducts can be tolerated near the building perimeter, the studies in Parts 4 and 5 demonstrate that tapered girders offer a substantial mass advantage over rolled sections (Tables 18 and 19). Where floor zone is a concern and duct alignment can be held close to the perimeter, the combination of asymmetry and taper modification offer more advantage than the fully tapered member (Table 19).

In structures, where services and floor steel are not inter-dependant, the prismatic girders studied in Part 5 could often prove viable at span-depth ratios which are close to those of rolled sections (Table 18).

It is perhaps worth noting that the tapered girders in the Type 2 framing of the Glasgow structure (7.4.1) have a number of features which differ from those used in the Imperial College test girder, but are more in accord with the design approach taken in these studies. These differences included:

- (i) A symmetric section, the studies showed little benefit from asymmetry (6.1.3 to 5).

- (ii) The steel profiled decking was broken at the primary beams (6.1.4).

- (iii) Two rows of "shop" installed shear studs, which represents a much higher degree of concrete participation than that assumed by the test girder (6.1.4). The Imperial College test girder had 67 shear studs, while the "Glasgow" girder had 180.

The "Glasgow" girder has been investigated in the studies in Part 4: Design Study No.46. From the Part 4: Results it can be seen that 180 studs represents $90\%C_o$ and that the design was governed by composite section deflection ($R_{s2} = 0.964$).

The study was made with the material properties, deck type, occupancy loading, and primary floor beam spacing used in the Glasgow building. These values differed considerably from those used in the studies as the "Standard Building Properties".

7.4.4 Warren Trusses with Hollow Structural Chords (Part 8)

If the depth of the floor zone is not a high priority, but the free alignment of air ducts is, it can be seen from Fig 43 and 44, and Tables 18 and 19 that the Warren Trusses studied in Part 8 are highly mass efficient and a sound choice for framing spans greater than 12 metres. The less obvious quality of trusses is their universal availability, since all fabricators will have experience in truss assembly.

In this context, it is perhaps worth noting that the structural steel for the Glasgow office complex was supplied by a fabricator in the North of England (Oh Caledonia !). While it is not likely that a truss design would have satisfied all the architectural and service requirements of this sophisticated building, it is perhaps reasonable to question why the built-up tapered girders were not supplied from Scottish sources.

APPENDIX 1

MEMBERS of the ADVISORY COMMITTEE

Dr.G.W. Owens	Senior Lecturer	Imperial College
Mr.T.M. Leiper	Research Fellow	
Mr.M. Balsara	Research Assistant	
Mr.G. West	Architects	Fitsroy Robinson Partnership
Mr.T.Poulson		Y. R. M.
Mr.D. Lazenby	Consulting Engineers	Andrews, Kent and Stone
Mr.D. Beare		Andrews, Kent and Stone
Mr.J. Mathys		H.L. Waterman & Partners
Mr.M. Glover		Ove Arup & Partners
Mr.P. Clarke		Rendel, Palmer & Tritton
Mr.J. Brew	Services Consultant	F.C. Foreman & Partners
Mr.R. Jones	Construction Manager	Property Project Control Ltd
Mr.A. Weller	Engineer	CONSTRADO
Mr.G. Taylor	Steel Fabricators	Graham Wood Structural Ltd
Mr.M. Harman		S.W. Farmer & Sons. Ltd
Mr.G. Raven		Ward Brothers
Mr.M. Solomon	Quantity Surveyor	Cyril Sweet & Partners

APPENDIX 2

COMPUTER HARDWARE and SOFTWARE used in DESIGN STUDIES

HARDWARE:

COMPUTER:

Hewlett Packard, HP 9000, Series 200, Model 216 with 1.25 MBytes of RAM and BASIC 3.0 language system.

DISC DRIVE:

Hewlett Packard, HP 9133D 15 Mbyte Winchester hard-disc and 3.5" double-sided flexible disc drive.

PRINTER:

Hewlett Packard Laser Jet+, Model 2686A.

PLOTTER:

Hewlett Packard, HP 7475A Graphics Plotter.

SOFTWARE:

SPREADSHEET:

Hewlett Packard "Context MBA": Release 2.4: Option 630.

Programme Discs: MBA1, MBA2, MBA3.

Data Discs:

Part No.:	2	3	4	5	6	7	8
Volume:	LSCFS:	LSCFS:	LSCFS:	LSCFS:	LSCFS:	LSCFS:	LSCFS:
Folder:	PART2:	PART3:	PART4:	PART5:	PART6:	PART7:	PART8:
Document:	UB	LPGC	LPGT	LPGM	UB_U	UB_R	TRUSS_H

GRAPHICS:

Hewlett Packard "Graphics Editor 200", version 3.0 with PASCAL 3.0 language system.

Programme Discs: BOOT(standard), BOOT(large system), FONTS, KEY.

Data Discs: Disc No.1, Volume:Files = Part1:FIG.1 to 26.

Disc No.2, Volume:Files = Part1:FIG.27 to 57.

APPENDIX 3

TABLE 1: DESIGN STRENGTHS of STRUCTURAL STEEL, REINFORCING FABRIC and PROFILED STEEL DECK

:-----:-----:-----:-----:-----:									
:		:		:STEEL: MATERIAL:		DESIGN STRENGTH :			
: MATERIAL		:SPECIFICATIONS:		GRADE:THICKNESS:		:-----:-----:-----:			
:		:		mm		:SYMBOLS:VALUES:UNITS:			
:-----:-----:-----:-----:-----:									
:STRUCTURAL STEEL	: BS4360(1979)	:	43	:	≤ 16	:	F_y	:	275 :N/mm ² :
:	:	:	43	:	≤ 40	:	:	:	265 : " :
:	:	:	50	:	≤ 16	:	:	:	355 : " :
:	:	:	50	:	≤ 63	:	:	:	340 : " :
:REINFORCING FABRIC:	BS4482(1985)	:	-	:	5 to 12	:	F_{ry}	:	460 : " :
:PROFILED DECK	: BS2989(1982)	:	-	:	0.9 & 1.2	:	F_{dy}	:	280 : " :
:-----:-----:-----:-----:-----:									
Other properties of steel: Modulus of elasticity - $E_s = 205 \text{ kN/mm}^2$									
Density of steel = 7850 kg/mm ³									

TABLE 2: DESIGN VALUES of CONCRETE

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APPENDIX 4

TABLE 3: DESIGN VALUES of STUD SHEAR CONNECTORS in SOLID SLABS

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:	:	STUD	FLGE:	:	CUBE STRENGTH OF CONCRETE	:	:	:	:
:STUD PROPERTIES:	SIZE	:	T _f	:	SYMB:	-----:-----:-----:-----:-----:-----:UNITS:			
:	:	mm	:	mm	:	:	NORMAL WEIGHT:	LIGHT-WEIGHT	:
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:	:	:	:	F _{cu}	:	30 : 40 : 50 : 25 : 30 : 40	:	N/mm ² :	:
:STATIC STRENGTH:	19 x 100:	:	:	Q _{sd}	:	100 :109 :119 :80.8:85.0:92.7	:	kN	:
:	:	16 x 100:	:	Q _{sd}	:	74 : 82 : 90 :59.5:62.9:69.7	:	kN	:
:DESIGN STRENGTH:	19 x 100:	> 7:	:	P _{sd}	:	80 : 87 : 95 :64.6:68.0:74.2	:	kN	:
:	:	16 x 100:	≤ 7:	P _{sd}	:	59 : 66 : 72 :47.6:50.3:55.8	:	kN	:
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									

For normal weight concrete $P_{sd} = 0.8(Q_{sd})$

For light-weight concrete $P_{sd} = 0.8(0.85Q_{sd})$

TABLE 4: REDUCTION FACTOR to the DESIGN STRENGTH of SHEAR CONNECTORS in
TYPE 1 BEAMS with COMPOSITE DECKS - K_{sd} (Fig 12)

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
:	:	:	DEPTH:	BREADTH:	STUD:	REDUCT:	STUDS IN RIB - N _r		
: DECK TYPE	:COMPANY:	T _d	:	B _r	:	H _{st} :	FACTOR:	-----:-----:-----:-----:-----:-----:	:
:	:	:	mm	:	mm	:	mm	:	1 : 2 : 3 : > 3 :
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									
: 1, Fig 12A	: CF 46	:	46	:	132	:	95	:	K _{sd} :1.000:1.000:1.000:1.000:
: 2, Fig 12B	:HOLORIB:	51	:	113	:	95	:	K _{sd} :1.000:1.000:0.938:0.938:	:
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:									

T_d, B_r and H_{st} are defined in Fig 12

Reduction factor = $K_{sd} = (0.85/\sqrt{N_r})(B_r/T_d)[(H_{st}/T_d) - 1] \leq 1$

APPENDIX 5

TABLE 5: CRITICAL SHEAR STRENGTH of an UNSTIFFENED WEB, V_{cr} (kN)
BS 4360: STEEL GRADE 50

:-----:-----:-----:														
: DEPTH :	$F_y = 355 \text{ N/mm}^2$							:	$F_y = 340 \text{ N/mm}^2$					:
: of :	:-----:-----:-----:													
: WEB(D_w) :	WEB THICKNESS - T_w (mm)							:	WEB THICKNESS - T_w (mm)					:
:-----:-----:-----:														
: mm :	5	6	7	8	9	10	:	5	6	7	8	9	10	
:-----:-----:-----:														
: 300 :	300	383	447	511	575	639	:	292	367	428	490	551	612	
: 350 :	307	429	522	596	671	746	:	300	417	500	571	643	714	
: 400 :	301	440	580	682	767	852	:	296	430	564	653	734	816	
: 450 :	278	440	597	754	863	959	:	278	431	582	732	826	918	
: 500 :	250	426	601	776	950	1065	:	250	421	588	755	918	1020	
: 550 :	227	393	593	785	977	1169	:	227	393	583	767	951	1122	
: 600 :	208	360	572	781	991	1201	:	208	360	566	767	967	1168	
: 650 :	192	332	528	766	993	1220	:	192	332	528	755	972	1190	
: 700 :	179	309	490	731	982	1227	:	179	309	490	731	966	1200	
: 750 :	167	288	457	683	959	1221	:	167	288	457	683	947	1198	
: 800 :	156	270	429	640	911	1203	:	156	270	429	640	911	1185	
: 850 :	147	254	404	602	858	1172	:	147	254	404	602	858	1160	
: 900 :	139	240	381	569	810	1111	:	139	240	381	569	810	1111	
: 950 :	132	227	361	539	767	1053	:	132	227	361	539	767	1053	
: 1000 :	125	216	343	512	729	1000	:	125	216	343	512	729	1000	
: 1050 :	119	206	327	488	694	952	:	119	206	327	488	694	952	
: 1100 :	114	196	312	465	663	909	:	114	196	312	465	663	909	
: 1150 :	109	188	298	445	634	870	:	109	188	298	445	634	870	
: 1200 :	104	180	286	427	608	833	:	104	180	286	427	608	833	
: 1250 :	100	173	274	410	583	800	:	100	173	274	410	583	800	
: 1300 :	96	166	264	394	561	769	:	96	166	264	394	561	769	
: 1350 :	93	160	254	379	540	741	:	93	160	254	379	540	741	
: 1400 :	89	154	245	366	521	714	:	89	154	245	366	521	714	
: 1450 :	86	149	237	353	503	690	:	86	149	237	353	503	690	
: 1500 :	83	144	229	341	486	667	:	83	144	229	341	486	667	
:-----:-----:-----:														

APPENDIX 6

TABLE 6: CRITICAL SHEAR STRENGTH of an UNSTIFFENED WEB, V_{cr} (kN)
BS 4360: STEEL GRADE 43

:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:																
: DEPTH :	$F_y = 275 \text{ N/mm}^2$							:	$F_y = 265 \text{ N/mm}^2$							:
: of :	:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:															
: WEB(D_w) :	WEB THICKNESS - T_w (mm)							:	WEB THICKNESS - T_w (mm)							:
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:																
: mm :	5	6	7	8	9	10	:	5	6	7	8	9	10	:		
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:																
: 300 :	248	297	347	396	446	495	:	239	286	334	382	429	477	:		
: 350 :	266	347	404	462	520	578	:	260	334	390	445	501	557	:		
: 400 :	270	378	462	528	594	660	:	225	369	445	509	572	636	:		
: 450 :	265	387	509	594	668	743	:	262	379	497	572	644	716	:		
: 500 :	250	388	523	659	743	825	:	250	381	512	636	716	795	:		
: 550 :	227	380	529	678	817	908	:	227	375	519	662	787	875	:		
: 600 :	208	360	526	688	851	990	:	208	360	518	674	831	954	:		
: 650 :	192	332	515	691	867	1043	:	192	332	509	678	848	1017	:		
: 700 :	179	309	490	685	874	1063	:	179	309	490	674	857	1039	:		
: 750 :	167	288	457	670	873	1076	:	167	288	457	662	858	1053	:		
: 800 :	156	270	429	640	863	1080	:	156	270	429	640	851	1060	:		
: 850 :	147	254	404	602	845	1075	:	147	254	404	602	836	1058	:		
: 900 :	139	240	381	569	810	1062	:	139	240	381	569	810	1048	:		
: 950 :	132	227	361	539	767	1040	:	132	227	361	539	767	1030	:		
: 1000 :	125	216	343	512	729	1000	:	125	216	343	512	729	1000	:		
: 1050 :	119	206	327	488	694	952	:	119	206	327	488	694	952	:		
: 1100 :	114	196	312	465	663	909	:	114	196	312	465	663	909	:		
: 1150 :	109	188	298	445	634	870	:	109	188	298	445	634	870	:		
: 1200 :	104	180	286	427	608	833	:	104	180	286	427	608	833	:		
: 1250 :	100	173	274	410	583	800	:	100	173	274	410	583	800	:		
: 1300 :	96	166	264	394	561	769	:	96	166	264	394	561	769	:		
: 1350 :	93	160	254	379	540	741	:	93	160	254	379	540	741	:		
: 1400 :	89	154	245	366	521	714	:	89	154	245	366	521	714	:		
: 1450 :	86	149	237	353	503	690	:	86	149	237	353	503	690	:		
: 1500 :	83	144	229	341	486	667	:	83	144	229	341	486	667	:		
:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:-----:																

APPENDIX 7

FORMULAE for COMPOSITE DECKS:

TYPE 1 DECK: CF 46 PROFILED STEEL DECK by PRECISION METAL FORMING

The relevant symbols and dimensions for Type 1 Deck are defined in Fig 12:A. The formulae developed in this appendix are applied in Parts 2 to 8: Table 3: Properties of Composite Deck.

Area of profile:

$$\begin{aligned} &= 0.5 \times 46(105 + 158) &= 6049 \text{ mm}^2 \\ &= 6049 \times 10^3 / 225 &= 26884 \text{ mm}^2/\text{m} \end{aligned}$$

Distance to centroid of profile from underside of deck:

$$= 46(2 \times 158 + 105) / 3(158 + 105) = 24.55 \text{ mm}$$

Second moment of area of profile about centroid:

$$= 46^3(158^2 + 4 \times 158 \times 105 + 105^2) / 36(158 + 105) = 1052201 \text{ mm}^4$$

Area of concrete in compression: (mm^2/m)

$$\begin{aligned} A_{cc} &= (T_c \times 10^3 + 26884) - \text{Type 2 Beam} \\ A_{cc} &= (T_c \times 10^3) - \text{Type 1 Beam} \end{aligned}$$

Area of concrete in longitudinal shear plane: (mm^2/m)

$$\begin{aligned} A_{cs} &= (T_c \times 10^3) - \text{Type 2 Beam} \\ A_{cs} &= (T_c \times 10^3 + 26884) - \text{Type 1 Beam} \end{aligned}$$

Distance from top to centroid of composite deck: (mm)

$$\begin{aligned} Y_{cd} &= [0.5T_c^2 \times 10^3 + 26884(T_o - 24.55)] / A_{cc} - \text{Type 2 Beam} \\ Y_{cd} &= 0.5T_c - \text{Type 1 Beam} \end{aligned}$$

Second moment of area of composite deck about centroid: (mm^4/m)

$$\begin{aligned} I_{cd} &= (0.083T_c^3 + T_c(Y_{cd} - 0.5T_c)^2 \\ &\quad + [6049(T_o - Y_{cd} - 24.55)^2 + 1052201] / 225) 10^3 - \text{Type 2 Beam} \\ I_{cd} &= (0.083T_c^3) 10^3 - \text{Type 1 Beam} \end{aligned}$$

APPENDIX 8

FORMULAE for COMPOSITE DECKS:

TYPE 2 DECK: SUPER HOLORIB STEEL DECK by RICHARD LEES LTD.

The relevant symbols and dimensions for Type 2 Deck are defined in Fig 12:B. The formulae developed in this appendix are applied in Parts 2 to 8: Table 3: Properties of Composite Deck.

Area of profile:

$$\begin{aligned} &= 0.5 \times 51(137.5 + 112.5) &= 6375 \text{ mm}^2 \\ &= 6375 \times 10^3 / 152.5 &= 41667 \text{ mm}^2/\text{m} \end{aligned}$$

Distance to centroid of profile from underside of deck:

$$= 51(2 \times 112.5 + 137.5) / 3(112.5 + 137.5) = 24.65 \text{ mm}$$

Second moment of area of profile about centroid:

$$= 51^3(112^2 + 4 \times 112 \times 137 + 137^2) / 36(112 + 137) = 1377175 \text{ mm}^4$$

Area of concrete in compression: (mm^2/m)

$$\begin{aligned} A_{cc} &= (T_c \times 10^3 + 41667) && \text{Type 2 Beam} \\ A_{cc} &= (T_c \times 10^3) && \text{Type 1 Beam} \end{aligned}$$

Area of concrete in longitudinal shear plane: (mm^2/m)

$$\begin{aligned} A_{cs} &= (T_c \times 10^3) && \text{Type 2 Beam} \\ A_{cs} &= (T_c \times 10^3 + 41667) && \text{Type 1 Beam} \end{aligned}$$

Distance from top to centroid of composite deck: (mm)

$$\begin{aligned} Y_{cd} &= [0.5T_c^2 \times 10^3 + 41667(T_o - 24.65)] / A_{cc} && \text{Type 2 Beam} \\ Y_{cd} &= 0.5T_c && \text{Type 1 Beam} \end{aligned}$$

Second moment of area of composite deck about centroid: (mm^4/m)

$$\begin{aligned} I_{cd} &= \{0.083T_c^3 + T_c(Y_{cd} - 0.5T_c)^2 \\ &\quad + [6375(T_o - Y_{cd} - 24.55)^2 + 1377175] / 225\} 10^3 && \text{Type 2 Beam} \\ I_{cd} &= (0.083T_c^3) 10^3 && \text{Type 1 Beam} \end{aligned}$$

APPENDIX 9

FORMULAE for UNIVERSAL ROLLED SECTIONS

The relevant symbols for Universal Rolled sections are defined in Fig 10. The formulae developed in this appendix are applied in Part 2: Tables 8 and 9, and Parts 6 and 7: Table 9.

Area of steel section: (mm²)

$$A_s = 2T_f B_b + T_w(D_b - 2T_f) + 0.8584r^2$$

Second moment of area of steel section about major axis: (mm⁴)

$$I_x = B_b T_f^3 / 6 + T_f B_b (D_b - T_f)^2 + T_w (D_b - 2T_f)^2 / 12 + 0.302r^4 + 0.8584r^2 (0.5D_w + 0.7767r)^2$$

Second moment of area of steel section about minor axis: (mm⁴)

$$I_y = T_f B_b^3 / 6 + (D_b - 2T_f) T_w^3 / 12 + 0.0302r^4 + 0.8584r^2 (0.5T_w + 0.2234r)^2$$

Radius of gyration of steel section about minor axis: (mm)

$$R_y = (I_y / A_s)^{0.5}$$

Elastic modulus of steel section about major axis: (mm³)

$$Z_x = 2I_x / D_b$$

Plastic modulus of steel section about major axis: (mm³)

$$S_x = T_f B_b (D_b - T_f) + 0.25T_w (D_b - 2T_f)^2 + 0.8584r^2 (0.5D_w + 0.7766r)$$

Torsion constant: (mm⁴)

$$J_t = 0.333[2B_b T_f^3 + D_w T_w^3 + (2r + 1.5T_f)(T_w + 0.4292r)^3]$$

Distance between shear centres of flanges: (mm)

$$H_s = D_b - T_f$$

Warping constant: (mm⁶)

$$H_w = 0.0417[H_s^2 T_f B_b^3]$$

APPENDIX 10

FORMULAE for MONO-SYMMETRIC LIGHT PLATE GIRDERS

The relevant symbols for Light Plate Girders are defined in Fig 10. The formulae developed in this appendix are applied in Parts 3, 4 and 5: Tables 8 and 9.

Area of steel section: (mm²)

$$A_s = B_b(T_{tf} + T_{bf}) + T_w D_w$$

Distance from top to ENA of steel section: (mm)

$$Y_{et} = \{B_b[0.5T_{tf}^2 + T_{bf}(D_b - 0.5T_{bf})] + T_w D_w(0.5D_w + T_{tf})\}/A_s$$

Second moment of area of steel section about major axis: (mm⁴)

$$I_x = B_b(T_{tf}^3 + T_{bf}^3)/12 + T_w D_w^3/12 + B_b T_{tf}(Y_{et} - 0.5T_{tf})^2 + B_b T_{bf}(D_b - Y_{et} - 0.5T_{bf})^2 + T_w D_w(T_{tf} + 0.5D_w - Y_{et})^2$$

Second moment of area of steel section about minor axis: (mm⁴)

$$I_y = [(T_{tf} + T_{bf})B_b^3 + D_w T_w^3]/12$$

Radius of gyration of steel section about minor axis: (mm)

$$R_y = (I_y/A_s)^{0.5}$$

Elastic modulus at top of steel section about major axis: (mm³)

$$Z_{xt} = I_x/Y_{et}$$

Distance from top to PNA of steel section: (mm)

$$Y_{pt} = [(0.5A_s - T_{tf}B_b)/T_w] + T_{tf}$$

Plastic modulus of steel section about major axis: (mm³)

$$S_x = B_b T_{tf}(Y_{pt} - 0.5T_{tf}) + B_b T_{bf}(D_b - Y_{pt} - 0.5T_{bf}) + 0.5T_w(Y_{pt} - T_{tf})^2 + 0.5T_w(D_b - Y_{pt} - T_{bf})^2$$

Torsion constant: (mm⁴)

$$J_t = 0.333[B_b(T_{tf}^3 + T_{bf}^3) + D_w T_w^3]$$

Distance between shear centres of flanges: (mm)

$$H_s = [D_b - 0.5(T_{tf} + T_{bf})]$$

Warping constant: (mm⁶)

$$H_w = 0.0833(H_s^2 T_{tf} T_{bf} B_b^3)/(T_{tf} + T_{bf})$$

APPENDIX 11

DEFINITION of SYMBOLS COMMON to APPENDICES 11 to 14 (Fig 22 to 25)

V_1	Shearing force at centreline of web opening, corresponding to point "1" on the interaction diagram.
M_1	Moment of resistance of perforated section at centreline of web opening, corresponding to point "1" on the interaction diagram.
M_o	Moment of resistance of perforated section under pure bending at centreline of web opening, corresponding to point "0" on the interaction diagram.
V_{u2}	Factored vertical shear for load case R_{u2}
V_{u3}	Factored vertical shear for load case R_{u3}
M_{rso}	Moment of resistance of steel section at web opening
M_{rco}	Moment of resistance of composite section at web opening
V_{1t}	Portion of V_1 carried at top tee
V_{1b}	Portion of V_1 carried at bottom tee
S_t	Depth of tee-section web above opening
S_b	Depth of tee-section web below opening
F_{yr}	design strength impaired by shear yield

The factors k_i , where $i = 1$ to 4, lie between zero and unity, and represent the portion of the flange, or the web, subject to tensile or compressive stress (Fig 24,25).

N_h	Number of shear studs within the length of web opening
R_{ch}	Resistance of shear studs within the length of web opening
T_{co}	Thickness of concrete in compression at web opening

APPENDIX 11 (cont'd)

FORMULAE for UNREINFORCED WEB OPENINGS in the STEEL SECTION

The formulae are derived from a paper by Redwood and Poubouras [38], and are applied in Part 6: Spreadsheet: Tables 12 to 14.

The assumed stress distribution at failure for a high shear-to-moment ratio, which corresponds to point "1" on the interaction diagram, is shown in Fig 24. The force in the concrete, R_{ch} , is ignored in this analysis. The important symbols are defined in Appendix 11.

Equilibrium above, below, and at the centreline of the opening gives the following relationships:

$$V_1 = V_{1t} + V_{1b}$$

$$P_1 = P_2 = P_3 = P_4$$

$$L_o V_{1t} = P_1 y_1 - P_2 y_2 \text{ at top tee} \quad (1)$$

$$L_o V_{1b} = P_3 y_3 - P_4 y_4 \text{ at btm tee} \quad (2)$$

At the high moment end of web opening:

$$M_1 + 0.5V_1 L_o = P_1(0.5H_o + y_1) + P_3(0.5H_o + y_3) \quad (3)$$

At the low moment end of web opening:

$$M_1 - 0.5V_1 L_o = P_2(0.5H_o + y_2) + P_4(0.5H_o + y_4) \quad (4)$$

From the addition of (3) and (4):

$$M_1 = 0.5\sum(P_i y_i) + 0.25H_o \sum(P_i), \text{ where } i = 1 \text{ to } 4 \quad (5)$$

$$V_{yt} = \text{ultimate vertical shear capacity of top tee} = 0.6F_y T_w S_t$$

$$V_{yb} = \text{ultimate vertical shear capacity of btm tee} = 0.6F_y T_w S_b$$

$$R_f = \text{resistance of flange of steel section} = F_y T_f B_b$$

R_{wtr} = resistance of web of top tee impaired by shear (Von Mises)

$$= T_w S_t (F_{yr}) = T_w S_t (F_y^2 - 3F_s^2)^{0.5} = (T_w^2 S_t^2 F_y^2 - 3T_w^2 S_t^2 F_s^2)^{0.5}$$

$$= (3V_{yt}^2 - 3V_{1t}^2)^{0.5} \text{ and } \beta_t = (R_{wtr}/R_f) \quad (6)$$

R_{wbr} = resistance of web of bottom tee impaired by shear (Von Mises)

$$= T_w S_b (F_{yr}) = T_w S_b (F_y^2 - 3F_s^2)^{0.5} = (T_w^2 S_b^2 F_y^2 - 3T_w^2 S_b^2 F_s^2)^{0.5}$$

$$= (3V_{yb}^2 - 3V_{1b}^2)^{0.5} \text{ and } \beta_b = (R_{wbr}/R_f) \quad (7)$$

APPENDIX 11 (cont'd)

From Fig 24, the stress resultants at the four corners are:

$$P_1 = R_f(2k_1 - 1) - R_{wtr} = R_f(2k_1 - 1 - \beta_t) \quad (8)$$

$$P_2 = R_f(1 - 2k_2) + R_{wtr} = R_f(1 - 2k_2 + \beta_t) \quad (9)$$

$$P_3 = R_f(2k_3 - 1) - R_{wbr} = R_f(2k_3 - 1 - \beta_b) \quad (10)$$

$$P_4 = R_f(1 - 2k_4) + R_{wbr} = R_f(1 - 2k_4 + \beta_b) \quad (11)$$

From equations (8) to (11) and $P_1 = P_2 = P_3 = P_4$:

$$k_1 = 0.5(\beta_t - \beta_b) + k_3 \quad (12)$$

$$k_2 = 0.5(\beta_t + \beta_b) + 1 - k_3 \quad (13)$$

$$k_4 = \beta_b + 1 - k_3 \quad (14)$$

From Fig 24, and assuming that S_t and S_b are much greater than T_f :

$$P_1 y_1 = R_f S_t (2k_1 - 1) - 0.5 R_{wtr} S_t = R_f S_t (2k_1 - 1 - 0.5\beta_t) \quad (15)$$

$$P_2 y_2 = R_f S_t (1 - 2k_2) + 0.5 R_{wtr} S_t = R_f S_t (1 - 2k_2 + 0.5\beta_t) \quad (16)$$

$$P_3 y_3 = R_f S_b (2k_3 - 1) - 0.5 R_{wbr} S_b = R_f S_b (2k_3 - 1 - 0.5\beta_b) \quad (17)$$

$$P_4 y_4 = R_f S_b (1 - 2k_4) + 0.5 R_{wbr} S_b = R_f S_b (1 - 2k_4 + 0.5\beta_b) \quad (18)$$

From equations (1), (15), (16) and then (12) and (13):

$$\begin{aligned} L_o V_{1t} &= R_f S_t [2(k_1 + k_2) - 2 - \beta_t] = R_f S_t [2(\beta_t + 1) - 2 - \beta_t] \\ &= R_f S_t (\beta_t) = S_t (R_{wtr}) \end{aligned}$$

$$V_{1t} = (S_t/L_o) R_{wtr} = (S_t/L_o) (3V_{yt}^2 - 3V_{1t}^2)^{0.5}, \text{ from (6)}$$

Squaring both sides:

$$V_{1t}^2 = (S_t/L_o)^2 (3V_{yt}^2 - 3V_{1t}^2), \text{ and if } K_t = 3(S_t/L_o)^2$$

$$V_{1t}^2 = K_t (3V_{yt}^2 - 3V_{1t}^2)/3$$

$$V_{1t}^2 = K_t V_{yt}^2 / (1 + K_t)$$

$$V_{1t} = V_{yt} [K_t / (1 + K_t)]^{0.5} \quad (19)$$

From equations (2), (17), (18) and then (14) the equivalent result to (19) is found for bottom tee:

$$V_{1b} = V_{yb} [K_b / (1 + K_b)]^{0.5} \quad (20)$$

The following result is used in Part 6: Spreadsheet: Table 12

$$V_1 = V_{1t} + V_{1b}, \text{ but less than } V_{yt} + V_{yb}$$

APPENDIX 11 (cont'd)

From equations (8) to (18), the maximum bending moment at the centreline of the web opening in terms of k_3 may be found:

$$\begin{aligned}
 M_1 &= 0.5\Sigma(P_i y_i) + 0.25H_o\Sigma(P_i), \text{ where } i = 1 \text{ to } 4 \\
 &= 0.5R_f[2(k_1-k_2)S_t + 2(k_3-k_4)S_b] + 0.25R_fH_o[2(k_1-k_2) + 2(k_3-k_4)] \\
 &= R_f[(2k_3 - 1 - \beta_b)S_t + (2k_3 - 1 - \beta_b)S_b + H_o(2k_3 - 1 - \beta_b)] \\
 &= R_f[2k_3(S_t + S_b + H_o) - \beta_b(S_t + S_b + H_o) - (S_t + S_b + H_o)] \\
 &= R_fD_b(2k_3 - \beta_b - 1), \text{ since } D_b \approx S_t + S_b + H_o \quad (21)
 \end{aligned}$$

$$= R_fD_b(1 - \beta_b) = R_fD_b[1 - (R_{wbr}/R_f)] \text{ if } k_3 = 1 \text{ (max)} \quad (22)$$

The maximum stress will be at the high moment end of the opening, and since the maximum value of k_1 or k_3 is unity it then follows from equation (12) that:

$$\begin{aligned}
 k_3 &= 1 \text{ if } \beta_b > \beta_t, \text{ ie. if } S_b > S_t \text{ and eccentricity } e \text{ is +ve (UP)} \\
 k_1 &= 1 \text{ if } \beta_b < \beta_t, \text{ ie. if } S_b < S_t \text{ and eccentricity } e \text{ is -ve (DOWN)}
 \end{aligned}$$

From equations (12) and (21) with $k_1 = 1$:

$$\begin{aligned}
 M_1 &= R_fD_b[2(1 - 0.5\beta_t + 0.5\beta_b) - \beta_b - 1] \\
 &= R_fD_b(1 - \beta_t) \text{ if } k_1 = 1 \text{ (max)} \\
 &= R_fD_b[1 - (R_{wtr}/R_f)] \quad (23)
 \end{aligned}$$

$$M_o = F_y S_{xo}, \text{ see Part 6: Spreadsheet: Table 10} \quad (24)$$

Equations (22),(23) and (24) are applied in Part 6: Spreadsheet: Table 13, and the resistance of the steel section is calculated in Table 14 from:

$$M_{rso} = M_1 + (M_o - M_1)[1 - (V_{u2}/V_1)^2]^{0.5}$$

APPENDIX 12

FORMULAE for REINFORCED WEB OPENINGS in the STEEL SECTION

The formulae are derived from a paper by Redwood [44], and are applied in Part 7: Spreadsheet: Tables 12 to 14.

The assumed stress distribution at failure for a high shear-to-moment ratio, which corresponds to point "1" on the interaction diagram, is shown in Fig 25. The force in the concrete, R_{ch} , is ignored in this analysis for the steel section. As can be seen from Fig 25:A, it is now assumed that normal stress in the web does not contribute to bending resistance (M_1). It is also assumed in the derivation of the formulae that T_f and T_r are small compared to S_t and S_b .

The important symbols are defined in Appendix 11.

Equilibrium above, below, and at the centreline of the opening gives the following relationships:

$$V_1 = V_{it} + V_{lb}$$

$$P_1 = P_2 = P_3 = P_4$$

$$L_o V_{1t} = P_1 y_1 - P_2 y_2 \text{ at top tee} \quad (1)$$

$$L_o V_{1b} = P_3 y_3 - P_4 y_4 \text{ at btm tee} \quad (2)$$

At the high moment end of web opening:

$$M_1 + 0.5V_1 L_o = P_1(0.5H_o + y_1) + P_3(0.5H_o + y_3) \quad (3)$$

At the low moment end of web opening:

$$M_1 - 0.5V_1 L_o = P_2(0.5H_o + y_2) + P_4(0.5H_o + y_4) \quad (4)$$

From the addition of (3) and (4):

$$M_1 = 0.5\sum(P_i y_i) + 0.25H_o \sum(P_i), \text{ where } i = 1 \text{ to } 4 \quad (5)$$

From Fig 25, the stress resultants at the four corners are:

$$P_1 = R_f + (1 - 2k_1)R_r \quad (6)$$

$$P_2 = (1 - 2k_2)R_f + R_r \quad (7)$$

$$P_3 = R_f + (1 - 2k_3)R_r \quad (8)$$

$$P_4 = (1 - 2k_4)R_f + R_r \quad (9)$$

From equations (6) to (9) and $P_1 = P_2 = P_3 = P_4$:

$$k_2 = k_1(R_r/R_f) \text{ from equations (6) and (7)} \quad (10)$$

$$k_4 = k_3(R_r/R_f) \text{ from equations (8) and (9)} \quad (11)$$

$$k_2 = k_4, \text{ and } k_1 = k_3 = 1 \text{ (max)} \quad (12)$$

APPENDIX 12 (cont'd)

From Fig 25:A and equations (1),(2) and then (10) to (12):

$$L_o V_{1t} = P_1 y_1 - P_2 y_2 = R_f S_t - (1 - 2k_2) R_f S_t = 2k_2 R_f S_t = 2k_1 R_r S_t \quad (13)$$

$$L_o V_{1b} = P_3 y_3 - P_4 y_4 = R_f S_b - (1 - 2k_4) R_f S_b = 2k_4 R_f S_b = 2k_3 R_r S_b \quad (14)$$

From addition of (13) and (14) and $k_1 = k_3 = 1$

$$V_1 = V_{1t} + V_{1b} = 2R_r(S_t + S_b)/L_o$$

and with $S_t + S_b \approx D_b - H_o$

$$V_1 = 2R_r(D_b - H_o)/L_o \text{ but less than } F_y T_w(D_b - H_o) = V_{yo} \quad (15)$$

Equation (15) is applied in Part 7: Spreadsheet: Table 12

From equations (5), the maximum bending moment at the centreline of the web opening is found with $k_1 = k_3$, and $k_2 = k_4 = R_r/R_f$:

$$M_1 = 0.5 \Sigma(P_i y_i) + 0.25 H_o \Sigma(P_i), \text{ where } i = 1 \text{ to } 4$$

$$\begin{aligned} \Sigma(P_i y_i) &= R_f S_t + (1 - 2k_2) R_f S_t + R_f S_b + (1 - 2k_4) R_f S_b \\ &= 2R_f(S_t + S_b) - 2k_2 R_f S_t - 2k_4 R_f S_b \\ &= 2R_f(S_t + S_b) - 2R_r(S_t + S_b) \end{aligned}$$

$$\Sigma(P_i) = 4(R_f - R_r)$$

$$\begin{aligned} M_1 &= 0.5[2(R_f - R_r)(S_t + S_b) + 0.25 H_o[4(R_f - R_r)]] \\ &= (R_f - R_r)(S_t + S_b + H_o) \\ &= D_b(R_f - R_r) \end{aligned} \quad (16)$$

$$M_o = F_y S_{xo}, \text{ see Part 7: Spreadsheet: Table 10} \quad (17)$$

Equations (16) and (17) are applied in Part 7: Spreadsheet: Table 13, and the resistance of the steel section is calculated in Table 14 from:

$$M_{rso} = M_1 + (M_o - M_1)[1 - (V_{u2}/V_1)^2]^{0.5}$$

APPENDIX 13

FORMULAE for UNREINFORCED WEB OPENINGS in the COMPOSITE SECTION

The formulae are derived from a paper by Redwood and Wong [40], and are applied in Part 6: Spreadsheet: Tables 16 to 19.

The assumed stress distribution at failure for a high shear-to-moment ratio, which corresponds to point "1" on the interaction diagram, is shown in Fig 24. The important symbols are defined in Appendix 11.

$$R_{ch} = N_h P_{sd}$$

$$T_{co} = T_c [R_{ch}/R_{c(min)}]$$

Equilibrium above, below, and at the centreline of the opening gives the following relationships:

$$V_1 = V_{1t} + V_{1b}$$

$$P_1 = P_2 = P_3 = P_4$$

$$L_o V_{1t} = P_1 y_1 - P_2 y_2 \text{ at top tee} \quad (1)$$

$$L_o V_{1b} = P_3 y_3 - P_4 y_4 \text{ at btm tee} \quad (2)$$

At the high moment end of web opening:

$$M_1 + 0.5V_1 L_o = P_1 (0.5H_o + y_1) + P_3 (0.5H_o + y_3) \quad (3)$$

At the low moment end of web opening:

$$M_1 - 0.5V_1 L_o = P_2 (0.5H_o + y_2) + P_4 (0.5H_o + y_4) \quad (4)$$

From the addition of (3) and (4):

$$M_1 = 0.5 \Sigma (P_i y_i) + 0.25 H_o \Sigma (P_i), \text{ where } i = 1 \text{ to } 4 \quad (5)$$

$$V_{yt} = \text{ultimate vertical shear capacity of top tee} = 0.6 F_y T_w S_t$$

$$V_{yb} = \text{ultimate vertical shear capacity of btm tee} = 0.6 F_y T_w S_b$$

$$R_f = \text{resistance of flange of steel section} = F_y T_f B_b$$

$$R_{wtr} = \text{resistance of web of top tee impaired by shear (Von Mises)}$$

$$= T_w S_t (F_{yr}) = T_w S_t (F_y^2 - 3F_s^2)^{0.5} = (T_w^2 S_t^2 F_y^2 - 3T_w^2 S_t^2 F_s^2)^{0.5}$$

$$= (3V_{yt}^2 - 3V_{1t}^2)^{0.5} \text{ and } \beta_t = (R_{wtr}/R_f) \quad (6)$$

$$R_{wbr} = \text{resistance of web of bottom tee impaired by shear (Von Mises)}$$

$$= T_w S_b (F_{yr}) = T_w S_b (F_y^2 - 3F_s^2)^{0.5} = (T_w^2 S_b^2 F_y^2 - 3T_w^2 S_b^2 F_s^2)^{0.5}$$

$$= (3V_{yb}^2 - 3V_{1b}^2)^{0.5} \text{ and } \beta_b = (R_{wbr}/R_f) \text{ also } \epsilon = (R_{ch}/R_f) \quad (7)$$

APPENDIX 13 (cont'd)

From Fig 24, the stress resultants at the four corners are:

$$P_1 = R_f(2k_1 - 1) - R_{wtr} + R_{ch} = R_f(2k_1 - 1 - \beta_t + \epsilon) \quad (8)$$

$$P_2 = R_f(1 - 2k_2) + R_{wtr} = R_f(1 - 2k_2 + \beta_t) \quad (9)$$

$$P_3 = R_f(2k_3 - 1) - R_{wbr} = R_f(2k_3 - 1 - \beta_b) \quad (10)$$

$$P_4 = R_f(1 - 2k_4) + R_{wbr} = R_f(1 - 2k_4 + \beta_b) \quad (11)$$

From equations (8) to (11) and $P_1 = P_2 = P_3 = P_4$:

$$k_1 = 0.5(\beta_t - \beta_b - \epsilon) + k_3 \quad (12)$$

$$k_2 = 0.5(\beta_t + \beta_b) + 1 - k_3 \quad (13)$$

$$k_4 = \beta_b + 1 - k_3 \quad (14)$$

From Fig 24, and assuming that S_t and S_b are much greater than T_f :

$$\begin{aligned} P_1 y_1 &= R_f S_t (2k_1 - 1) - 0.5 R_{wtr} S_t + R_{ch} (S_t + T_o - 0.5 T_{co}) \\ &= R_f [2k_1 S_t - S_t - 0.5 \beta_t S_t + \epsilon (S_t + T_o - 0.5 T_{co})] \end{aligned} \quad (15)$$

$$P_2 y_2 = R_f (S_t - 2k_2 S_t + 0.5 \beta_t S_t) \quad (16)$$

$$P_3 y_3 = R_f (2k_3 S_b - S_b - 0.5 \beta_b S_b) \quad (17)$$

$$P_4 y_4 = R_f (S_b - 2k_4 S_b + 0.5 \beta_b S_b) \quad (18)$$

The shear carried by the bottom tee is found from equations (2),(17), (18) and then (14):

$$\begin{aligned} L_o V_{1b} &= R_f S_b [2(k_3 + k_4) - 2 - \beta_b] = R_f S_b [2(\beta_b + 1) - 2 - \beta_b] \\ &= R_f S_b (\beta_b) = S_b (R_{wbr}) \end{aligned}$$

$$V_{1b} = (S_b/L_o) R_{wbr} = (S_b/L_o) (3V_{yb}^2 - 3V_{1b}^2)^{0.5}, \text{ from (7)}$$

Squaring both sides:

$$V_{1b}^2 = (S_b/L_o)^2 (3V_{yb}^2 - 3V_{1b}^2), \text{ and if } K_b = 3(S_b/L_o)^2$$

$$V_{1b}^2 = K_b (3V_{yb}^2 - 3V_{1b}^2)/3$$

$$V_{1b}^2 = K_b V_{yb}^2 / (1 + K_b)$$

$$V_{1b} = V_{yb} [K_b / (1 + K_b)]^{0.5}, \text{ same as result found in Appendix 12} \quad (19)$$

The shear carried by the top tee is found from equations (1),(15),(16) and then (12) and 13:

$$\begin{aligned} L_o V_{1t} &= R_f [2(k_1 + k_2) S_t - 2S_t - \beta_t + \epsilon (S_t + T_o - 0.5 T_{co})] \\ &= R_f [2(\beta_t + 1 - 0.5\epsilon) S_t - 2S_t - \beta_t S_t + \epsilon (S_t + T_o - 0.5 T_{co})] \\ &= R_f [\beta_t S_t + \epsilon (T_o - 0.5 T_{co})] \\ &= S_t R_{wtr} + R_{ch} (T_o - 0.5 T_{co}) \end{aligned}$$

APPENDIX 13 (cont'd).

$$\begin{aligned}
 L_o V_{1t} &= S_t (3V_{yt}^2 - 3V_{1t}^2)^{0.5} + R_{ch}(T_o - 0.5T_{co}) \\
 &= S_t V_{yt} [3 - 3(V_{1t}/V_{yt})^2]^{0.5} + R_{ch}(T_o - 0.5T_{co}) \\
 (L_o V_{1t}/S_t V_{yt}) &= [3 - 3(V_{1t}/V_{yt})^2]^{0.5} + [R_{ch}(T_o - 0.5T_{co})/S_t V_{yt}]
 \end{aligned}$$

Now if $A_1 = L_o/S_t$, and $B_1 = R_{ch}(T_o - 0.5T_{co})/S_t V_{yt}$

$$\begin{aligned}
 A_1 (V_{1t}/V_{yt}) &= [3 - 3(V_{1t}/V_{yt})^2]^{0.5} + B_1 \\
 [A_1 (V_{1t}/V_{yt}) - B_1]^2 &= 3 - 3(V_{1t}/V_{yt})^2 \\
 A_1^2 (V_{1t}/V_{yt})^2 - 2A_1 B_1 (V_{1t}/V_{yt}) + B_1^2 - 3 + 3(V_{1t}/V_{yt})^2 &= 0 \\
 (3 + A_1^2) (V_{1t}/V_{yt})^2 - 2A_1 B_1 (V_{1t}/V_{yt}) + B_1^2 - 3 &= 0 \\
 (V_{1t}/V_{yt}) &= \{2A_1 B_1 + [4A_1^2 B_1^2 - 4(3 + A_1^2)(B_1^2 - 3)]^{0.5}\} / 2(3 + A_1^2) \\
 V_{1t} &= V_{yt} [A_1 B_1 + (3A_1^2 - 3B_1^2 + 9)^{0.5}] / (3 + A_1^2) \quad (20) \\
 V_1 &= V_{1t} + V_{1b}, \text{ but less than } V_{yt} + V_{yb} \quad (21)
 \end{aligned}$$

Equations (19), (20), (21) are applied in Part 6: Spreadsheet: Table 18, and V_{1b} has the value used for the steel section in Table 12.

From equations (8) to (18), the maximum bending moment at the centreline of the web opening in terms of k_3 may be found:

$$\begin{aligned}
 M_1 &= 0.5 \sum (P_i y_i) + 0.25 H_o \sum (P_i), \text{ where } i = 1 \text{ to } 4 \\
 0.5 \sum (P_i y_i) &= 0.5 R_f [2(k_1 - k_2)S_t + 2(k_3 - k_4)S_b + \epsilon(S_t + T_o - 0.5T_{co})] \\
 &= 0.5 R_f [(4k_3 - 2 - 2\beta_b - \epsilon)S_t + (4k_3 - 2 - 2\beta_b)S_b + \epsilon(\text{Term})] \\
 &= R_f [(2k_3 - 1 - \beta_b)S_t + (2k_3 - 1 - \beta_b)S_b + 0.5\epsilon(T_o - .5T_{co})]
 \end{aligned}$$

APPENDIX 13 (cont'd)

$$\begin{aligned}
 0.25H_o\Sigma(P_i) &= 0.25H_oR_f(2k_1 - 2k_2 + 2k_3 - 2k_4 + \epsilon) \\
 &= 0.25H_oR_f(4k_3 - 2\beta_b - 2 - \epsilon + 4k_3 - 2\beta_b - 2 + \epsilon). \\
 &= H_oR_f(2k_3 - \beta_b - 1) \\
 M_1 &= R_f(2k_3 - \beta_b - 1)(S_t + S_b + H_o) + 0.5R_{ch}(T_o - 0.5T_{co}) \\
 &= R_fD_b(2k_3 - \beta_b - 1) + 0.5R_{ch}(T_o - T_{co}) \\
 &= R_fD_b[1 - (R_{wbr}/R_f)] + 0.5R_{ch}(T_o - T_{co}), \text{ if } k_3 = 1 \quad (22)
 \end{aligned}$$

M_o is calculated from first principles in Part 6: Spreadsheet: Tables 16 and 17. Equation (22) is applied in Table 18, and the resistance of the composite section is calculated in Table 19 from:

$$M_{rco} = M_1 + (M_o - M_1)[1 - (V_{u3}/V_1)^2]^{0.5}$$

APPENDIX 14

FORMULAE for REINFORCED WEB OPENINGS in the COMPOSITE SECTION

The formulae are derived by the writer by the extrapolation of Redwood's method [44] for the steel section to the composite section, and are applied in Part 7: Spreadsheet: Tables 16 to 19.

The assumed stress distribution at failure for a high shear-to-moment ratio, which corresponds to point "1" on the interaction diagram, is shown in Fig 25. As can be seen from Fig 25:A, it is assumed that the normal stress in the web does not contribute to the bending resistance (M_1). It is also assumed in the derivation of the formulae that T_f and T_r are small compared to S_t and S_b .

The important symbols are defined in Appendix 11.

$$\begin{aligned} R_{ch} &= N_h P_{sd} \\ T_{co} &= T_c [R_{ch}/R_{c(min)}] \\ R_r &= F_y A_r \end{aligned}$$

Equilibrium above, below, and at the centreline of the opening gives the following relationships:

$$V_1 = V_{it} + V_{1b}$$

$$P_1 = P_2 = P_3 = P_4$$

$$L_o V_{1t} = P_1 y_1 - P_2 y_2 \text{ at top tee} \quad (1)$$

$$L_o V_{1b} = P_3 y_3 - P_4 y_4 \text{ at btm tee} \quad (2)$$

At the high moment end of web opening:

$$M_1 + 0.5V_1 L_o = P_1(0.5H_o + y_1) + P_3(0.5H_o + y_3) \quad (3)$$

At the low moment end of web opening:

$$M_1 - 0.5V_1 L_o = P_2(0.5H_o + y_2) + P_4(0.5H_o + y_4) \quad (4)$$

From the addition of (3) and (4):

$$M_1 = 0.5\Sigma(P_i y_i) + 0.25H_o \Sigma(P_i), \text{ where } i = 1 \text{ to } 4 \quad (5)$$

From Fig 25, the stress resultants at the four corners are:

$$P_1 = R_f + (1 - 2k_1)R_r + R_{ch} \quad (6)$$

$$P_2 = (1 - 2k_2)R_f + R_r \quad (7)$$

$$P_3 = R_f + (1 - 2k_3)R_r \quad (8)$$

$$P_4 = (1 - 2k_4)R_f + R_r \text{ and } \beta_r = R_r/R_f, \text{ and also } \epsilon = R_{ch}/R_f \quad (9)$$

APPENDIX 14 (cont'd)

From equations (6) to (9) and $P_1 = P_2 = P_3 = P_4$:

$$k_3 = k_1 - 0.5\epsilon/\beta_r \quad \text{from equations (6) and (8)} \quad (10)$$

$$k_2 = k_1\beta_r - 0.5\epsilon \quad \text{from equations (6) and (7)} \quad (11)$$

$$k_4 = k_1\beta_r - 0.5\epsilon \quad \text{from equations (6) and (9) and } k_2 = k_4 \quad (12)$$

From Fig 25:A and equations (1),(2) and then (10) to (12):

$$\begin{aligned} L_o V_{1t} &= P_1 y_1 - P_2 y_2 \\ &= R_f S_t - (1 - 2k_2)R_f S_t + R_{ch}(S_t + T_o - 0.5T_{co}) \\ &= R_f[2k_2 S_t + \epsilon(S_t + T_o - 0.5T_{co})] \\ &= R_f[2(k_1\beta_r - 0.5\epsilon)S_t + \epsilon(S_t + T_o - 0.5T_{co})] \\ &= R_f[2\beta_r S_t + \epsilon(T_o - 0.5T_{co})] \quad \text{with } k_1 = 1 \text{ (max)} \end{aligned}$$

$$V_{it} = 2R_r(S_t/L_o) + R_{ch}(T_o - 0.5T_{co})/L_o \quad (13)$$

$$\begin{aligned} L_o V_{1b} &= P_3 y_3 - P_4 y_4 = R_f S_b - (1 - 2k_4)R_f S_b = 2k_4 R_f S_b \\ &= 2R_f(k_1\beta_r - 0.5\epsilon)S_b \end{aligned}$$

$$V_{1b} = 2R_r(S_b/L_o) - R_{ch}(S_b/L_o) \quad \text{with } k_1 = 1 \text{ (max)} \quad (14)$$

From addition of (13) and (14):

$$V_1 = V_{1t} + V_{1b} = 2R_r(S_t + S_b)/L_o + R_{ch}(T_o - 0.5T_{co} - S_b)/L_o$$

and with $S_t + S_b \approx D_b - H_o$

$$V_1 = 2R_r(D_b - H_o)/L_o + R_{ch}(T_o - 0.5T_{co} - S_b)/L_o \quad (15)$$

Equation (15) is applied in Part 7: Spreadsheet: Table 18

APPENDIX 14 (cont'd)

The maximum bending moment at the centreline of the web opening is found with equations (15) and then (6) to (12):

$$M_1 = 0.5\Sigma(P_i y_i) + 0.25H_o\Sigma(P_i), \text{ where } i = 1 \text{ to } 4$$

$$\begin{aligned} 0.5\Sigma(P_i y_i) &= 0.5R_f[S_t + (1 - 2k_2)S_t + S_b + (1 - 2k_4)S_b + \epsilon(S_t + T_o - 0.5T_{co})] \\ &= 0.5R_f[(2S_t + 2S_b) - 2k_2S_t - 2k_4S_b + \epsilon(S_t + T_o - 0.5T_{co})] \\ &= 0.5R_f[(2S_t + 2S_b)(1 - k_2) + \epsilon(S_t + T_o - 0.5T_{co})] \\ &= 0.5R_f[(2S_t + 2S_b)(1 - \beta_r + 0.5\epsilon) + \epsilon(S_t + T_o - 0.5T_{co})] \\ &= R_f(S_t + S_b) - R_r(S_t + S_b) + R_{ch}(S_t + .5S_b + .5T_o - .25T_{co}) \end{aligned}$$

$$\begin{aligned} .25H_o\Sigma(P_i) &= .25H_o[4R_f + 2R_r - 4k_2R_f - 2k_3R_r + R_{ch}] \\ &= .25H_o[4R_f + 2R_r - 4(\beta_r - .5\epsilon)R_f - 2(1 - .5\epsilon/\beta_r)R_r + R_{ch}] \\ &= H_o(R_f - R_r) + H_oR_{ch} \end{aligned}$$

$$\begin{aligned} M_1 &= (R_f - R_r)(S_t + S_b + H_o) + R_{ch}(H_o + S_t + .5S_b + .5T_o - .25T_{co}) \\ &= (R_f - R_r)D_b + R_{ch}(D_b - 0.5S_b) + 0.5R_{ch}(T_o - 0.5T_{co}) \end{aligned} \quad (16)$$

M_o is calculated from first principles in the Part 7: Spreadsheet: Table 16. Equation (16) is applied in Table 18, and the resistance of the composite section is calculated in Table 19 from:

$$M_{rco} = M_1 + (M_o - M_1)[1 - (V_{u3}/V_1)^2]^{0.5}$$

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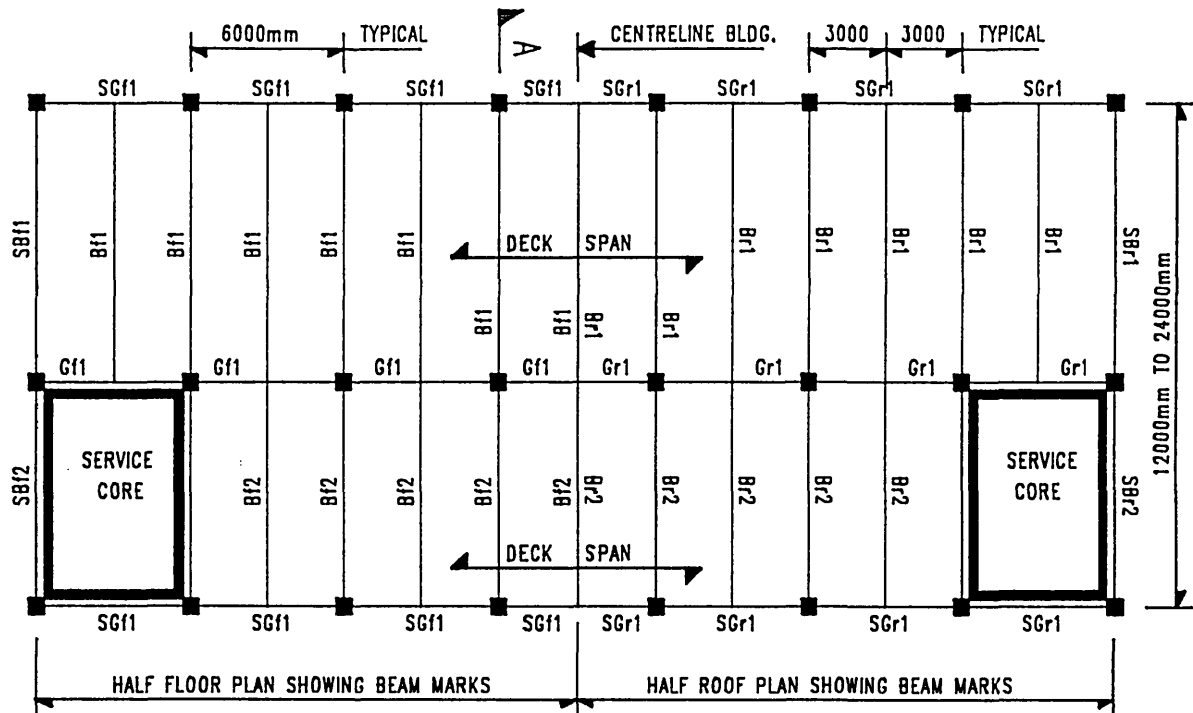
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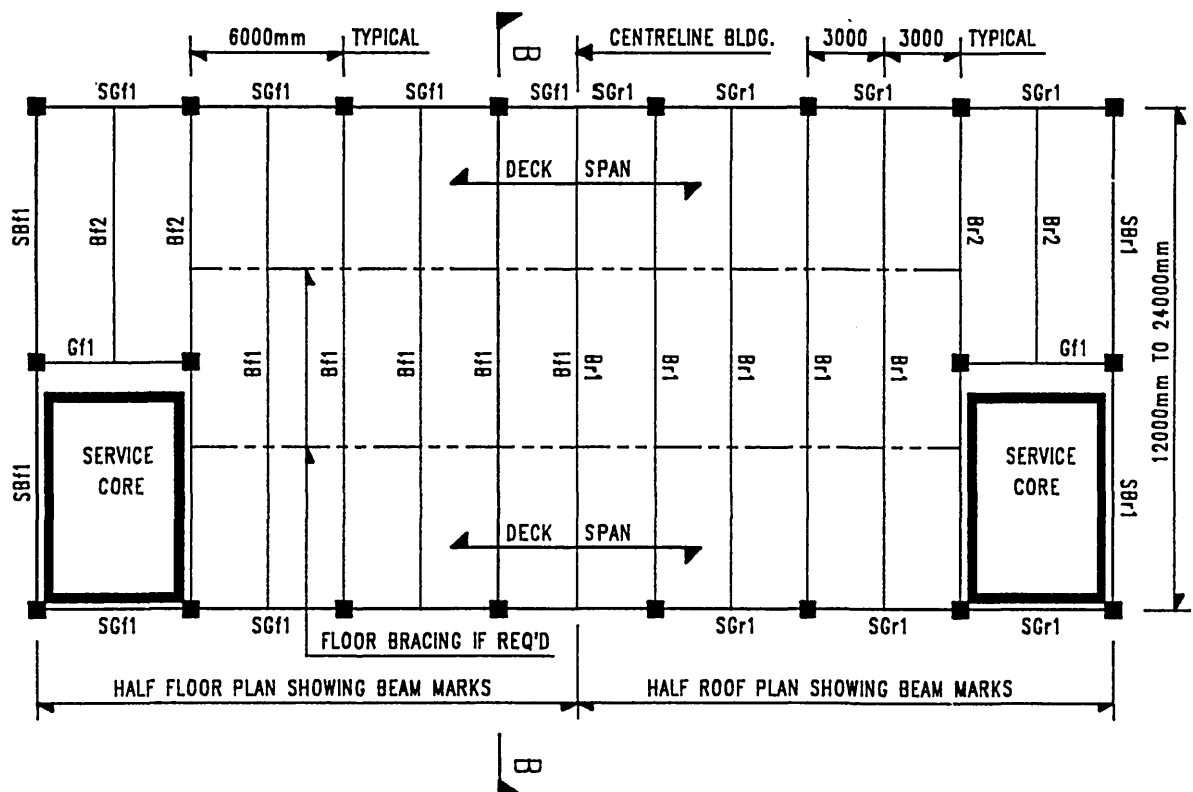
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FIG.1

PLAN A: A CONVENTIONAL FLOOR PLAN WITH OFF-CENTRE INTERNAL COLUMNS AND SHORT SPAN TYPE 1 PRIMARY BEAMS.



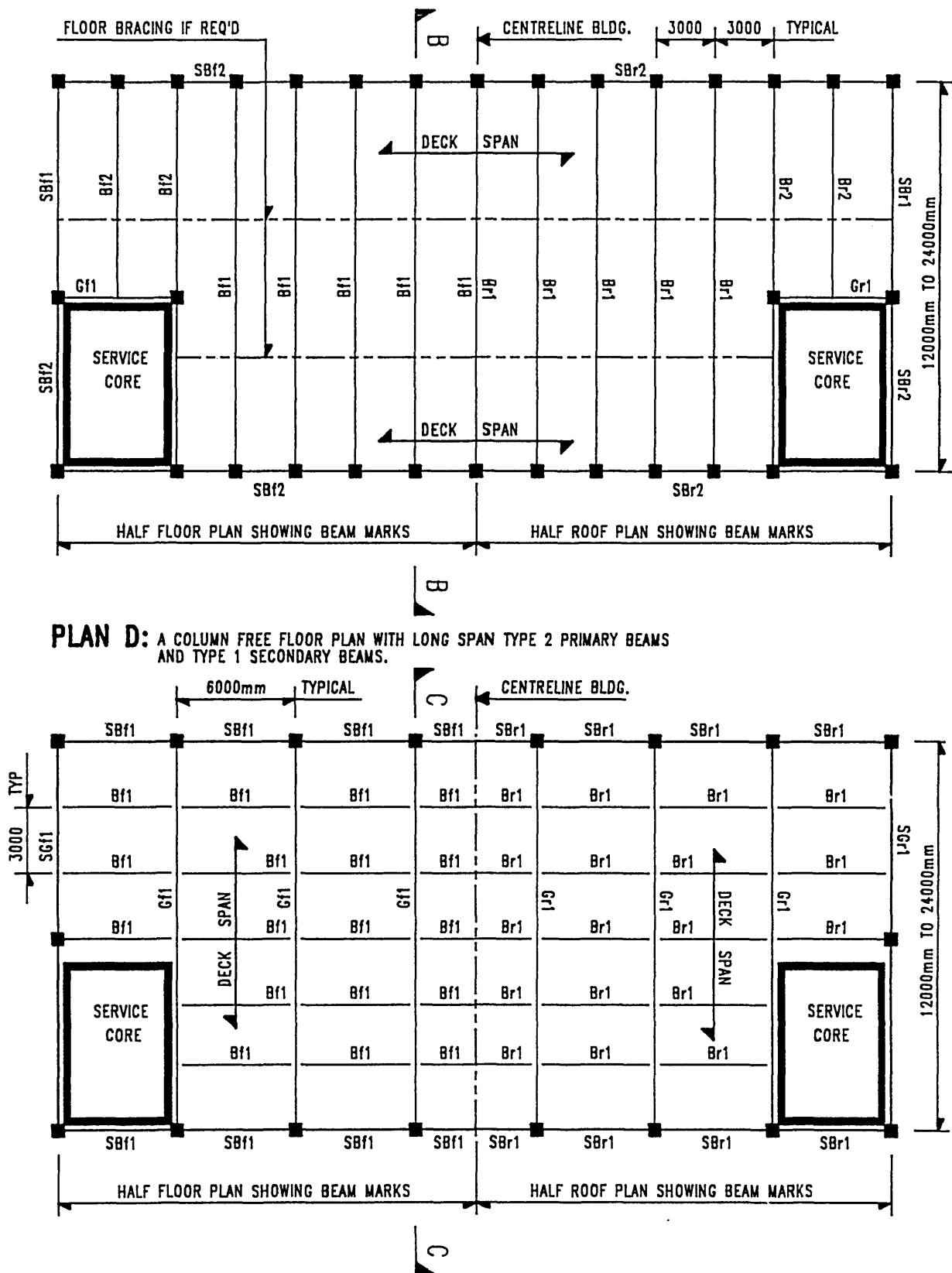
PLAN B: COLUMN FREE FLOOR PLAN WITH LONG SPAN TYPE 1 PRIMARY BEAMS.



FLOOR PLANS C & D.

FIG.2

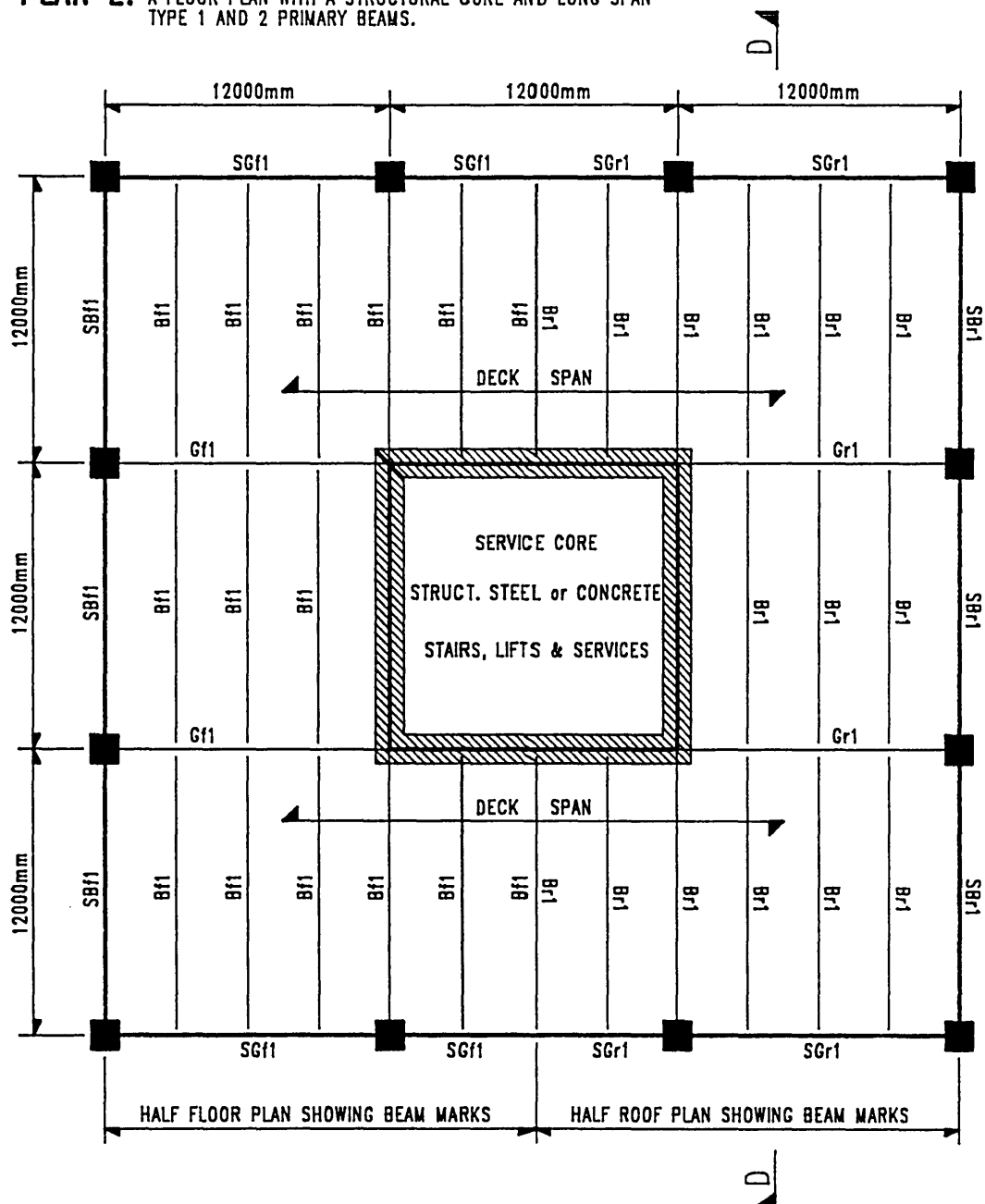
PLAN C: A COLUMN FREE FLOOR PLAN WITH LONG SPAN TYPE 1 BEAMS AND PERIMETER COLUMNS AT 3000mm CENTRES.



FLOOR PLAN E.

FIG.3

PLAN E: A FLOOR PLAN WITH A STRUCTURAL CORE AND LONG SPAN TYPE 1 AND 2 PRIMARY BEAMS.



KEY TO BEAM MARK SYSTEM:

- TYPE 1 BEAM: A BEAM SUPPORTING UNIFORMLY DISTRIBUTED LOADING AND WITH THE DECK PROFILE RUNNING TRANSVERSELY.

TYPE 2 BEAM: A BEAM WITH INTERCONNECTED SECONDARY MEMBERS AND WITH THE DECK PROFILE RUNNING PARALLEL.
- Bf1: AN INTERNAL TYPE 1 FLOOR BEAM.

Bbr1: AN INTERNAL TYPE 1 ROOF BEAM.

Gf1: AN INTERNAL TYPE 2 FLOOR BEAM.

Gr1: AN INTERNAL TYPE 2 ROOF BEAM.
- SBr1: AN EXTERNAL (SPANDREL) TYPE 1 FLOOR BEAM.

SBr1: AN EXTERNAL (SPANDREL) TYPE 1 ROOF BEAM.

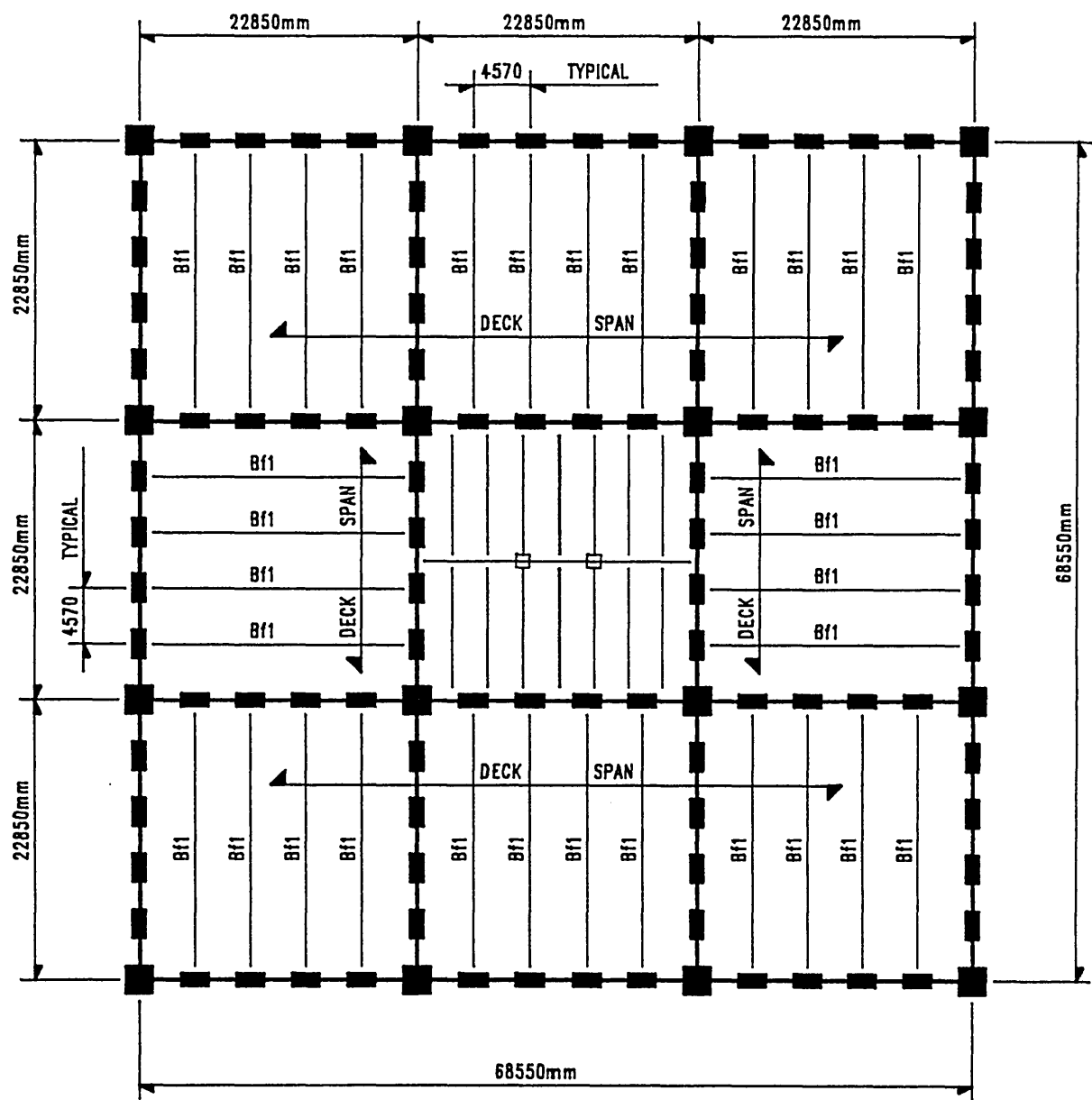
SGf1: AN EXTERNAL (SPANDREL) TYPE 2 FLOOR BEAM.

SGr1: AN EXTERNAL (SPANDREL) TYPE 2 ROOF BEAM.

FLOOR PLAN F.

FIG.4

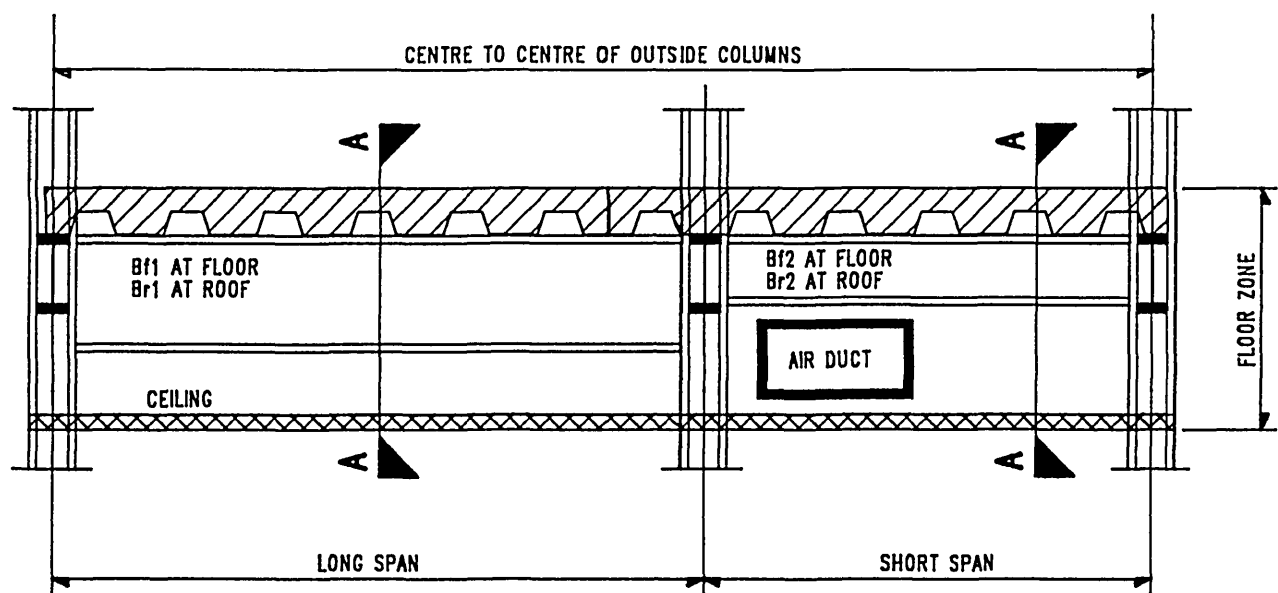
PLAN F: THE FLOOR PLAN OF THE SEARS TOWER, CHICAGO, ILLINOIS.



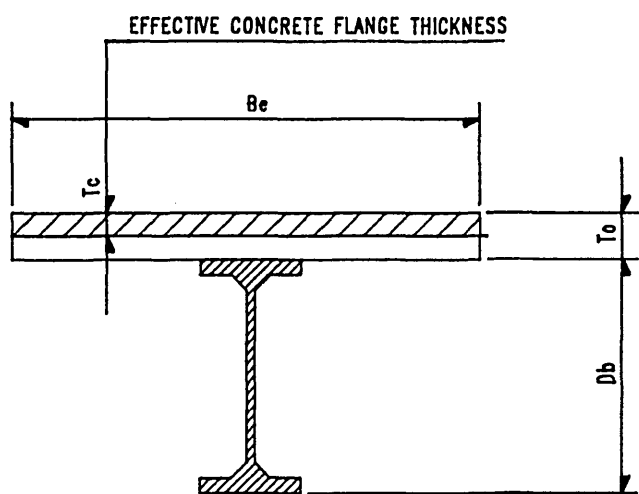
- NOTES:
- 1) ALL FLOOR MEMBERS AT THE EIGHT EXTERNAL CELLS ARE TYPE 1 TRUSSES (MARK Bf1).
 - 2) THE TRUSSES HAVE A WARREN TYPE WEB-FRAMING CONFIGURATION AND SPAN 22850mm.
 - 3) THE TRUSSES HAVE A DEPTH OF 1016mm AND A SPAN/DEPTH RATIO OF 22.5.
 - 4) THE COMPOSITE DECK IS 140mm THICK, SPANS 4570mm, AND USES A 75mm TRAPEZOIDAL BLENDED METAL DECK PROFILE.
 - 5) THE LATERAL STIFFNESS OF THE SEARS TOWER IS DERIVED FROM BOTH THE EXTERIOR AND INTERIOR COLUMNS. THIS FRAMING SYSTEM IS CALLED "BUNDLED TUBE".

CROSS-SECTION at FLOOR PLAN A (FIG.1) using
UNIVERSAL SECTIONS WITHOUT WEB OPENINGS (PART 2)

FIG.5

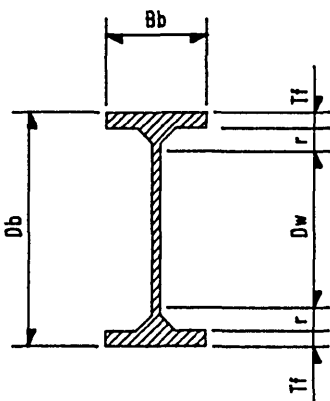


SECTION "A - A" FLOOR PLAN A (FIG.1)



SECTION "A - A"

COMPOSITE SECTION

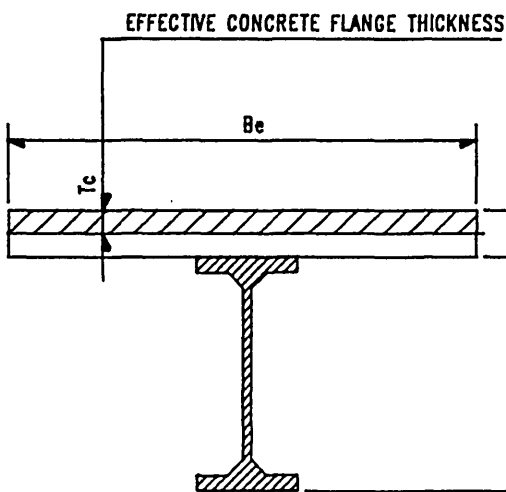
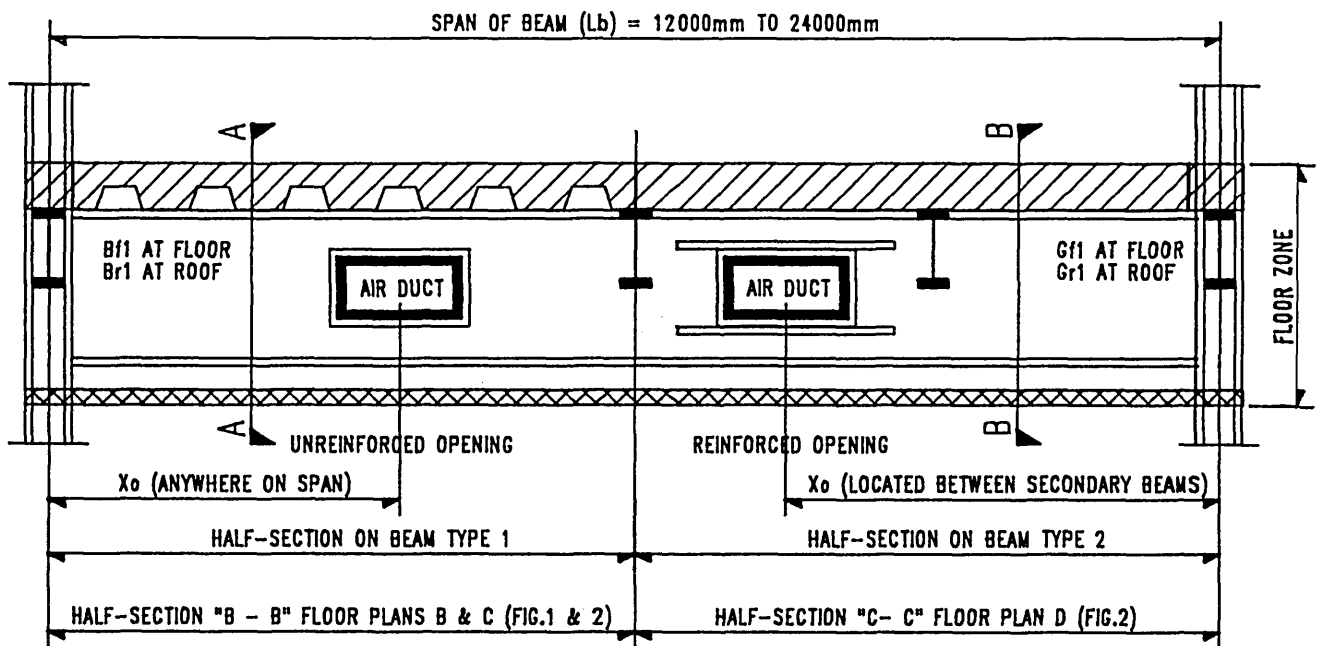


STEEL SECTION

BEAM TYPE 1.

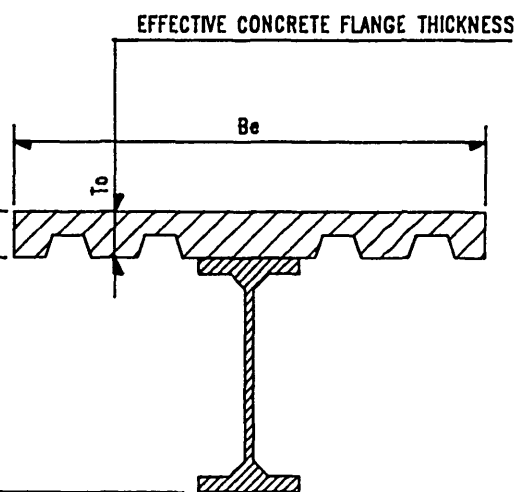
**CROSS-SECTION at FLOOR PLANS B, C & D (FIG.1 & 2) using
UNIVERSAL SECTIONS WITH WEB OPENINGS (PARTS 6 & 7)**

FIG.6



SECTION "A - A"
COMPOSITE SECTION

BEAM TYPE 1.



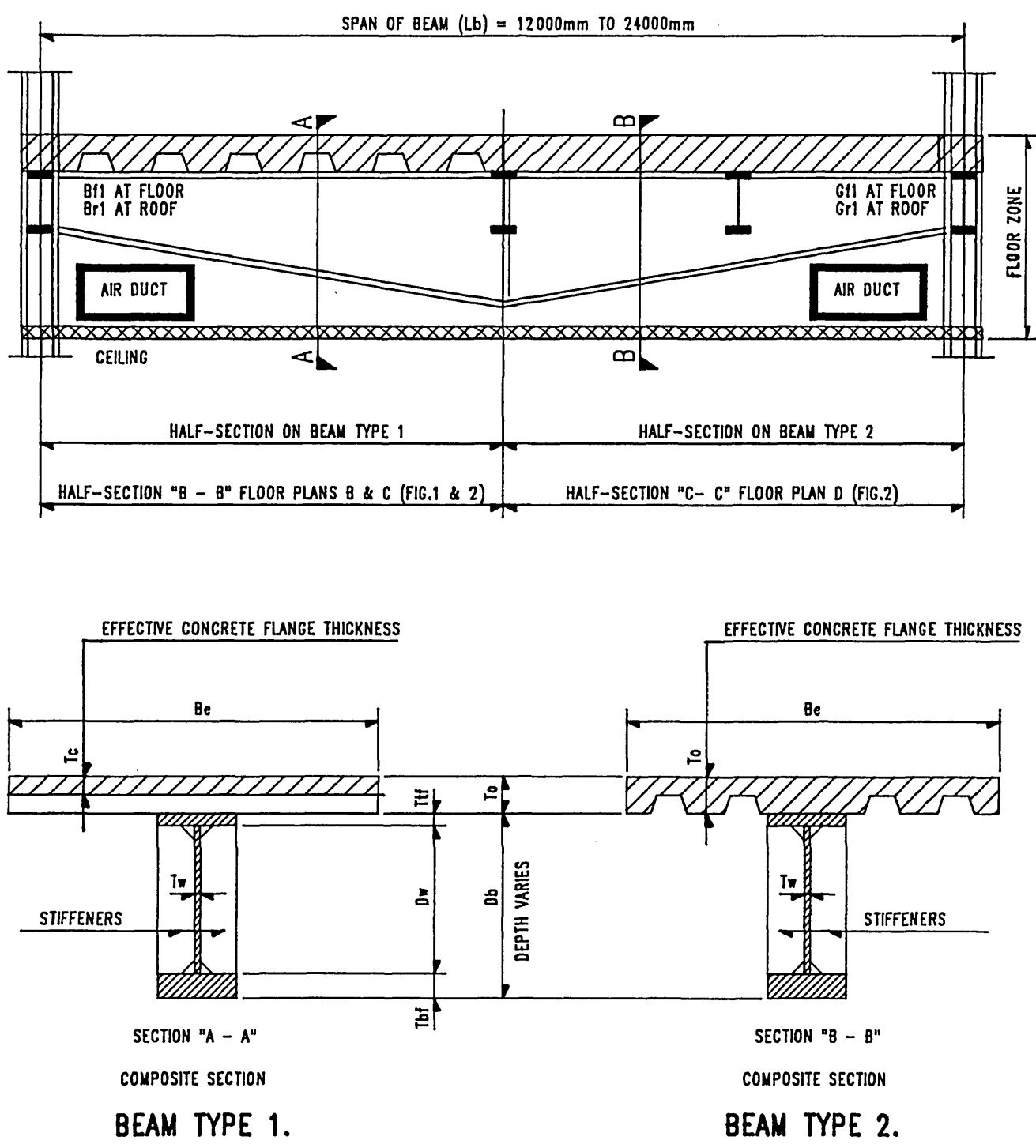
SECTION "B - B"
COMPOSITE SECTION

BEAM TYPE 2.

NOTE: THE MANUAL AND COMPUTER DESIGN PROCEDURES FOR REINFORCED AND UNREINFORCED WEB OPENINGS ARE GIVEN IN PART 6 & 7: DESIGNSTUDIES ON UNIVERSAL ROLLED SECTIONS with WEB OPENINGS UB(U) and UB(R)

CROSS-SECTION at FLOOR PLANS B, C & D (FIG.1 & 2) using
 LIGHT PLATE GIRDERS WITH A CONSTANT TAPER (LPG(T)) - (PART 4)

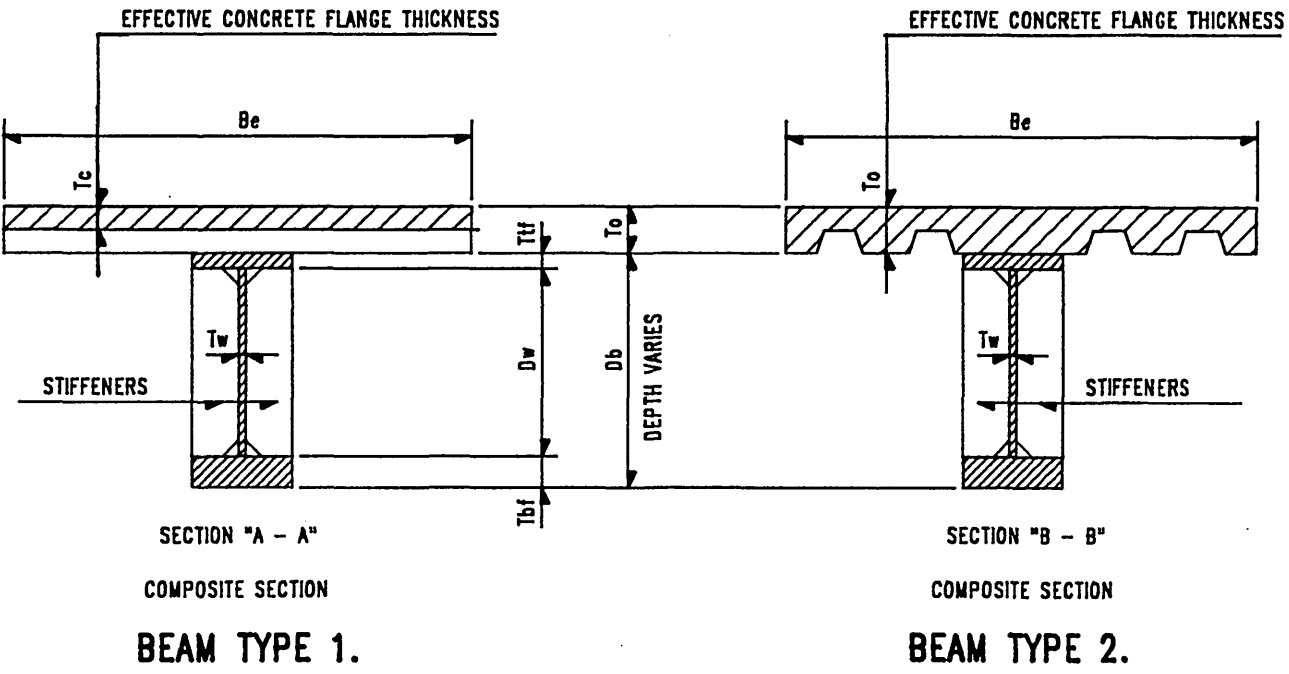
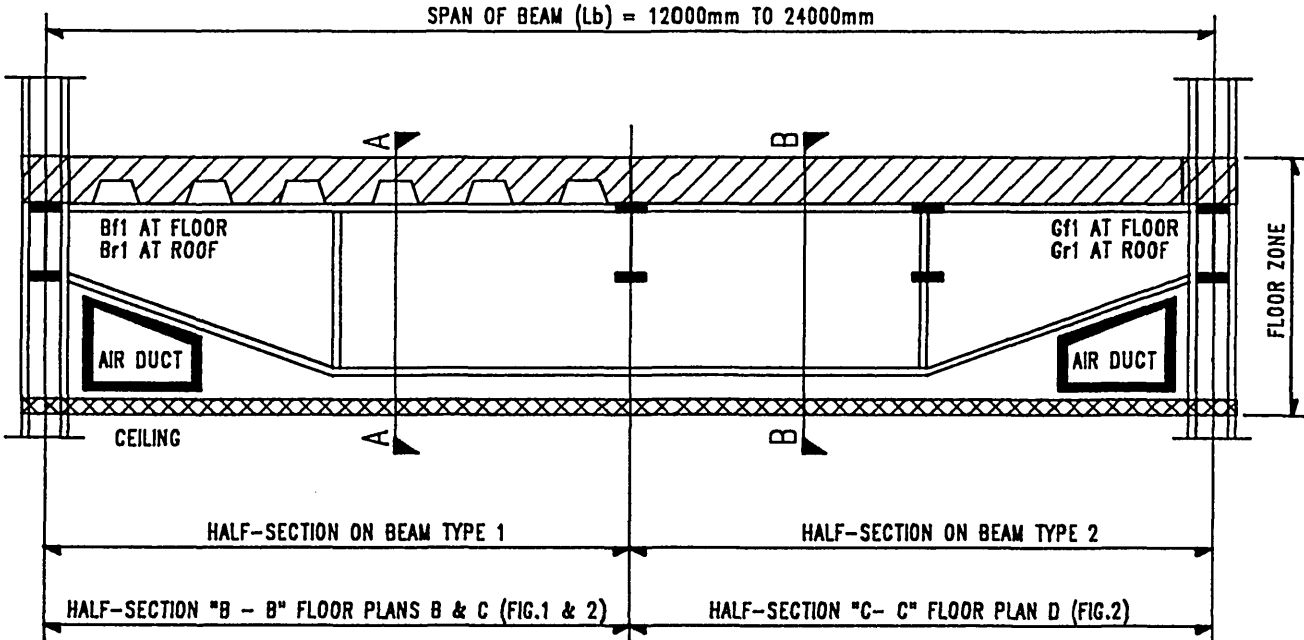
FIG.7



NOTE: 1) THE MANUAL AND COMPUTER DESIGN PROCEDURES FOR THESE TAPERED BEAMS ARE GIVEN IN
 PART 4: DESIGN STUDIES ON LIGHT PLATE GIRDERS WITH A CONSTANT TAPER (LPG(T)).
 2) VERTICAL STIFFENERS AT CENTRELINE ONLY.

**CROSS-SECTION at FLOOR PLANS B, C & D (FIG.1 & 2) using
LIGHT PLATE GIRDERS WITH A MODIFIED TAPER (LPG(M)) – (PART 5)**

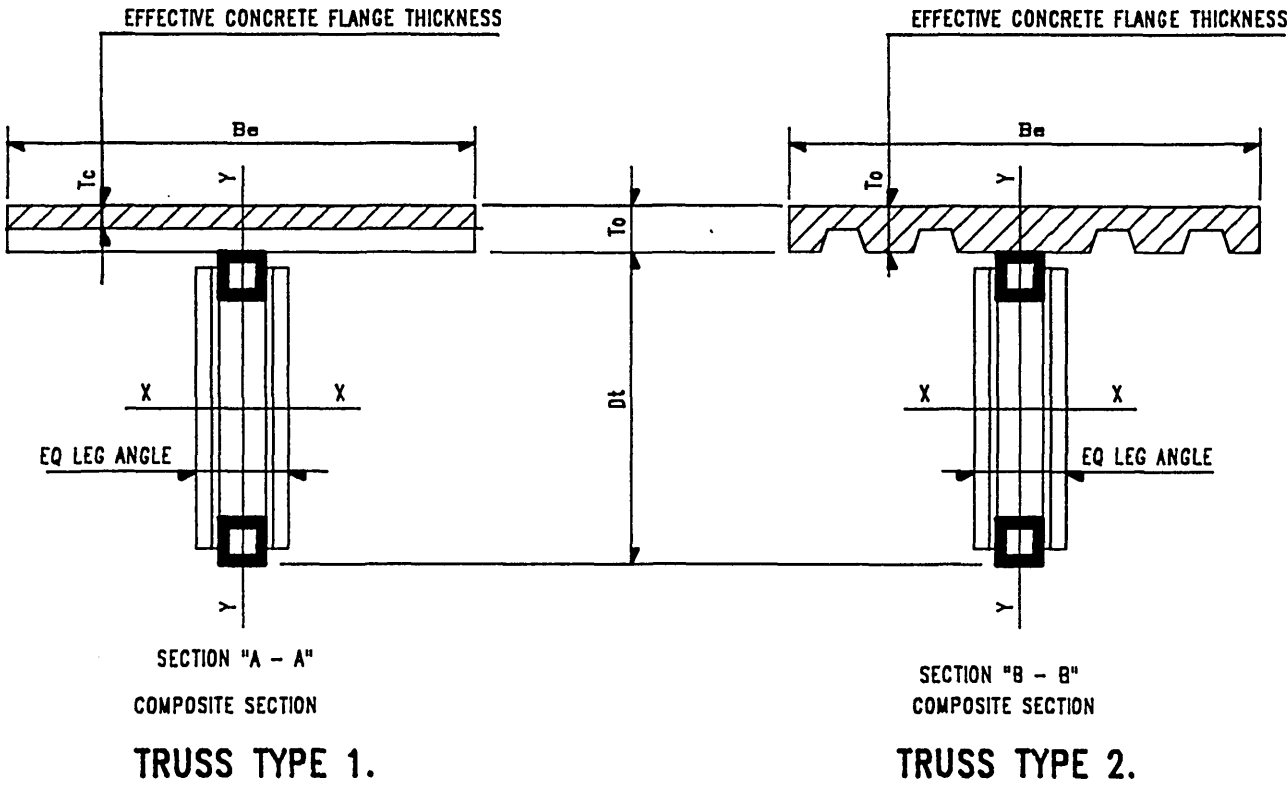
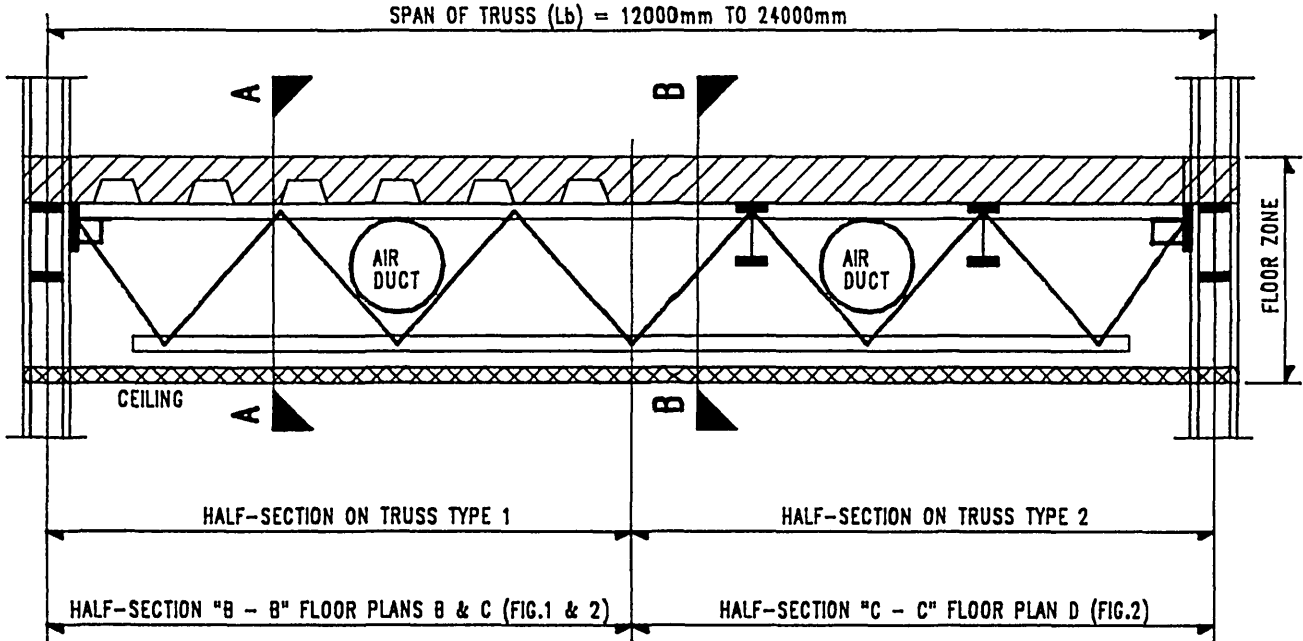
FIG.8



- NOTE: 1) THE MANUAL AND COMPUTER DESIGN PROCEDURES FOR THESE TAPERED BEAMS ARE GIVEN IN
PART 5: DESIGN STUDIES ON LIGHT PLATE GIRDERS WITH A MODIFIED TAPER (LPG(M)).
- 2) VERTICAL STIFFENERS AT END OF TAPER ONLY.

CROSS-SECTION at FLOOR PLANS B,C & D (FIG.1 & 2) using WARREN TRUSSES with H.S.S. CHORDS (TRUSS(H)) - (PART 8)

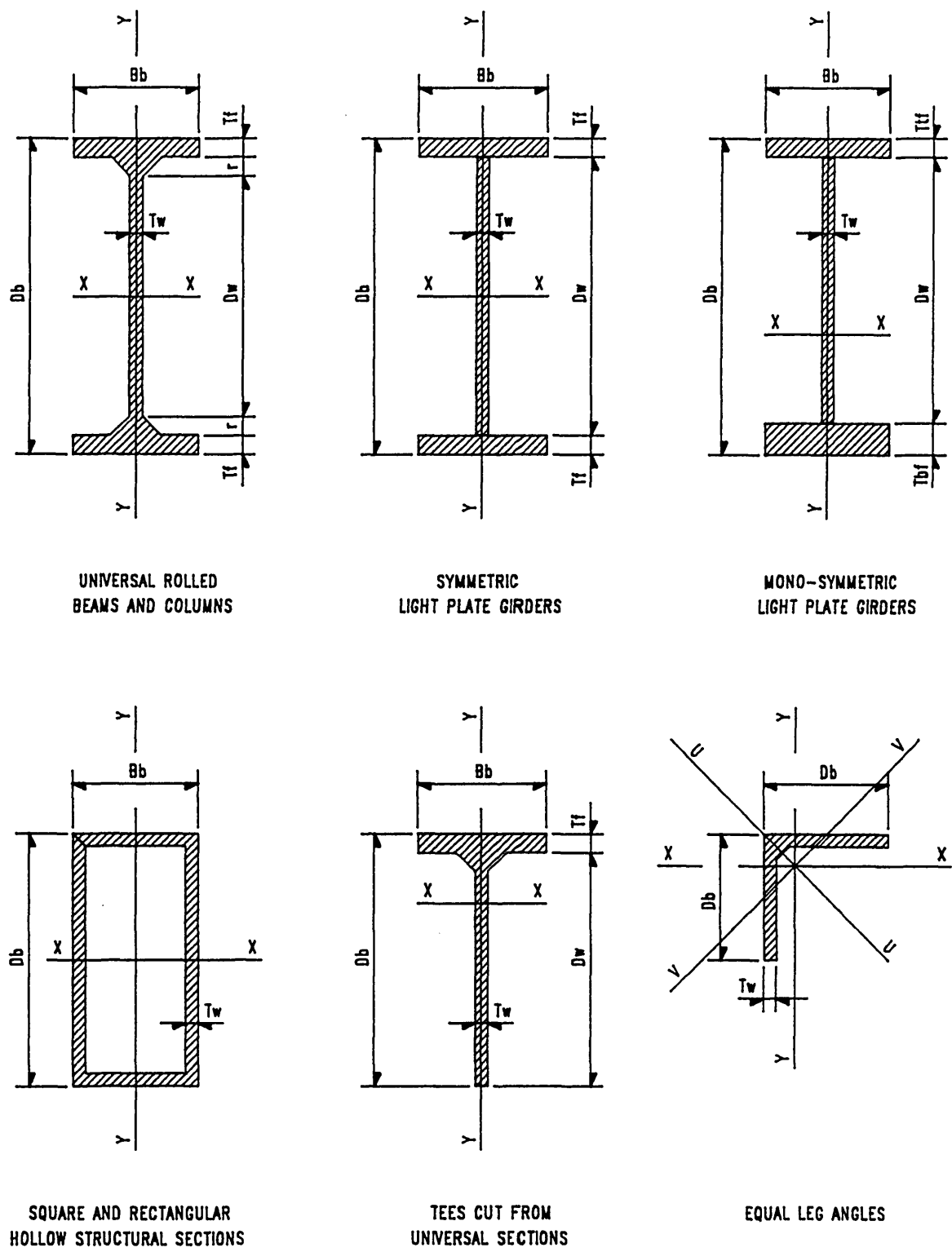
Fig.9



NOTE: THE MANUAL AND COMPUTER DESIGN PROCEDURES FOR WARREN TRUSSES ARE GIVEN IN
PART 8: DESIGN STUDIES ON WARREN TRUSSES WITH HSS CHORDS (TRUSS(H)).

DIMENSIONS of STEEL SECTIONS.

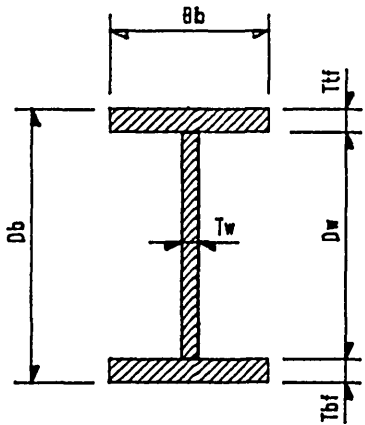
FIG.10



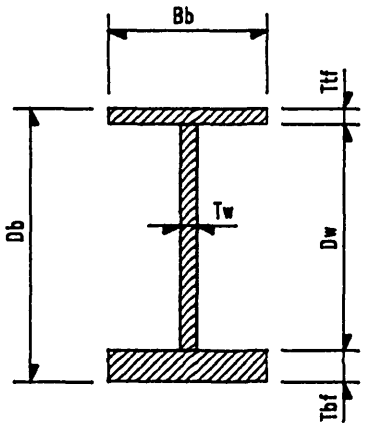
NOTE: THE DEFINITIONS OF SYMBOLS GIVEN IN THIS FIGURE ARE USED IN ALL THE DESIGN STUDIES.

**LIGHT PLATE GIRDERS
BEAM SYMMETRY TYPES 1, 2 & 3.**

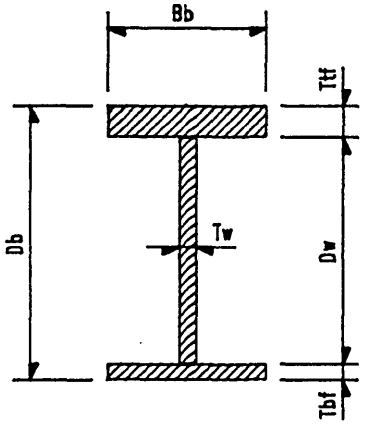
FIG.11



TYPE 1: SYMMETRIC SECTION, $T_{tf}=T_{bf}$ and Bb constant.



TYPE 2: MONOSYMMETRIC SECTION, $T_{tf}<T_{bf}$ and Bb constant.

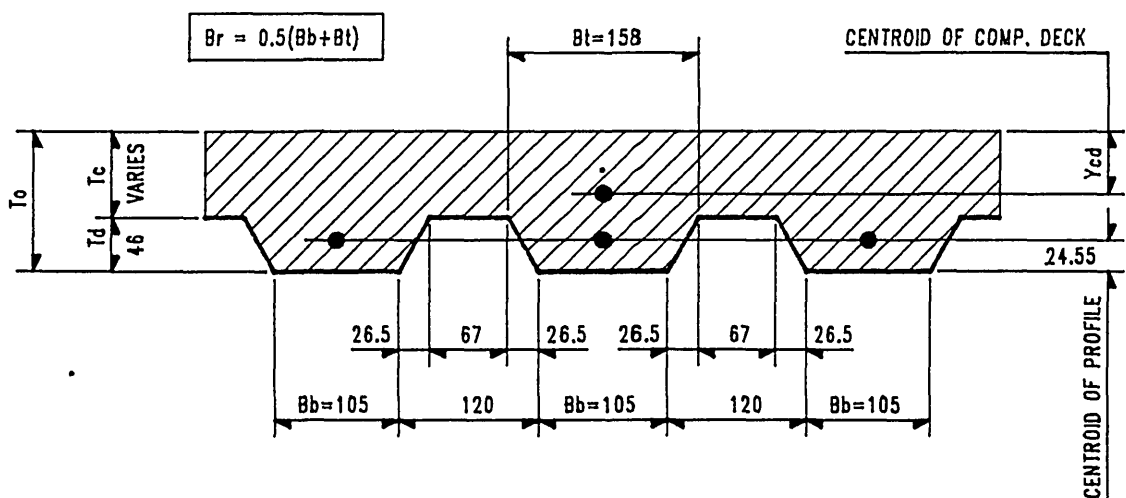


TYPE 3: MONOSYMMETRIC SECTION, $T_{tf}>T_{bf}$ and Bb constant.

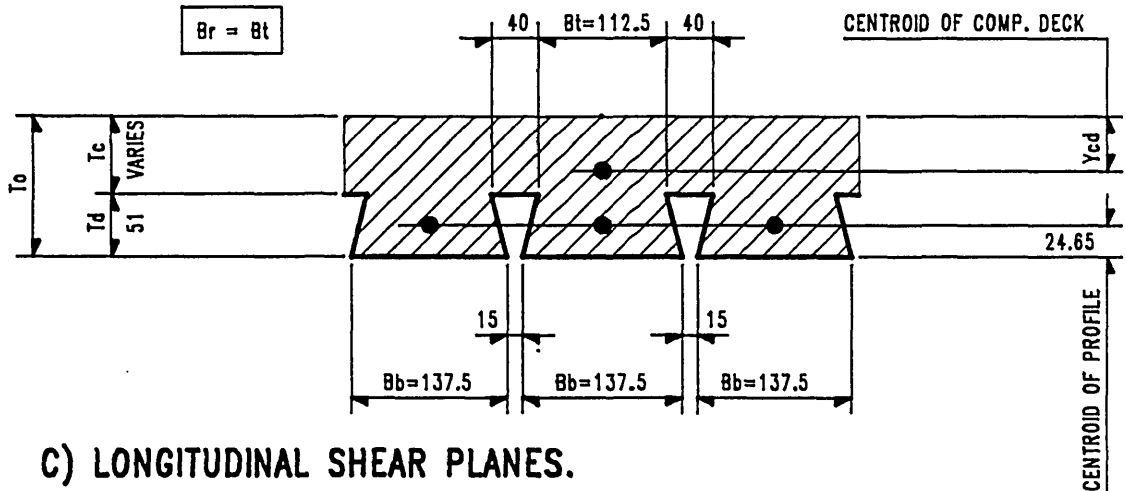
COMPOSITE SLAB with PROFILED DECKS TYPE 1 & 2
and LONGITUDINAL SHEAR PLANES.

FIG.12

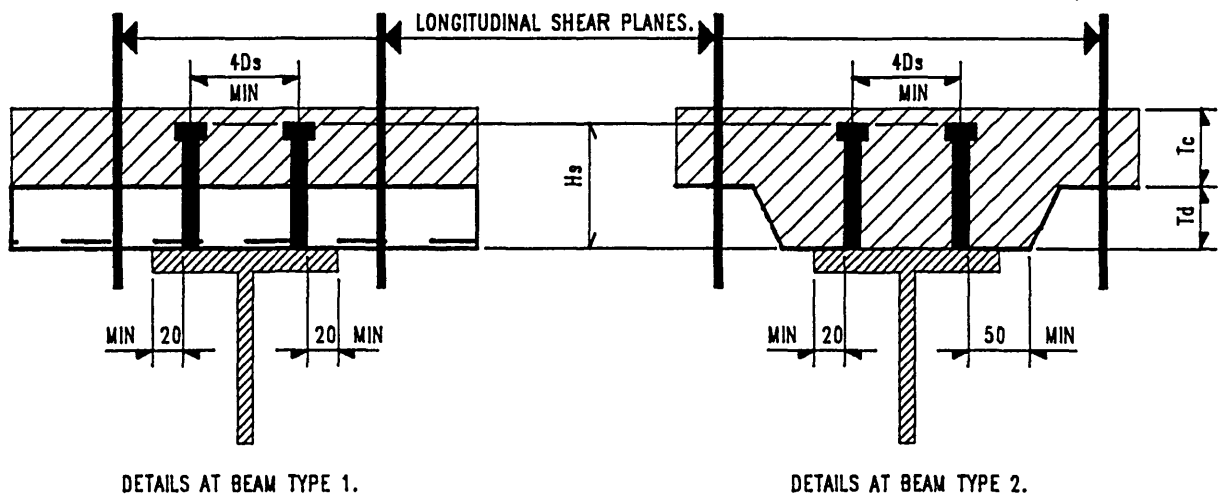
A) DECK TYPE 1: CF46 PROFILED STEEL DECK BY PRECISION METAL FORMING LTD.



B) DECK TYPE 2: SUPER HOLORIB PROFILED STEEL DECK BY RICHARD LEES.



C) LONGITUDINAL SHEAR PLANES.

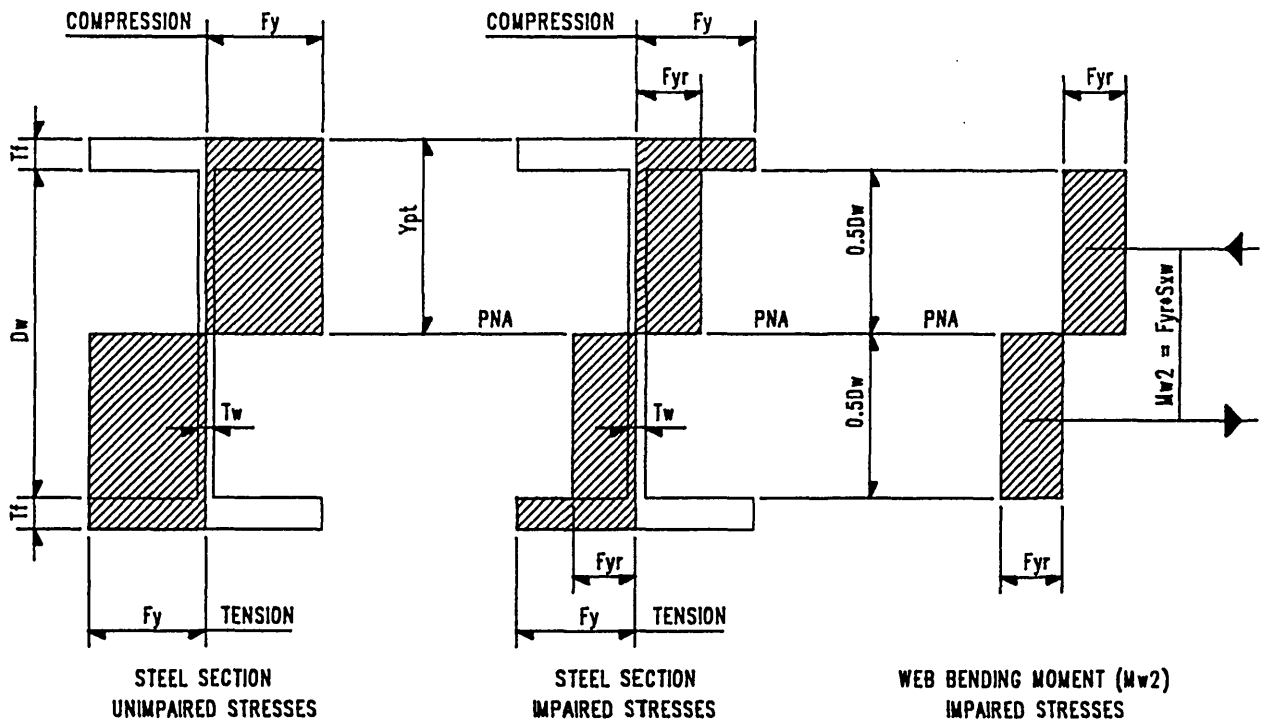


- NOTE: 1) ALL DIMENSIONS ARE IN MILLIMETRES.
2) AREA OF SHEAR PLANE AT TYPE 1 BEAMS IS THE GROSS AREA OF COMPOSITE DECK.
3) AREA OF SHEAR PLANE AT TYPE 2 BEAMS IS THE AREA OF DECK ABOVE PROFILE.

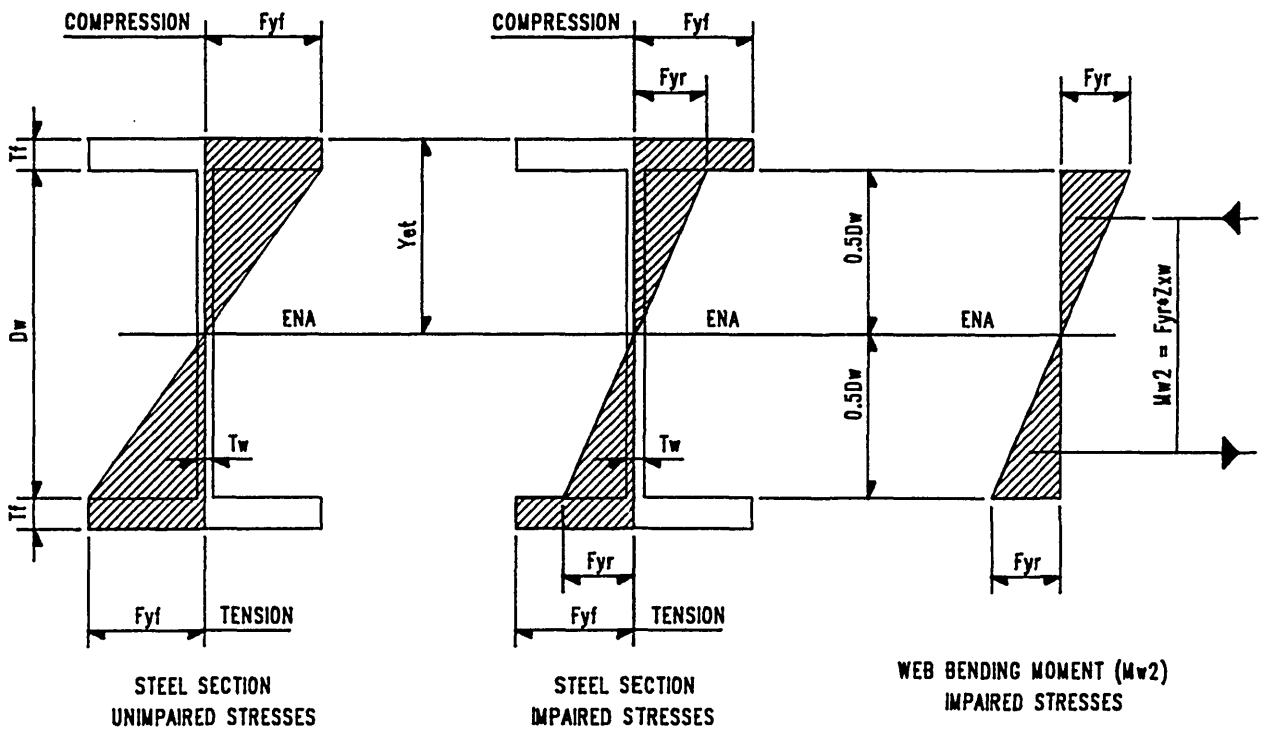
**STRENGTH of STEEL SECTION with WEB IMPAIRMENT
in SYMMETRIC LIGHT PLATE GIRDERS (PARTS 3, 4, 5)**

FIG.13

A) COMPACT LIGHT PLATE GIRDERS.



B) NON-COMPACT LIGHT PLATE GIRDERS.

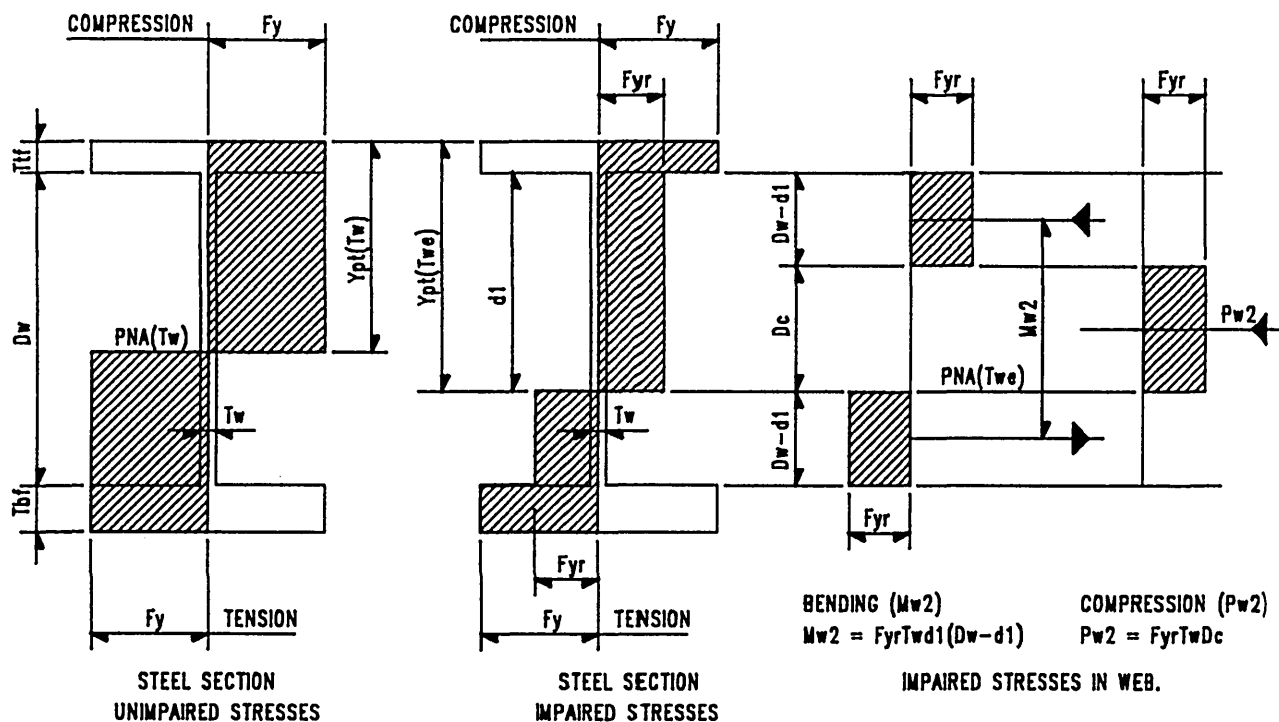


- NOTE: 1) IF THE FLANGE IS COMPACT IN A NON-COMPACT SECTION $F_{yf} = F_y$.
 2) F_{yf} = DESIGN STRENGTH OF SLENDER TOP FLANGE.
 3) F_{yr} = DESIGN STRENGTH IMPAIRED BY WEB BUCKLING.

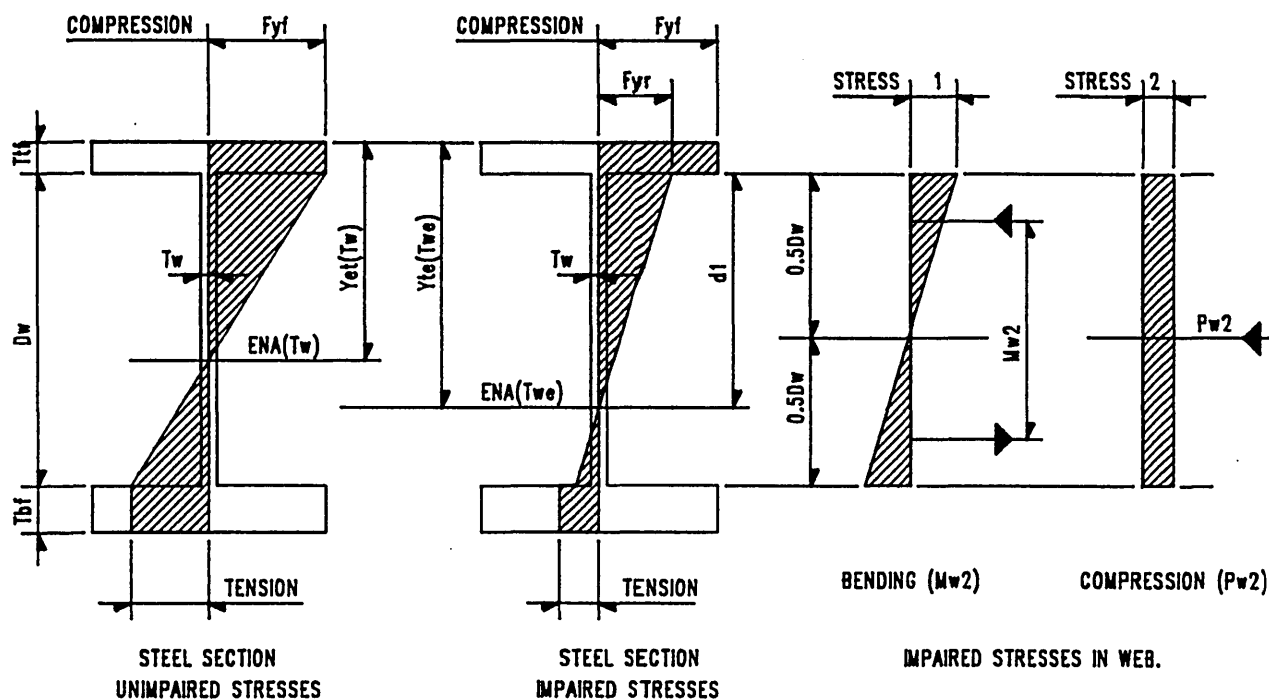
STRENGTH of STEEL SECTION with WEB IMPAIRMENT in MONO-SYMMETRIC LIGHT PLATE GIRDERS (PARTS 3,4,5)

FIG.14

A) COMPACT LIGHT PLATE GIRDERS.



B) NON-COMPACT LIGHT PLATE GIRDERS.



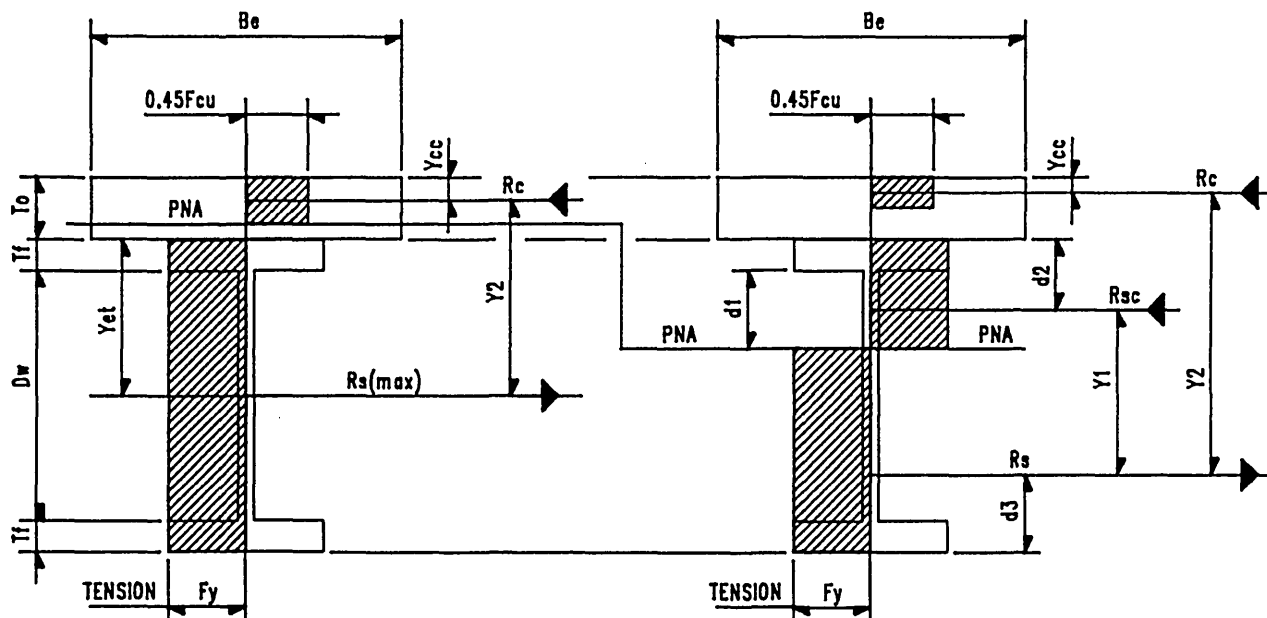
NOTE: 1) IF THE FLANGE IS COMPACT IN A NON-COMPACT SECTION $F_{yf} = F_y$.
2) F_{yf} = DESIGN STRENGTH OF SLENDER TOP FLANGE.
3) F_{yr} = DESIGN STRENGTH IMPAIRED BY WEB BUCKLING.

4) STRESS 1 = $0.5F_{yr}(D_w/d_1)$
5) STRESS 2 = $F_{yr}(1 - 0.5(D_w/d_1))$

STRENGTH of COMPOSITE SECTION and the EFFECT of PARTIAL SHEAR CONNECTION (PARTS 2 to 8)

FIG.15

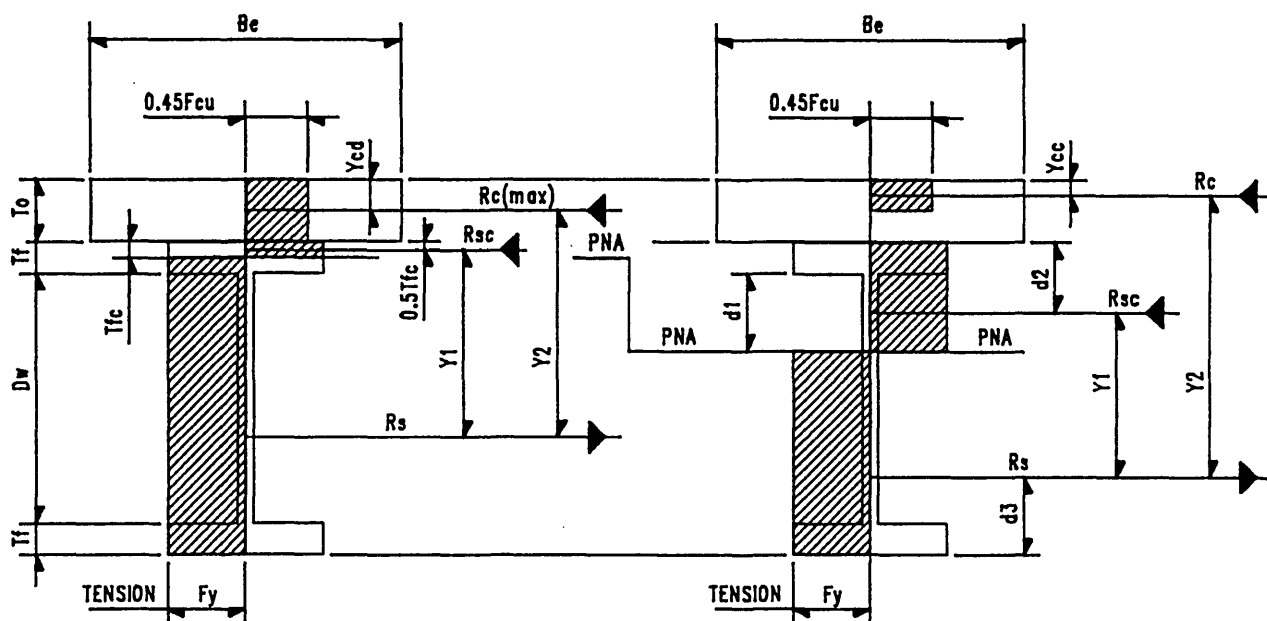
A) CASE 1: PNA lies in CONCRETE.



i) STRESS DISTRIBUTION WITH 100% CONNECTION.

ii) STRESS DISTRIBUTION WITH 50% CONNECTION.

B) CASE 2: PNA lies in TOP FLANGE of STEEL SECTION.



i) STRESS DISTRIBUTION WITH 100% CONNECTION.

ii) STRESS DISTRIBUTION WITH 50% CONNECTION.

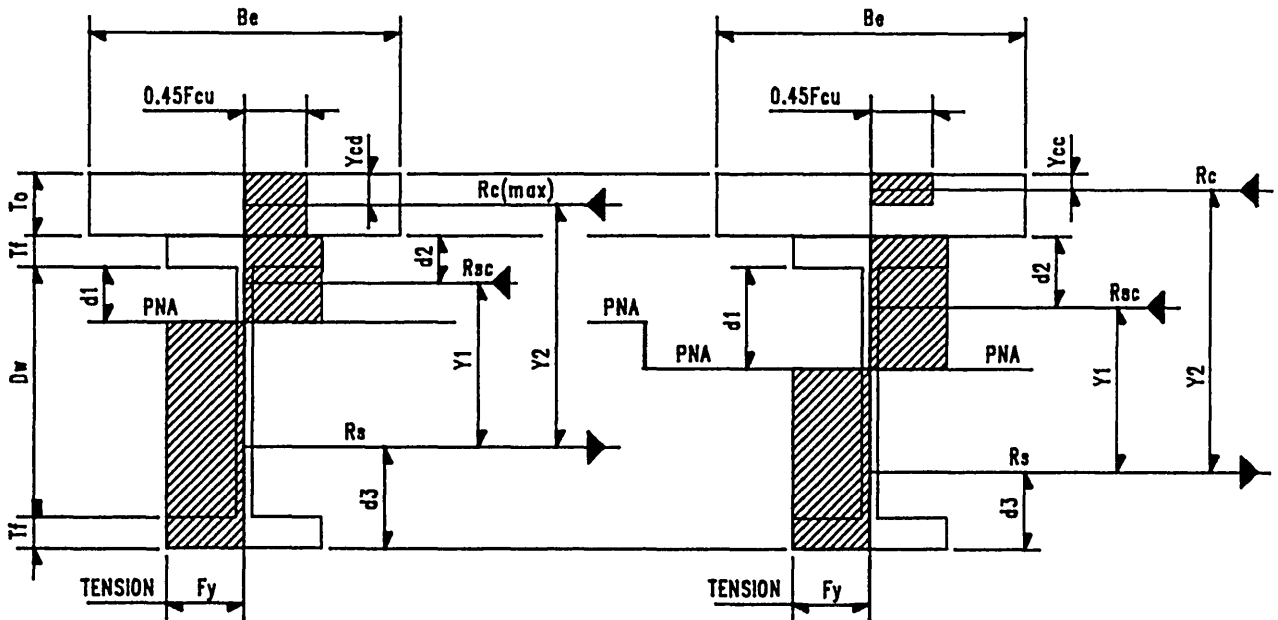
NOTE: 1) $R_c(max)$ = AVAILABLE RESISTANCE OF CONCRETE.
2) $R_s(max)$ = AVAILABLE RESISTANCE OF STEEL SECTION.
3) R_{sc} = RESISTANCE OF STEEL STRESS BLOCK IN COMPRESSION.

4) PNA = PLASTIC NEUTRAL AXIS.

STRENGTH of COMPOSITE SECTION and the EFFECT of PARTIAL SHEAR CONNECTION (PARTS 2 to 8)

FIG.16

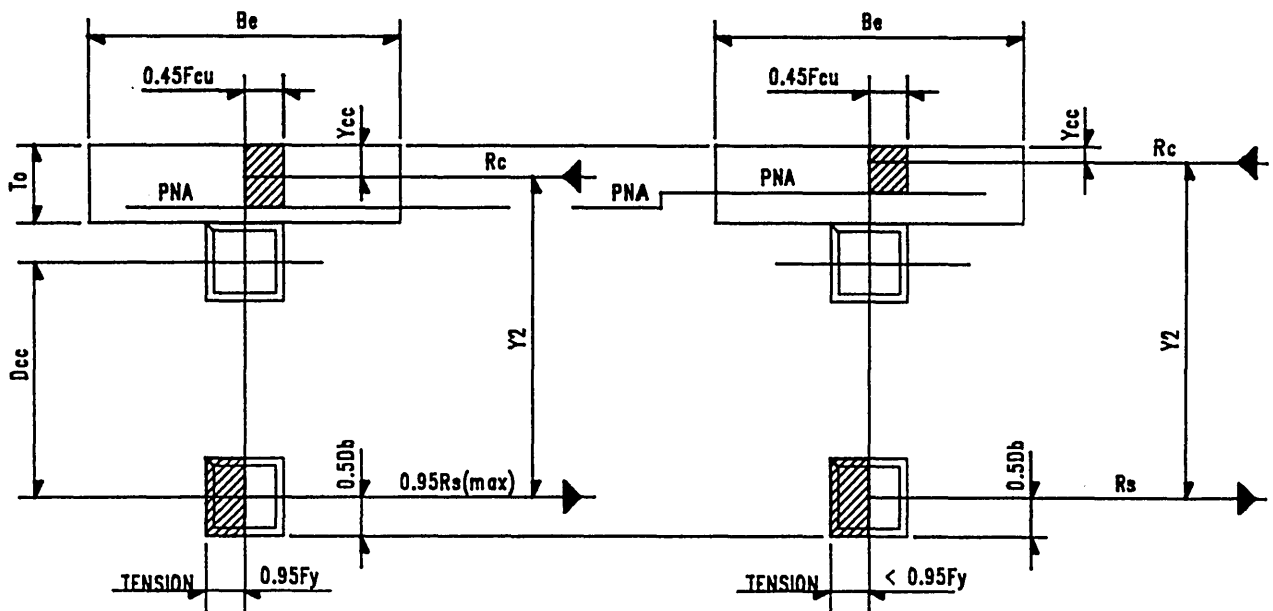
A) CASE 3: PNA lies in WEB of STEEL SECTION.



i) STRESS DISTRIBUTION WITH 100% CONNECTION.

ii) STRESS DISTRIBUTION WITH 50% CONNECTION.

B) CASE 1: TRUSS – PNA always lies in CONCRETE.



i) STRESS DISTRIBUTION WITH 100% CONNECTION.
AT MID-SPAN

ii) STRESS DISTRIBUTION WITH 50% CONNECTION.
AT INTERMEDIATE POINT ON SPAN

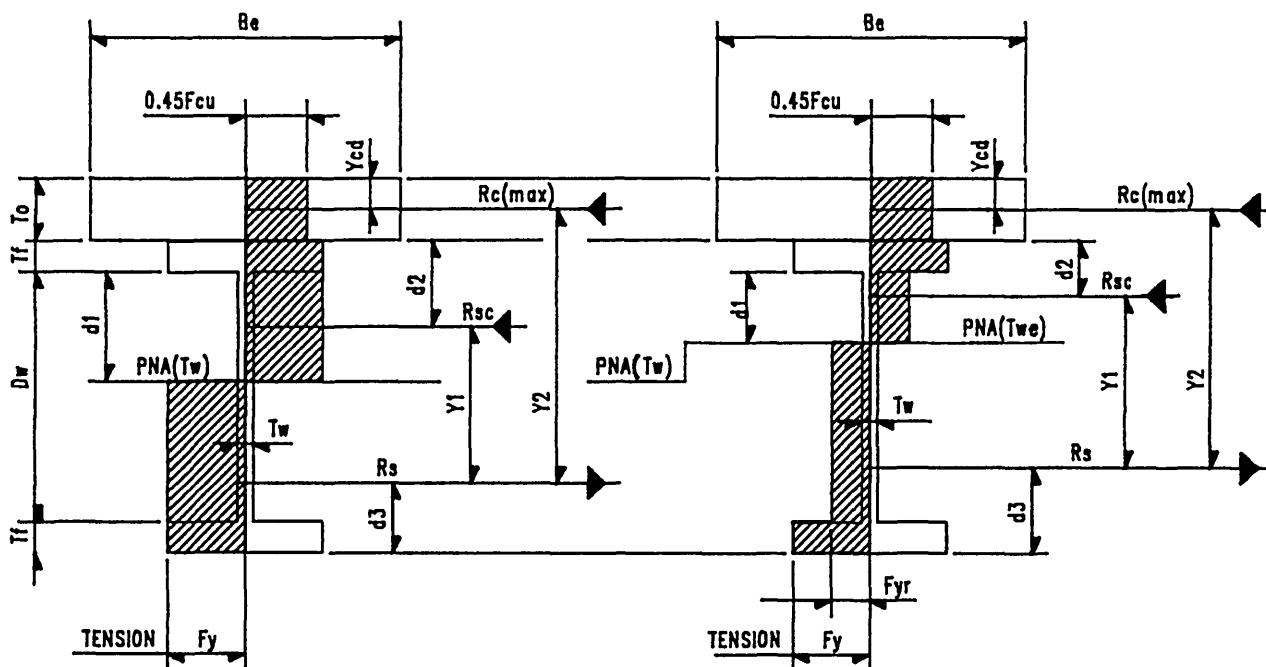
- NOTE:
- 1) $R_c(max)$ = AVAILABLE RESISTANCE OF CONCRETE.
 - 2) $R_s(max)$ = AVAILABLE RESISTANCE OF STEEL SECTION.
 - 3) R_{sc} = RESISTANCE OF STEEL STRESS BLOCK IN COMPRESSION.

4) PNA = PLASTIC NEUTRAL AXIS.

STRENGTH of COMPOSITE SECTION with WEB IMPAIRMENT in LIGHT PLATE GIRDERS (PARTS 3,4,5)

FIG.17

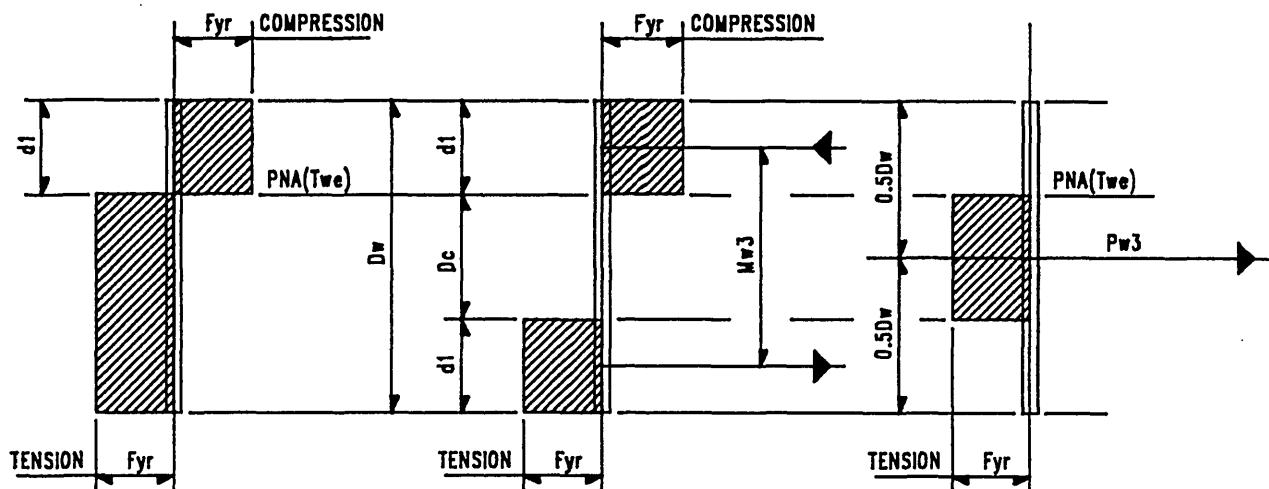
A) CASE 3: PNA lies in WEB of STEEL SECTION.



i) STRESS DISTRIBUTION WITH 100% CONNECTION AND UNIMPAIRED WEB.

ii) STRESS DISTRIBUTION WITH 100% CONNECTION AND IMPAIRED WEB.

B) IMPAIRED WEB STRESS COMPONENTS.



IMPAIRED STRESSES IN WEB.

WEB BENDING MOMENT (M_{w3})
 $M_{w3} = F_y T_w d_1 (D_w - d_1)$

WEB COMPRESSION FORCE (P_{w3})
 $P_{w3} = F_y T_w D_c$

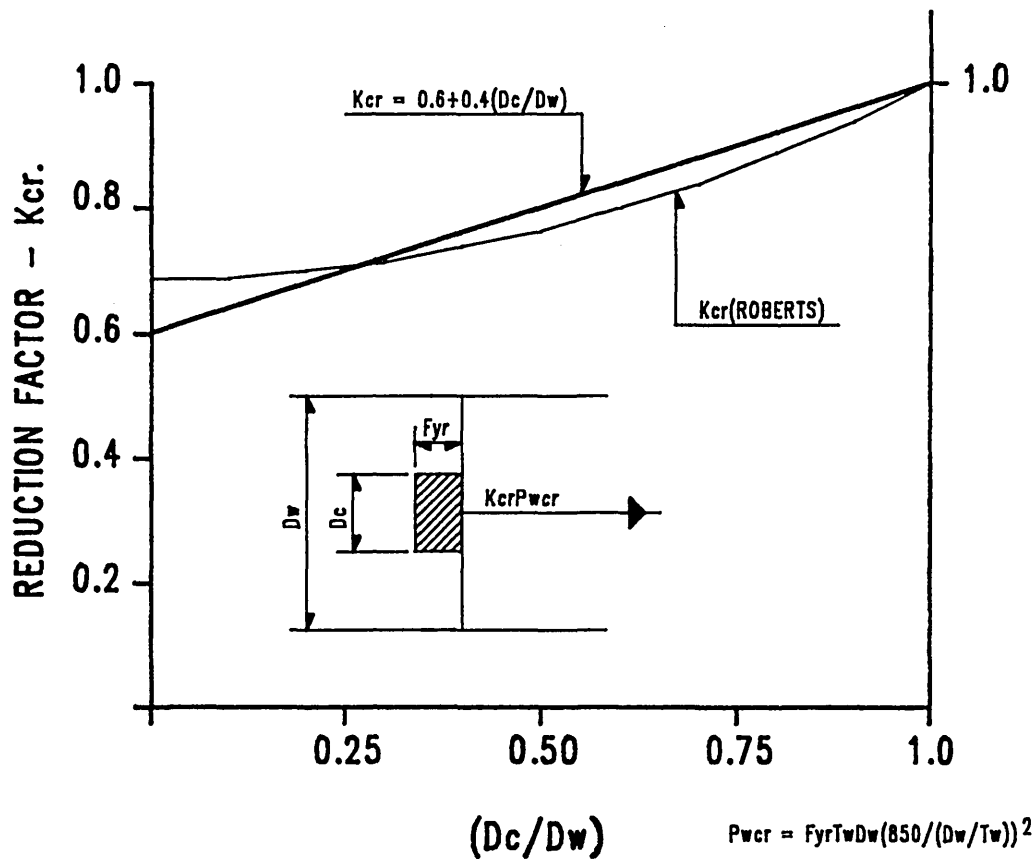
- NOTE:
- 1) $R_c(max)$ = AVAILABLE RESISTANCE OF CONCRETE.
 - 2) $R_s(max)$ = AVAILABLE RESISTANCE OF STEEL SECTION.
 - 3) R_{sc} = RESISTANCE OF STEEL STRESS BLOCK IN COMPRESSION.

4) PNA = PLASTIC NEUTRAL AXIS.

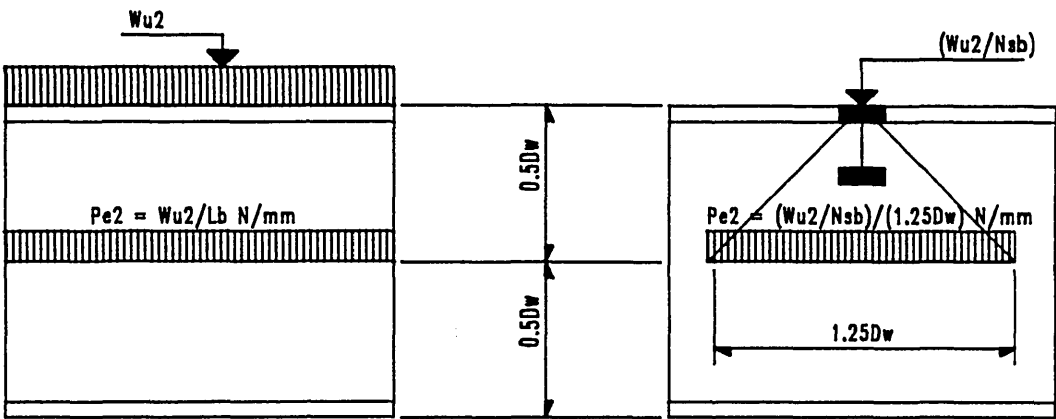
CRITICAL AXIAL STRENGTH REDUCTION FACTOR (K_{cr})
and EDGE LOADING (Pe_2)

FIG.18

A) REDUCTION FACTOR (K_{cr}).



B) EDGE LOADING (Pe_2).



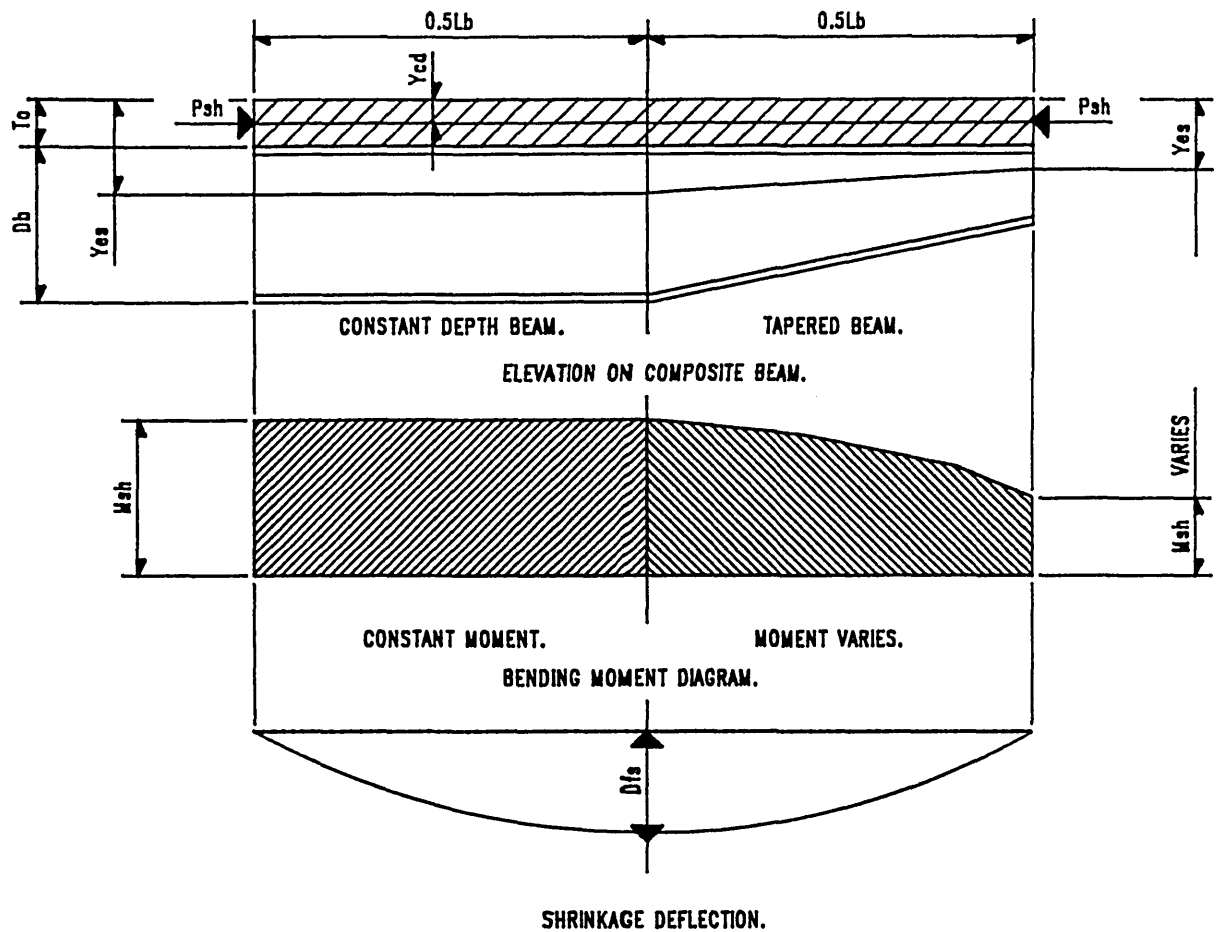
BEAM TYPE 1.

BEAM TYPE 2.

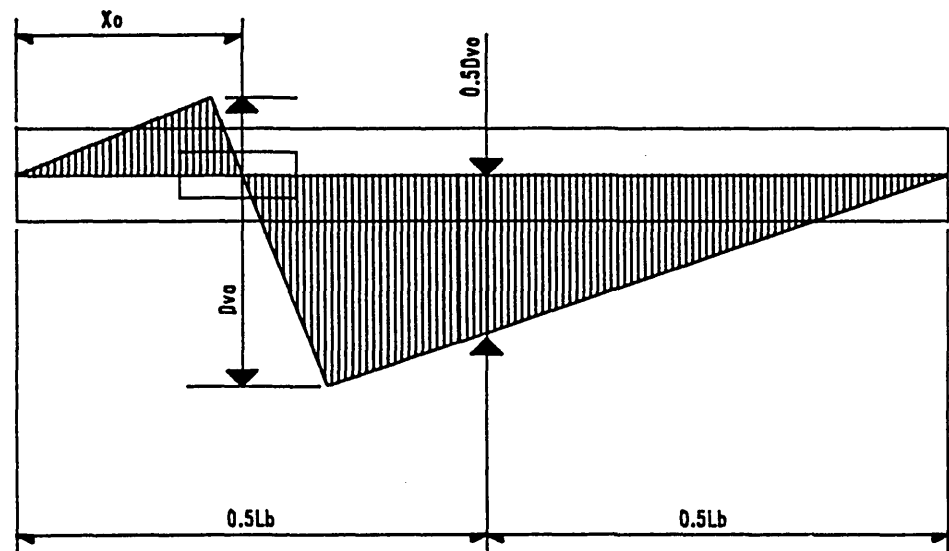
SHRINKAGE DEFLECTION of COMPOSITE BEAMS (D_{fs})
and VIERENDEEL DEFLECTION at WEB OPENINGS (D_{vo})

FIG.19

A) SHRINKAGE DEFLECTION (D_{fs}).

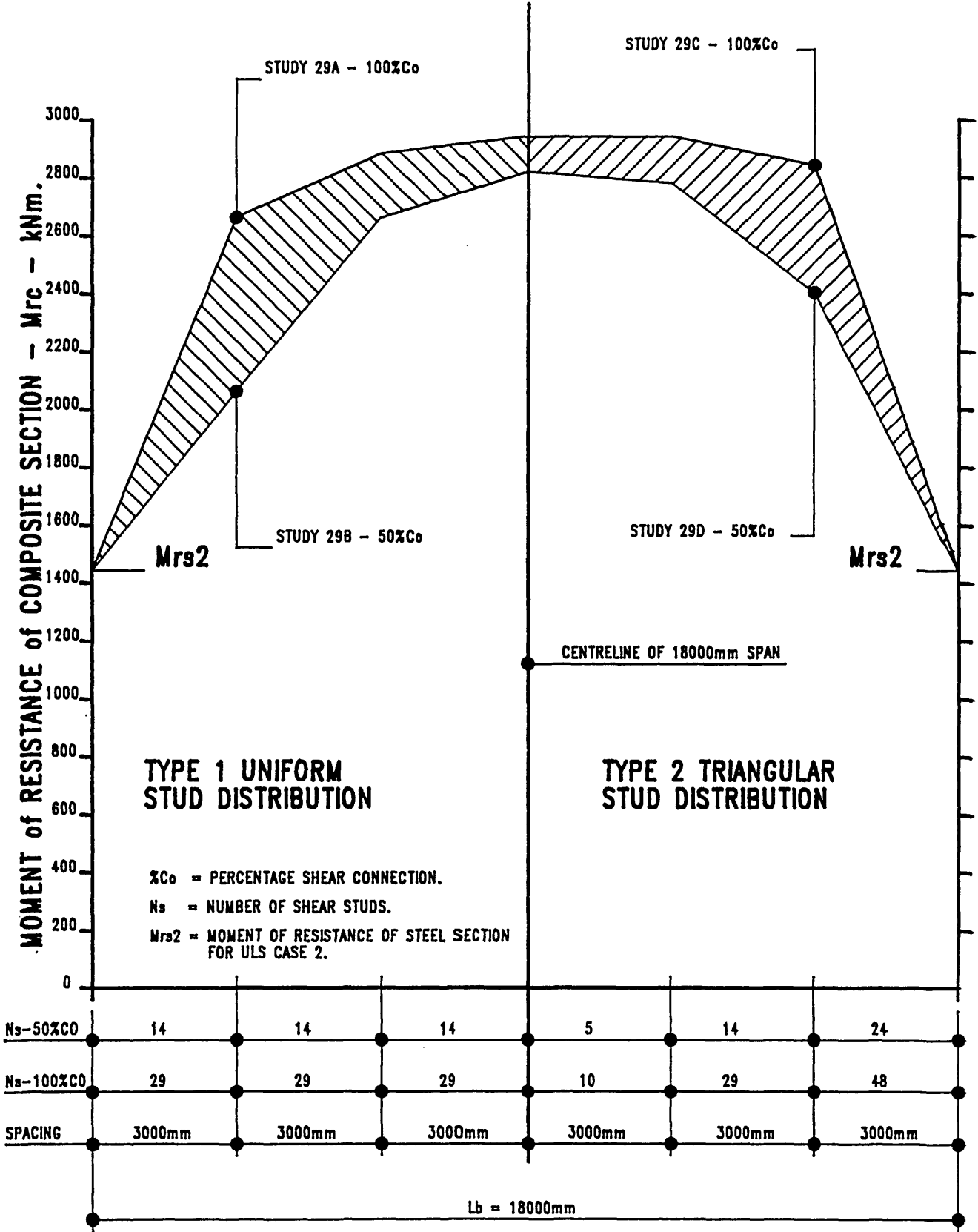


B) VIERENDEEL DEFLECTION (D_{vo}).



EFFECTS OF STUD DISTRIBUTION and %Co. on Mrc.
for TYPE 2 FLOOR BEAMS (PART 3: LPG(C): STUDY 19:A,B,C,D)

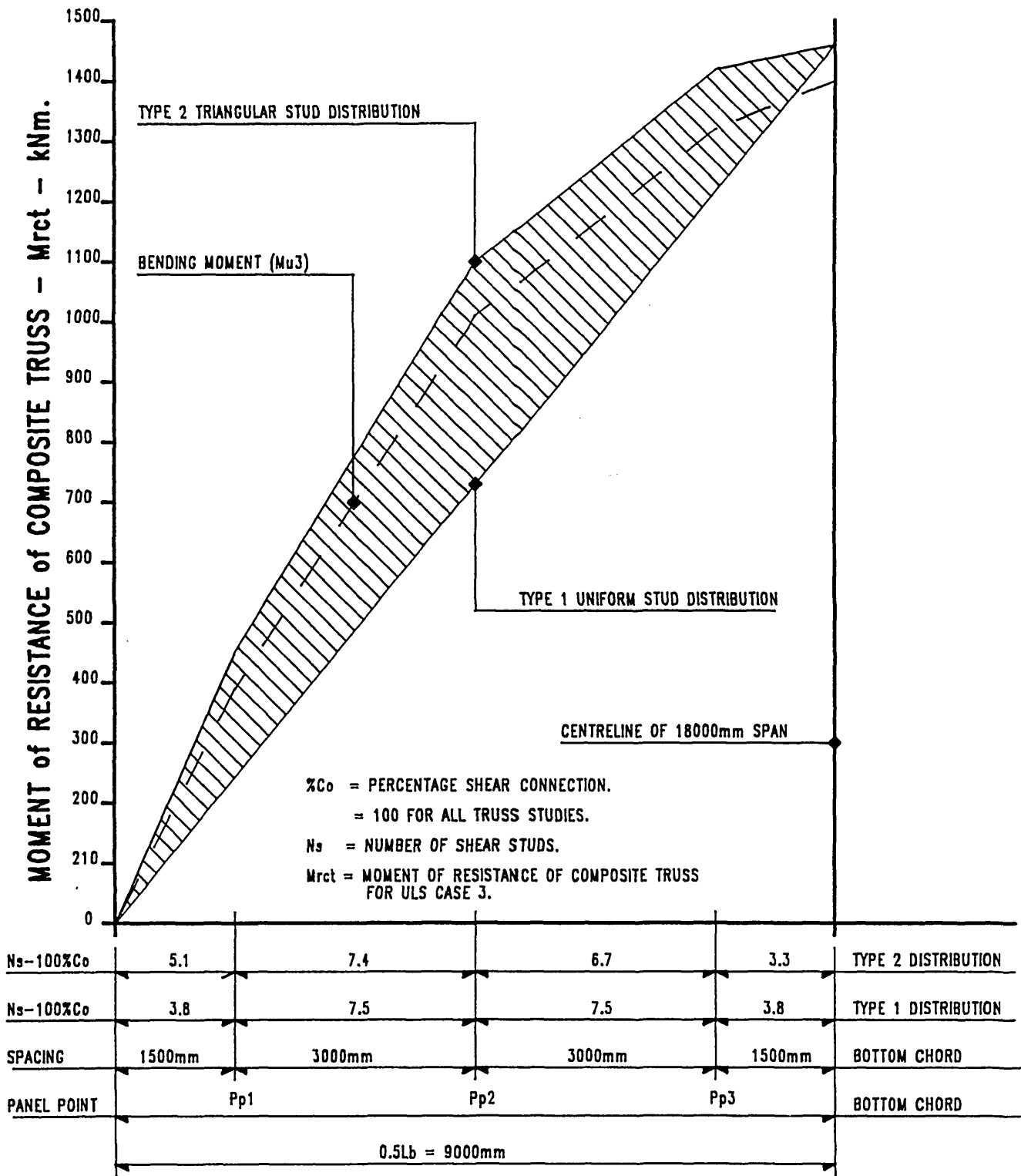
FIG.20



EFFECTS OF STUD DISTRIBUTION on M_{rct}
for a TYPE 1 FLOOR BEAM (PART 8: TRUSS(H))

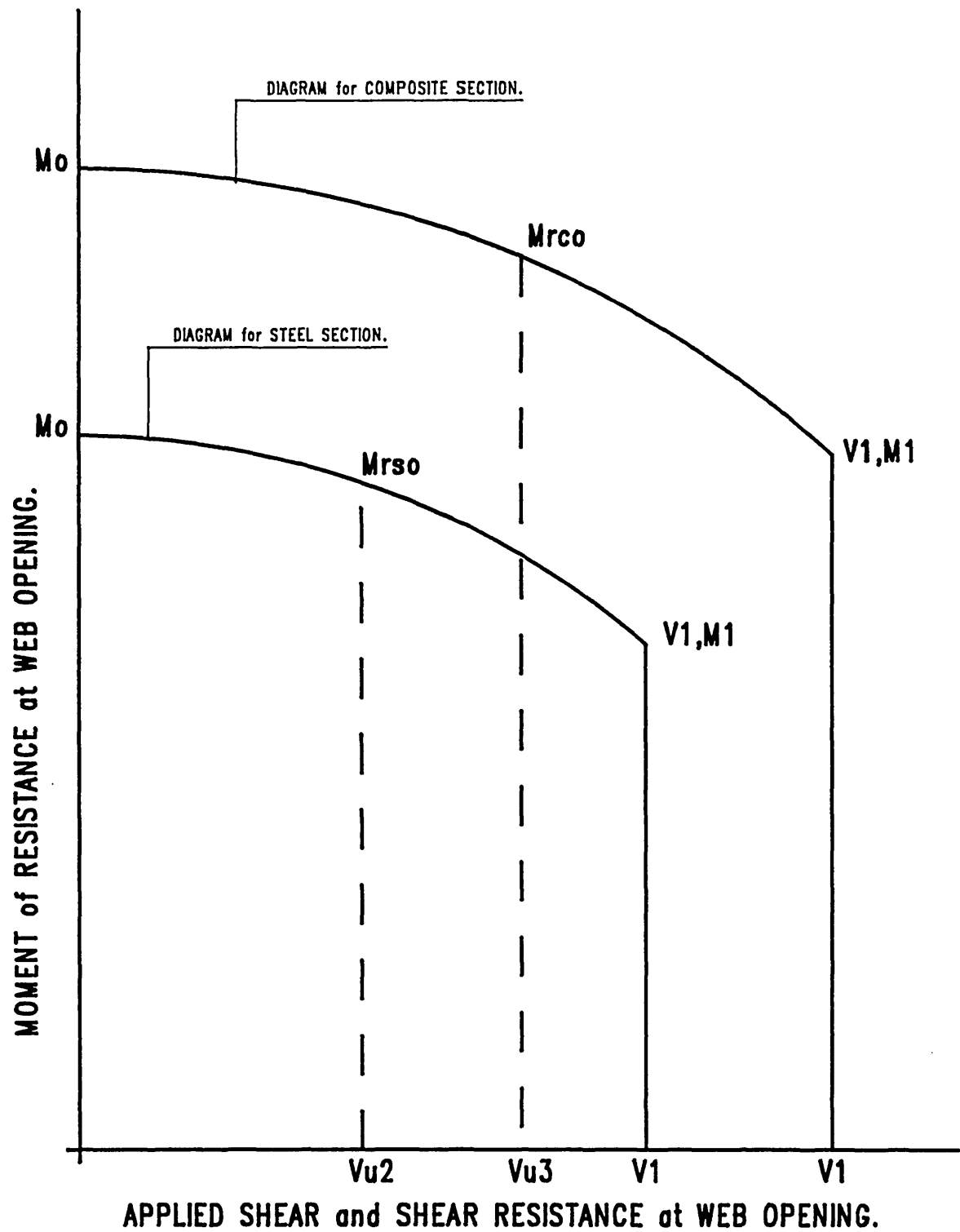
FIG.21

NOTE: VALUES ARE TAKEN FROM PART 8: STUDY NO.5 TABLES 14 & 24.



MOMENT-SHEAR INTERACTION DIAGRAM for STEEL
and COMPOSITE SECTIONS at WEB OPENINGS (PARTS 6 & 7).

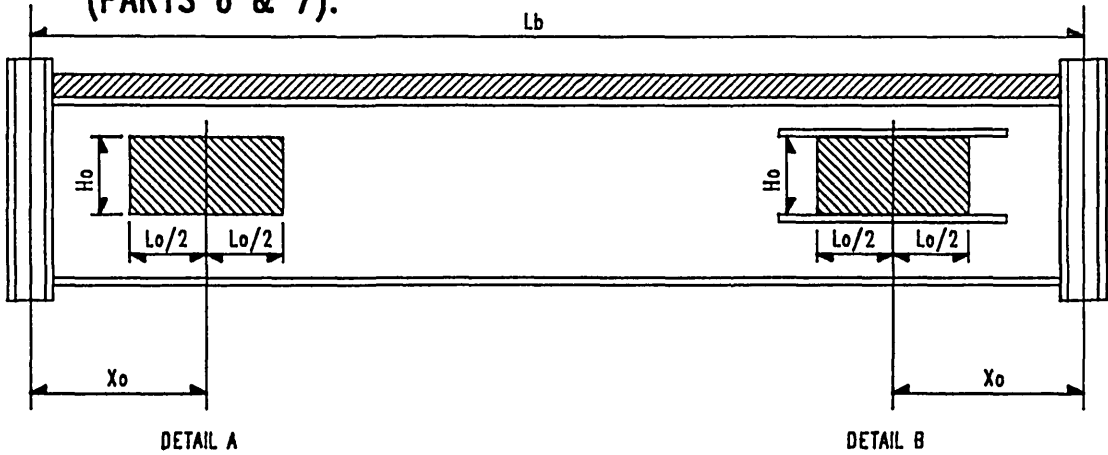
FIG.22



- NOTES: 1) $Mrso$ = MOMENT of RESISTANCE of STEEL SECTION at WEB OPENING.
2) $Mrco$ = MOMENT of RESISTANCE of COMPOSITE SECTION at WEB OPENING.

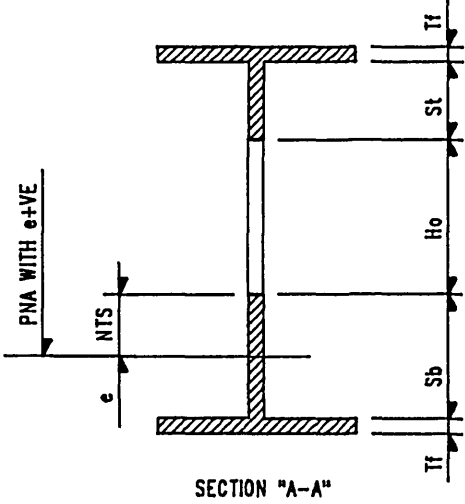
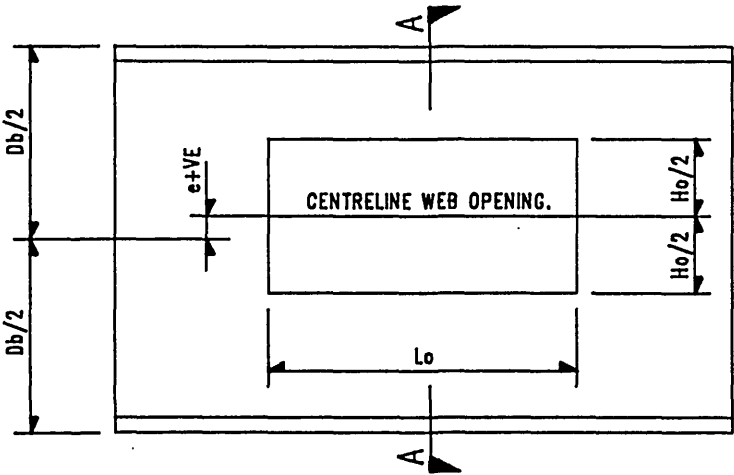
DETAILS at UNREINFORCED and REINFORCED WEB OPENINGS
(PARTS 6 & 7).

FIG.23

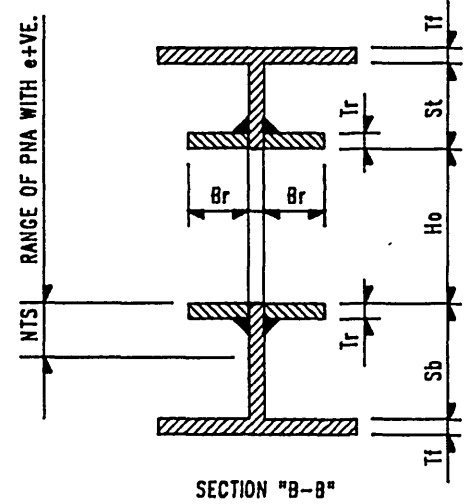
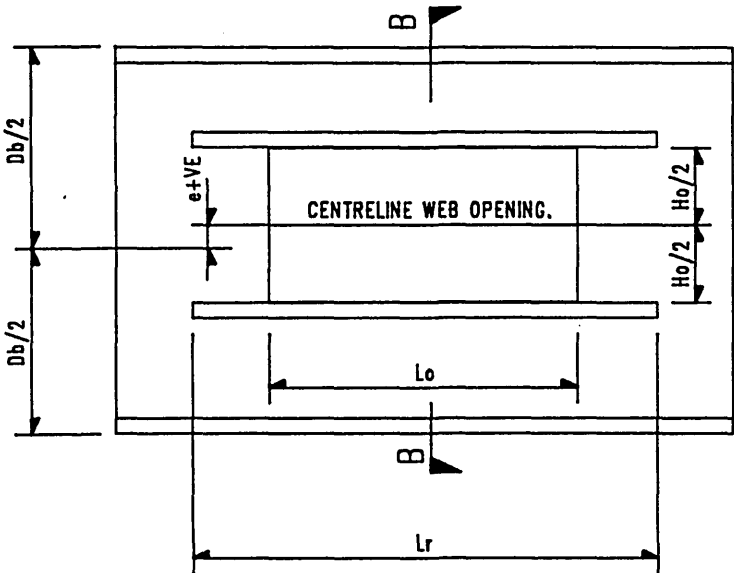


ELEVATION on PERFORATED BEAM.

DETAIL A: UNREINFORCED OPENING. (CONCRETE NOT SHOWN)



DETAIL B: REINFORCED OPENING. (CONCRETE NOT SHOWN)



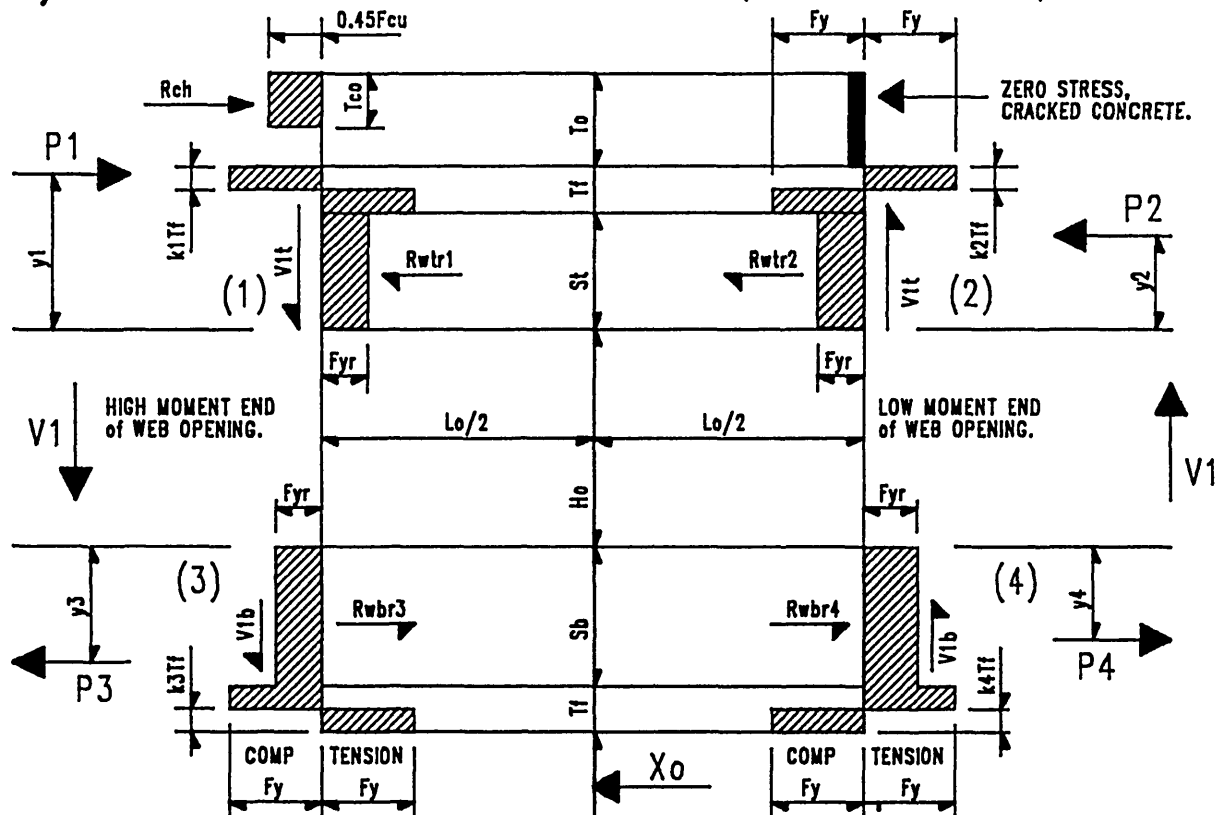
NOTE: e = ECCENTRICITY of WEB OPENING from MID-DEPTH of STEEL SECTION (+VE UP).

FIGURES

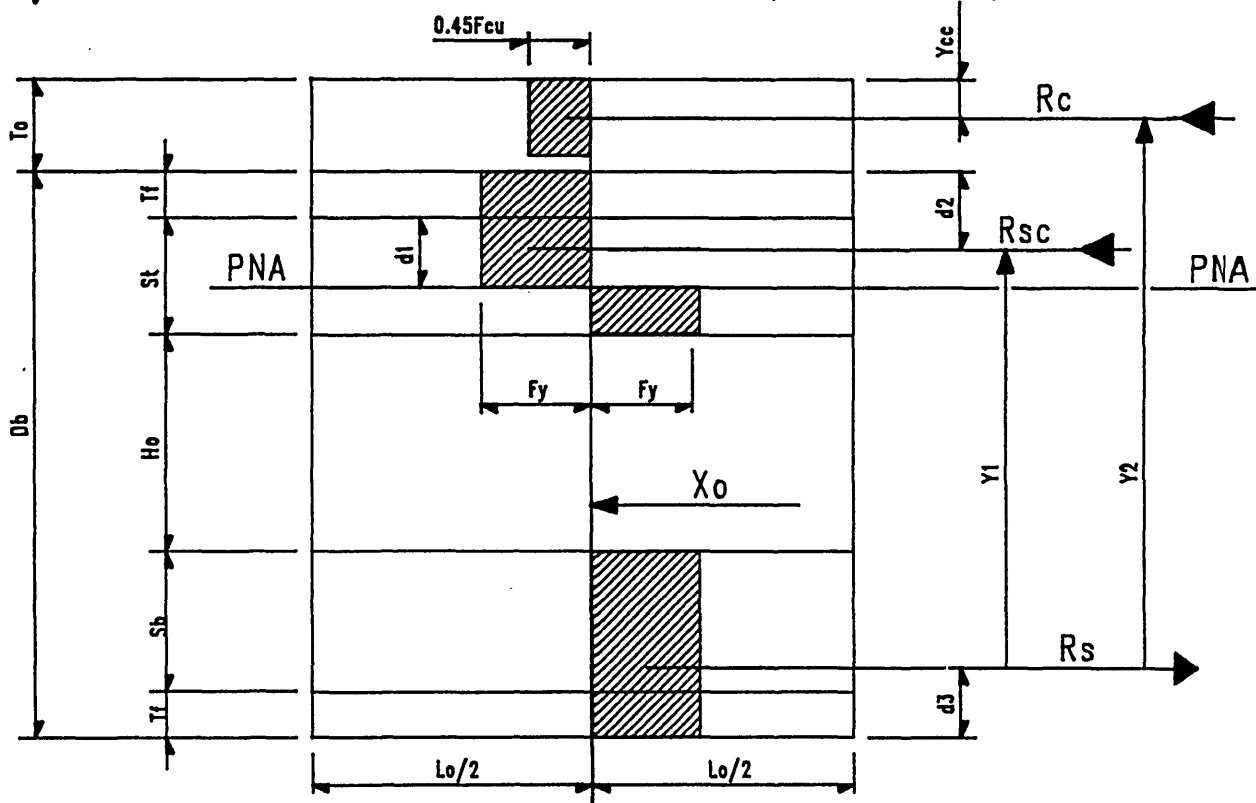
ASSUMED ULTIMATE STRESS DISTRIBUTIONS for COMPOSITE SECTION at UNREINFORCED WEB OPENINGS (PART 6)

FIG.24

A) STRESS DISTRIBUTION with HIGH SHEAR. (USED to CALCULATE V_1 and M_1).



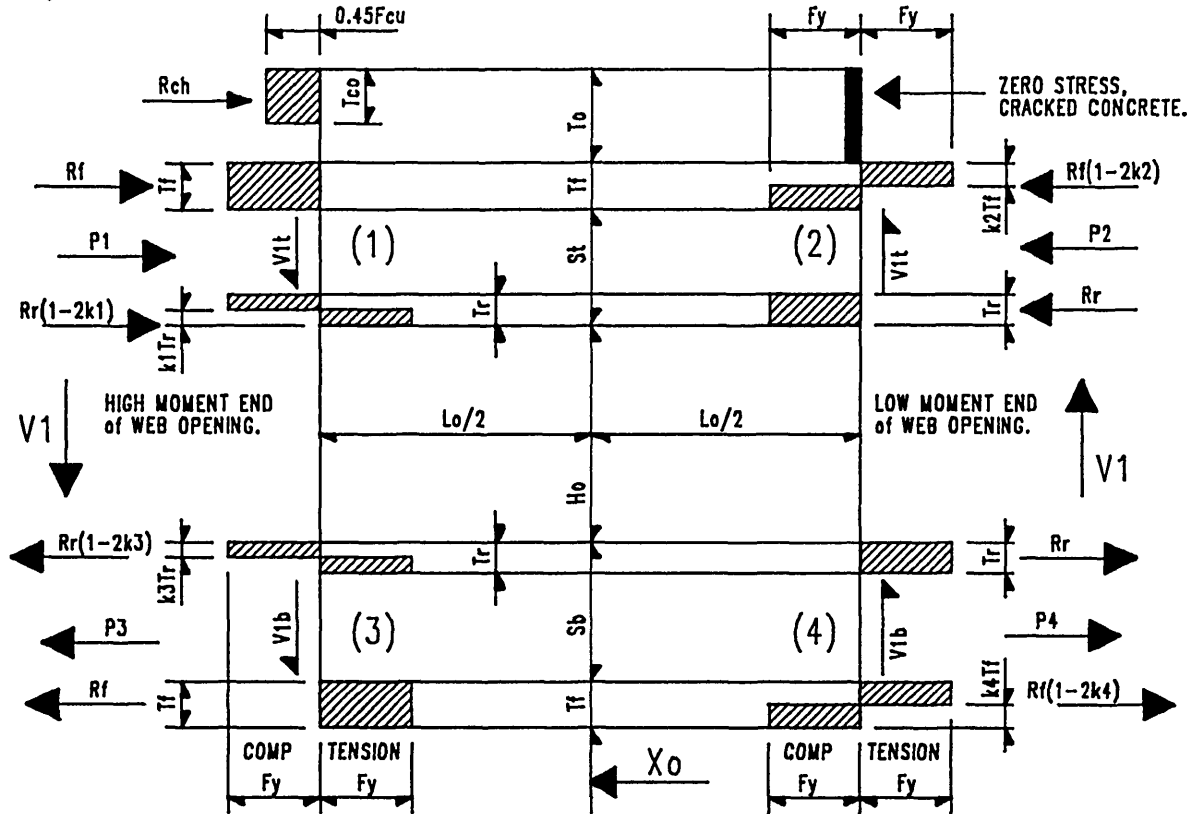
B) STRESS DISTRIBUTION with ZERO SHEAR. (USED to CALCULATE M_0)



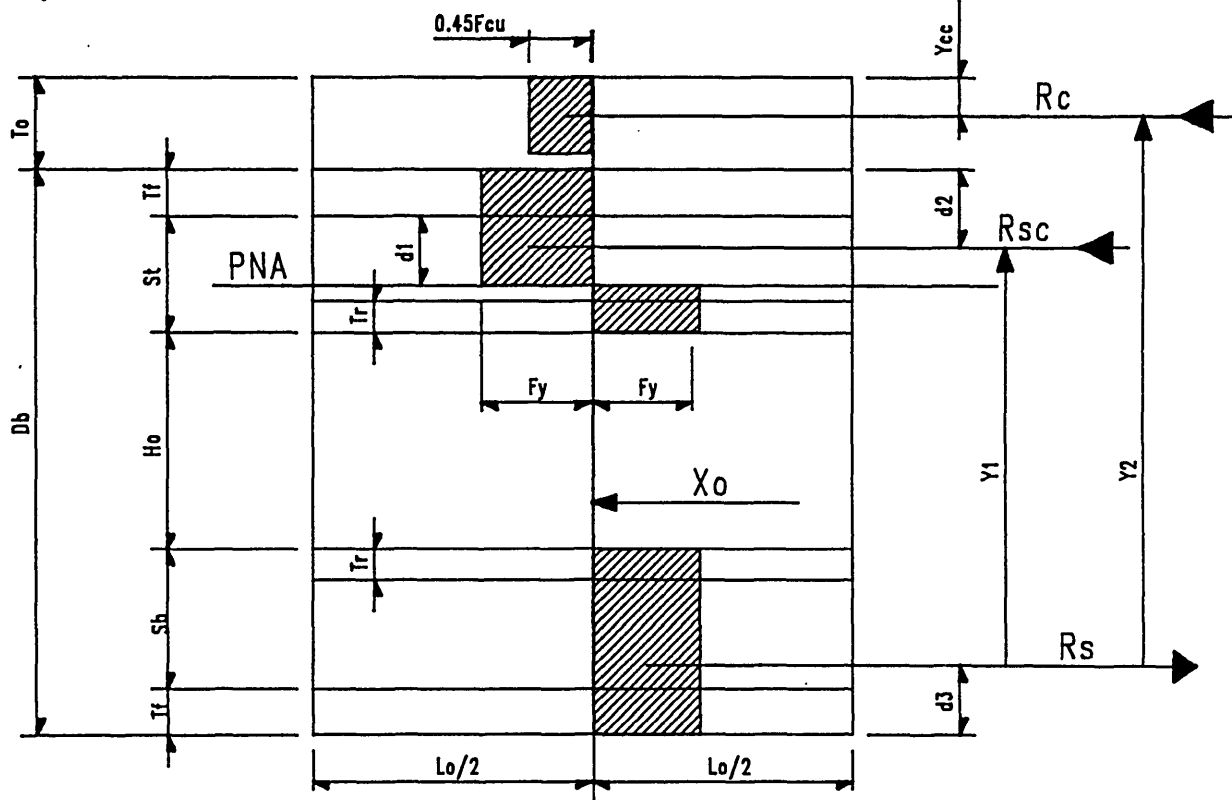
ASSUMED ULTIMATE STRESS DISTRIBUTIONS for
COMPOSITE SECTION at REINFORCED WEB OPENINGS (PART 7)

FIG.25

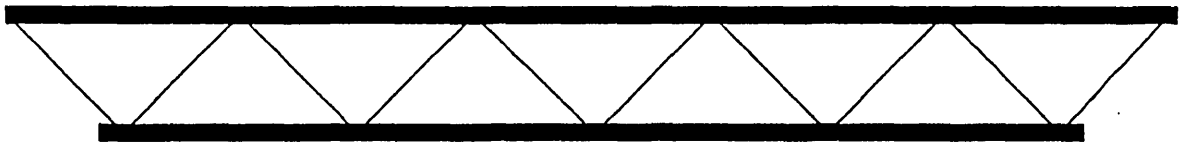
A) STRESS DISTRIBUTION with HIGH SHEAR. (USED to CALCULATE V_1 and M_1).



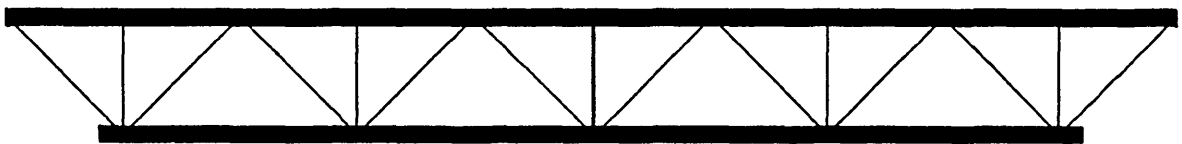
B) STRESS DISTRIBUTION with ZERO SHEAR. (USED to CALCULATE M_o)



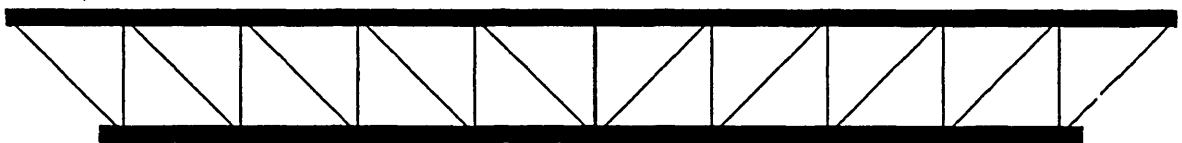
A) WARREN TRUSS.



B) MODIFIED WARREN TRUSS WITH VERTICALS.

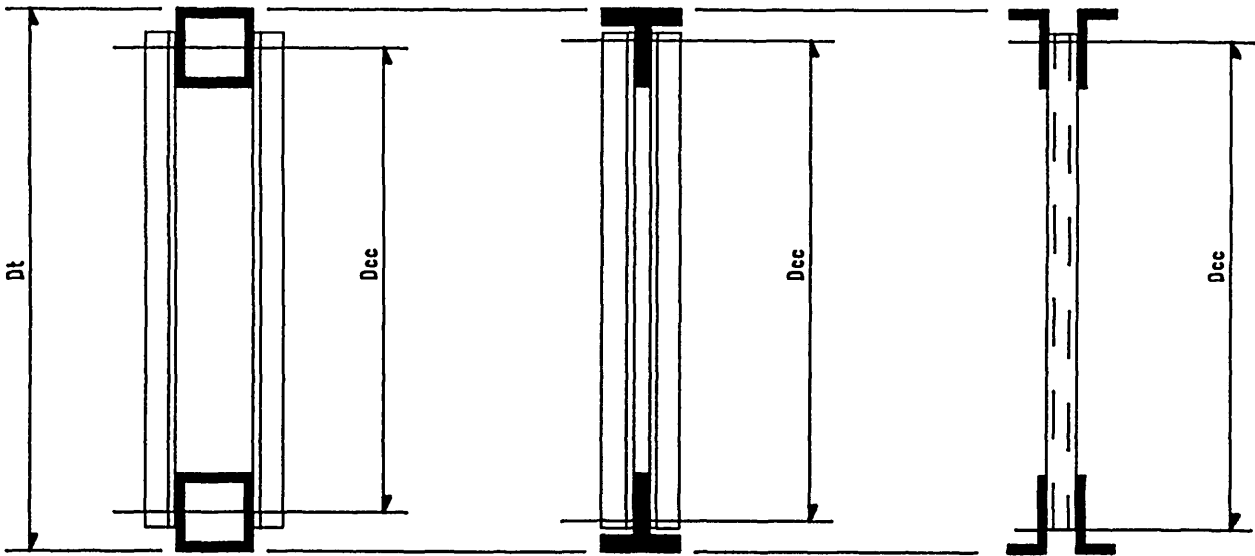


C) PRATT TRUSS.



D) TRUSS with VIERENDEEL PANEL at MID-SPAN.

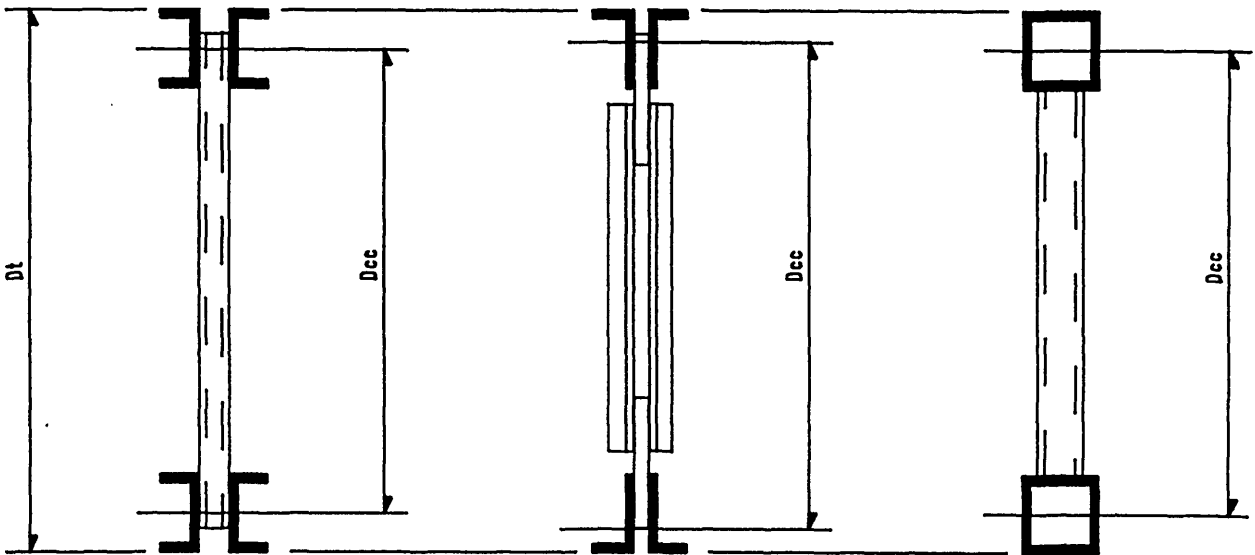




A) HOLLOW STRUCTURAL CHORDS and UNBATTENED ANGLE DIAGONALS.

B) ROLLED TEE CHORDS and BATTENED ANGLE DIAGONALS.

C) DOUBLE ANGLE CHORDS and HOLLOW STRUCTURAL DIAGONALS.



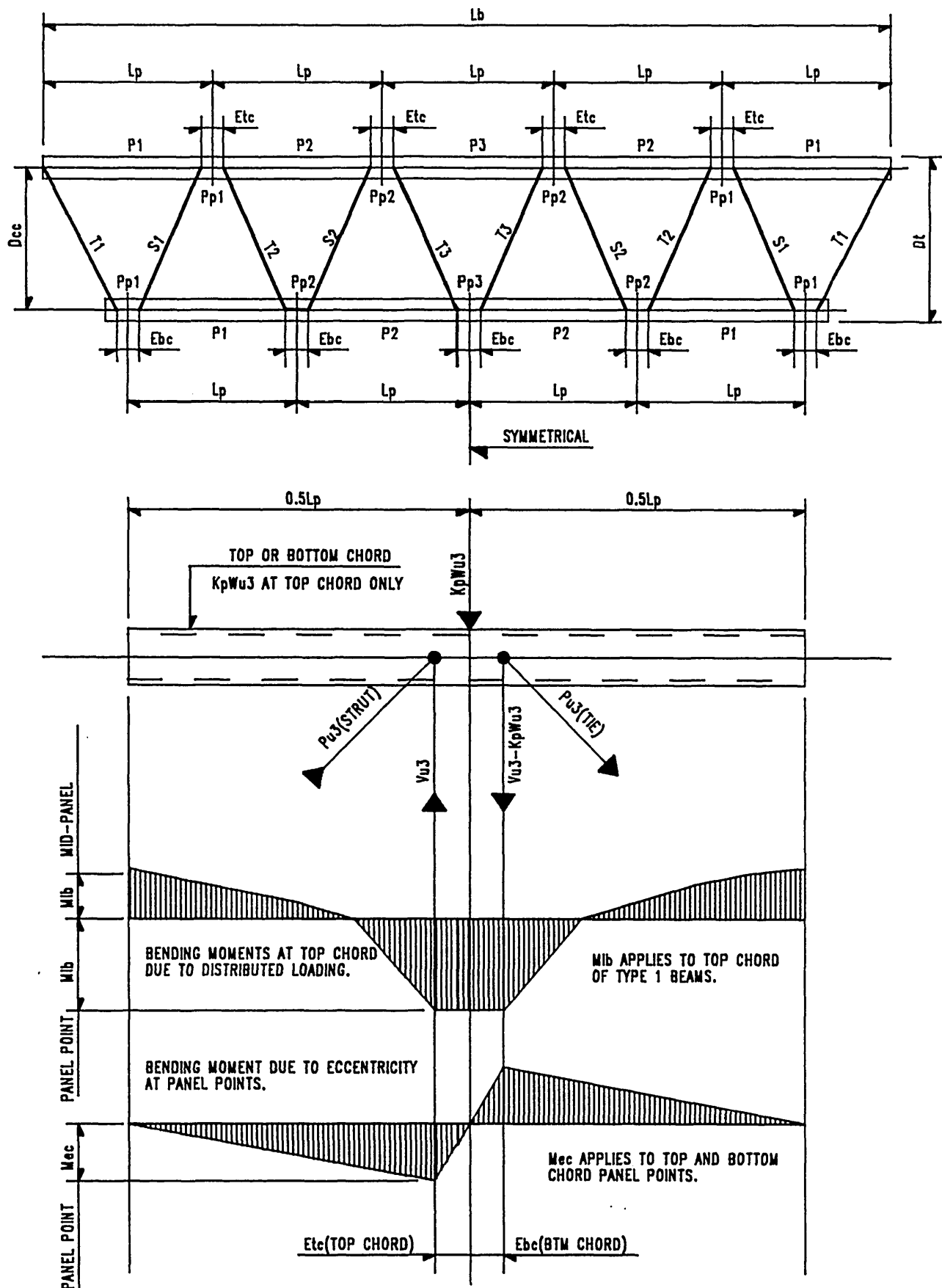
D) DOUBLE CHANNEL CHORDS and HOLLOW STRUCTURAL DIAGONALS.

E) GUSSETED DOUBLE ANGLE CHORDS with BATTENED ANGLE DIAGONALS.

F) HOLLOW STRUCTURAL CHORDS and DIAGONALS.

TRUSS KEY DIAGRAM and LOCAL FORCES at PANEL POINTS
(PART 8)

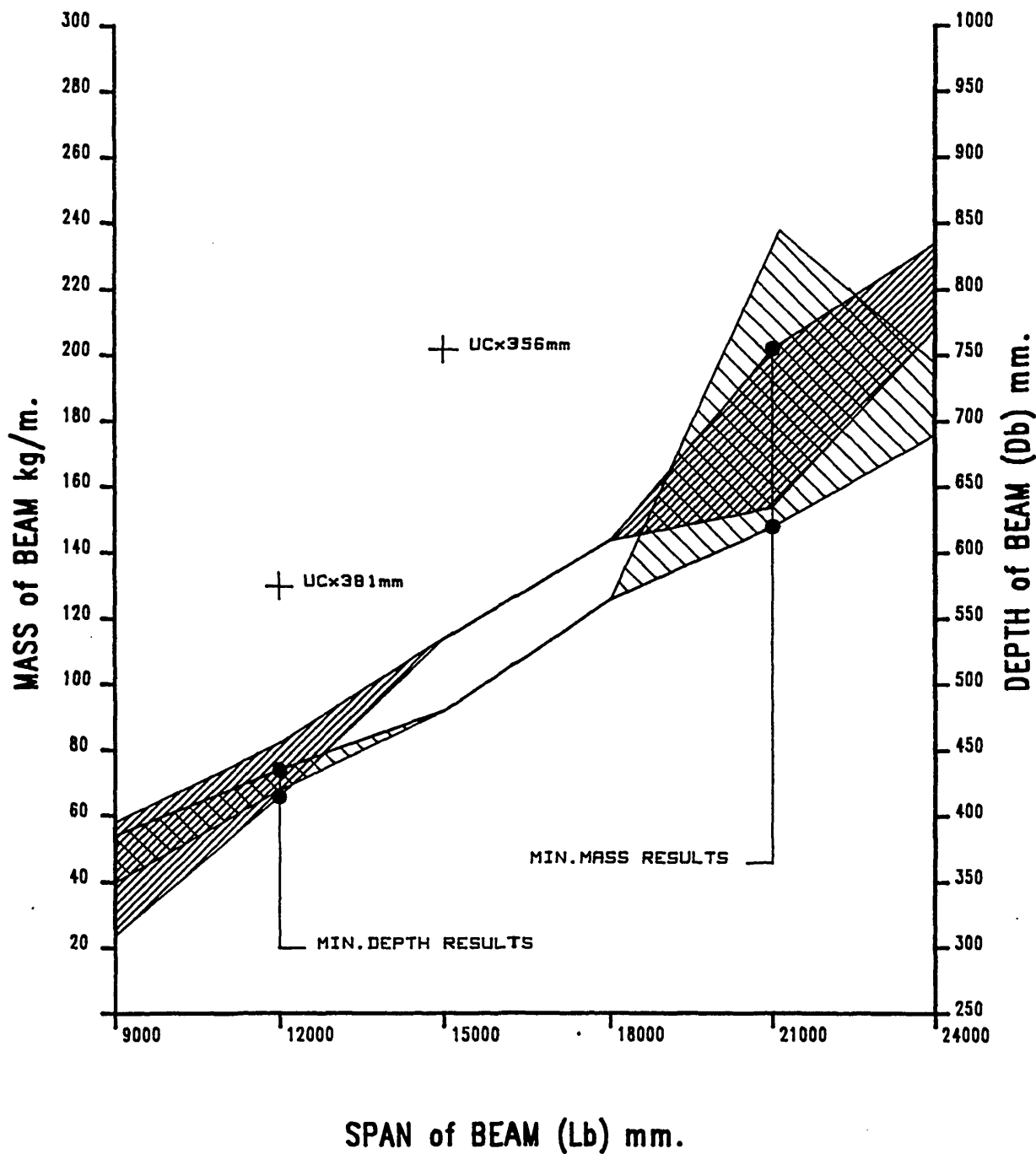
FIG.28



FIGURES

MIN. MASS and DEPTH STUDIES on UNIVERSAL BEAMS
TYPE 1 ROOF BEAMS – Br1 (PART 2)

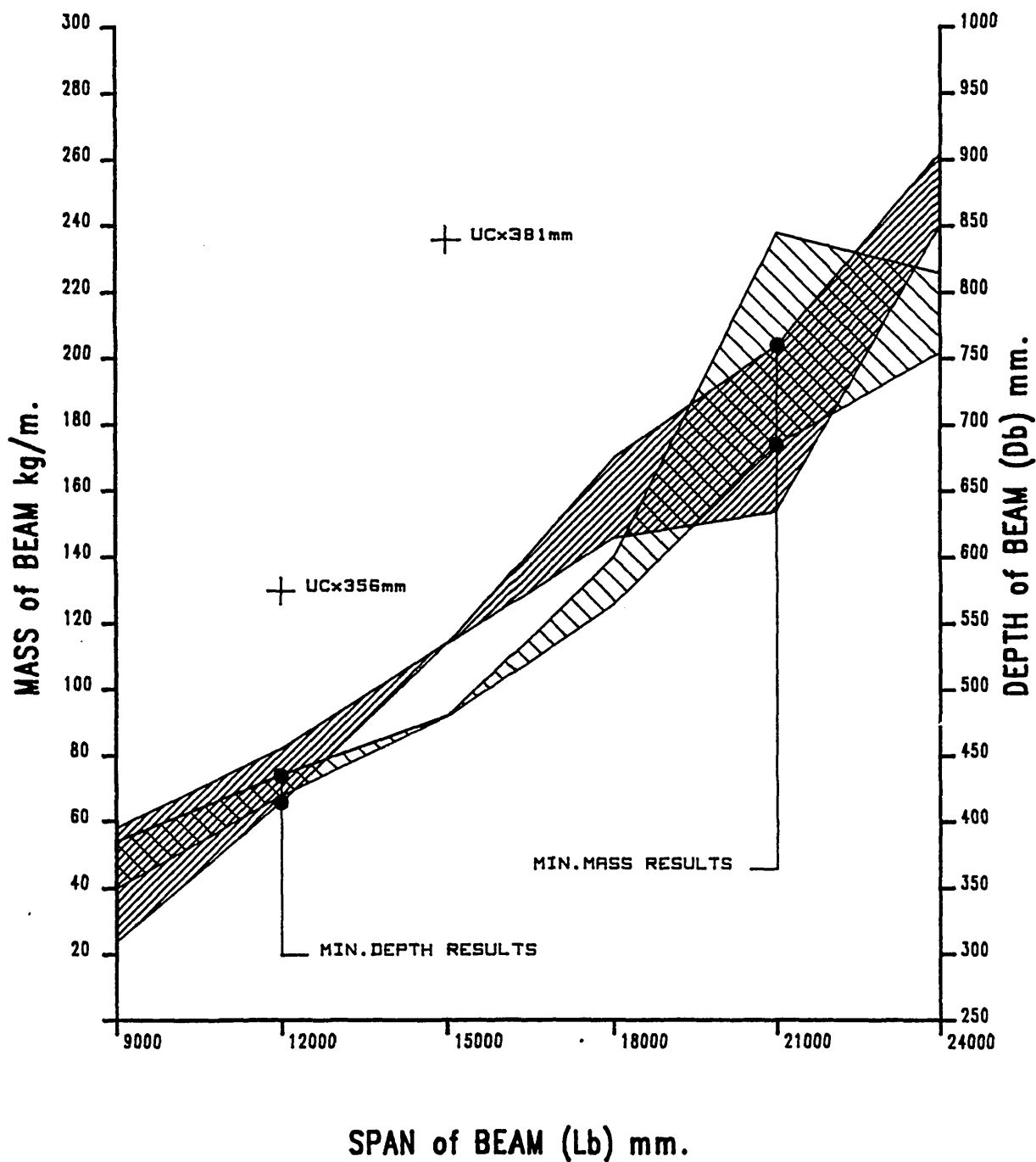
FIG.29



NOTE:- VALUES ARE TAKEN FROM RESULTS of DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES.

MIN. MASS and DEPTH STUDIES on UNIVERSAL BEAMS
TYPE 1 FLOOR BEAMS - Bf1 (PART 2)

Fig.30



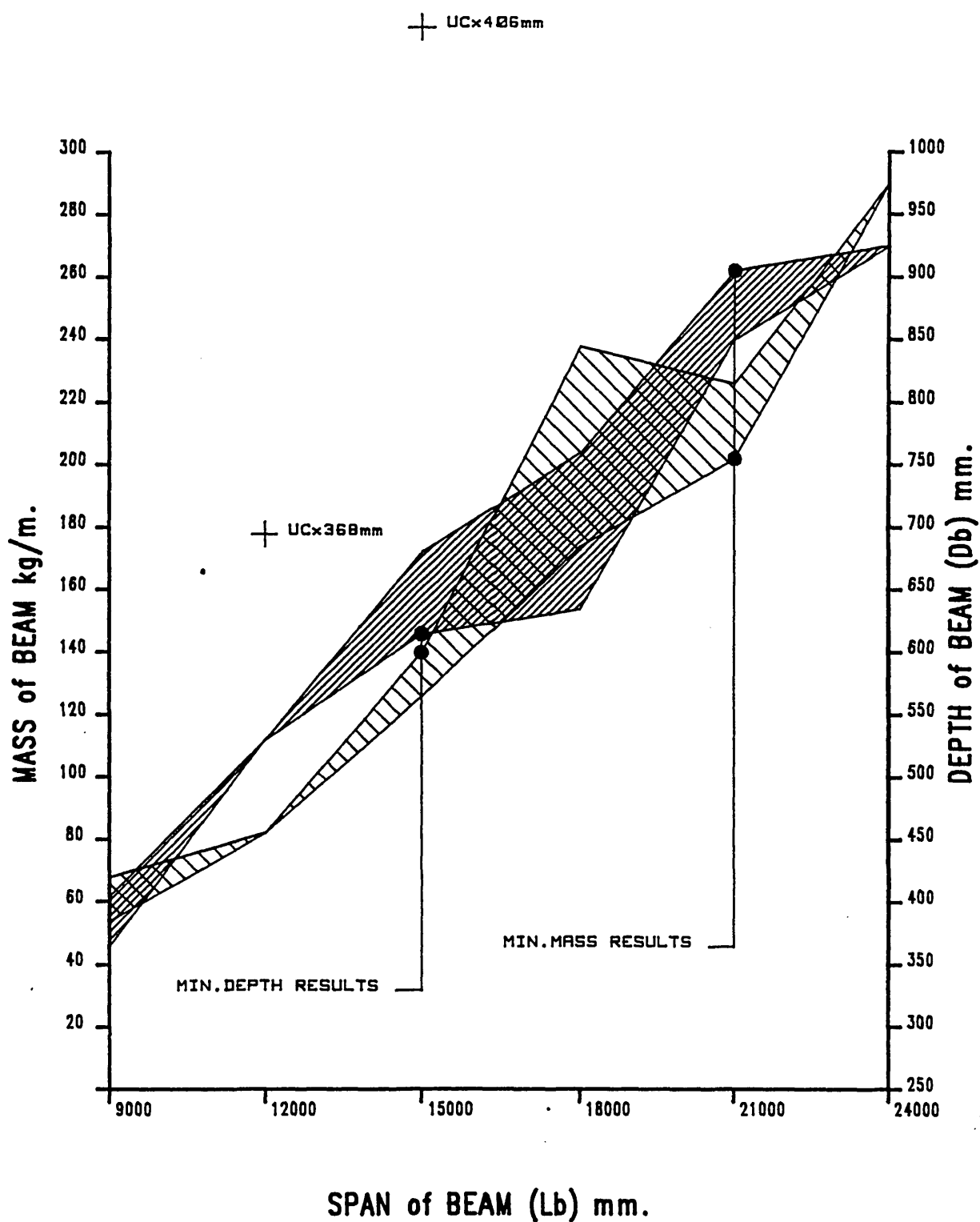
PLOT IN TERMS OF BEAM MASS.

PLOT IN TERMS OF BEAM DEPTH.

NOTE:- VALUES ARE TAKEN FROM PART 2:RESULTS OF DESIGN STUDIES
TABLE 2: DEFINITION of STUDIES.

MIN. MASS and DEPTH STUDIES on UNIVERSAL BEAMS
TYPE 2 ROOF BEAMS – Gr1 (PART 2)

FIG.31



PLOT IN TERMS OF BEAM MASS.

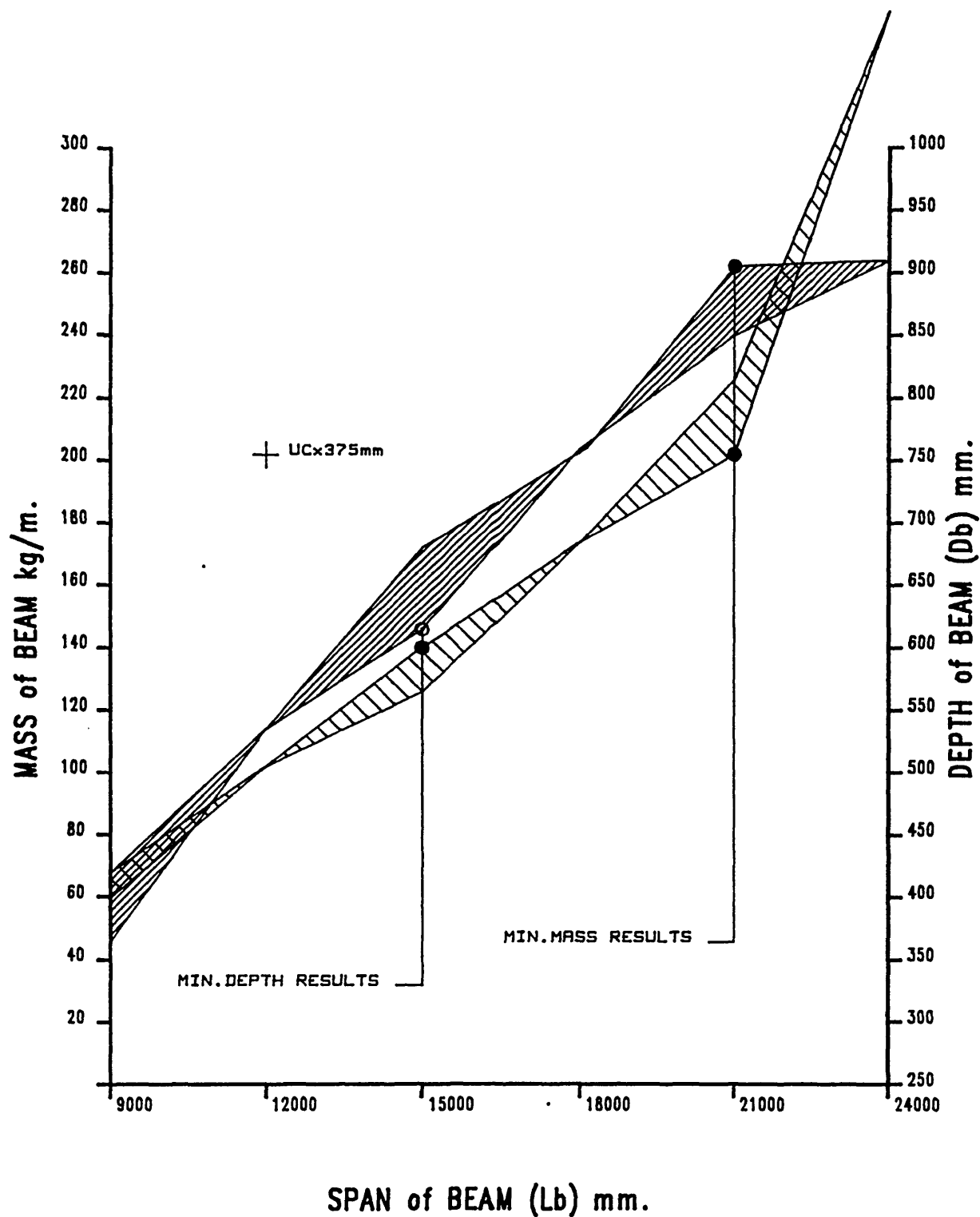


PLOT IN TERMS OF BEAM DEPTH.

NOTE:- VALUES ARE TAKEN FROM PART 2: RESULTS OF DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES.

MIN. MASS and DEPTH STUDIES on UNIVERSAL BEAMS
TYPE 2 FLOOR BEAMS – Gf1 (PART 2)

FIG.32



PLOT IN TERMS OF BEAM MASS.

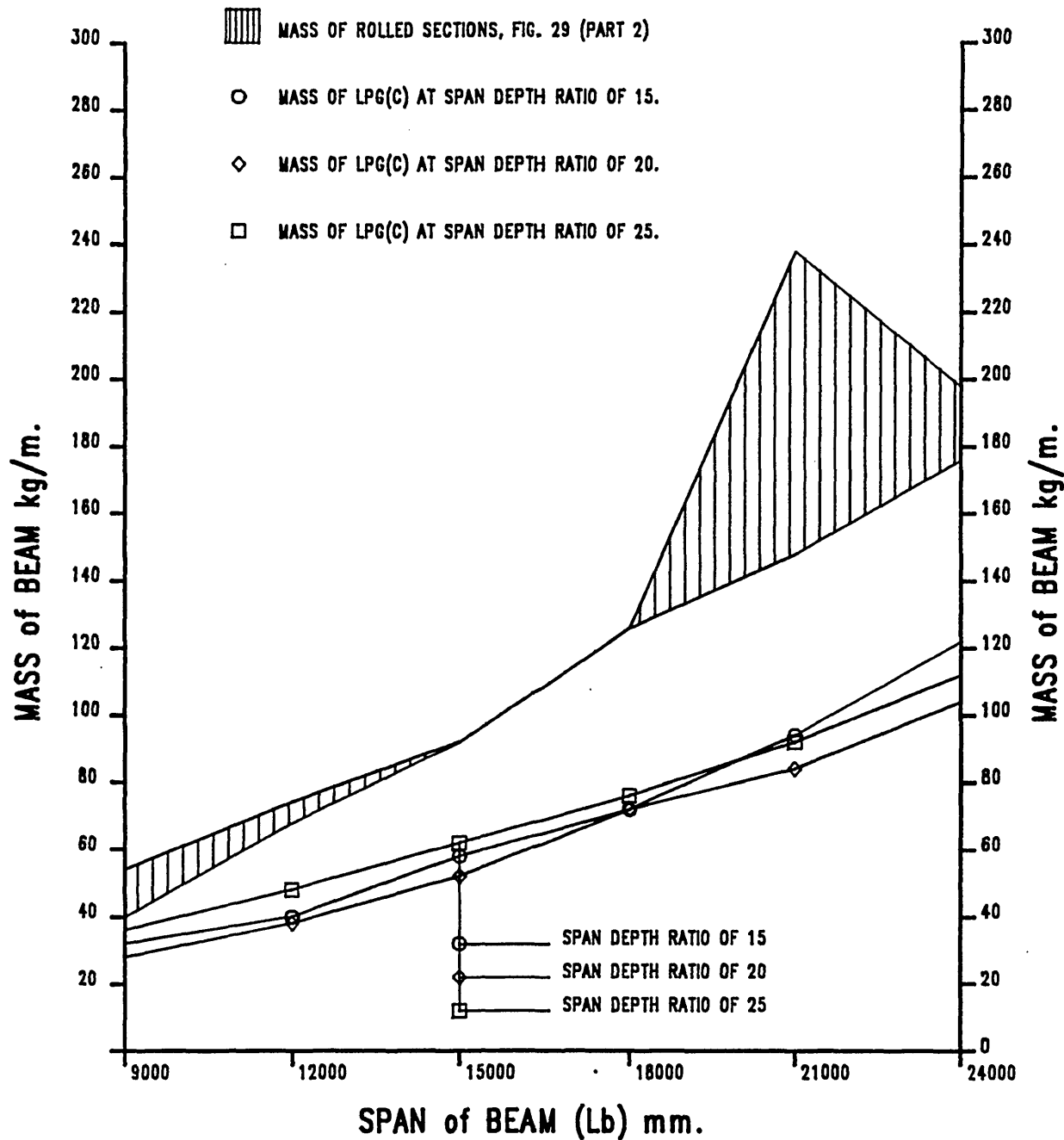


PLOT IN TERMS OF BEAM DEPTH.

NOTE:- VALUES ARE TAKEN FROM PART 2: RESULTS OF DESIGN STUDIES
TABLE 2: DEFINITION of STUDIES.

MASS of LPG(C) TYPE 1 ROOF BEAMS (Br1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 3)

FIG.33

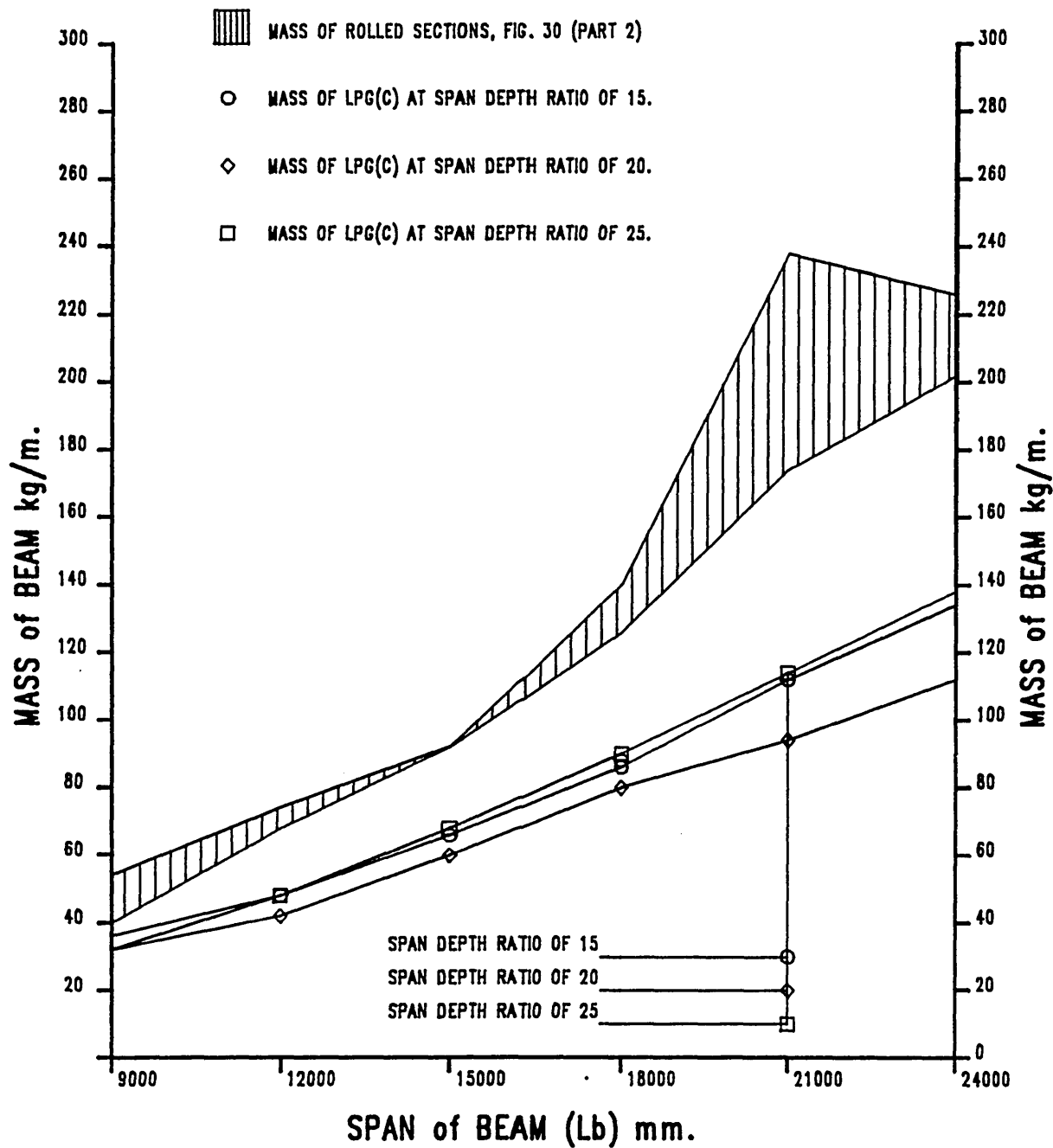


NOTES:- 1. VALUES FOR LPG(C) ARE TAKEN FROM PART 3: RESULTS OF DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES - ROOF BEAMS (Br1).

2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 29 (PART 2)

MASS of LPG(C) TYPE 1 FLOOR BEAMS (Bf1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 3)

FIG.34

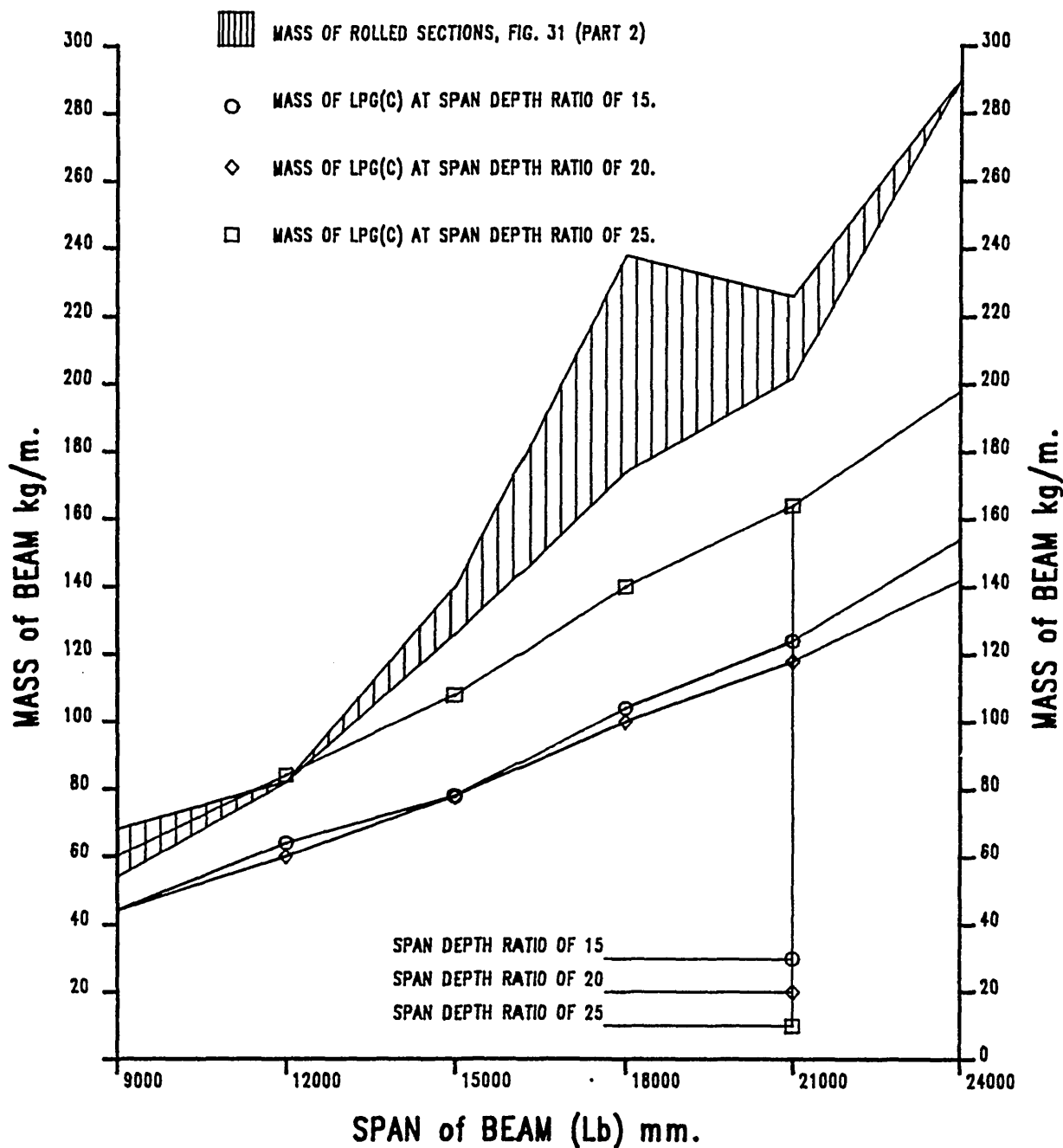


NOTES:- 1. VALUES FOR LPG(C) ARE TAKEN FROM PART 3: RESULTS OF DESIGN STUDIES
TABLE 2: DEFINITION OF STUDIES - FLOOR BEAMS (Bf1).

2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 30 (PART 2)

MASS of LPG(C) TYPE 2 ROOF BEAMS (Gr1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 3)

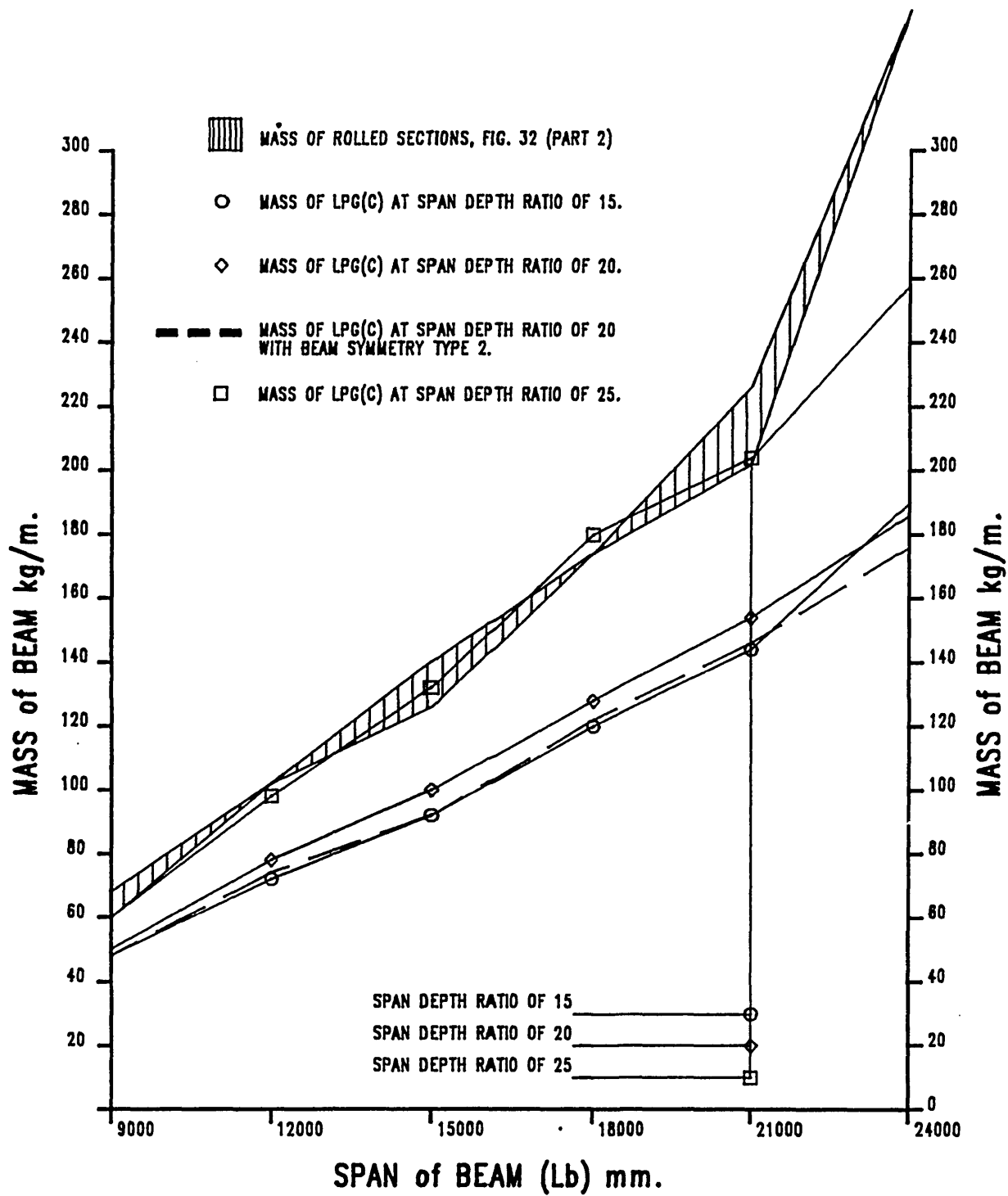
FIG.35



NOTES:- 1. VALUES FOR LPG(C)₂ ARE TAKEN FROM PART 3: RESULTS OF DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES - ROOF BEAMS (Gr1).
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 31 (PART 2)

MASS of LPG(C) TYPE 2 FLOOR BEAMS (Gf1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 3)

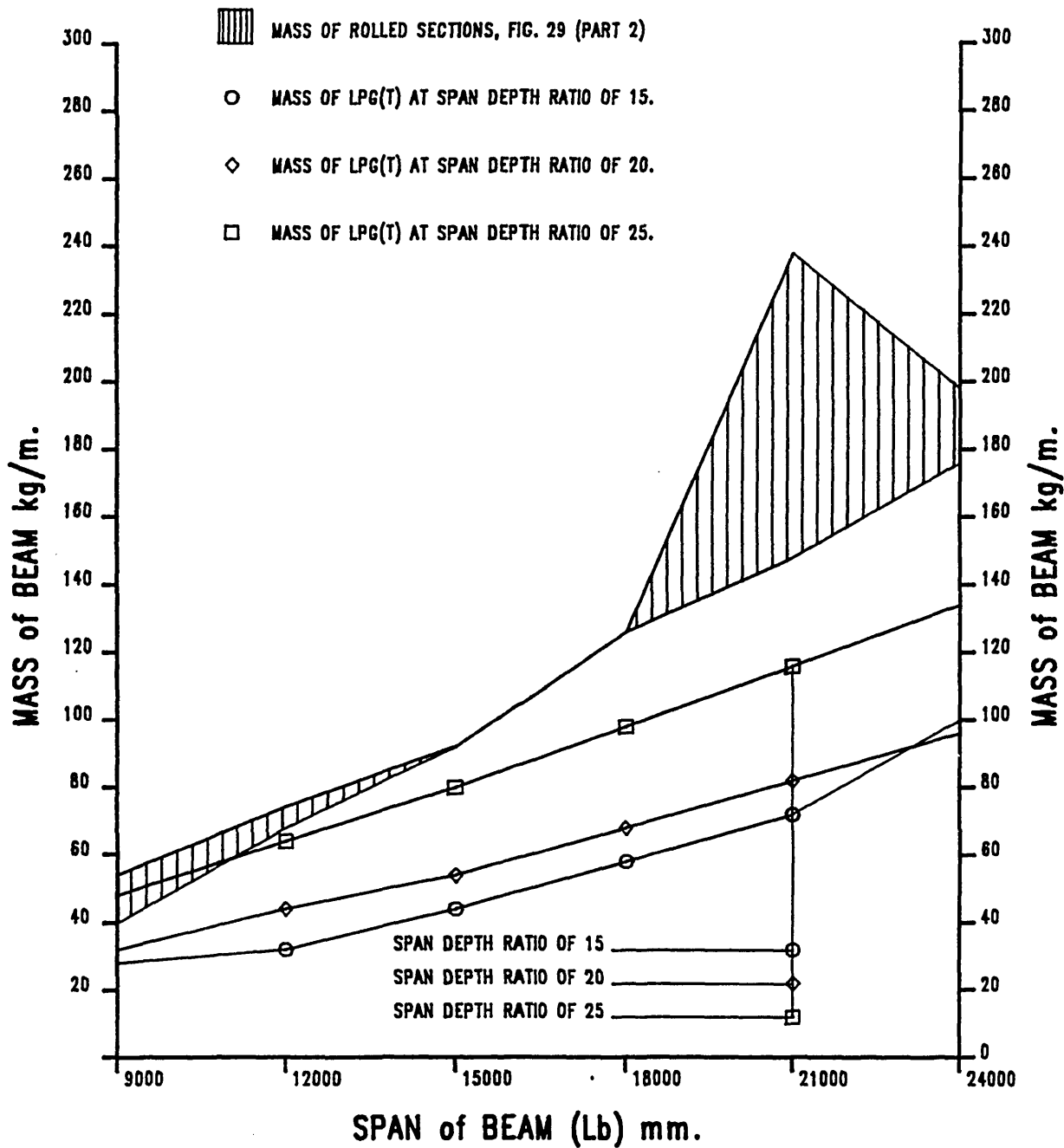
FIG.36



NOTES:- 1. VALUES FOR LPG(C) ARE TAKEN FROM PART 3: RESULTS OF DESIGN STUDIES
TABLE 2: DEFINITION of STUDIES - FLOOR BEAMS (Gf1).
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 32 (PART 2)

MASS of LPG(T) TYPE 1 ROOF BEAMS (Br1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 4)

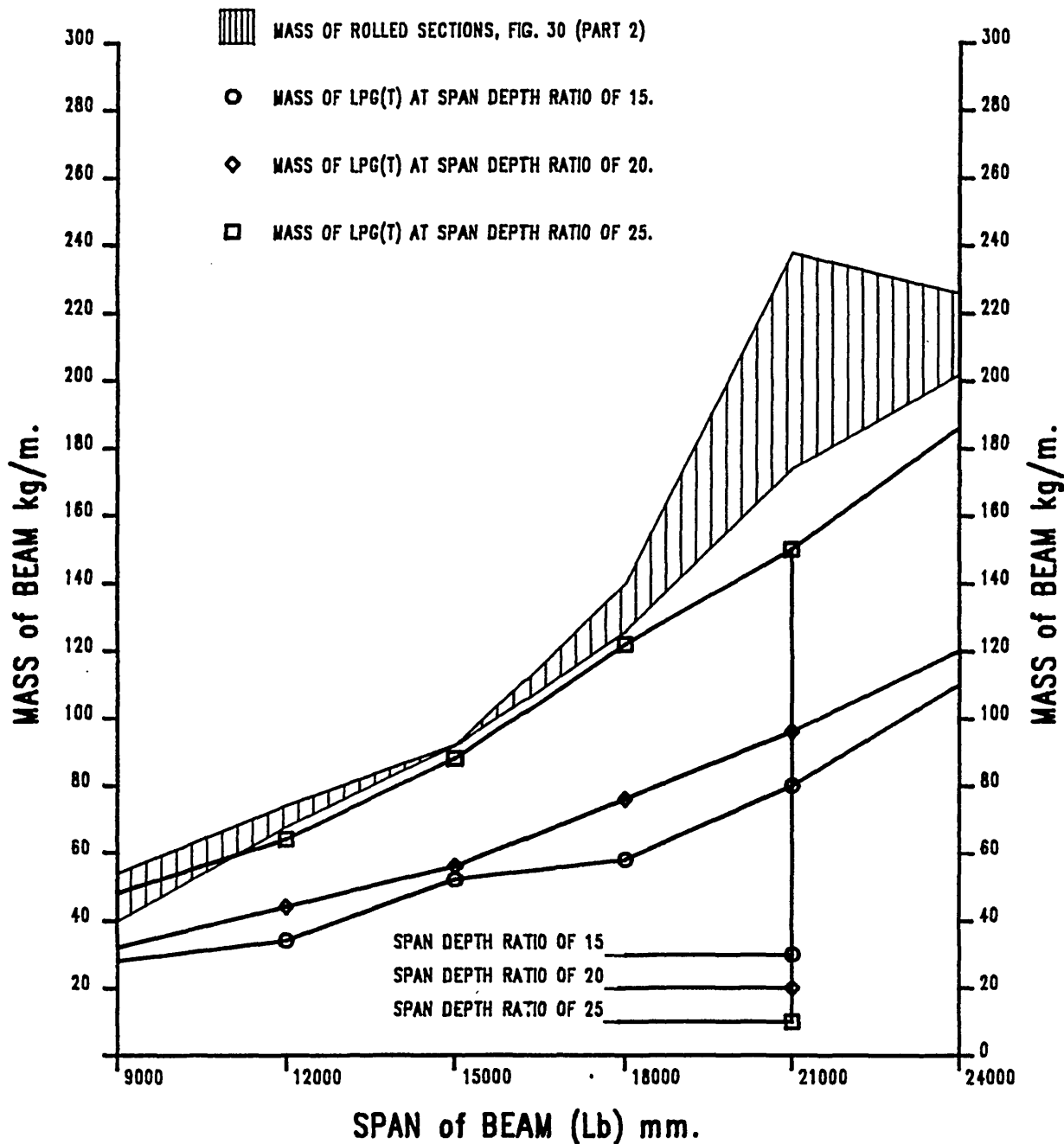
Fig.37



NOTES:- 1. VALUES FOR LPG(T)s ARE TAKEN FROM PART 4: RESULTS OF DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES - ROOF BEAMS (Br1).
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 29 (PART 2)

MASS of LPG(T) TYPE 1 FLOOR BEAMS (Bf1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 4)

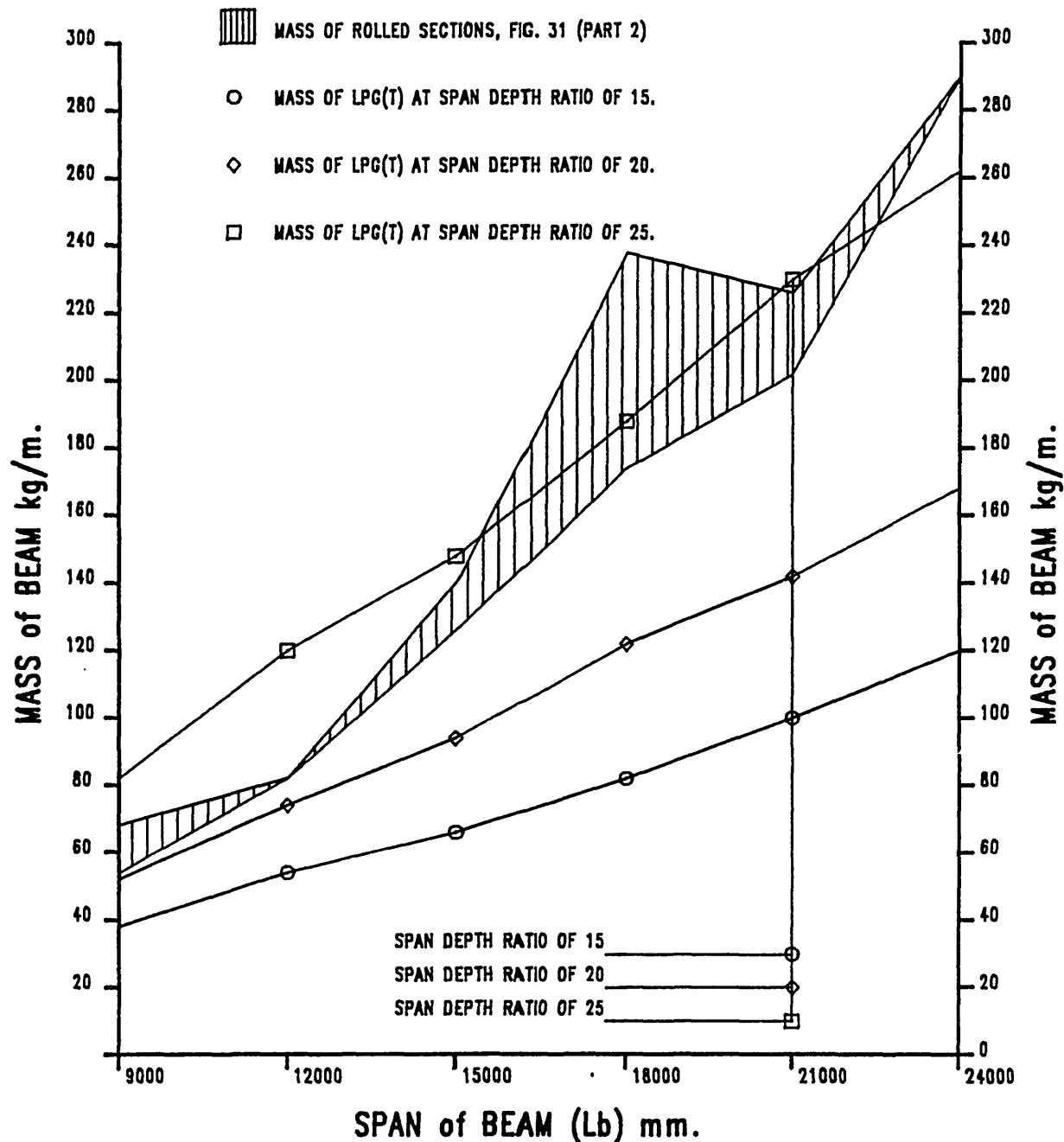
FIG.38



NOTES:- 1. VALUES FOR LPG(T) ARE TAKEN FROM PART 4: RESULTS OF DESIGN STUDIES
TABLE 2: DEFINITION of STUDIES - FLOOR BEAMS (Bf1).
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 30 (PART 2)

MASS of LPG(T) TYPE 2 ROOF BEAMS (Gr1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 4)

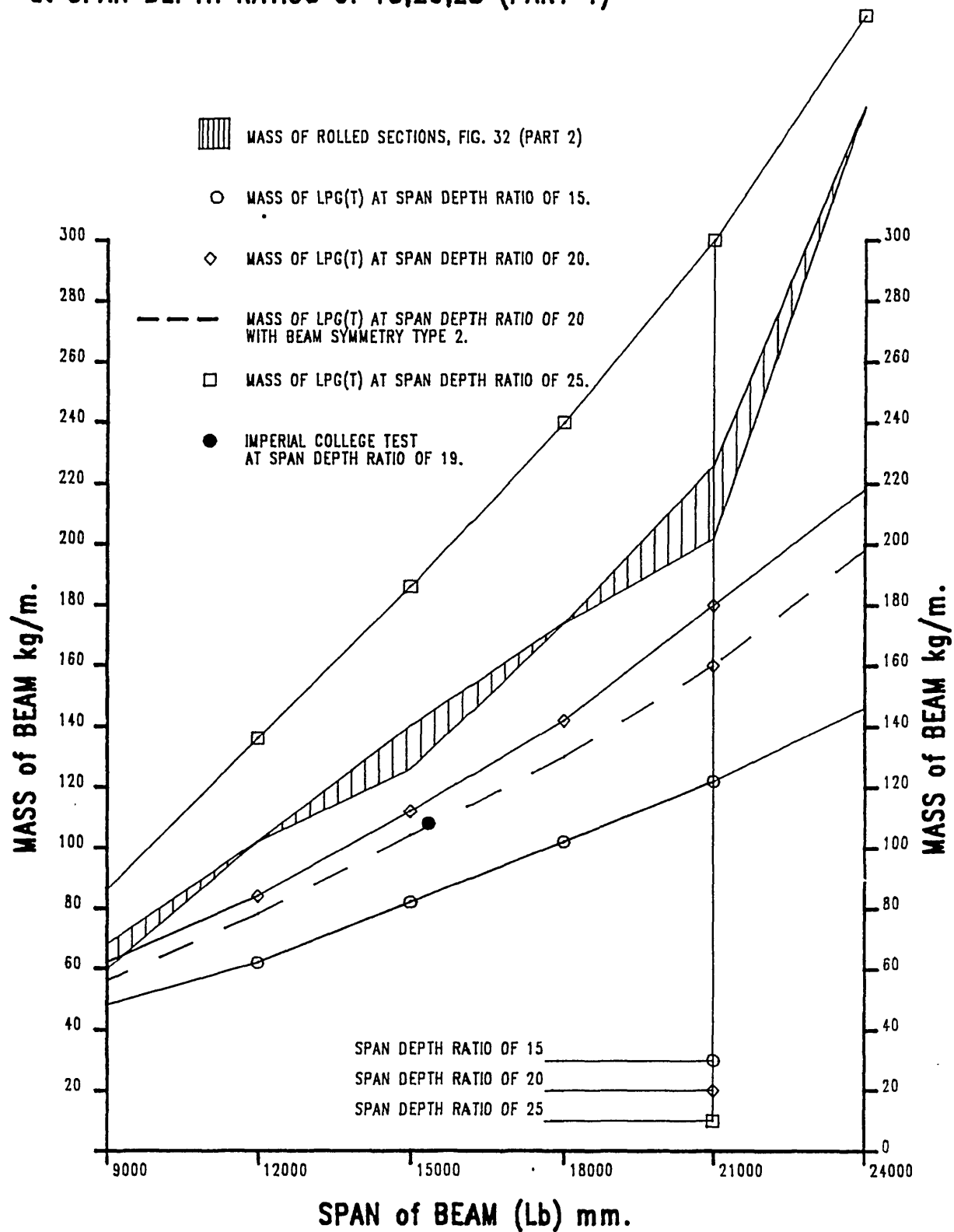
FIG.39



NOTES:- 1. VALUES FOR LPG(T) ARE TAKEN FROM PART 4: RESULTS OF DESIGN STUDIES.
TABLE 1: DEFINITION of STUDIES - ROOF BEAMS (Gr1).
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES PLOTTED ABOVE ARE GIVEN IN FIG. 31 (PART 2)

MASS of LPG(T) TYPE 2 FLOOR BEAMS (Gf1) at SPAN DEPTH RATIOS of 15,20,25 (PART 4)

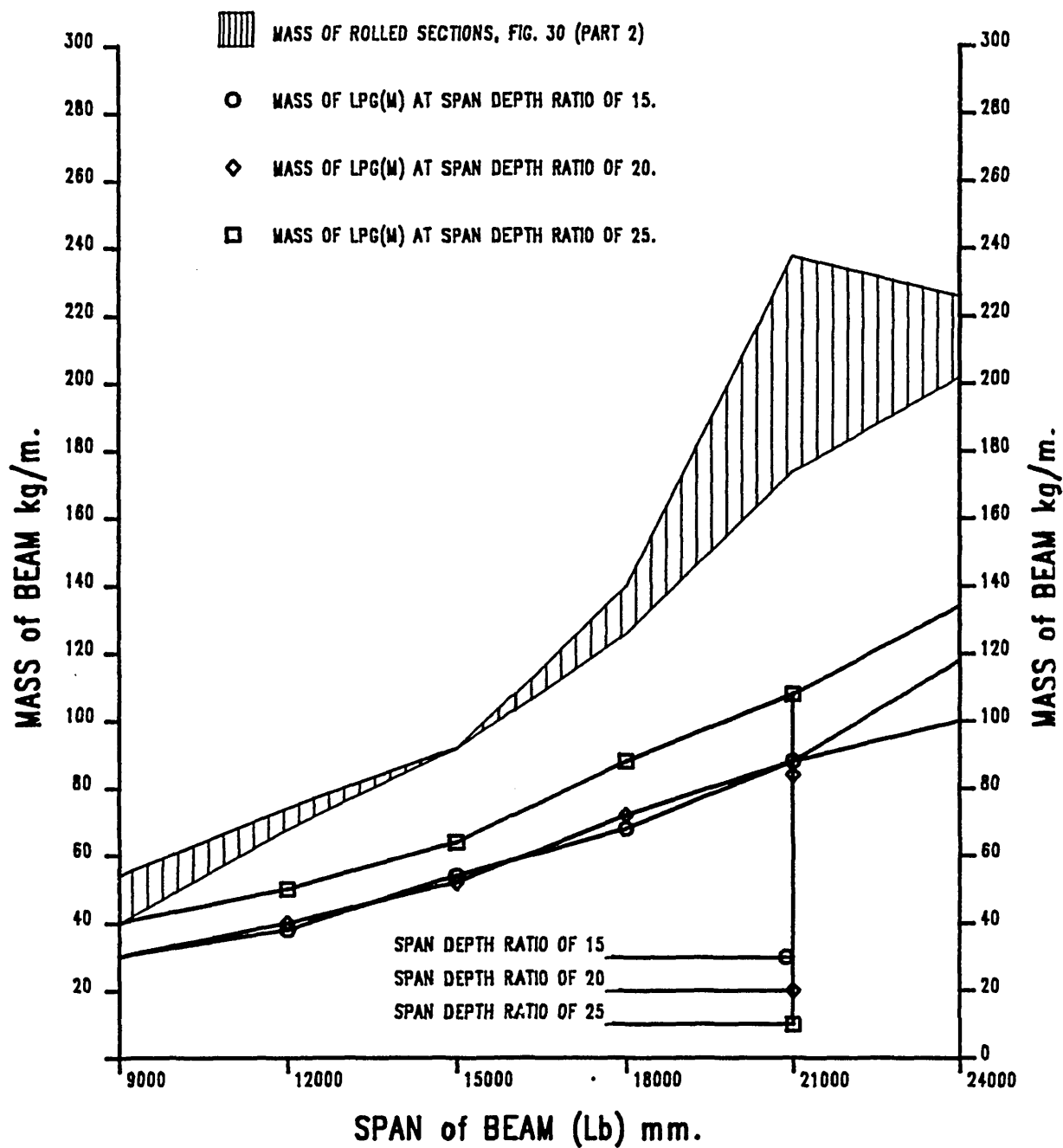
FIG.40



NOTES:- 1. VALUES FOR LPG(T)_s ARE TAKEN FROM PART 4: RESULTS OF DESIGN STUDIES
TABLE 2: DEFINITION OF STUDIES - FLOOR BEAMS (Gf1).
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 32 (PART 2)

MASS of LPG(M) TYPE 1 FLOOR BEAMS (Bf1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 5)

FIG.41

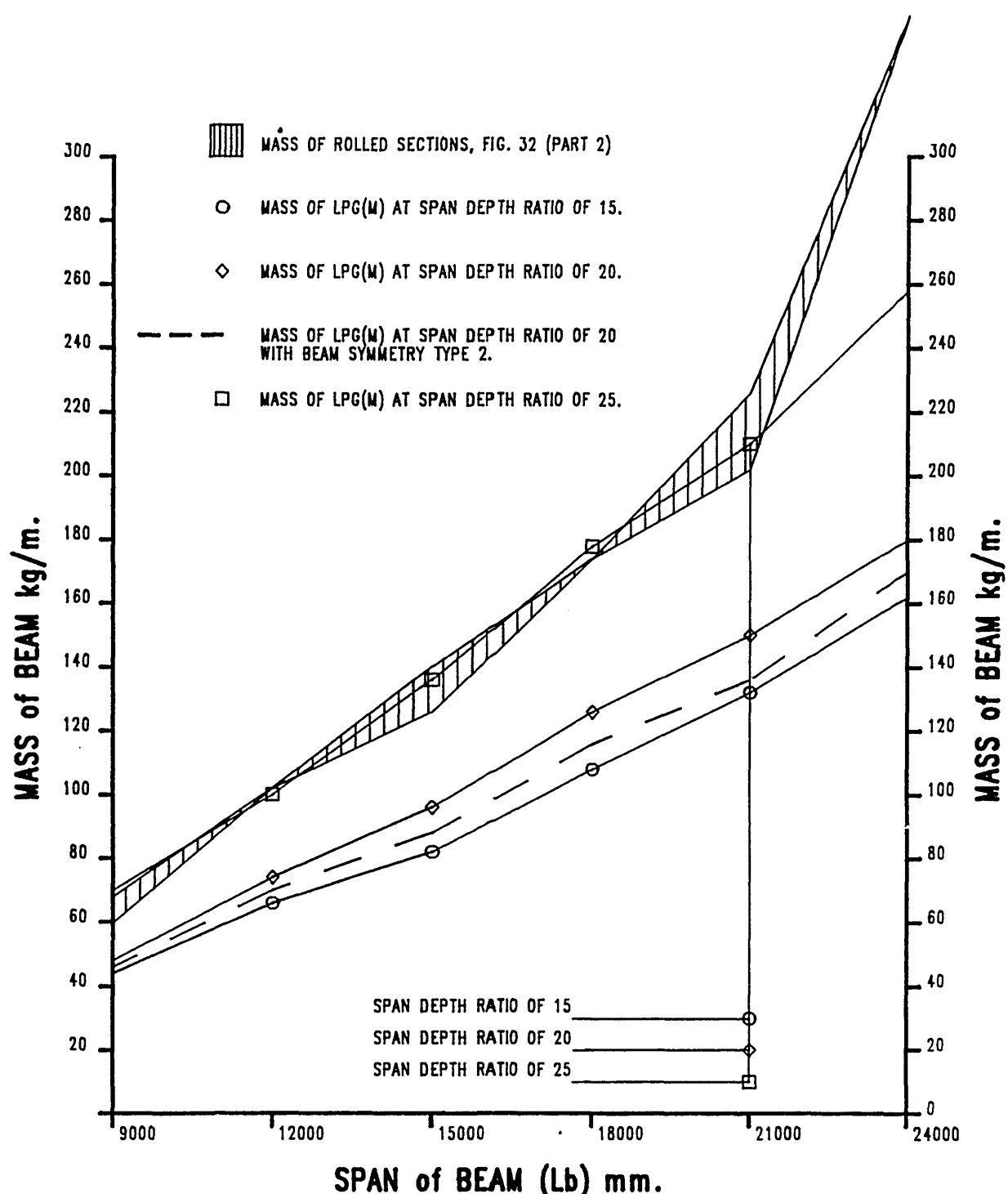


NOTES:- 1. VALUES FOR LPG(M)s ARE TAKEN FROM PART 5: RESULTS OF DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES - FLOOR BEAMS (Bf1).

2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 30 (PART 2)

**MASS of LPG(M) TYPE 2 FLOOR BEAMS (Gf1)
at SPAN DEPTH RATIOS of 15,20,25 (PART 5)**

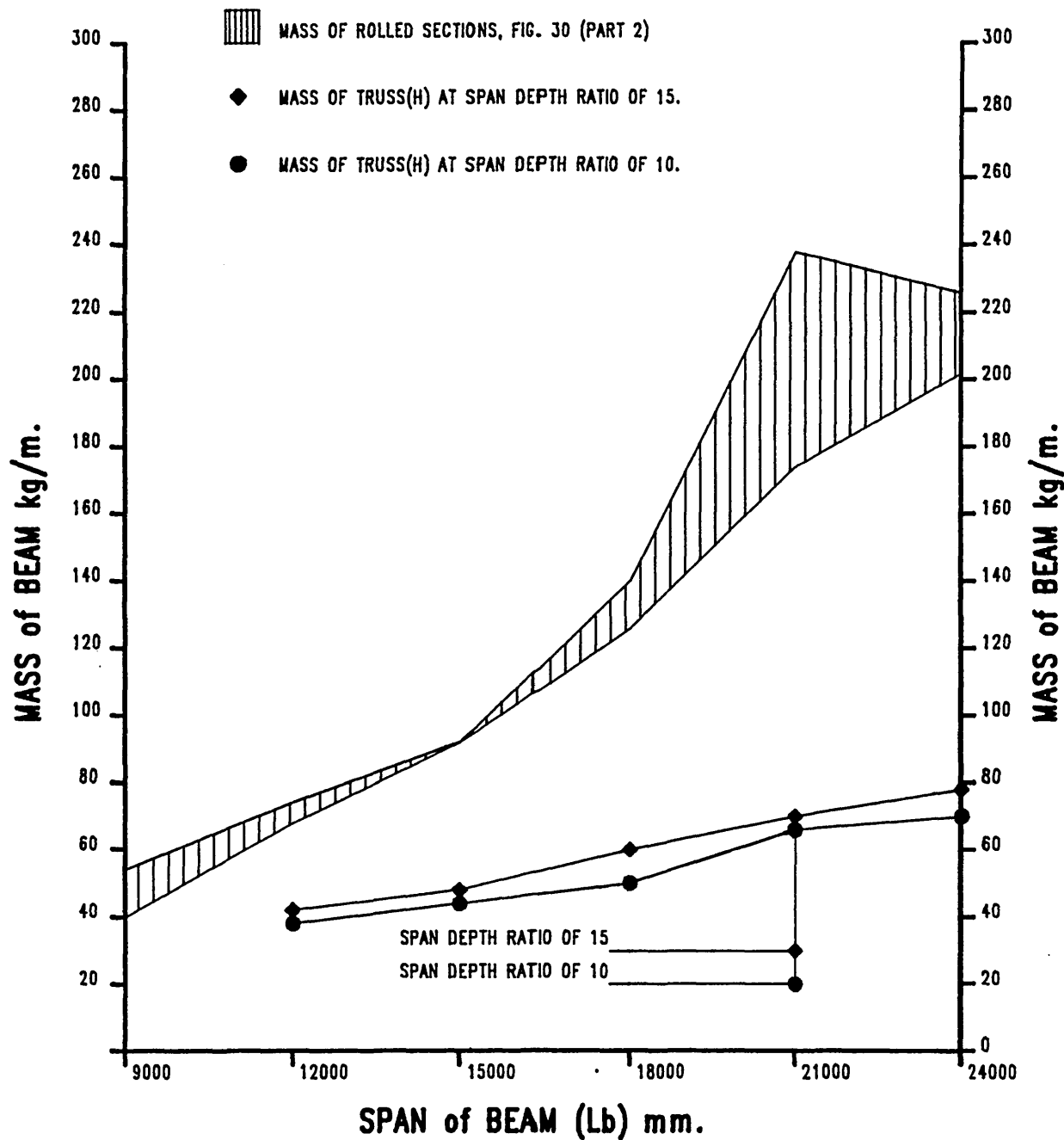
FIG.42



NOTES:- 1. VALUES FOR LPG(M)_s ARE TAKEN FROM PART 5: RESULTS OF DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES - FLOOR BEAMS (Gf1).
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 32 (PART 2)

**MASS OF TRUSS(H) TYPE 1 FLOOR BEAMS (Bf1)
at SPAN DEPTH RATIOS of 10 and 15 (PART 8).**

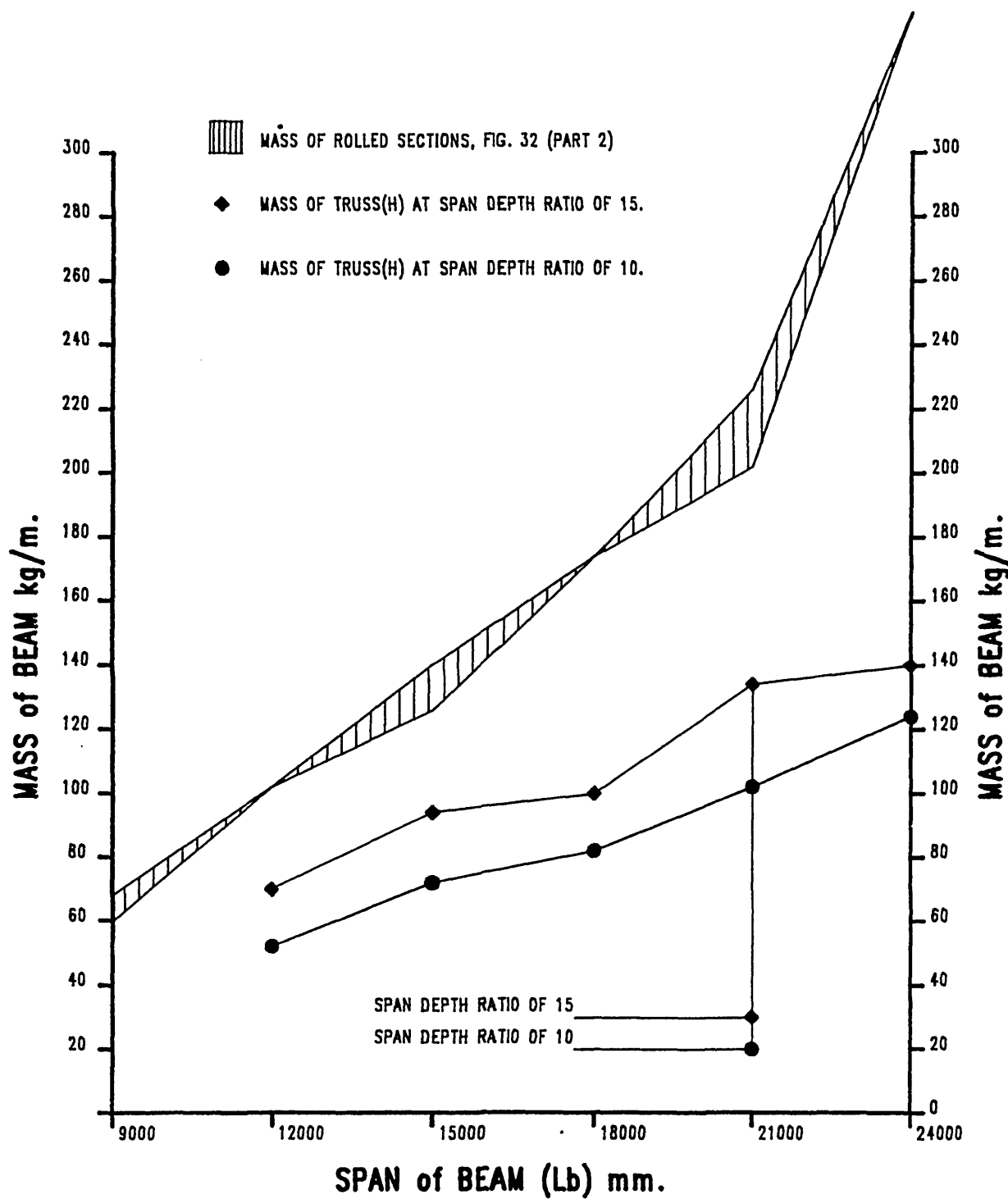
FIG.43



NOTES:- 1. VALUES FOR TRUSS(H) ARE TAKEN FROM PART 8: RESULTS OF DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES - TYPE 1 AND 2 FLOOR TRUSSES.
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 30 (PART 2)

**MASS of TRUSS(H) TYPE 2 FLOOR BEAMS (Gf1)
at SPAN DEPTH RATIOS of 10 and 15 (PART 8)**

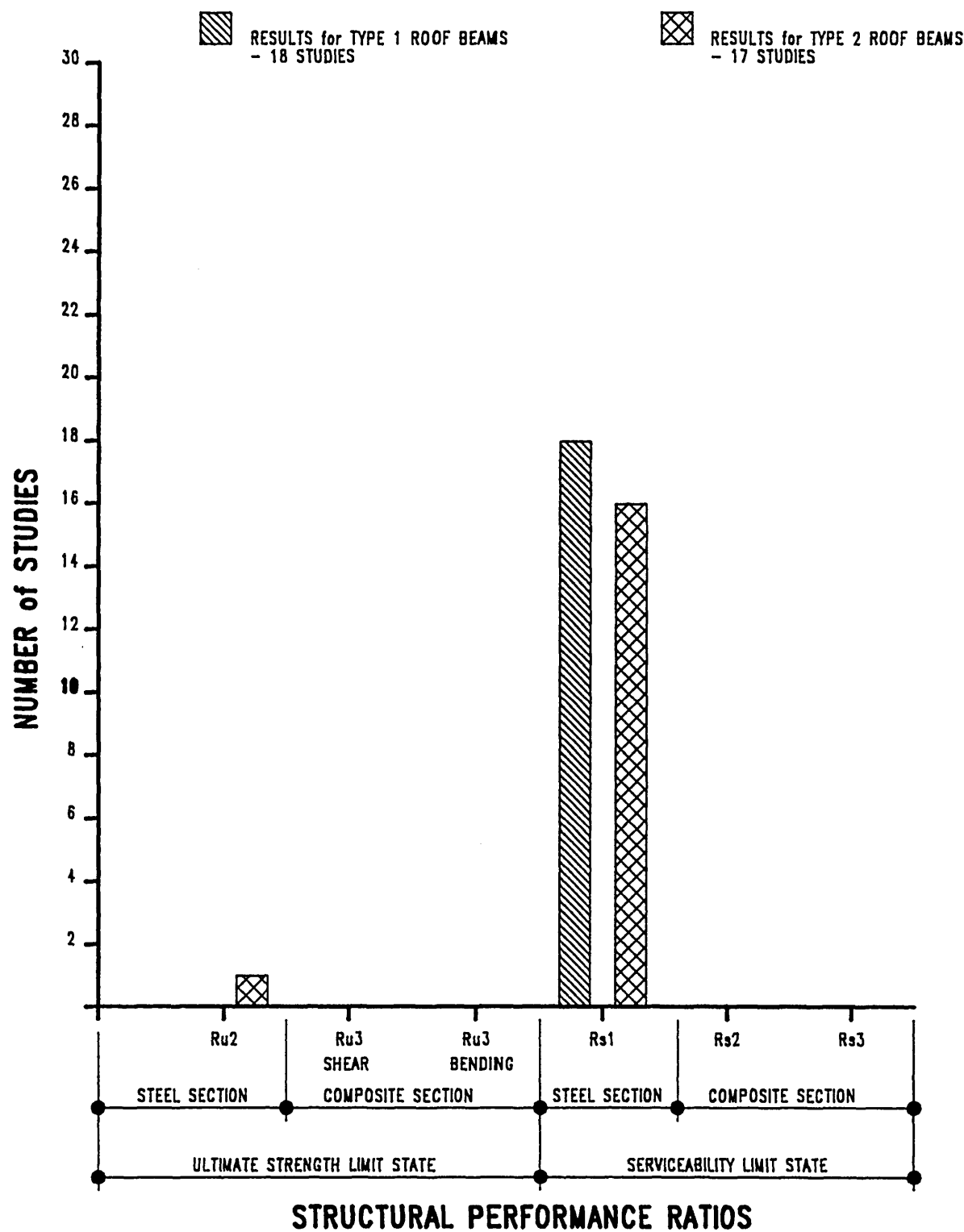
FIG.44



NOTES:- 1. VALUES FOR TRUSS(H) ARE TAKEN FROM PART 8: RESULTS OF DESIGN STUDIES
TABLE 1: DEFINITION of STUDIES - TYPE 1 AND 2 FLOOR TRUSSES.
2. ROLLED SECTION DEPTHS CORRESPONDING TO THE MASSES
PLOTTED ABOVE ARE GIVEN IN FIG. 32 (PART 2)

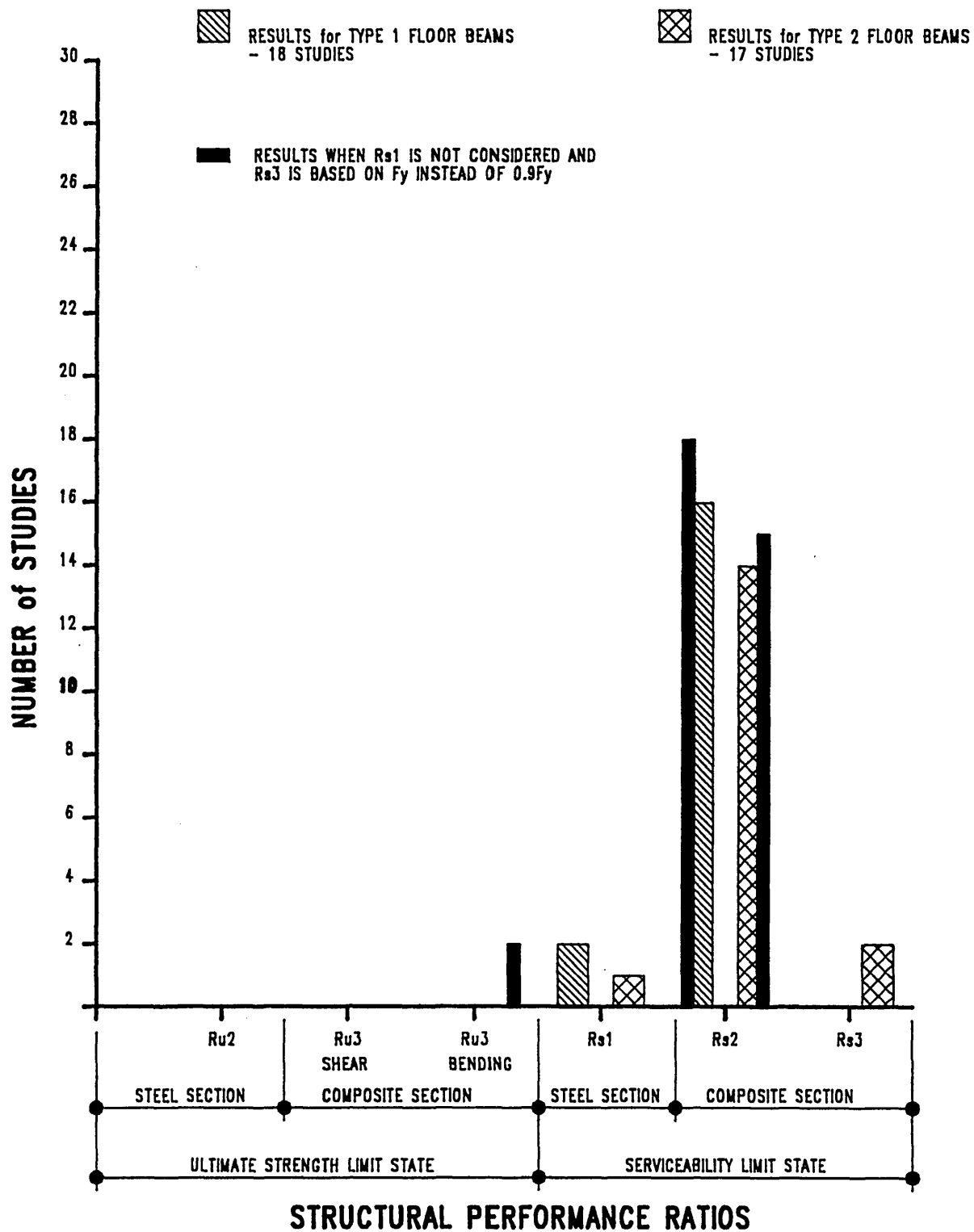
BAR CHART of GOVERNING STRUCTURAL PERFORMANCE RATIOS
for UNIVERSAL SECTIONS TYPE 1 & 2 ROOF BEAMS (PART 2)

FIG.45



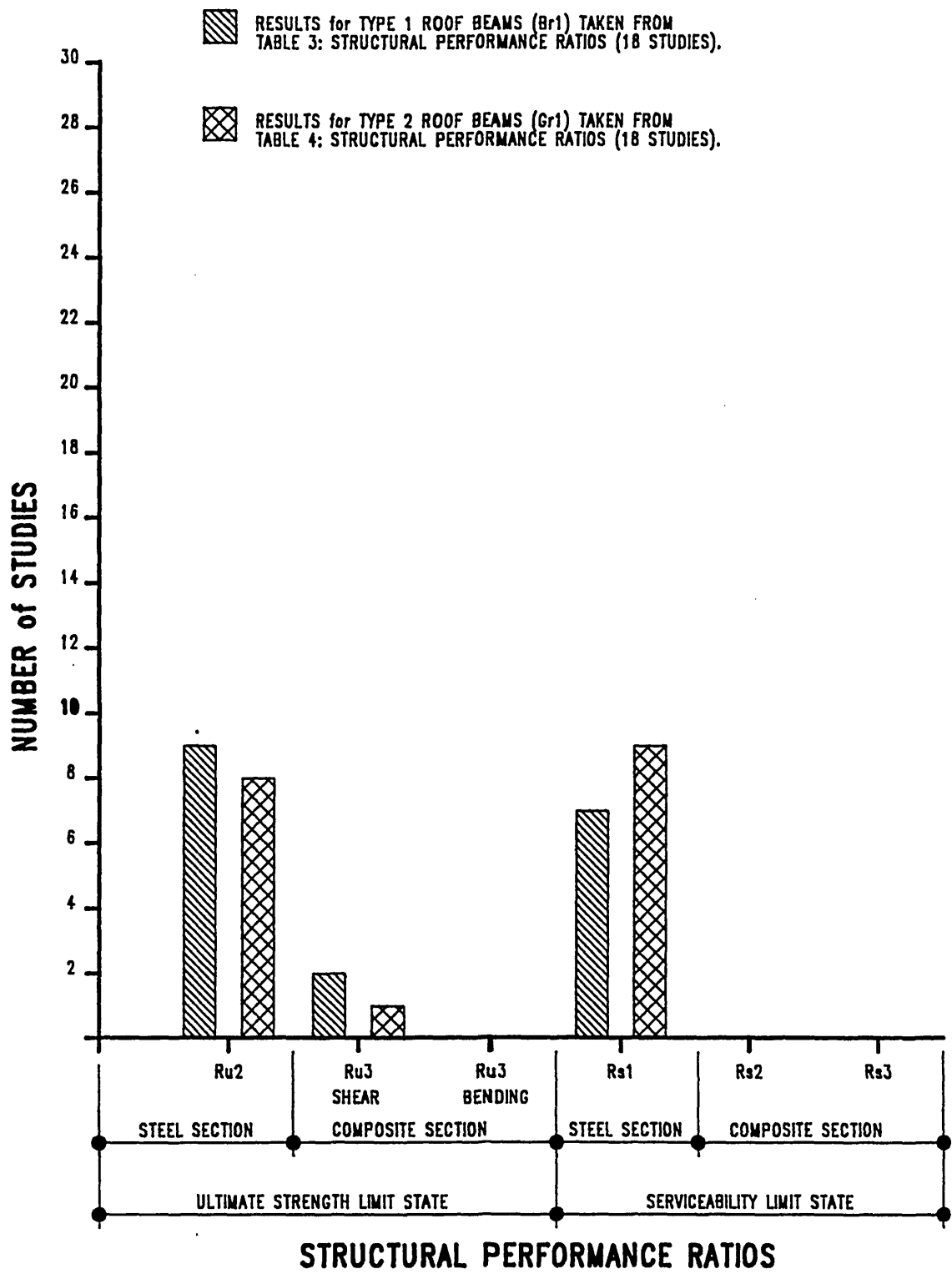
BAR CHART of GOVERNING STRUCTURAL PERFORMANCE RATIOS
for UNIVERSAL SECTIONS TYPE 1 & 2 FLOOR BEAMS (PART 2)

FIG.46



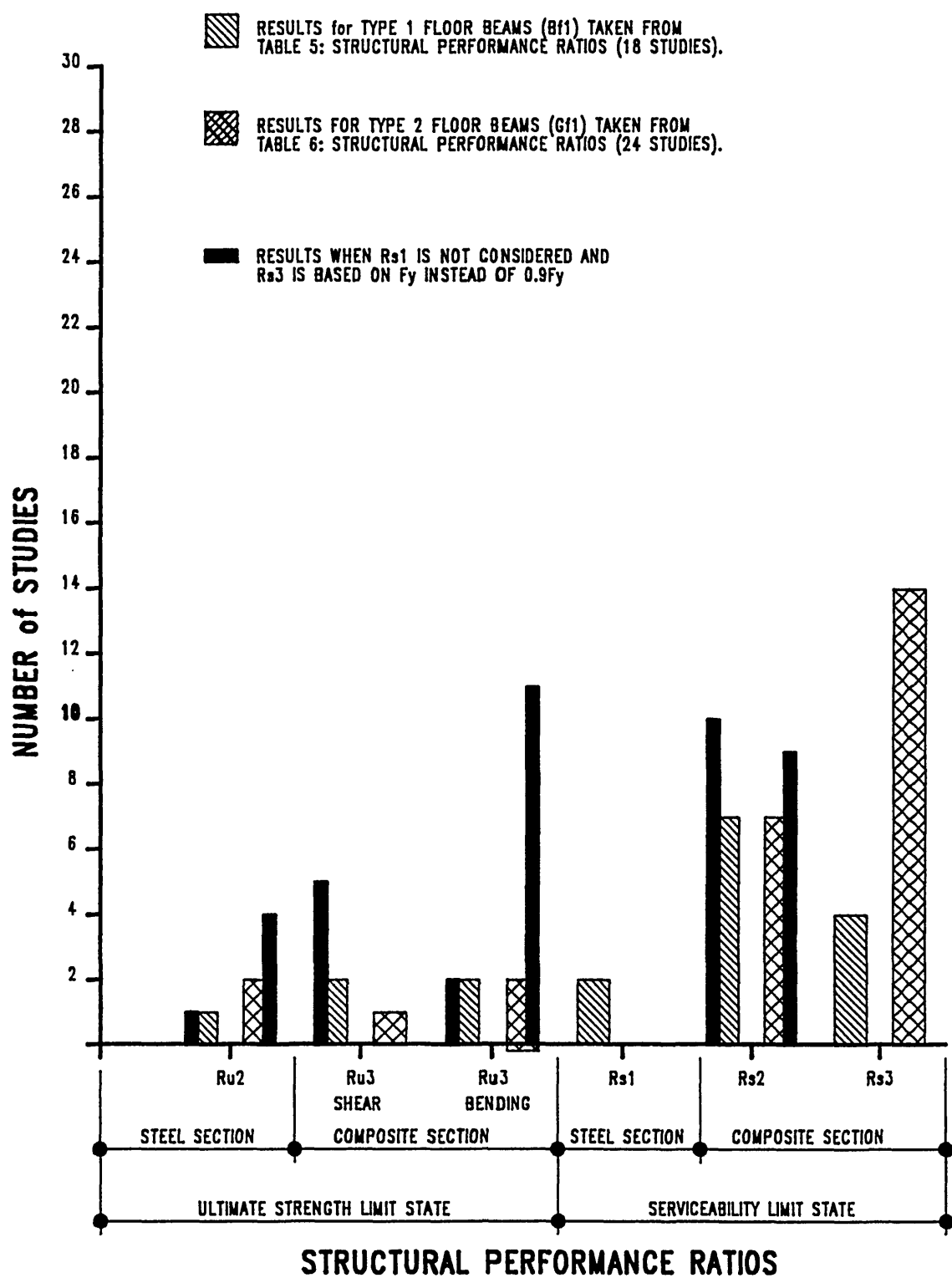
BAR CHART of GOVERNING STRUCTURAL PERFORMANCE RATIOS
for LPG(C) TYPE 1 & 2 ROOF BEAMS (PART 3)

FIG.47



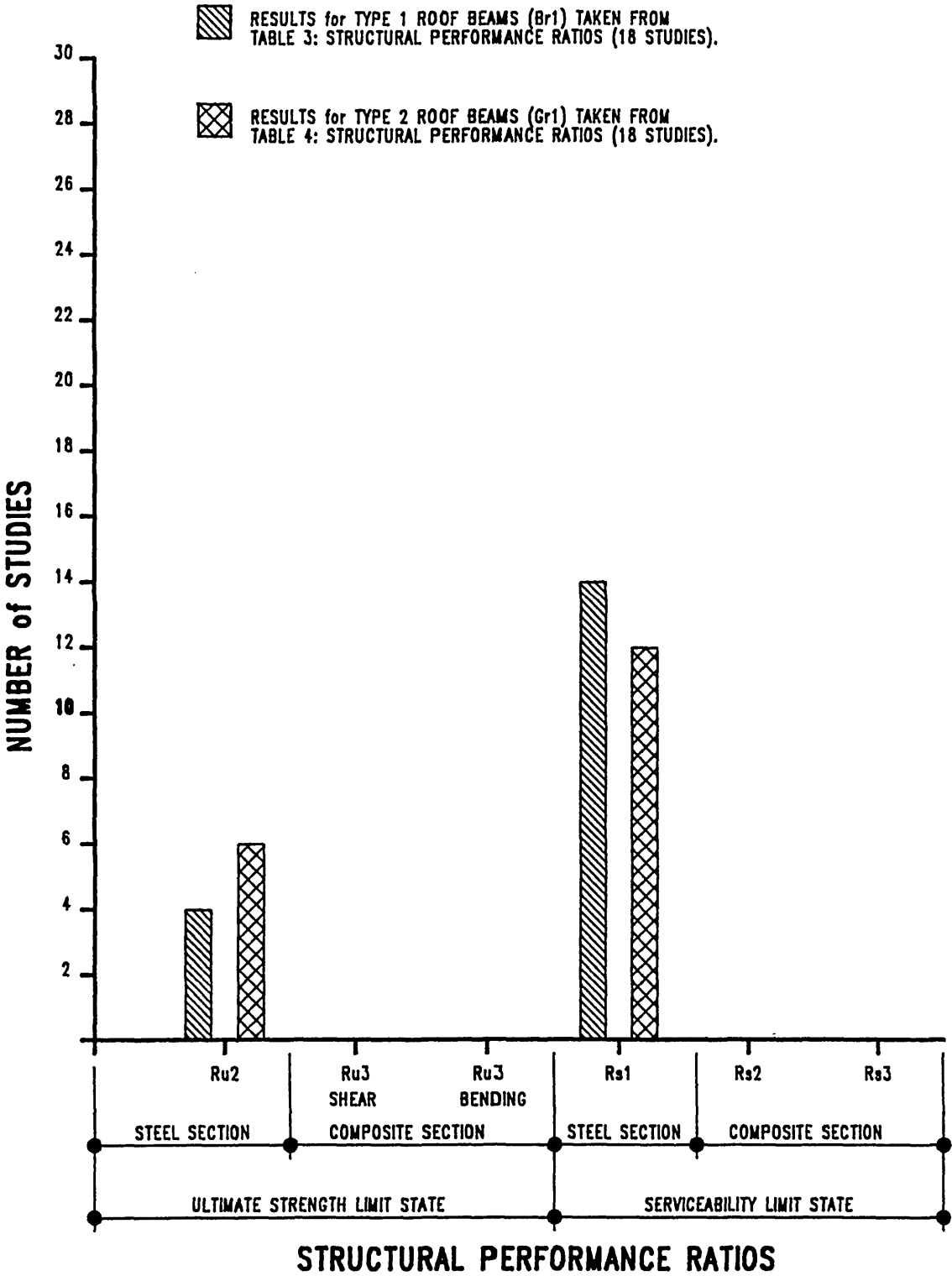
BAR CHART of GOVERNING STRUCTURAL PERFORMANCE RATIOS
for LPG(C) TYPE 1 & 2 FLOOR BEAMS (PART 3)

FIG.48



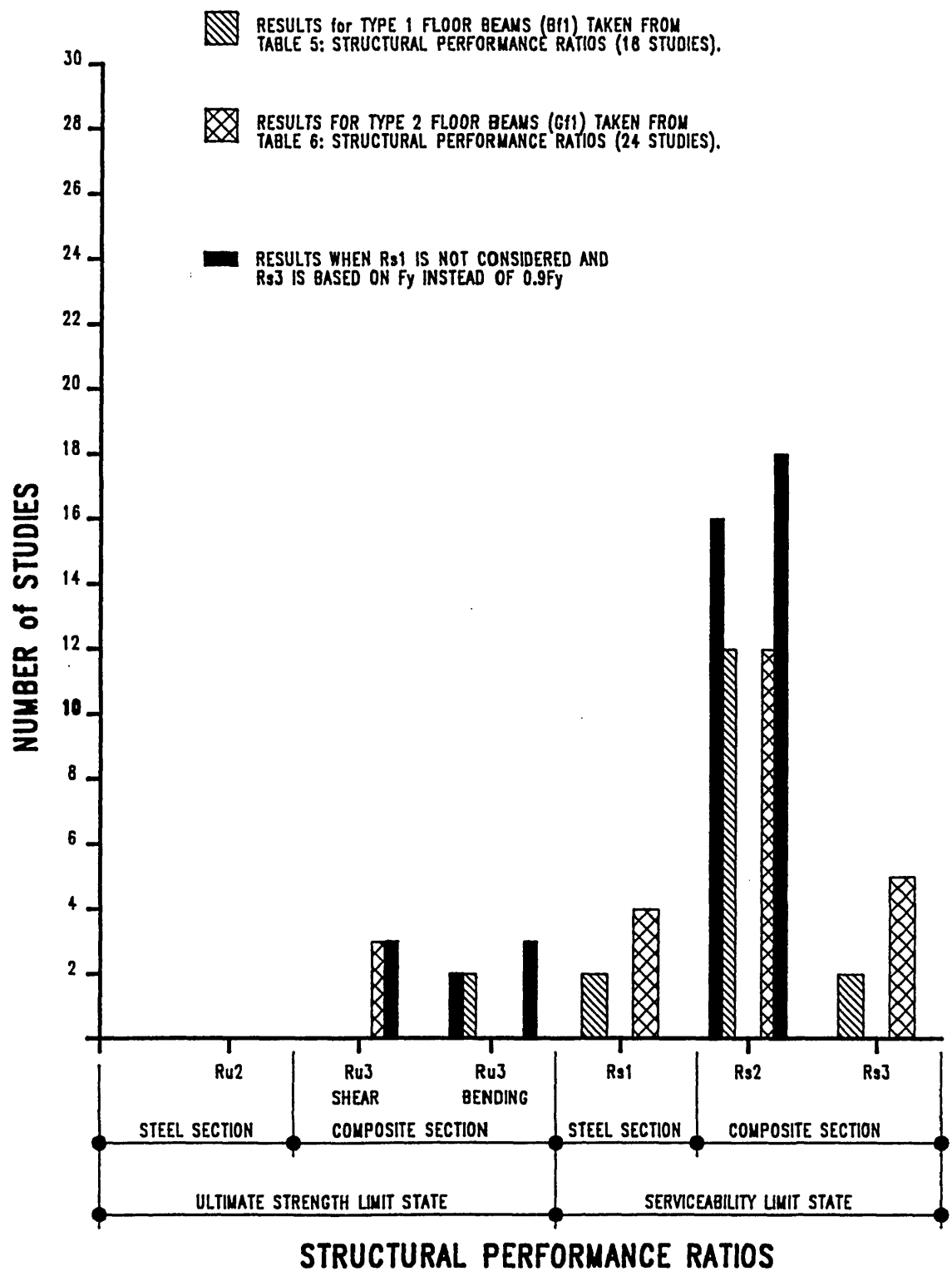
**BAR CHART of GOVERNING STRUCTURAL PERFORMANCE RATIOS
for LPG(T) TYPE 1 & 2 ROOF BEAMS (PART 4)**

FIG.49



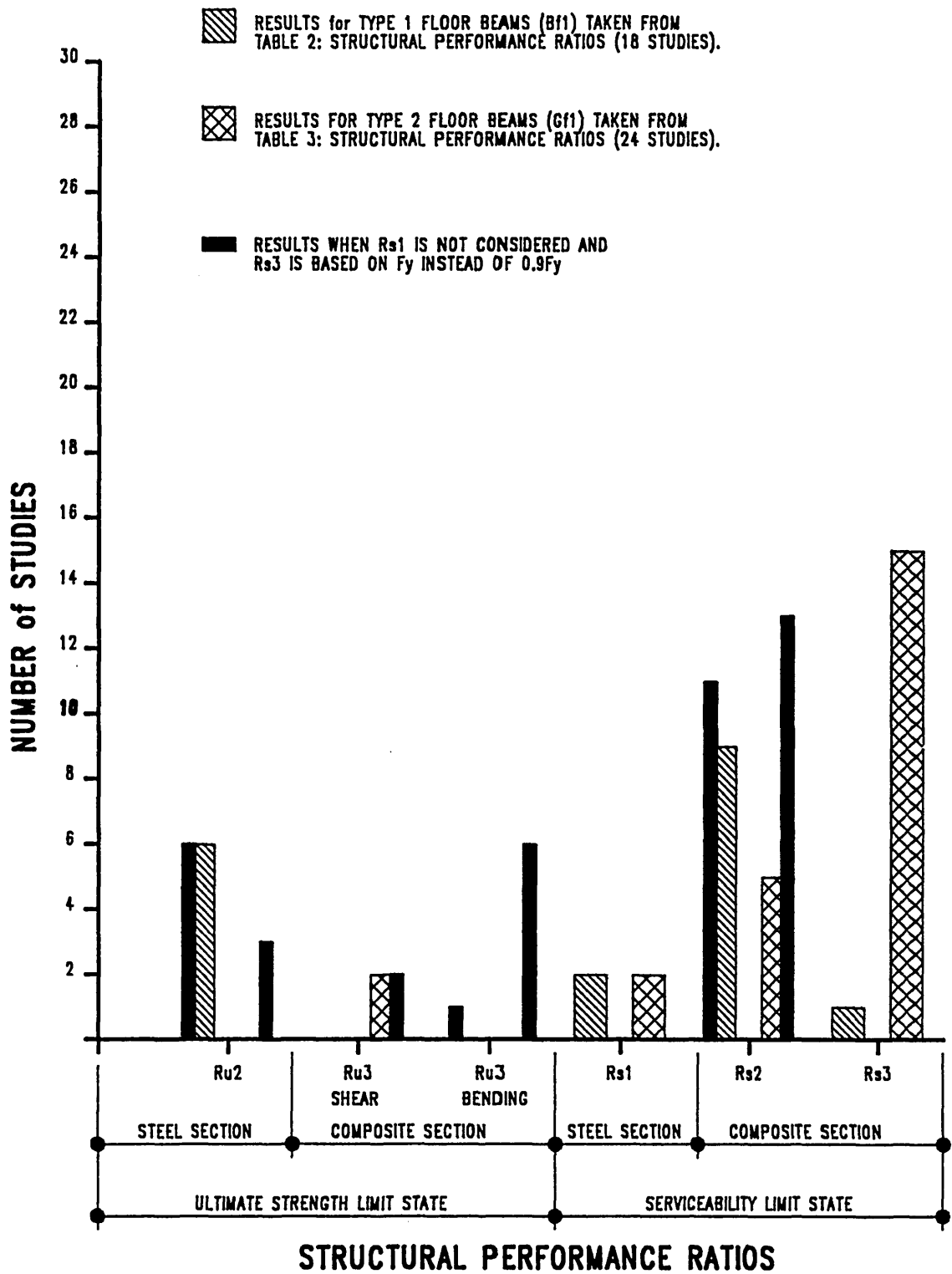
BAR CHART of GOVERNING STRUCTURAL PERFORMANCE RATIOS
for LPG(T) TYPE 1 & 2 FLOOR BEAMS (PART 4)

FIG.50



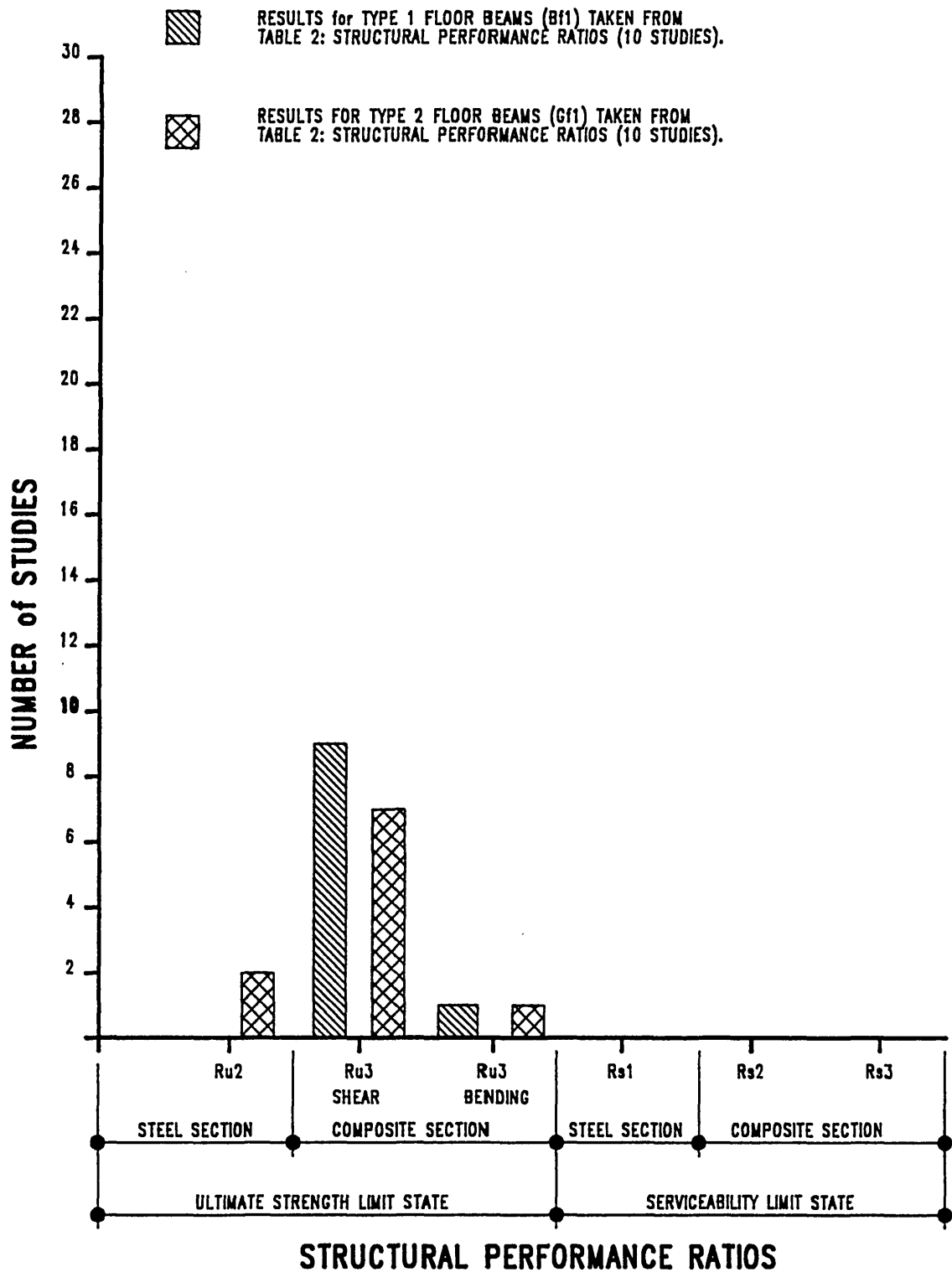
**BAR CHART of GOVERNING STRUCTURAL PERFORMANCE RATIOS
for LPG(M) TYPE 1 & 2 FLOOR BEAMS (PART 5)**

FIG.51



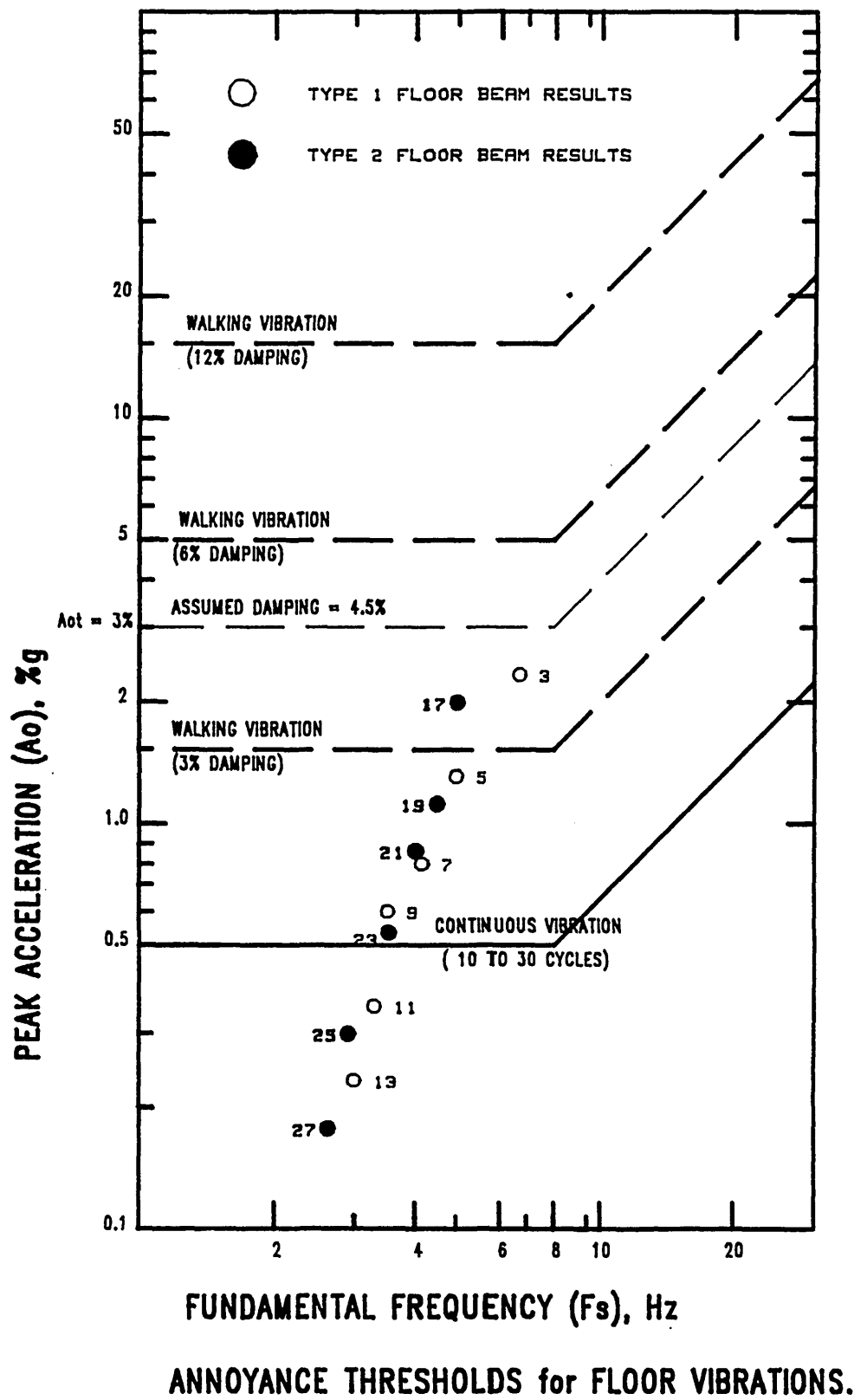
BAR CHART of GOVERNING STRUCTURAL PERFORMANCE RATIOS
for TRUSS(H) TYPE 1 & 2 FLOOR BEAMS (PART 8)

FIG.52



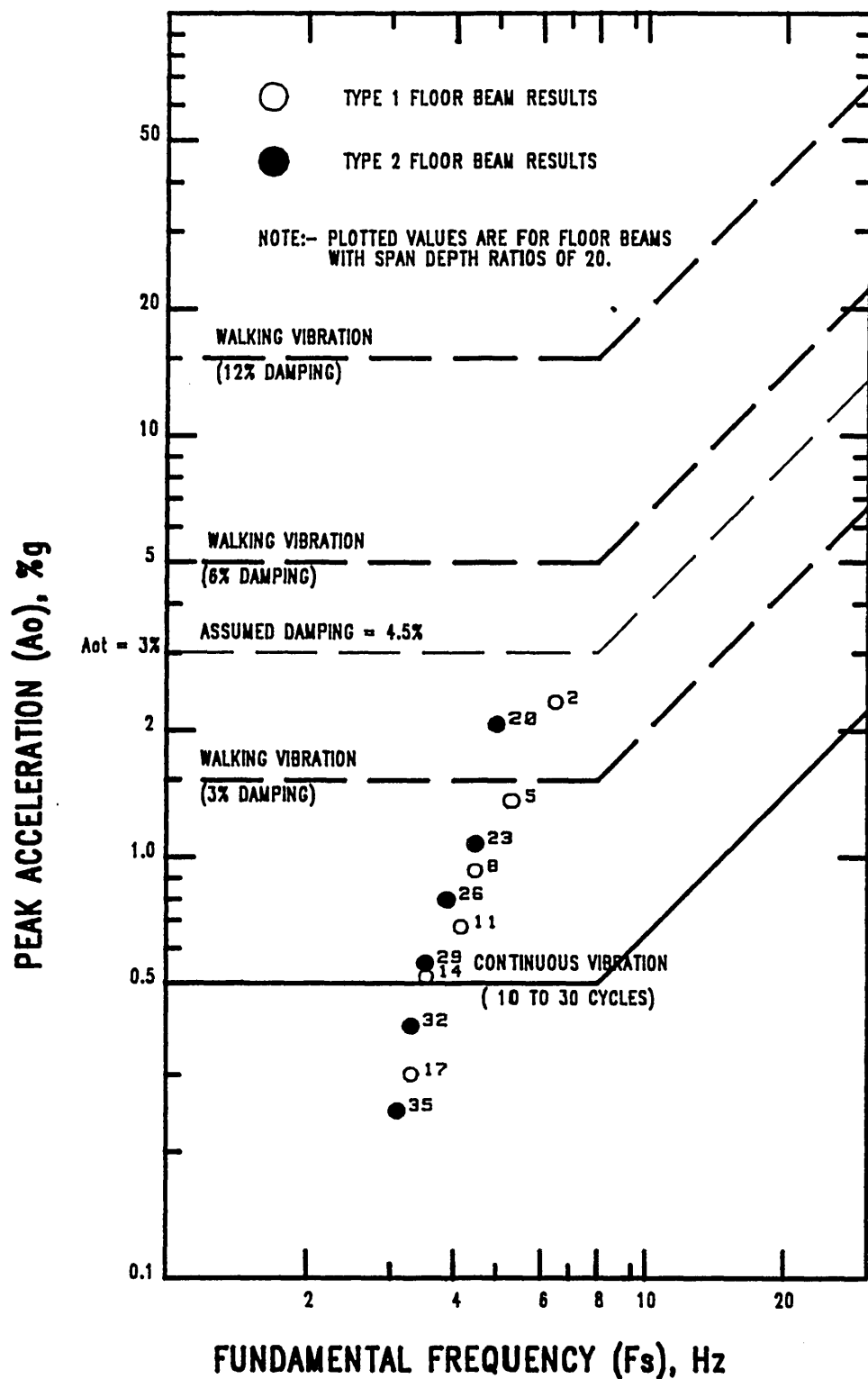
VIBRATION ASSESSMENT of UNIVERSAL SECTIONS
TYPE 1 and 2 FLOOR BEAMS (PART 2)

FIG.53



VIBRATION ASSESSMENT of LIGHT PLATE GIRDERS
of CONSTANT DEPTH TYPE 1 & 2 FLOOR BEAMS (PART 3)

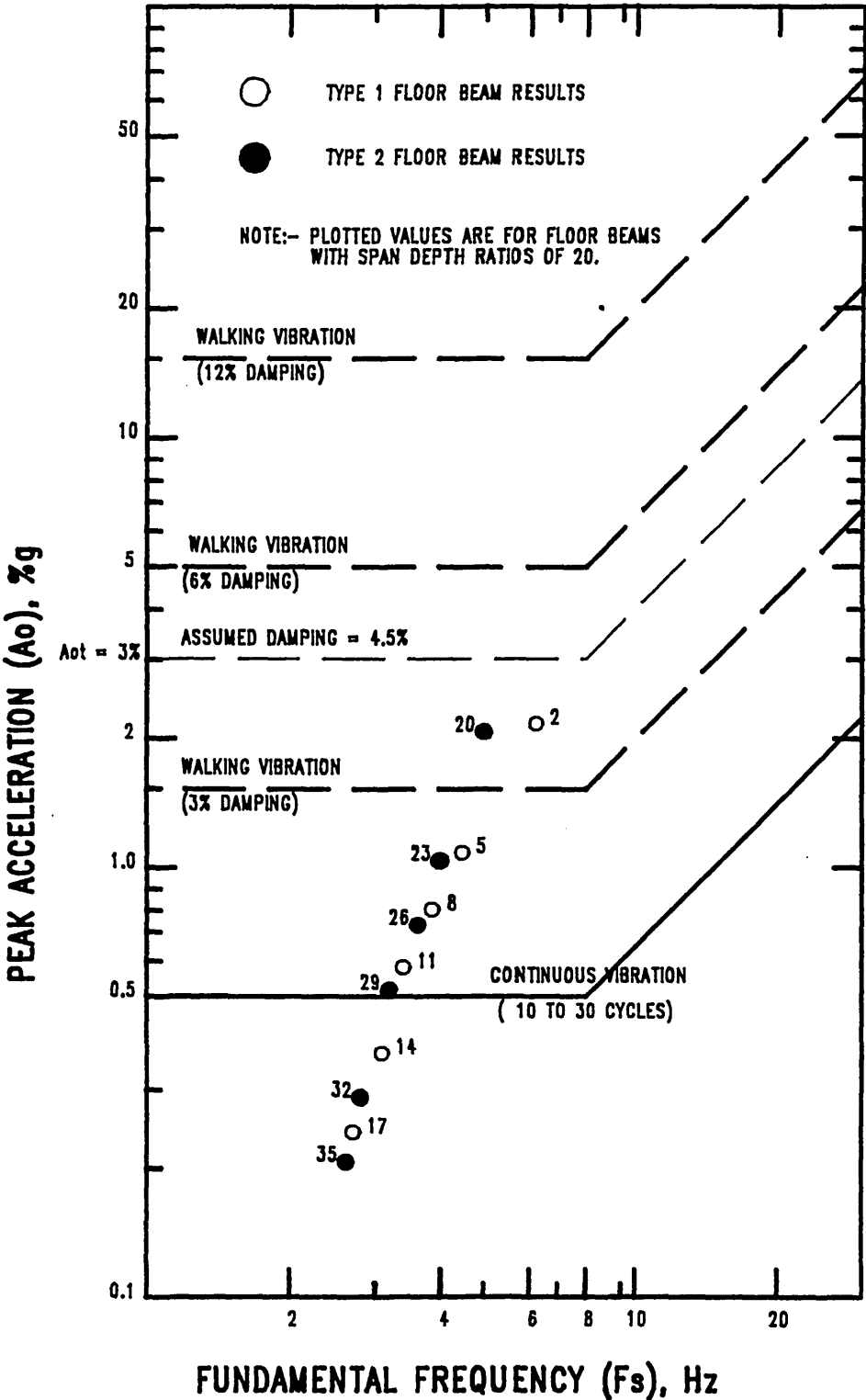
FIG.54



ANNOYANCE THRESHOLDS for FLOOR VIBRATIONS.

VIBRATION ASSESSMENT of LIGHT PLATE GIRDERS
with a CONSTANT TAPER TYPE 1 & 2 FLOOR BEAMS (PART 4)

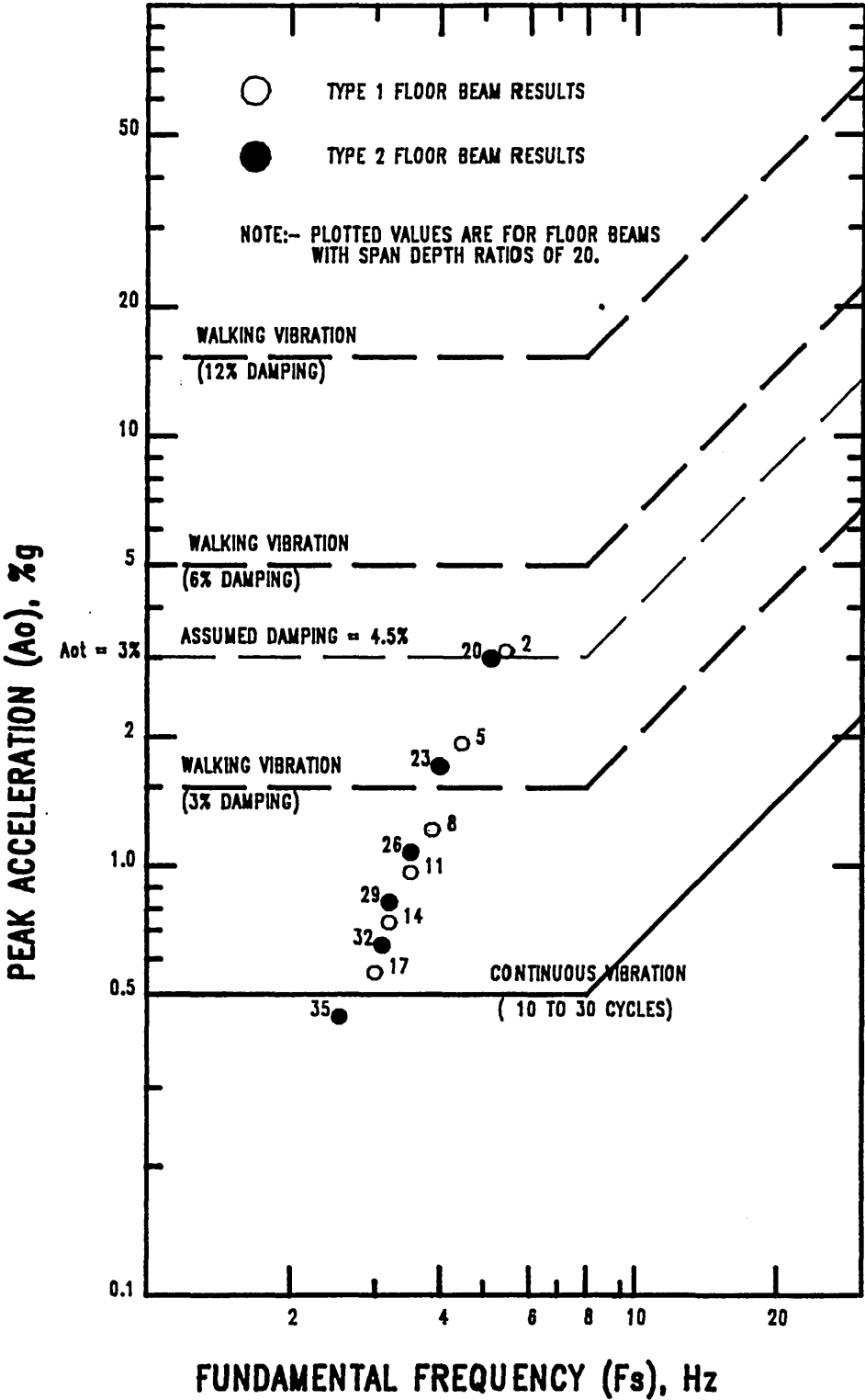
FIG.55



ANNOYANCE THRESHOLDS for FLOOR VIBRATIONS.

VIBRATION ASSESSMENT of LIGHT PLATE GIRDERS
with a MODIFIED TAPER TYPE 1 & 2 FLOOR BEAMS (PART 5)

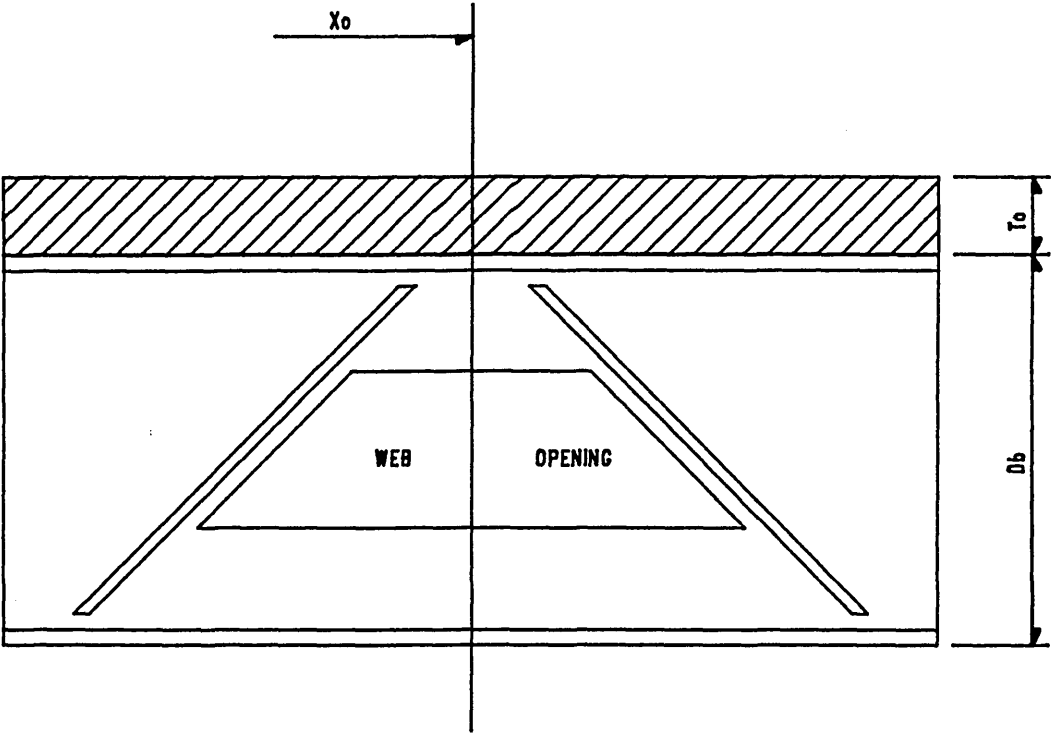
FIG.56



ANNOYANCE THRESHOLDS for FLOOR VIBRATIONS.

**TRIANGULATED REINFORCEMENT in LIGHT PLATE GIRDERS
with WEB OPENINGS**

FIG.57



ELEVATION ON LIGHT PLATE GIRDER