

Structural detailing in concrete

2nd edition

A comparative study of British, European
and American codes and practices

M. Y. H. Bangash

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Preface

Preface to the first edition, 1992

A number of books on various aspects of concrete design and detailing have been published but this is believed to be the first comprehensive detailing manual. The aim of this book is to cover a wide range of topics, so simplifying and reducing the work required to prepare structural drawings and details in reinforced, prestressed, precast and composite concrete.

The book initially provides a list of extracts from relevant codes and current practices. Where drawings are carried out using imperial units, a conversion table is provided to change them into SI units.

The book is divided into eight sections: Section I deals with the general requirements for structural detailing in concrete, basic drafting criteria and the properties of materials. Section II is devoted entirely to the structural detailing of beams and slabs. Section III covers reinforced concrete detailing of stairs and staircases. A comprehensive description is given of the detailing of reinforced concrete columns, frames and walls in Section IV. The reader is also referred for more information to the later section on integrated structures.

Section V covers prestressed concrete systems with some basic structural detailing of beams and anchorages. Again the reader is referred to other sections, in particular Section VIII regarding the use of prestressed tendon elements in integrated structures. Section VI presents structural detailing in composite construction, precast concrete elements, joints and connections.

Section VII includes basic structural detailing of reinforced concrete foundations and earth-retaining structures. An effort is made to include a number of foundation drawings so that the reader can appreciate the quality and design required for a specific job.

Students of civil and structural engineering who have worked through to this part of the book will have acquired the background necessary to draw the majority of reinforced, prestressed, precast and composite concrete structures commonly encountered in professional practice. To assist the reader in his/her completion of drawings, an unusually large number of drawings have been incorporated into the text since they are generally the principal communication between the structural engineer/designer, architect, builder and client.

Case studies in Section VIII include the structural detailing of the following special structures in concrete:

- reinforced concrete beam/slab bridge deck
- culvert bridge super and substructures
- continuous reinforced concrete girder deck
- reinforced concrete box bridge deck
- open spandrel arch bridge—reinforced and prestressed
- reinforced concrete rigid frame bridge details
- composite/steel—concrete bridge deck
- reinforced concrete rigid frame bridge
- bridge bearings and substructural layouts
- samples of reinforced concrete cylindrical shells, hyperbolic shells:

- groin type hyperbolic paraboloid shells and domes, water retaining structures and silos with elevated towers, nuclear shelter
- pressure and containment vessels for nuclear power plants, gas and oil installations and cells for offshore platforms
- hydroelectric and irrigation/hydraulics structures, spillways, piers, intakes, switch yard foundations, electric manholes, chutes, gates, tunnels and culverts.

An increasing emphasis has been placed on the role of the designer in planning reinforcement and structural details so that the detailer can do his/her work thoroughly without having to complete the design himself/herself. Improved methods and standards presented in the text should result in better construction and reduced costs.

The book will serve as a useful text for teachers preparing a syllabus for technician and graduate courses. Each major section has been fully explained to permit the book to be used by practising engineers and postgraduate students, particularly those facing the formidable task of having to design/detail complicated structures for specific contracts and research assignments. Contractors will also find this book useful in the preparation of construction drawings.

M. Y. H. Bangash

Preface to the second edition

This concrete detailing manual has been prepared to provide practical and up-to-date information on many aspects of concrete construction, and is intended for educators, designers, draftsmen and detailers, and all others who have an interest in structural concrete work.

The text covers the full scope of structural detailing in the UK, Europe and the USA, starting with the fundamentals of drawing, continuing with drafting practice and conventional methods of detailing components, and concluding with a number of case studies.

The first edition of the text was based on the British Standard codes and practices. However, in the past decade or so there has been an increase in international multipurpose concrete construction, and engineers on both sides of the Atlantic (and elsewhere in the World) showed a desire for European and American codes and practices to be included in this book. This task, taken upon himself by the author, proved gigantic, especially the incorporation of the newly developed Eurocode 2. Several organizations dealing in British, European and American codes were approached and their advice was sought in the preparation of this second edition.

Those who have used the first edition will find the main headings of the various sections unchanged. The introduction to each section is as given in the first edition. However, each section has typical explanatory notes and drawings with up-to-date information on developmental methods. In some sections only minimal alteration was required, while in others a complete revision was needed. Each section was expanded with codified methods for drafting and detailing concrete structures based on European and American practices. The second edition of this text, therefore, covers the full scope of structural detailing in the UK, Europe and the USA.

Section I now encompasses all general requirements for concrete structures based on the three practices. Section II, on reinforced concrete beams and slabs, now includes deep beams. Geometric staircases are now included in Section III. Based on the three practices, columns, frames and walls are discussed in Section IV. Details on prestressed concrete are given in Section V,

in which special provision is made for Eurocode 2. Section VI is mostly unchanged.

Section VII, concerning concrete foundations and earth-retaining structures, has been modified in the light of current provisions indicated in Eurocode 2 and the ACI/ASCE codes. Pile foundations are examined in detail, and new sections on machine foundations, caissons, rafts and retaining walls have been included. In Section VIII, which presents a series of case studies, a great deal of modification is introduced. The bridge section, VIII.1, has been extended to cover the three practices. VIII. 2 is a new section covering at greater length conventional concrete building details. Similarly, a new section under VIII.3 deals with structural details of stadia, arenas and grandstands. Section VIII.4, on water-retaining structures and silos, has minor alterations. Section VIII.5, on bomb protective structures, also contains new material, while Section VIII.6 is mostly unchanged. Section VIII.7 covers concrete shells, chimneys and towers and includes a new section on tower design/detailing. Section VIII.7 of the first edition, on hydroelectric and irrigation structures, was excluded from this edition for market reasons.

This book should serve both as a primer for trainee detailers and as a reference manual for more experienced personnel. Engineers, architects and contractors working in countries on different continents will find the comparative study in the book useful both for reference and for practical use when preparing drawings to different codes for specific countries.

M. Y. H. Bangash

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Metric conversions

Overall geometry

Spans	1 ft = 0.3048 m
Displacements	1 in. = 25.4 mm
Surface area	1 ft ² = 0.0929 m ²
Volume	1 ft ³ = 0.0283 m ³
	1 yd ³ = 0.765 m ³

Structural properties

Cross-sectional dimensions	1 in. = 25.4 mm
Area	1 in. ² = 645.2 mm ²
Section modulus	1 in. ³ = 16.39 × 10 ³ mm ³
Moment of inertia	1 in. ⁴ = 0.4162 × 10 ⁶ mm ⁴

Material properties

Density	1 lb/ft ³ = 16.03 kg/m ³
Modulus of elasticity and stress	1 lb/in. ² = 0.006895 MPa
	1 kip/in. ² = 6.895 MPa

Loadings

Concentrated loads	1 lb = 4.448 N
	1 kip = 1000 lbf = 4.448 kN
Density	1 lb/ft ³ = 0.1571 kN/m ³
Linear loads	1 kip/ft = 14.59 kN/m
Surface loads	1 lb/ft ² = 0.0479 kN/m ²
	1 kip/ft ² = 47.9 kN/m ²

Prefixes in SI units

G = giga	10 ⁹
M = Mega	10 ⁶
k = kilo	10 ³
m = milli	10 ⁻³
Pa = Pascal	

I. General requirements for structural detailing in concrete

I.1. Introduction

This section gives general requirements for structural detailing in concrete. A slight departure from these requirements can be expected because each project is different. Individual structural engineers and designer detailers also influence the style of working drawings and schedules. Moreover, structural detailing in concrete can vary since it can be considerably affected by external requirements including those of authorities such as gas, electricity, water, municipal, etc. In this section all major general requirements are given which are based on the British Codes, European Codes and the American Codes related to concrete.

I.2. Drafting practice based on British codes

Full drawings are prepared by structural engineers acting as consultants as part of the tender documentation. The architects are involved in the preparation of the site and other general arrangement plans. The main contractors are involved in the preparation of temporary work drawings, including shoring and formwork. During the contract, drawings are sometimes modified by minor amendments and additional details. These drawings are generally updated as the projects progress. The drawings, which are distributed to other engineers including those providing services and to contractors, are prints taken from the original drawings made on tracing paper, called *negatives*. These negatives are provided with thick borders as a precaution against tearing. Plastic film on the other hand gives a smooth hard wearing surface. Almost all drawings are done in ink. A typical drawing sheet contains the following data in the panel on the right-hand side of the drawing.

<i>Starting from the top</i>	<i>Example</i>
NOTES	Specification, etc.
REVISION . . .	751/10 Rev D (details of amendments)
NAME OF THE ENGINEER	Bangash Consultants
NAME OF THE CLIENT/ ARCHITECT	Bangash Family Estate
DRAWING TITLE . . .	BANGASH ESTATE CENTRE FOUNDATION LAYOUT
SCALES/DRAWN BY/DATE . . .	1 : 20, 1 : 50, 1 : 100/Y. Bangash/ 13 July 1992
	Underneath the name of the person and the date
DRAWING NUMBER	The drawing numbers may run in sequence such as 751 or 1, 2, 3 or 100, 101, etc.

The International Standard Organisation (ISO) recommends A or B ranges for paper sizes and most common are A1 (594×841 mm) and B1 (707×1000 mm), for structural detailing in concrete A2 (420×594 mm) size is recommended. For small sketches and detailing and specifications, design teams and contractors use A4-sized (210×297 mm) sheets. All major drawings and site plans carry the north sign.

I.2.1. Drawing instruments

The most general instruments required for good drawings are: a drawing board, woodcase pencils, clutch pencils, automatic pencils, technical drawing pens, erasers, scales, set squares, templates and stencils. A description of these is excluded from this text as they are well known.

I.2.2. Linework and dimensioning

Drawings consist of *plan*, *elevation* and *section*. The structure is viewed 'square on' to give a series of plans, elevations and sections. The two basic types are: first-angle projection and third-angle projection. Dimensioning varies from country to country. Some examples are given later on in this section and in other sections of the book.

I.2.2.1. Line thickness

The following line thicknesses (based on ISO line thickness) are recommended for concrete drawings:

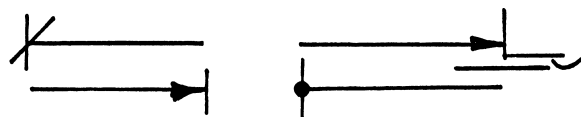
		Colour code
General arrangement drawings	0.35 mm	Yellow
Concrete outlines on reinforcement drawings	0.35 mm	Yellow
Main reinforcing bars	0.70 mm	Blue
Links/stirrups	0.35–0.70 mm	—
Dimension lines and centre lines	0.25 mm	White

The line thickness increases in the ratio $1:\sqrt{2}$, for example, $0.25 \sqrt{2} = 0.35$ etc.

I.2.2.2. Dimensioning

As stated in Section I.2.2.1, dimension lines of 0.25 mm thickness are shown in several ways. Some are given below. A gap is necessary between the dimension line and the structural grid. Dimensions are given in different ways. In SI units, dimensions are given as follows in various countries:

Britain (BS 1192)	All major dimensions shown, say, 1700 for 1700 mm
Codes:	
Sweden	1700 mm rather than 1700
Switzerland	1.700 m
Italy	1700
Japan	1.7 m and 1700
Germany	1.7 m



USA	Major dimensions in ft (feet), smaller dimensions in inches
Pakistan/India	Same as USA on some projects in metres and millimetres.

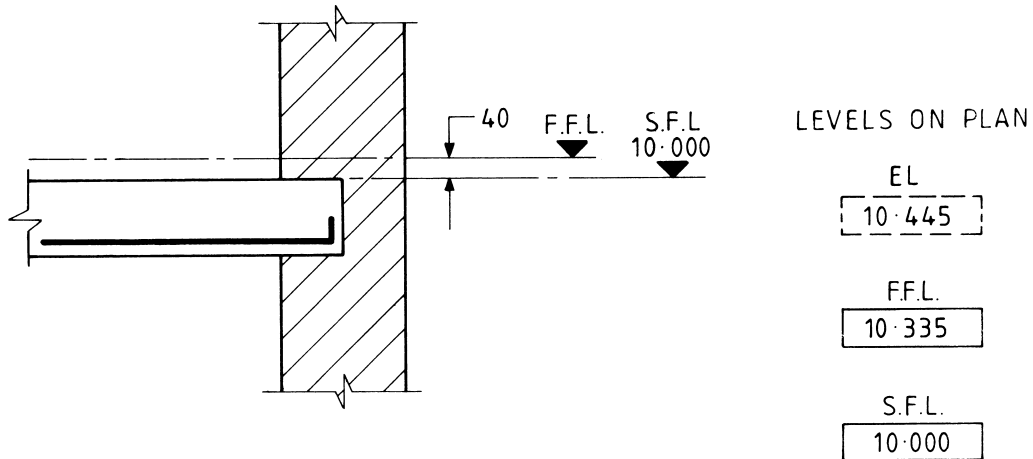
I.2.3. Grids and levels

A point on the drawing can be located by a grid reference. A grid is a series of vertical and horizontal lines on the plan of the structure. They are sometimes called *building grids*. They may not have identical spacings but it

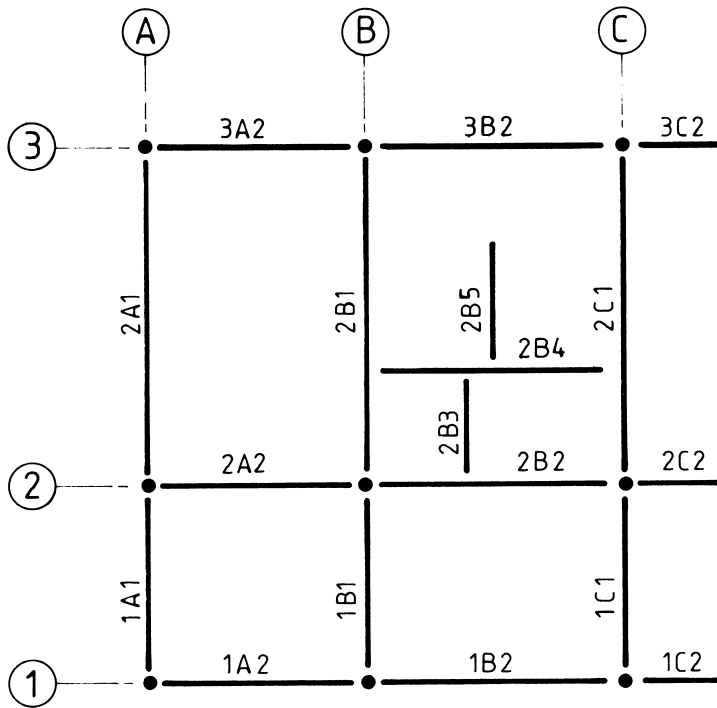
GRIDS AND LEVELS

SHEET NO. I.1

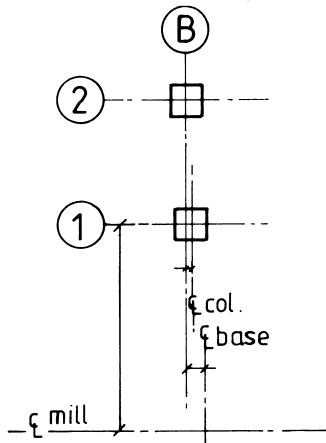
(a) Levels on section and elevation



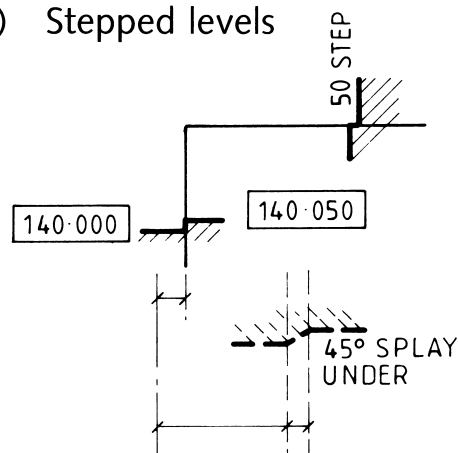
(b) Grid lines and reference system



(c) Columns and grid lines



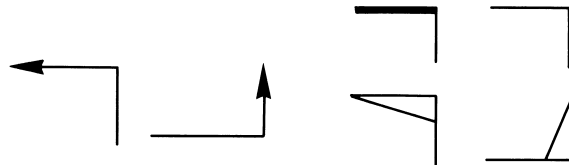
(d) Stepped levels



is preferable that the spacing is constant in the same row between the grid lines. The grid lines are identified by letters and numbers. On sections and elevations, various levels are marked. Typical examples are shown in Sheet No. I.1 for grids and levels and a proper notation is shown for reference beams and columns.

I.2.4. Sections and elevation marker

The exact style cannot easily be determined as it varies from country to country. In a way, it is not important what style is used, as long as it is simple and clear. The markers are located on the plane of the section or elevation with indicators pointing in the direction of the view. The section markers must be shown in the correct direction and the letters must read from the bottom of the drawing. Some of them are shown later on various drawings and details in this



book either with horizontal and vertical thick lines or arrow heads of the types shown. In some important cases two thick lines are shown. Where sections are indicated they are marked as shown below.

X-X

A-A

Similar markers can be seen on different drawings. The author has deliberately changed these markers on drawings to give the reader a choice of any marker that he or she wishes to adopt.

I.2.5. Symbols and abbreviations

aggregate	agg	centres	crs
bitumen	bit	centre to centre	c/c
blockwork	blk	centre line	⊕
brickwork	bwk	finished floor level	FFL
building	bldg	structural floor level	SFL
column	col	average	av
concrete	conc	external	ext
damp proof course/ membrane	dpc/dpm	figure	FIG or fig
diameter	dia, Ø	internal	int
drawing	drg	holes	hls or HOLES
elevation	EL	radius	rad
foundation	fdn	inside/outside dia	id/od
full size	FS	sheet	sh
setting out point	SOP	horizontal/vertical	hor/vert
setting out line	SOL	not to scale	NTS or nts
near face	NF	bottom	B or b
far face	FF	top	T or t
each face	EF	existing level	
each way	EW	(plan) × 100 000	
square	sq	section ∇ ^{100 000}	
		metre	m

GENERAL REQUIREMENTS FOR STRUCTURAL DETAILING IN CONCRETE

right hand	rh	millimetre	mm
sketch	sk	minimum	min
number	no. or NO.		
approximately	approx		
specifications	spec		
pockets	PKT		
kerb	KERB		
nib	NIB		

With reference to reinforcement

Far face outer layer F1

Far face second layer F2

Near face outer layer N1

Near face outer second layer N2

Bottom/top face outer layer B1/T1 or b1/t1

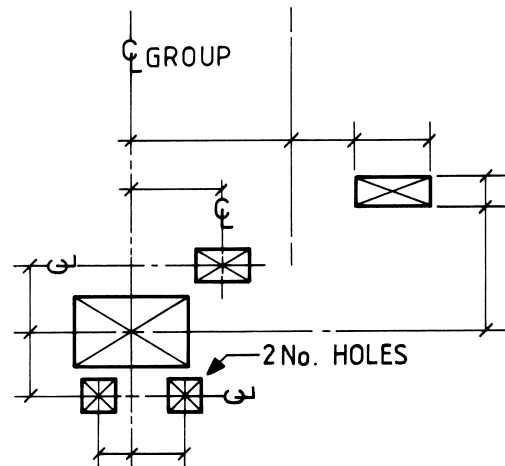
Bottom/top face second layer B2/T2 or b1/t2

I.2.6. Holes, pockets, recesses, nibs and kerbs (curbs)

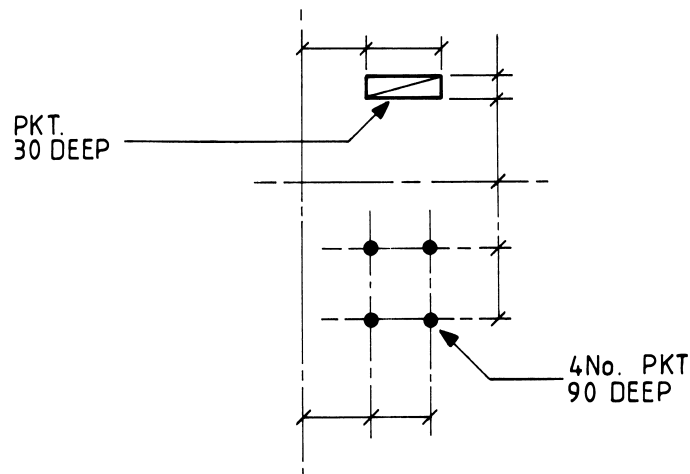
They are either shown as thin cross-lines or single diagonal lines with appropriate symbols. A typical example is shown on Sheet No. I.2.

HOLES, RECESSES, NIBS AND KERBS

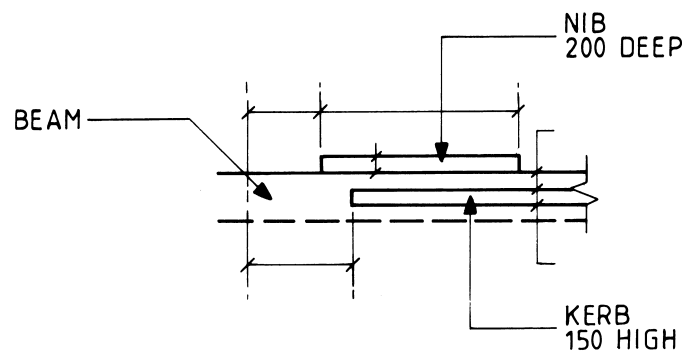
(a) Holes in concrete structures



(b) Pockets and recesses



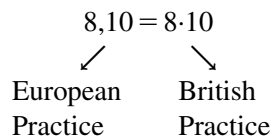
(c) Nibs and kerbs on beams



I.3. Drafting practice based on Eurocode 2

Many of the current British Codes of Practice are geared to those adopted by the Europeans and by Eurocode (EC2). Many practical differences do exist in areas where the Europeans could not abandon their longstanding practices and wish to emphasise them in the Code. The drawing sizes indicated in Section I.2 are identical and so are the linework and the dimensioning. The grids and levels with small changes are almost identical to the British codes. Sections and elevation markers are different, as explained later on in the text. At present, there is no convention adopted on the true representations of symbols and abbreviations. The reader can see these changes on the noted drawings based on EC2. The grids and levels shown in Sheet No. I.1 are basically the same under the concrete code EC2. Variations to these are identified on the sample drawings for EC2 as shown in this section.

Sheet No. I.3 shows a typical ground floor plan on which familiar grid lines are drawn. All walls and columns are marked with thick black lines and black square rectangles respectively. The black circles on the outside of the boundary lines are circular large columns supporting the cantilever zones of the building. The internal columns and their axes are oriented to suit the design and architectural appearances. The comma sign ‘,’ shows the European longstanding practice for a decimal. Hence:



All staircases shown are familiar to British/American practices. The section A–A indicated on the plan shown in Sheet No. I.3 in broken lines without arrowheads can be considered as one of the marked differences in practice. There is no reason why the local symbols cannot replace this one. This drawing is marked ‘1’ which is not a British practice. Another totally different drawing (Sheet No. I.4) shows a portion of a first floor plan with beams and girders in white and rectangular columns in black. Various intended sections are marked. Typical sections A–A and D–D are shown with reinforcement details marked ‘10’ for identification. Contrary to the British and American practice, the European practice shows A–A and D–D on top of the details’ numbers. The dimensions are marked with \bullet — rather than the arrow \leftarrow . All identified sections, such as No. 10, are given detailed descriptions separately on the drawing. The walls are shaded generally. All small dimensions on the sections are in ‘cm’. The following indicate a comparative representation of reinforcement bars with spacings, if any:

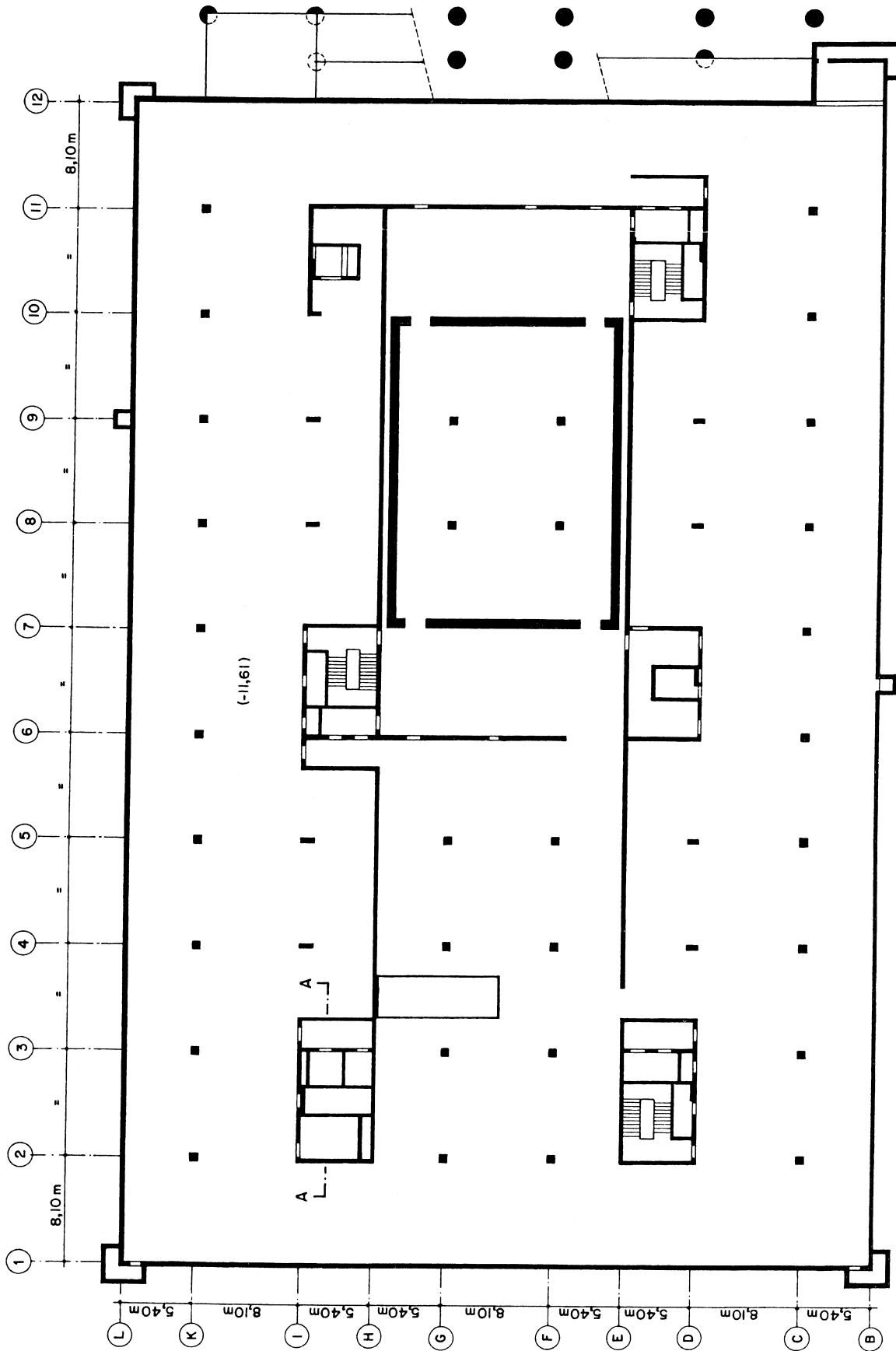
European (EC2)	British equivalent
Ø14/25 cm	T16-250 (No. 14 does not exist)
4Ø24	4T25 (No. 24 does not exist)

Sheet No. I.5 shows sectional elevations of a building with some component details. As shown in the identification No. 2, all columns and floors below ground level are blackened. The foundation pads are kept white and so are adjacent structures.

It is interesting to show some sectional elevations on Sheet No. I.6. All columns, beams, slabs and foundation structures are left shaded. Where the centre lines shown by a cross flange are of the same ‘black colour’, the elevations on both sides are a mirror image. The European practice for unsymmetrical elevations are marked by cross flags with black and white, the

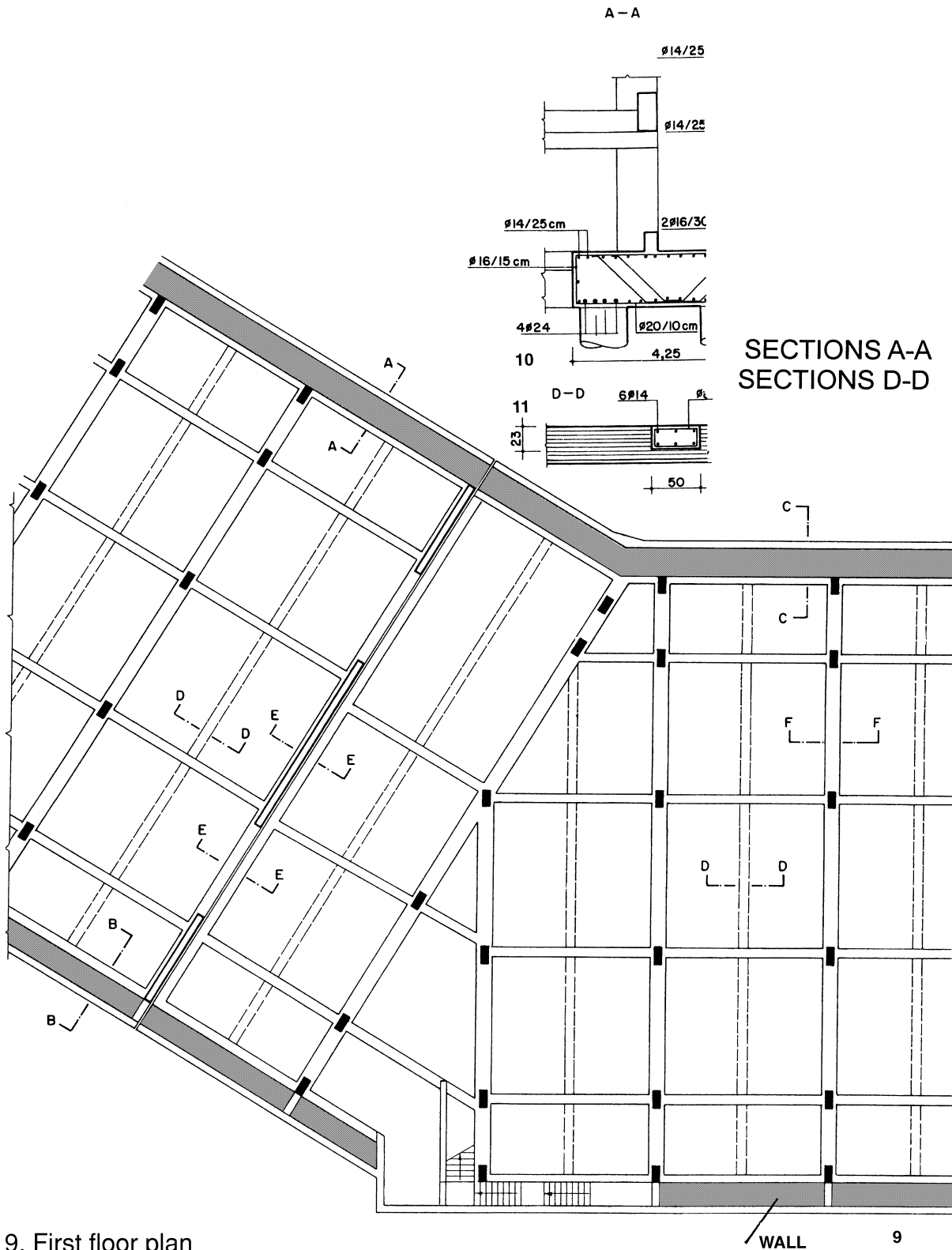
**TYPICAL FIRST FLOOR PLAN
(BASED ON EC2 AND EUROPEAN PRACTICES)**

SHEET NO. I.3



**TYPICAL FIRST FLOOR WITH TYPICAL STRUCTURAL DETAILS
(BASED ON EC2)**

SHEET NO. I.4



9. First floor plan
(beams, columns, slabs, stairs and walls)

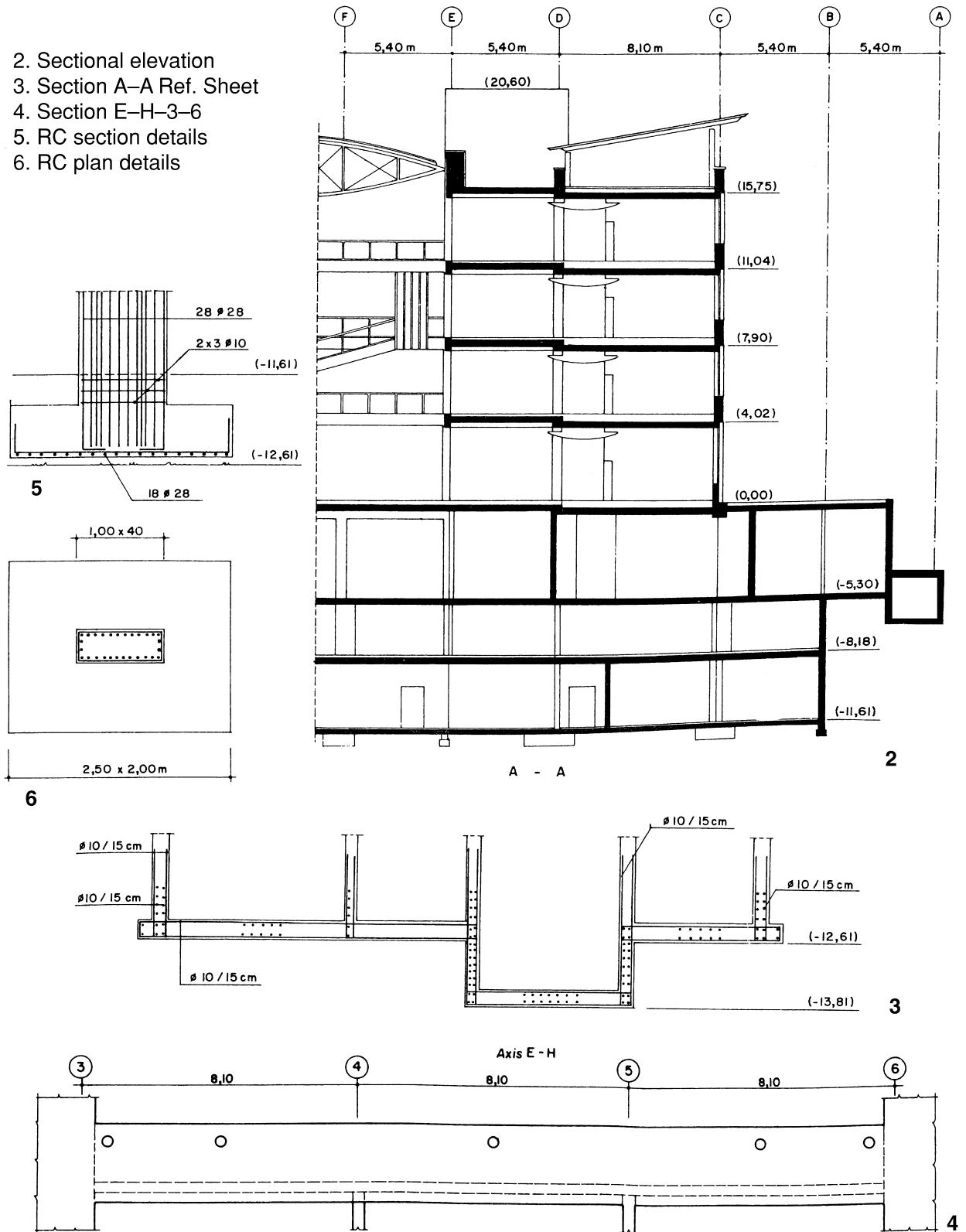
10. Ground floor slab reinforcement details

11. Beam details

SECTIONAL ELEVATION OF A BUILDING WITH STRUCTURAL DETAILS (BASED ON EC2)

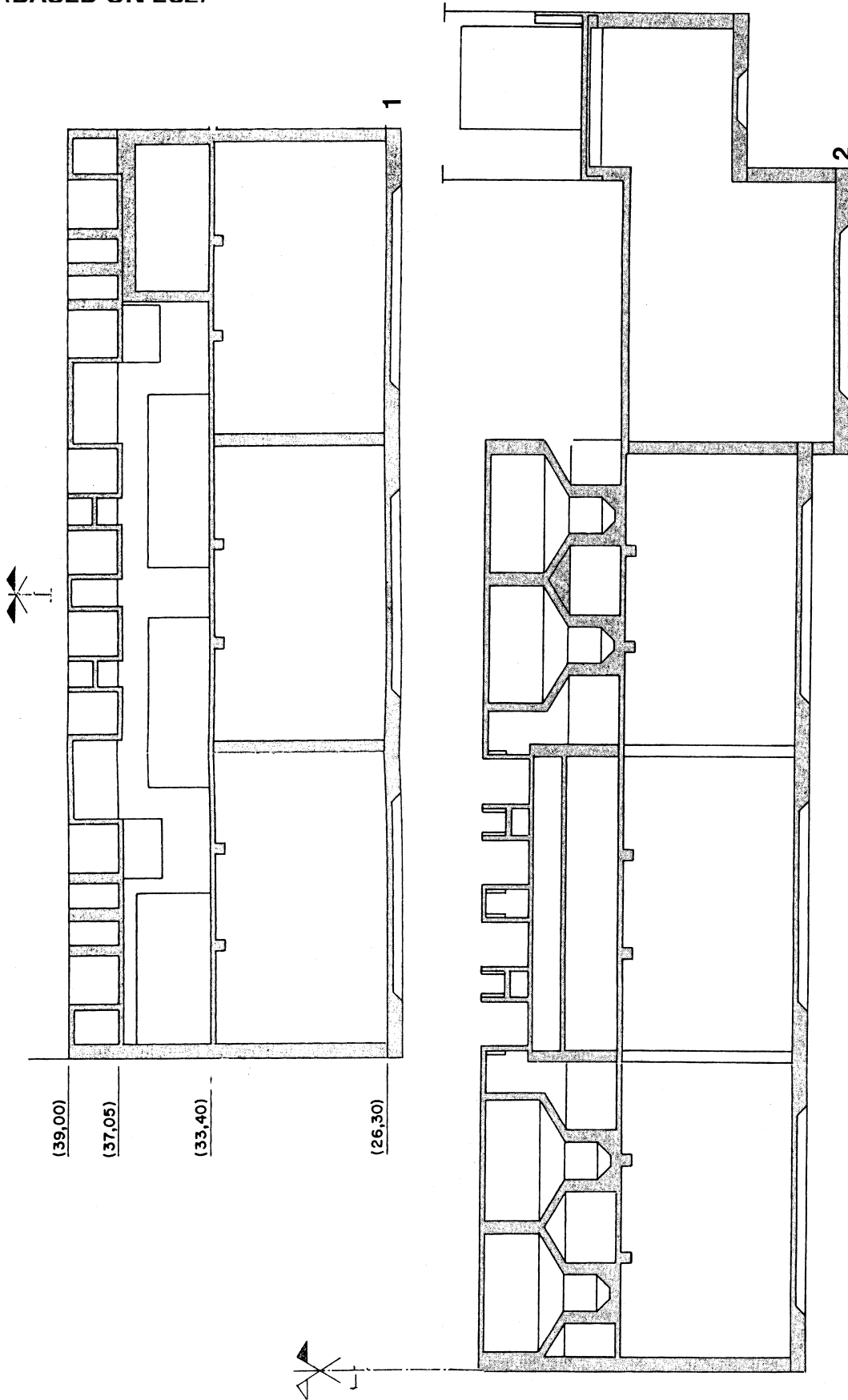
SHEET NO. I.5

- 2. Sectional elevation
- 3. Section A-A Ref. Sheet
- 4. Section E-H-3-6
- 5. RC section details
- 6. RC plan details



**SECTIONAL ELEVATIONS WITH LEVELS AND CENTRE LINES
(BASED ON EC2)**

SHEET NO. I.6



elevational sections are not identical and hence cannot be termed a mirror image.

I.4. Drafting practice based on American codes

Placing drawings are working drawings for fabrication and for the placing of reinforcing steel. These drawings may comprise bar lists, schedules, bending details, placing details, and placing plans or elevations. They may be prepared entirely manually or may include a computer printout.

Placing drawings are prepared to the same general standards as engineering drawings. A broad layout is shown on Sheet No. I.7. Drawings usually show a plan, elevations, sections and details of a structure, accompanied by schedules for footings, columns, beams and slabs. The plan should be drawn in the upper left corner of the sheet.

Placing drawings, ordinarily prepared by the fabricator, show details for fabrication and for the placing of reinforcement. They are not for use in building form-work (except joist forms when these are supplied by the same fabricator) and consequently the only required dimensions are those necessary for the proper location of the reinforcement. Building dimensions are shown on the placing drawing only if it is necessary to locate the reinforcement properly, since the detailer becomes responsible for the accuracy of dimensions when they are given. The placing drawings must be used with the contract (engineering) drawings. Bending details may be shown on a separate list instead of on the drawings.

On receipt of the engineering drawings, the fabricator takes the following steps.

1. Prepare placing drawings (including bending details).
2. Obtain engineer's, architect's or contractor's approval, if required.
3. Prepare bar lists (shop lists) and fabricate the reinforcement.
4. Provide coated bars if specified.

The detailer is responsible for carrying out the instructions on the contract documents. When coated reinforcing bars are detailed along the uncoated reinforcing bars, the coated reinforcing bars should be identified in some manner such as with a suffix E or G, or with an asterisk (*) and a note stating that all reinforcing bars marked as such are to be epoxy-coated or galvanized. Epoxy-coated reinforcing bars listed with uncoated reinforcing bars in schedules or Bills of Materials should also be marked with E or *. The designation G is appropriate for galvanized reinforcing bars.

The reinforcement of floors and many other parts of structures can best be shown in tabular form, commonly referred to as a schedule. The schedule is a compact summary of all the bars complete with the number of pieces, shape and size, lengths, marks, grades, coating information, and bending details from which shop orders can be easily and readily written. While these schedules usually include the bending details for bent bars, separate bending detail schedules may be used. Placing drawings must show the size, shape, grade and location of coated and uncoated bars in the structure, including bar supports, if supplied by the fabricator. They also serve as the basis for preparing bar lists.

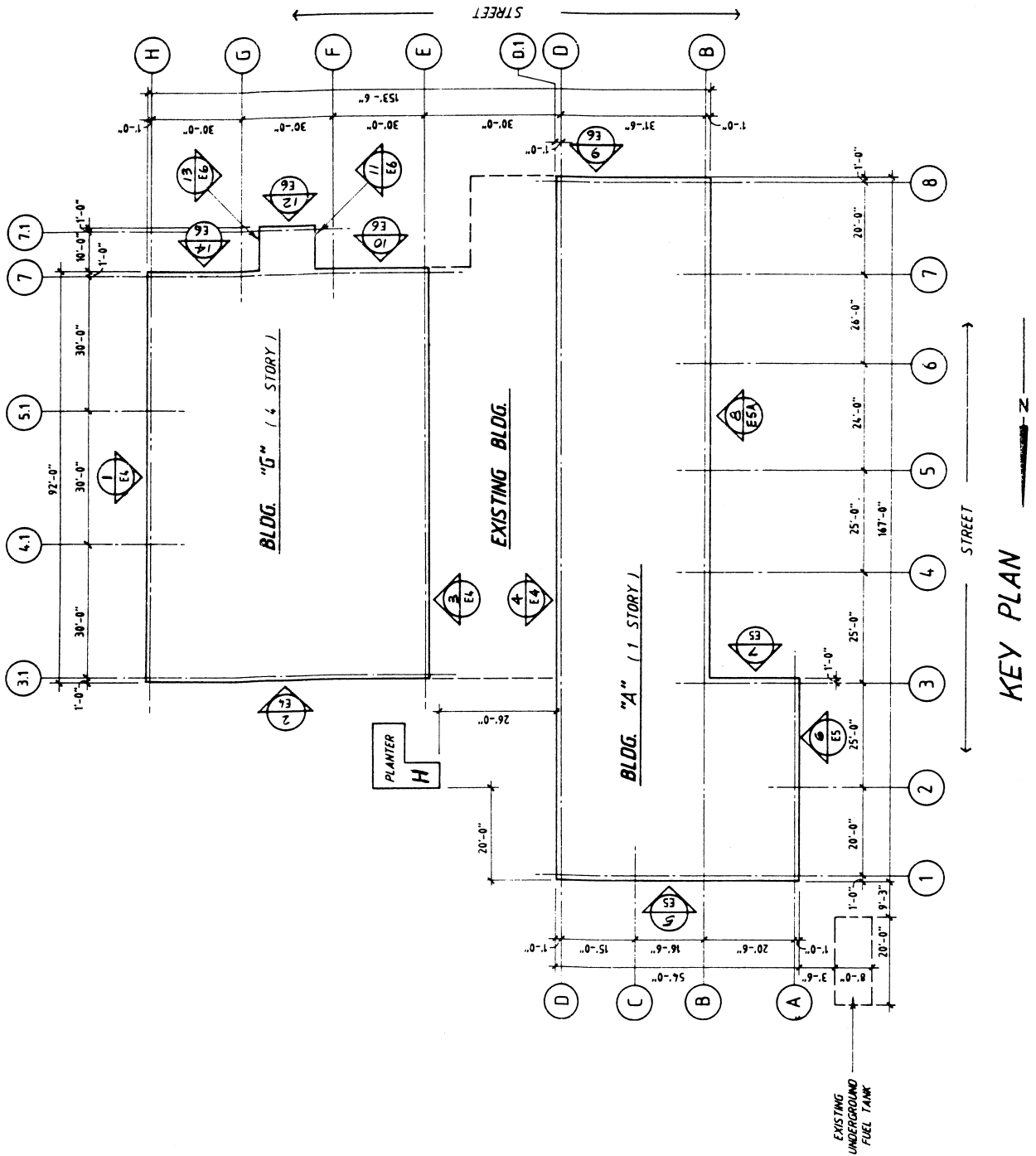
To assure proper interpretation of the engineering drawings and the contractor's requirements, the fabricator's placing drawings are usually submitted for approval to the contractor before shop fabrication is begun.

Slabs, joists, beams, girders and sometimes footings that are alike on engineering drawings are given the same designation mark. Where possible, the same designations shall be used on the placing drawings as on the

**LAYOUT OF AN EXISTING BUILDING
(BASED ON ACI/PCI/ASCE CODES)**

SHEET NO. I.7

KEY: 1 ft. = 0.3048 m
1 in. = 25.4 mm



engineering drawings. When members that are alike on the engineering drawings are slightly different on the placing drawings, a suffix letter is added to the designation to differentiate the numbers. If part of the beams marked 2B3 on the engineering drawing actually differ from the others, the placing drawing would show part of the beams as 2B3 and the others as 2B3A. In concrete joist floors there may be so many variations from the basic joists shown on the engineering drawings that it is necessary to change the basic designations (as, for example, from prefix J to prefix R, for rib).

Columns, and generally footings, are numbered consecutively or are designated by a system of coordinates on the engineering drawings. The same designations shall be used on placing drawings.

The described systems of marking designate individual concrete members of a structure. Reinforcing bars must be individually identified on placing drawings. Only bent bars are given a mark to assist the reinforcing bar placer in selecting the proper bars for each member. The straight bar size and length is its own identification.

Reinforcement in elements of a structure may be drawn on placing drawings either on the plan, elevation, or section, or may be listed in a schedule. It is acceptable practice to detail footings, columns, beams and slabs in schedules. There is no standard format for schedules. They take the place of a drawing, such as a beam elevation, and must clearly indicate to the reinforcing bar placer exactly where and how all the material listed must be placed.

I.4.1. Drawing preparation

The effectiveness of a drawing is measured in terms of how well it communicates its intent. To the user, an erection or production drawing is a set of instructions in the form of diagrams and text. With this thought in mind, the drafter can improve the presentation by observing the following.

1. Make all notes on the drawings brief, clear and explicit, leaving no chance for misunderstanding. Use commands.
2. Make all views and lettering large enough to be clearly legible.
3. Emphasize the specific items for which the drawing is intended. (For instance, when drawing reinforcing tickets, show the outline of the panel with light lines but show the reinforcement and reinforcing designations with dark lines.)
4. If drawings will be reduced photographically, use broader lines and larger lettering.
5. Do not allow the drawings or details to become crowded. Use additional drawings and additional large-scale details when necessary.
6. Highlight special purpose notes so that they are clearly evident (i.e. ERECTOR NOTE!).
7. Use cross references to other erection drawings as required.

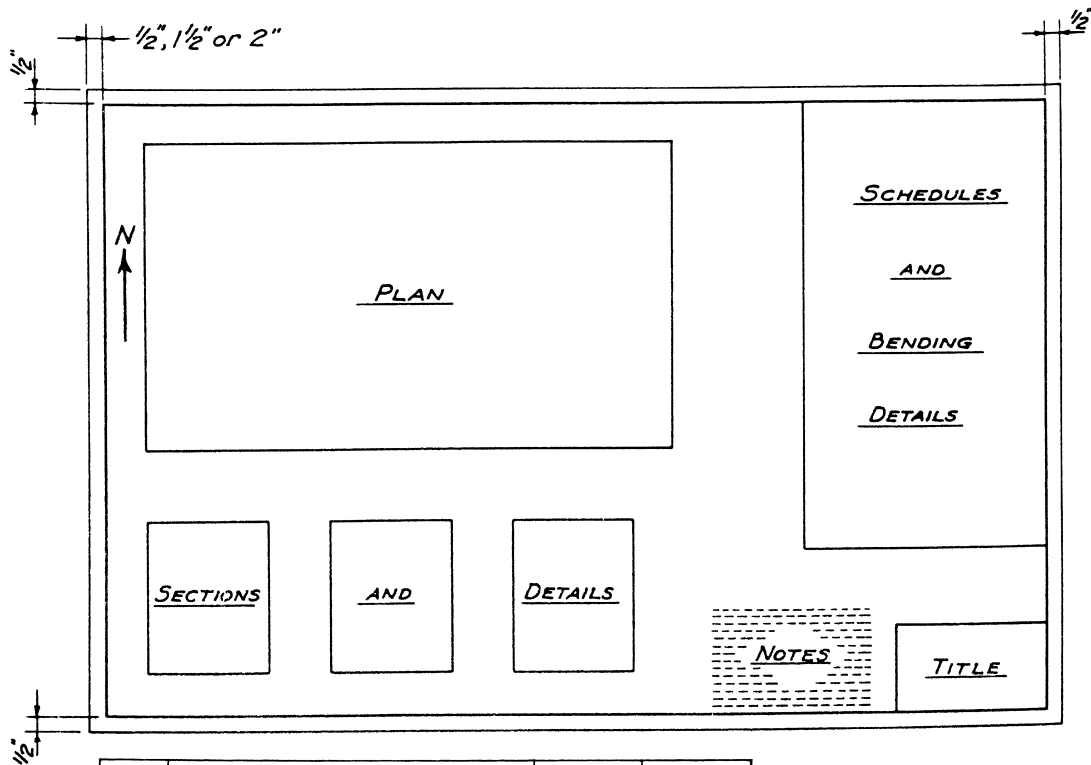
The drafter should become familiar with the following standards in order to use them properly in the preparation of precast concrete drawings:

- (a) general information
- (b) tolerances
- (c) drawing symbols
- (d) graphic symbols
- (e) finish designations
- (f) welding symbols and charts.

All drawings should have a title block, which is usually pre-printed in its lower right-hand corner (see Sheet No. I.8). The following information is recommended for inclusion in the title block.

RECOMMENDED LAYOUT FOR PLACING DRAWINGS

SHEET NO. I.8



NO.	REVISION	BY	DATE
			JOB:
ARCH.		ENG.	
CONTRACTOR		DWN.	ENG.
PROJECT		CKD.	DATE
TITLE:			SHT.

NOTE
 1 inch=25.4 mm
 1 ft=0.3048 m

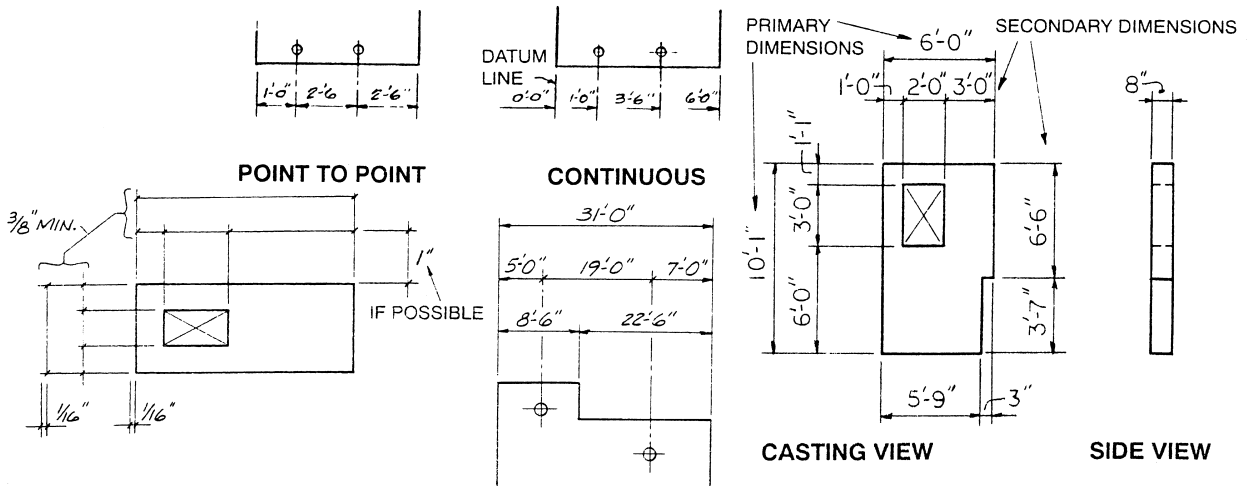


Table I.1. Size and scale of drawing

Type	Prefix	Size	Scale
Erection drawing	E	24×36	1/8 in. min
Keyplan and general notes	K or E	24×36	1/8 in. or proportion
Elevations	E	24×36	1/8 in., 3/16 in., 3/4 in.
Erection plans	E	24×36	1/8 in., 1/4 in.
Sections	E or S	24×36	1/2 in., 3/4 in., 1 in.
	E or S	8 $\frac{1}{2}$ ×11	1 $\frac{1}{2}$ in., 3 in.
Connection details	E or CD	24×36	1/2 in., 3/4 in., 1 in.
	E or CD	8 $\frac{1}{2}$ ×11	1 $\frac{1}{2}$ in., 3 in.
Anchor layouts	E or AL	24×36	1/8 in., 3/16 in., 1/4 in.
Hardware details	H	8 $\frac{1}{2}$ ×11	3/4 in., 1 in., 1 $\frac{1}{2}$ in., 3 in.
Piece drawings	PD or use piece mark	18×24 11×17	1/2 in., 3/4 in., 1 in. or proportion itself
Shape drawings	SH	18×24	3/4 in., 1 in. or proportion
Reinforcing tickets	R	18×24	3/4 in., 1 in. or proportion
	R	11×17	proportion
Handling details	HD	8 $\frac{1}{2}$ ×11	Proportion
	HD	11×17	Proportion

1. Descriptive title for drawing.
2. Name and location of project.
3. Architect's, engineer's and general contractor's names.
4. Name, address and phone number of precast concrete manufacturer.
5. Initials of drafter.
6. Initials of checker.
7. Date of issuance.
8. Job number.
9. Number of each sheet.
10. Revision block.

Refer to Table I.1 for the size and scale of drawings.

1.4.1.1. Drawing instruments

Refer to Section I.2.1.

1.4.1.2. Line work and dimensioning

Dimensioning

All dimensions and arrowheads should be made using a style that is legible, uniform and capable of rapid execution. Two types of dimensioning methods are used within the precast concrete industry. They are *point-to-point* and *continuous* dimensioning (see Sheet No. I.8). Point-to-point relates to the technique of dimensioning from point 'a' to point 'b', point 'b' to point 'c', etc. Continuous dimensioning relates to the technique of referring the location of all points back to the same reference. While this technique minimizes the possibility of cumulative errors in locating items, it requires subtraction to find the distance between any two points, which increases the possibility of drafting errors.

The following dimensioning practices cover most conditions normally encountered: always give all three primary (overall for height, length and thickness) dimensions; primary dimensions should be placed outside of the views and on the outermost dimension line; secondary dimensions should be placed between the view itself and the primary dimensions (see Sheet No. I.8).

1.4.1.3. Lettering

All letters and numbers should be distinct in form to avoid confusion between symbols such as 3 and 5, 3 and 8, 2 and Z, 5 and S, 6 and C, 6 and G, 8 and B, 0 and D, U and V, etc.

The height and boldness of letters and numerals should be in proportion to the importance of the note or dimension. For titles, 3/16 in. to 1/4 in. is recommended, while 1/8 in. should be used for notes and dimensions (Sheet No. I.9). Individual preference should dictate the use of either vertical or slanted lettering, however, only one style should be used on a drawing. Often a firm will establish a policy on the lettering type to be used. Also, refer to the project specifications for requirements, since occasionally future microfilming requirements may dictate the lettering style to be used.

Use of guide lines is recommended for lettering. Guide lines should be lightweight lines that will not reproduce when the drawing is printed. The use of non-print lead should be considered.

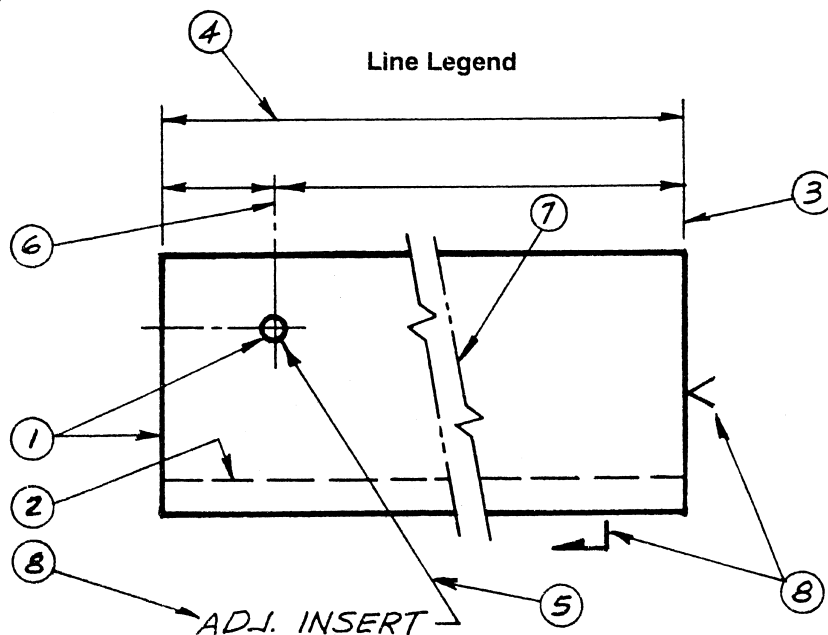
LETTERING AND SYMBOLS

SHEET NO. I.9

A B C D E F G H I J K L M N
 O P Q R S T U V W X Y Z &
 1 2 3 4 5 6 7 8 9 0 $\frac{1}{2}$ $\frac{3}{4}$ $\frac{15}{16}$ $\frac{7}{8}$ $\frac{19}{32}$
 a b c d e f g h i j k l m n o p q
 r s t u v w x y z $\frac{1}{2}$ $\frac{15}{16}$ $\frac{7}{8}$ $\frac{1}{64}$

A B C D E F G H I J K
 L M N O P Q R S T U
 V W X Y Z 1 2 3 4 5 6
 7 8 9 0 . , : ; = ÷ +
 - ± @ & * ? # x " % ' !
 () [] ° ! ¢ \$ % /

Lettering and symbols and lettering styles (adapted from AISC practice)



1.4.1.4. Scales and lines

All erection drawings should be drawn to scale. Production drawings generally cannot be drawn to scale since techniques to speed up the process are often used, however, they should be proportionately correct.

All line work falls into one of the following eight categories: object, hidden, extension, dimension (primary and secondary), leader, centre, break and symbols. Varying line weight (density) helps to differentiate between types of lines on a drawing, providing increased clarity and ease of interpretation.

Line weight can be varied by making repeated strokes on a line or by using different weight leads. When using ink, line weight is controlled through the use of different pen points. Dimension and extension lines, while being the lightest (thinnest) lines on the drawing, must be dense enough to reproduce clearly when multi-generation copies are made. Sheet No. I.9 illustrates the appearance of each type of line as they relate to one another on a drawing, and the recommended weight for each line. The following symbols are used in drawings:

No.	Line type	Lead weight	Pen size
1.	Object	H,HB	#2 (0.60)
2.	Hidden	2H,3H	#00 (0.30)
3.	Extension	4H	#000 (0.25)
4.	Dimension	4H	#000 (0.25)
5.	Leader	4H	#000 (0.25)
6.	Center	4H	#000 (0.25)
7.	Break	2H,3H	#00 (0.30)
8.	Symbol	3H	#0 (0.35)

Note: Special leads are used for Mylars.

1.4.1.5. Grids and levels

The American practice is identical to the one described in Section I.2.3.

1.4.1.6. Sections and elevation marker

Refer to Section I.2.4 and the contents are identical for the American practices.

1.5. Holes, pockets, recesses, nibs and kerbs (curbs)—based on Eurocode 2

They are either shown as thin cross-lines or single diagonal lines with appropriate symbols. A typical example is shown on Sheet No. I.2. Using the British codes and practices, the layout of holes in concrete structures with respect to the centre line of a group is given in Sheet No. I.2 section (a). It is important to give each hole its respective centrelines. Where pockets and recesses are considered, the pockets are given certain depths, as shown in section (b) of Sheet No. I.2. Where ribs and kerbs on beams need to be shown, appropriate dimensions for the depth of rib and the height of kerb are shown in section (c) of Sheet No. I.2.

With small variations in dimensions. the details given on Sheet No. I.2 are acceptable to the European codes and to the American codes and practices.

I.6. Reinforcement size, cover, spacings and dimensional tolerance

I.6.1. British practice

A standard range of bars and sizes is available for use in reinforced concrete. They may be hot-rolled (mild steel, high yield steel) or cold-worked (high yield steel). Bars are made in a range of diameters from 8 to 40 mm. Special sizes of 6 and 50 mm are seldom available. The specification for steel, covers chemical composition, tensile strength, ductility, bond strength, weldability and cross-sectional area. It is important to compare these bars with the American system bars (Table I.2). It is useful in case the drawings are done using American steels.

Table I.2. Bar sizes

Bars											
Britain. Europe, Japan. Russia: bar types (mm)	8	10	12	16	20	25	32	40			
USA. Canada, S. America: bar types (mm) denoted by # or no.		#3	#4	#5	#6	#7 (22 mm)	#8 (29 mm)	#9 (35 mm)	#10 (43 mm)	#11 (57 mm)	#14 (57 mm)
Area (mm ²)	50	78	113	201	314	387	491	645	804	1006	1257
										1452	2581

I.6.1.1. Spacing and arrangement of bars

Bars are spaced on the basis of a number of factors which include beam sizes, aggregate sizes, spacers, concrete cover and many others including requirements imposed by other services. Sheet No. I.10 gives a summary of spacing and arrangement of bars. Both single and group bars are shown. A number of other combinations are possible. When bars of different diameters are used, they tend to be grouped in similar sizes. Some of them are:

10, 12, 16; 16, 20; 20, 25; 16, 20, 25; 20, 25, 32.

I.6.1.2. Cover to reinforcement

The distance between the outermost bars and the concrete face is termed the cover. The cover provides protection against corrosion, fire and other accidental loads. For the bond to be effective an effective cover is needed. Various concrete codes allow grouping or bundling of bars and in such a case the perimeter around a bundle determines the equivalent area of a 'single bar'. The cover also depends on the grade of concrete and the full range of exposure conditions.

Table I.3 gives the nominal cover for such conditions. For concrete against water and earth faces, the cover shall be at least 75 mm.

SPACING AND ARRANGEMENT OF BARS

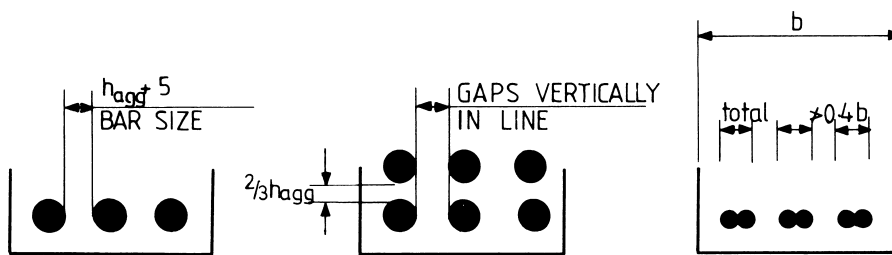
SHEET NO. I.10

(a) Spacing of bars

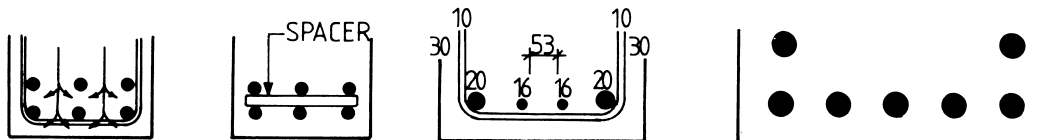
INDIVIDUAL	TWINNED		BUNDLED
$b \geq \frac{2}{3} h_{agg}$	$b \geq h_{agg} + 5mm$	$b \geq \frac{2}{3} h_{agg}$	$b \geq h_{agg} + 15mm$
$a \geq \phi_e$ and $a \geq h_{agg} + 5mm$ h_{agg} IS THE MAX SIZE OF THE COARSE AGGREGATE			$a \geq \phi_e$ and $a \geq h_{agg} + 15mm$

(b) Bar arrangements

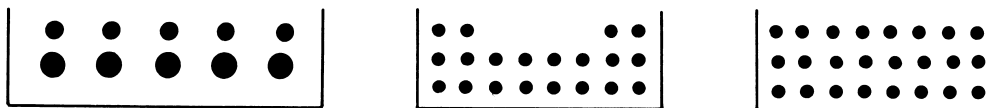
(i)



(ii)



(iii)



(iv)

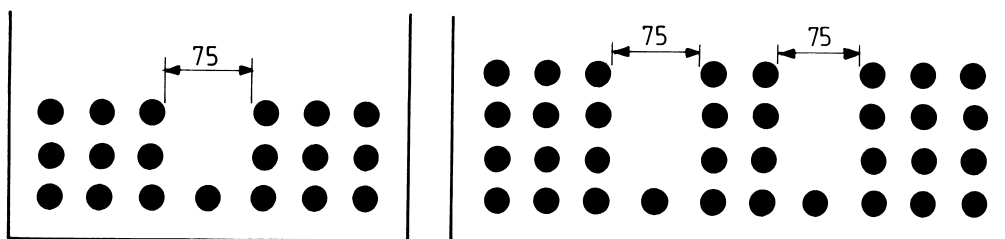


Table I.3. Nominal cover based on BS 8110

Conditions of exposure	Nominal cover: mm*				
	Mild	25	20	20†	20†
Moderate	—	35	30	25	20
Severe	—	—	40	30	25
Very severe	—	—	50+	40‡	30
Extreme	—	—	—	60‡	50
Water/cement ratio	0.65	0.60	0.55	0.50	0.45
Concrete grade	c30	c35	c40	c45	c50

* All values in the table are for h_{agg} maximum aggregate size of 20 mm.

† To be reduced to 15 mm provided $h_{agg} > 15$ mm.

‡ Air-entrainment should be used when concrete is subject to freezing.

I.6.1.3. Dimensional tolerance and spacers

Dimensional tolerance should be allowed at several stages in reinforced concrete detailing, e.g. bar bending, provision of shutter and fixing of reinforcement.

On-site minimum cover = nominal cover – tolerance of 5 mm.

Spacers as shown in Sheet No. I.10 are needed to achieve the required cover between bars and the shutter. They are cast into the concrete. There are different types of spacers. They are normally plastic or concrete, but spacers in the form of steel chairs are also used. They serve to support the steel. All spacers must prevent the dislodgement of the reinforcement cage. They can be used for vertical bars in walls and columns and are clipped into the bars.

I.6.2. Eurocode 2 DD ENV 1992-1-1: 1992

The detailing requirements are mainly governed by bond-related phenomena, which are significantly influenced by:

- the surface characteristics of the bars (plain, ribbed)
- the shape of the bars (straight, with hooks or bends)
- the presence of welded transverse bars
- the confinement offered by concrete (mainly controlled by the size of the concrete cover in relation to the bar diameter)
- the confinement offered by non-welded transverse reinforcement (such as links)
- the confinement offered by transverse pressure.

The rules governing detailing allow for the above factors. Particular emphasis is placed on the need for adequate concrete cover and transverse reinforcement to cater for tensile stresses in concrete in regions of high bond stresses.

Bond stresses for plain bars are related to the cylinder strength of concrete f_{ck} ; those for high-bond bars are a function of the tensile strength of concrete f_{ctk} .

The guidance for detailing of different types of member includes requirements for minimum areas of reinforcement. This is stipulated in order to (a) prevent a brittle failure, (b) prevent wide cracks, and (c) resist stresses arising from temperature effects, shrinkage and other restrained actions.

In this section, the main features of the detailing requirements are arranged in a practical order and discussed.

Table I.4 gives the reinforcement bar sizes and other relevant details including bar parameters (see Sheet No. I.11).

**BAR AREAS AND SPACING
(BASED ON BRITISH CODES)**

SHEET NO. I.11

Table I.4. Sectional areas of groups of bars (mm²)

Sectional areas of groups of bars: mm²

Bar size: mm	Number of bars									
	1	2	3	4	5	6	7	8	9	10
6	28.3	56.6	84.9	113	142	170	198	226	255	283
8	50.3	101	151	201	252	302	352	402	453	503
10	78.5	157	236	314	393	471	550	628	707	785
12	113	226	339	452	566	679	792	905	1020	1130
16	201	402	603	804	1010	1210	1410	1610	1810	2010
20	314	628	943	1260	1570	1890	2200	2510	2830	3140
25	491	982	1470	1960	2450	2950	3440	3930	4420	4910
32	804	1610	2410	3220	4020	4830	5630	6430	7240	8040
40	1260	2510	3770	5030	6280	7540	8800	10100	11300	12600

Perimeters and weights of bars

Bar size: mm	6	8	10	12	16	20	25	32	40
Perimeter: mm	18.85	25.1	31.4	37.7	50.2	62.8	78.5	100.5	125.6
Weight: kg/m	0.222	0.395	0.616	0.888	1.579	2.466	3.854	6.313	9.864

Bar weight based on density of 7850 kg/m³

Sectional areas per metre width for various spacings: mm²

Bar size: mm	Spacing of bars									
	50	75	100	125	150	175	200	250	300	
6	566	377	283	226	189	162	142	113	94.3	
8	1010	671	503	402	335	287	252	201	168	
10	1570	1050	785	628	523	449	393	314	262	
12	2260	1510	1130	905	754	646	566	452	377	
16	4020	2680	2010	1610	1340	1150	1010	804	670	
20	6280	4190	3140	2510	2090	1800	1570	1260	1050	
25	9820	6550	4910	3930	3270	2810	2450	1960	1640	
32	16 100	10 700	8040	6430	5360	4600	4020	3220	2680	
40	25 100	16 800	12 600	10 100	8380	7180	6280	5030	4190	

A_{sw}/s for varying stirrup diameter and spacing

Stirrup diameter: mm	Stirrup spacing: mm										
	85	90	100	125	150	175	200	225	250	275	300
8	1.183	1.118	1.006	0.805	0.671	0.575	0.503	0.447	0.402	0.366	0.335
10	1.847	1.744	1.57	1.256	1.047	0.897	0.785	0.698	0.628	0.571	0.523
12	2.659	2.511	2.26	1.108	1.507	1.291	1.13	1.004	0.904	0.822	0.753
16	4.729	4.467	4.02	3.216	2.68	2.297	2.01	1.787	1.608	1.462	1.34

Note: A_{sw} is based on the cross-sectional area of two legs of the stirrup.

I.6.2.1. Cover to bar reinforcement (see also Sheet No. I.11) spacing of bars

Minimum diameters of bends

Although this is not stated explicitly, the diameters of bends specified in Tables I.4a and b in Sheet No. I.12 relate to fully stressed bars; linear interpolation is permissible for other stress levels.

Bond

Bond conditions—Two bond conditions (good and poor) are defined. These take note of the likely quality of concrete as cast, and are illustrated in Sheet No. I.13.

**COVER FOR REINFORCEMENT AND EXPOSURE CLASSES
(BASED ON EC2 CODE)**

SHEET NO. I.12

Table I.4a. Cover requirements to bar reinforcement

Minimum cover: mm	Exposure class								
	1	2(a)	2(6)	3	4(a)	4(6)	5(a)	5(6)	5(c)
	15	20	25	40	40	40	25	30	40

For concrete cast directly against the ground, minimum cover = 75 mm.

For concrete cast against prepared ground, minimum cover = 40 mm.

The minimum cover should be increased by an allowance for tolerance, which is normally in the range 5–10 mm.

The cover should never be less than \varnothing or \varnothing_n (> 40 mm), or ($\varnothing + 5$ mm) or ($\varnothing_n + 5$ mm) where aggregates larger than 32 mm are used (\varnothing is the diameter of the bar, \varnothing_n is the equivalent diameter for a bundle of bars).

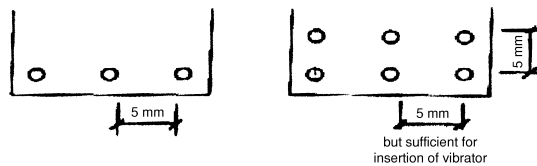


Table I.4b. Exposure classes related to environmental conditions

Exposure class	Examples of environmental conditions	
1. Dry environment	Interior of buildings for normal habitation of offices*	
2. Humid environment	(a) Without frost	Interior of buildings where humidity is high (e.g. laundries) Exterior components Components in non-aggressive soil and/or water
	(b) With frost	Exterior components exposed to frost Components in non-aggressive soil and/or water and exposed to frost Interior components when the humidity is high and exposed to frost
3. Humid environment with frost and de-icing salts	Interior and exterior components exposed to frost and de-icing agents	
4. Seawater environment	(a) Without frost	Components completely or partially submerged in seawater, or in the splash zone Components in saturated salt air (coastal area)
	(b) With frost	Components partially submerged in seawater or in the splash zone and exposed to frost Components in saturated salt air and exposed to frost

The following classes may occur alone or in combination with the above classes

5. Aggressive chemical environment†	(a)	Slightly aggressive chemical environment (gas, liquid or solid) Aggressive industrial atmosphere
	(b)	Moderately aggressive chemical environment (gas, liquid or solid)
	(c)	Highly aggressive chemical environment (gas, liquid or solid)

* This exposure class is valid only as long as during construction the structure or some of its components is not exposed to more severe conditions over a prolonged period of time.

† Chemically aggressive environments are classified in ISO/DP 9690. The following equivalent exposure conditions may be assumed:

Exposure class 5a: ISO classification A1G, A1L, A1S

Exposure class 5b: ISO classification A2G, A2L, A2S

Exposure class 5c: ISO classification A3G, A3L, A3S

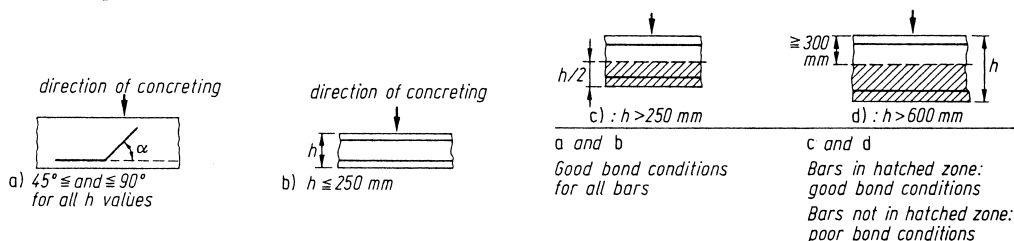
Table I.5 Minimum diameters for hooks, bends and loops

	Bar diameter	
	$\varnothing < 20 \text{ mm}$	$\varnothing \geq 20 \text{ mm}$
Plain bars $f_{yk} = 250$	$2.5\varnothing$	$5\varnothing$
High-bond bars $f_{yk} = 460$	$4\varnothing$	$7\varnothing$

Minimum diameters for bent-up bars or other curved bars

	Minimum cover perpendicular to the plane of curvature		
	$> 100 \text{ mm and } > 7\varnothing$	$> 50 \text{ mm and } > 3\varnothing$	$\leq 50 \text{ mm and } \leq 3\varnothing$
Plain bars $f_{yk} = 250$	$10\varnothing$	$10\varnothing$	$15\varnothing$
High-bond bars $f_{yk} = 460$	$10\varnothing$	$15\varnothing$	$20\varnothing$

Definition of good bond conditions



Figures (a) and (b) show good bond conditions for all bars. Figures (c) and (d) show good bond conditions for bars in hatched zones, and poor bond conditions for bars in zones that are not hatched.

Ultimate bond stress — see Table I.6. Where bond conditions are ‘poor’, the values in Table I.6 should be multiplied by 0.7.

Table I.6 Values of ultimate bond stress

$f_{ck} : \text{N/mm}^2$	Ultimate bond stress $f_{bd} : \text{N/mm}^2$	
	Plain bars	High-bond bars $\varnothing \leq 32$
12	0.9	1.6
16	1.0	2.0
20	1.1	2.3
25	1.2	2.7
30	1.3	3.0
35	1.4	3.4
40	1.5	3.7
45	1.6	4.0
50	1.7	4.3

I.7. ACI/ASTM/ASCE and American practices

Reinforced concrete's unlimited variety in shape and form can be safely and economically achieved only through the use of standardized materials. In the earlier days of reinforced concrete, an extremely wide variety of proprietary reinforcing material was available, but obvious advantages have led to a high degree of standardization in modern reinforcing materials. In the United States, the American Society for Testing and Materials (ASTM) has produced standards that govern both the form and materials of modern reinforcing steel.

Standard deformed reinforcing bar sizes are designated by bar numbers. The nominal bar diameter of a deformed bar is the diameter of a plain round bar having the same mass per metre (weight per foot) as the deformed bar. The actual maximum diameter is always larger than the nominal diameter, due to the deformations. This increase is always neglected in design, except for the cases of sleeves or couplings that must fit over the bar when the actual maximum diameter must be used. Table I.7 shows the nominal specification dimensions for deformed reinforcing bars.

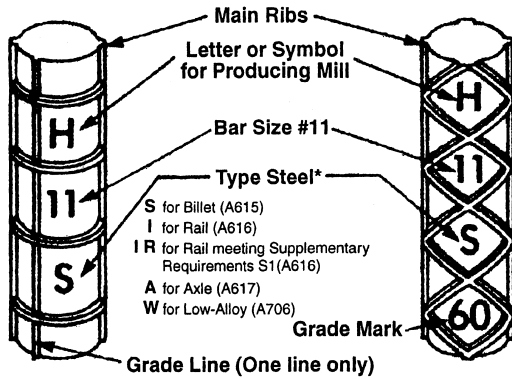
The proper method of designating the size of a standard deformed bar is by its 'bar number'. On a drawing, Bill of Material, invoice or bar tag, the bar number is preceded by the conventional number symbol (#). When more than one bar of the same size is indicated, the number of bars precedes the size marking; thus '6-#13' ('6-#4') indicates six deformed bars of size number 13 (4), and '12-#25' ('12-#8') would refer to 12 deformed bars of size number 25 (8).

Plain round steel bars, which were the first form of reinforcement, are presently used as column spirals, as expansion joint dowels, and in the fabrication of bar mats. The requirements for welded plain bar or rod mats are prescribed by ASTM Specification A704/A704M, The AASHTO Bridge Specifications, which permit the use of plain bars for ties. Specification A305 is now obsolete, since the deformation requirements have been incorporated into the ASTM reinforcing bar specifications A615/A615M and A706/A706M.

Standard reinforcing bars are rolled with protruding ribs or deformations. A deformed steel reinforcing bar is shown on Sheet No. I.14. These deformations serve to increase the bond and eliminate slippage between the bars and the concrete.

Reinforcing bars are produced to ASTM standards in several minimum yield strengths or grades. Grade in this context is the minimum yield strength expressed in units of megapascals (kips per in.²). For example, Grade 420 (60) designates a reinforcing bar with a minimum yield strength of 420 MPa (60 ksi). Table I.8 lists the standard reinforcing bar grades that are used and a summary of the important physical property requirements. Grade 420 (60) billet-steel bars conforming to ASTM Specification A615/A615M are currently the most widely used. A615/A615M prescribes requirements for certain mechanical properties.

ASTM SPECIFICATIONS FOR BARS



*Bars marked with an S and W meet both A615 and A706
GRADE 60

Typical deformed reinforcing bars showing identification marks. (Courtesy of Concrete Reinforcing Steel Institute.)

Table I.7. ASTM Standard Metric Reinforcing Bars

Bar Size ^b	Nominal Dimensions ^a		
	Diameter mm [in.]	Cross-Sectional Area, mm ² [in. ²]	Weight kg/m [lbs/ft]
#10 [#3]	9.5 [0.375]	71 [0.11]	0.560 [0.376]
#13 [#4]	12.7 [0.500]	129 [0.20]	0.944 [0.668]
#16 [#5]	15.9 [0.625]	199 [0.31]	1.552 [1.043]
#19 [#6]	19.1 [0.750]	284 [0.44]	2.235 [1.502]
#22 [#7]	22.2 [0.875]	387 [0.60]	3.042 [2.044]
#25 [#8]	25.4 [1.000]	510 [0.79]	3.973 [2.670]
#29 [#9]	28.7 [1.128]	645 [1.00]	5.060 [3.400]
#32 [#10]	32.3 [1.270]	819 [1.27]	6.404 [4.303]
#36 [#11]	35.8 [1.410]	1006 [1.56]	7.907 [5.313]
#43 [#14]	43.0 [1.693]	1452 [2.25]	11.38 [7.65]
#57 [#18]	57.3 [2.257]	2581 [4.00]	20.24 [13.60]

^aEquivalent inch-pound bar sizes are the designations enclosed within brackets.

^bThe equivalent nominal dimensions of inch-pound bars are the values enclosed within brackets.

Table I.8. ASTM Specifications—Bar Sizes, Grades, and Tensile and Bending Properties

Type of Steel and ASTM Specification	Bar Sizes	Grade	Minimum Yield Strength MPa [psi]	Minimum Tensile Strength MPa [psi]	Minimum Percentage of Elongation in 203.2 mm [8 in.]	Cold Bend Test Pin Diameter (d = nominal diameter of bar)
Billet-Steel A615/A615M-96a	#10 - #19 [#3 - #6]	300 [40]	300 [40,000]	500 [70,000]	#10 [#3] 11 #13, #16, #19 12 #4, #5, #6] 12	#10, #13, #16 [#3, #4, #5] 3.5d #19 [#6] 5d
	#10-#36, #43, #57 [#3-#11, #14, #18]	420 [60]	420 [60,000]	620 [90,000]	#10, #13, #16, #19 [#3, #4, #5, #6] 9 #22, #25 [#7, #8] 8 #29, #32, #36, #43, #57 [#9, #10, #11, #14, #18] .7	#10, #13, #16 [#3, #4, #5] 3.5d #19, #22, #25 [#6, #7, #8] 5d #29, #32, #36 [#9, #10, #11] 7d #43, #57 (90°) [#14, #18 (90°)] 9d
	#19-#36, #43, #57 [#6-#11, #14, #18]	520 [75]	520 [75,000]	690 [100,000]	#19, #22, #25 [#6, #7, #8] 7 #29, #32, #36, #43, #57 [#9, #10, #11, #14, #18] .6	#19, #22, #25 [#6, #7, #8] 5d #29, #32, #36 [#9, #10, #11] 7d #43, #57 (90°) [#14, #18 (90°)] 9d
Low-Alloy Steel A706/A706M-96b	#10-#36, #43, #57 [#3-#11, #14, #18]	420 [60]	420 [60,000]	550 [80,000]	#10, #13, #16, #19 [#3, #4, #5, #6] 14 #22, #25, #29, #32, #36 [#7, #8, #9, #10, #11] . 12 #43, #57 [#14, #18] 10	#10, #13, #16 [#3, #4, #5] 3d #19, #22, #25 [#6, #7, #8] 4d #29, #32, #36 [#9, #10, #11] 6d #43, #57 [#14, #18] 8d

Bar sizes and other data enclosed within brackets are for inch-pound bars.

For low-alloy steel rebars, ASTM A706/A706M-96b prescribes a maximum yield strength of 540 MPa [78,000 psi] and tensile strength must be 1.25 times the actual yield strength.

Bend tests are 180° except ASTM A615/A615M-95a permits 90° for bar sizes #43 and #57 [#14 and #18].

I.7.1. Cover and spacings

Ample concrete protection, called cover, must be provided for the steel reinforcing. Cover is measured as the distance from the outside face of the concrete to the edge of a reinforcing bar. For reinforcement near surfaces not exposed to the ground or to weather, cover should be not less than $\frac{3}{4}$ in. (19 mm) for slabs, walls and joists, and 1.5 in. (38 mm) for beams, girders and columns. Where formed surfaces are exposed to earth or weather, the cover should be 1.5 in. (38 mm) for No. 5 bars and smaller $\frac{3}{4}$ and 2 in. (51 mm) for No. 6 to No. 18 bars. For foundation construction poured directly against ground without forms, cover should be 3 in. (76 mm).

Where multiple bars are used in members (which is the common situation), there are both upper and lower limits for the spacing of the bars. Lower limits are intended to permit adequate development of the concrete-to-steel stress transfers and to facilitate the flow of the wet concrete during pouring. For columns, the minimum clear distance between bar is specified as 1.5 times the bar diameter or a minimum of 1.5 in. for other situations, the minimum is one bar diameter or a minimum of 1 in. (25 mm).

For walls and slabs, maximum centre-to-centre bar spacing is specified as three times the wall or slab thickness or a maximum of 18 in. This applies to reinforcement required for computed stresses. For reinforcement that is required for control of cracking due to shrinkage or temperature change, the maximum spacing is five times the wall or slab thickness or a maximum of 18 in. (457 mm).

I.8. Steel fabric for reinforcement of concrete

I.8.1. British practice: BS 4483 (1998)

I.8.1.1. Definitions

For the purposes of this British Standard the following definitions apply.

Batch

Quantity of fabric of one type or steel grade presented for examination and test at one time.

Bundle

Two or more sheets of fabric bound together.

Transverse wire

Reinforcing element perpendicular to the manufacturing direction of the fabric.

Longitudinal wire

Reinforcing element in the manufacturing direction of the fabric.

Welded fabric

Arrangement of longitudinal and transverse bars or wires of the same or different diameter and length, arranged substantially at right angles to each other, and factory electrical resistance welded together by machine at the points of intersection.

Nominal size

Diameter of a circle with an area equal to the cross-sectional area of the wire.

Length

Length of sheet is the longest side of a sheet of fabric, irrespective of the manufacturing direction.

Pitch

Pitch of fabric is the centre-to-centre distance of wires in a sheet of fabric.

Width

Width of sheet is the shortest side of a sheet of fabric, irrespective of the manufacturing direction.

1.8.1.2. Dimensions

The dimensions of the individual wires shall conform to the appropriate British Standard, i.e. BS 4449 and BS 4482, except for D49 wrapping fabric of 2.5 mm diameter.

The combination of mesh, wire size, wire grade and sheet dimensions for welded steel fabric shall be specifiable in accordance with annex B, or, for the preferred range of standard fabric types, shall be as specified on Sheet No. I.15. The combination shall conform to the tolerances specified.

1.8.1.3. Cross-sectional area and mass

The cross-sectional area and mass of an individual sheet shall be derived from the specified dimensions of the sheet, the nominal wire sizes and the specified pitches for the wires.

The cross-sectional area and mass per square metre of the preferred range of standard fabric types shall be as specified in Table I.9 in Sheet No. I.16.

The actual cross-sectional area and mass of welded steel fabric shall conform to the tolerances specified in Clause 10.

1.8.1.4. Fabric classification

For reference and ordering purposes, the notation specified in BS 4466 for concrete reinforcement shall be used as a general basis for describing and classifying sheets of fabric.

1.8.1.5. Tolerances on mass, dimensions and pitch

Mass

The tolerance on the specified mass of the fabric per square metre shall be $\pm 6\%$.

Dimensions

The tolerance on the specified linear dimensions of the longitudinal and transverse wires in a sheet shall be ± 25 mm or 0.5%, whichever is the greater.

Pitch

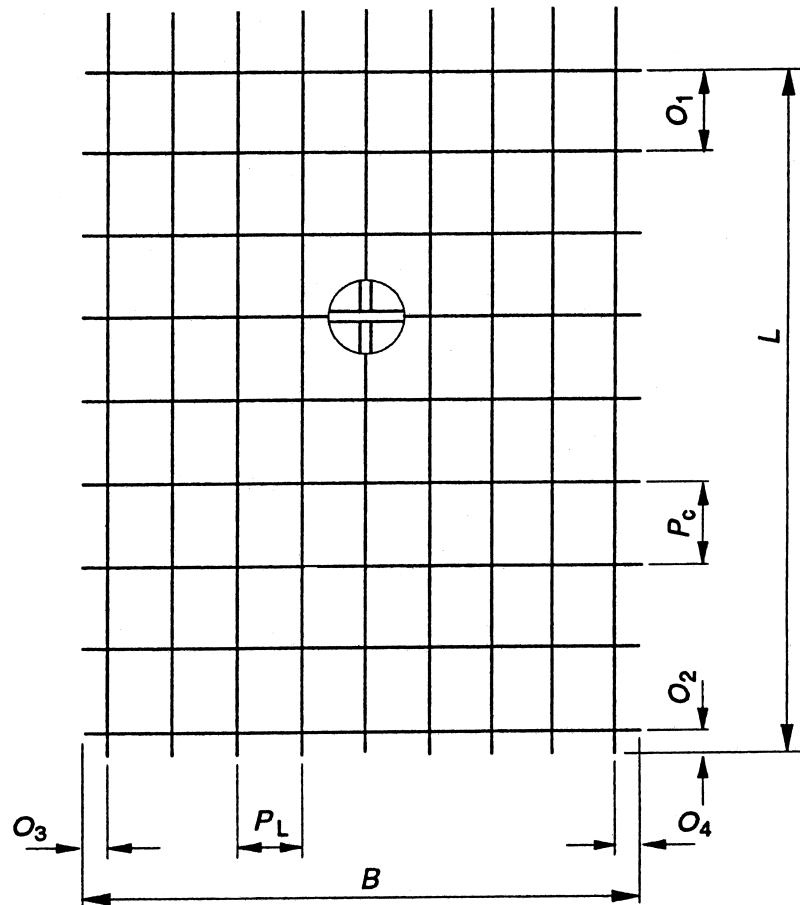
The deviation on the pitch of adjacent wires shall not exceed 15 mm or 7.5% of the nominal pitch, whichever is the greater.

Sheet Nos I.15 and I.16 show the fabric notation and a preferred range designated fabric and stock sheet size.

Fabric notation

Key

- L length of longitudinal wires
- B length of cross wires
- O_1, O_2 overhangs of longitudinal wires
- O_3, O_4 overhangs of cross wires
- P_L pitch of longitudinal wires
- P_C pitch of cross wires



Fabric reinforcement is manufactured to BS 4483 requirements. There are four types of fabric, made from hard drawn mild steel wire of $f_y = 485 \text{ N/mm}^2$ or from cold-worked high yield bars:

- (a) Square mesh fabric regular bars of lightweight (A Type). They are used in walls and slabs.
- (b) Structural fabric main wires 100 mm crs (B Type), cross-wires 200 mm crs.
- (c) Long mesh fabric main wires 100 mm crs (C Type), cross-wires 400 mm crs.
- (d) Wrapping fabric lightweight square mesh (D Type) encased conditions for fire resistance mainwire cross-sectional area 252 mm^2 ; $f_y = 250 \text{ N/mm}^2$.

FABRIC TYPES AND OTHER DATA

SHEET NO. I.16

Table I.9a. Preferred range of designated fabric types and stock sheet size

Fabric reference	Logitudinal wires			Transverse wires			
	Nominal wire size mm	Pitch mm	Area mm ² /m	Nominal wire size mm	Pitch mm	Area mm ² /m	Mass kg/m ²
Square mesh:							
A393	10	200	393	10	200	393	6.16
A252	8	200	252	8	200	252	3.95
A193	7	200	193	7	200	193	3.02
A142	6	200	142	6	200	142	2.22
A98	5	200	98	5	200	98	1.54
Structural mesh:							
B1131	12	100	1131	8	200	252	10.9
B785	10	100	785	8	200	252	8.14
B503	8	100	503	8	200	252	5.93
B385	7	100	385	7	200	193	4.53
B283	6	100	283	7	200	193	3.73
B196	5	100	196	7	200	193	3.05
Long mesh:							
C785	10	100	785	6	400	70.8	6.72
C636	9	100	636	6	400	70.8	5.55
C503	8	100	503	5	400	49	4.34
C385	7	100	385	5	400	49	3.41
C283	6	100	283	5	400	49	2.61
Wrapping mesh:							
D98	5	200	98	5	200	98	1.54
D49	2.5	100	49	2.5	100	49	0.77
Stock sheet size	Length 4.8 m			Width 2.4 m			Sheet area 11.52 m ²
NOTE Values are subject to the tolerances specified in clause 10.							

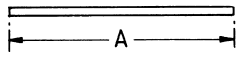
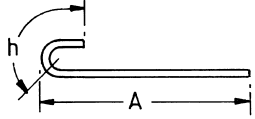
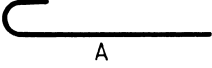
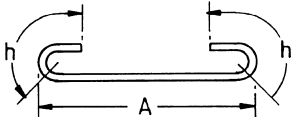
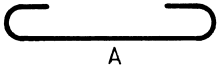
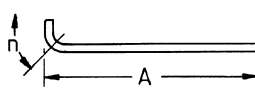
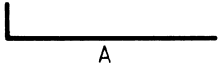
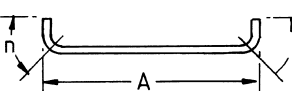
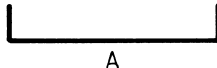
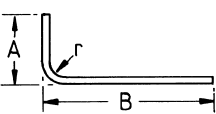
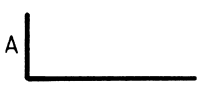
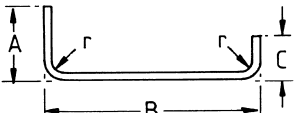
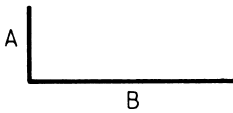
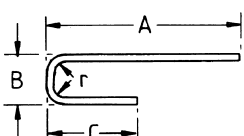
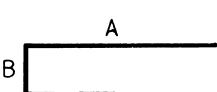
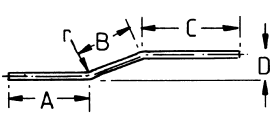
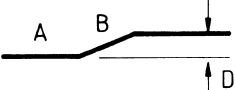
Table I.9b. Purpose-made fabric example

Wires	No. per sheet	Size mm	Length m	Mass kg/m	Mass/sheet kg	No. of sheets	Total mass	
Longitudinal	12	8	5000	0.395	23.700	15	0.355	
Cross	27	6	2100	0.222	12.587		0.189	
Fabric to BS 4483 Bending to BS 4466					36.287		0.544	

I.9. Bar shape codes

I.9.1. British practice: BS 4449, BS 4482, BS 4483 and BS 6744

The standard shapes for the bending of reinforcing bars are generally given specific numbers called shape codes. They are listed on Sheet Nos I.17 to I.21. Where construction demands a special shape not available in these sheets, a special shape code 99 of any form should be used. The shape codes are defined by two digit numbers. In the tables the number of shape code is given first. The next is the method of measurement of bending dimensions. The total length of bar measured along the centre line is given in the third column. The last column indicates a sketch and dimensions, which are intended to be given in schedule.

SHAPE CODE	METHOD OF MEASUREMENT OF BENDING DIMENSIONS	TOTAL LENGTH OF BAR (L) MEASURED ALONG CENTRE LINE	SKETCH & DIMENSIONS TO BE GIVEN IN SCHEDULE
20		A	STRAIGHT
32		A + h	
33		A + 2h	
34		Where the overall dimension of the bob is critical, use shape code 37 A + n	
35		A + 2n Where the overall dimension of either bob is critical, do not use this shape code	
37		$A + (B) - \frac{1}{2} r - d$ This formula is approximate Where r is greater than the minimum value in Table 3 in Sheet No. I.25 use shape code 51	
38		$A + B + (C) - r - 2d$	
OR			
41		WHERE D IS AT LEAST 2φ A + B + C $A + B + C - r - 2φ$ IF THE ANGLE TO THE HORIZONTAL > 45°	

Note 1. r indicates the minimum value in Table 3 in Sheet No. I.25.

Note 2. The dimensions in parentheses are the free dimensions.

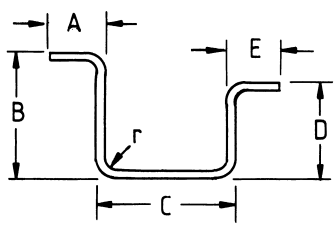
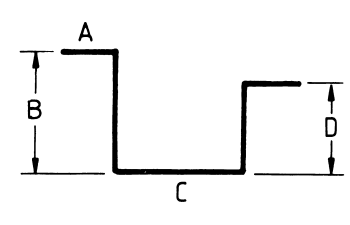
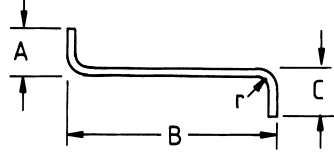
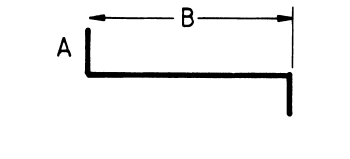
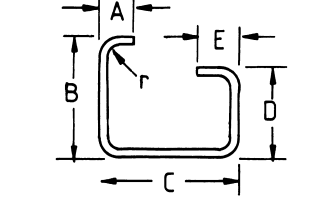
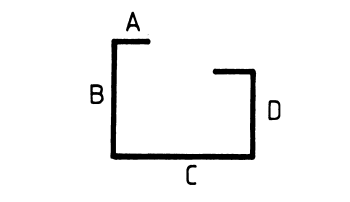
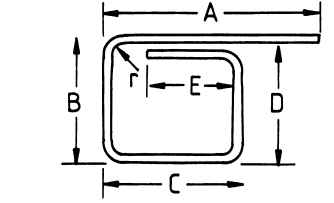
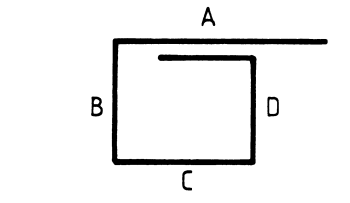
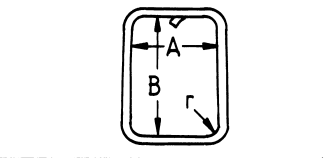
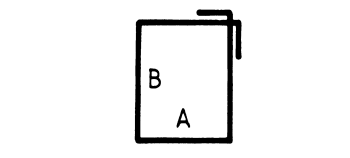
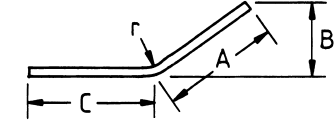
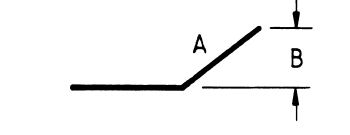
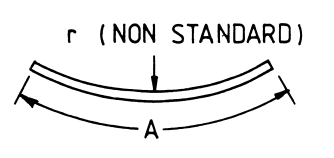
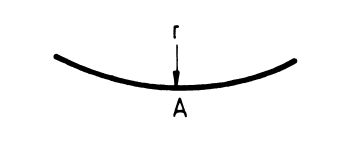
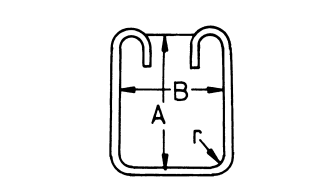
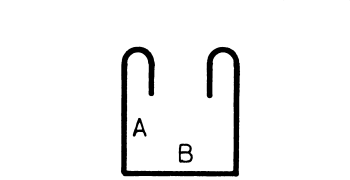
Note 3. To avoid separate equations for each steel grade and bending radius, simplified total length formulae are used for shape codes 61 and 82. These formulae are necessarily approximate.

Note 4. The length formula is approximate and when bending angles exceed 45° the length should be calculated more accurately allowing for the difference between the specified overall dimensions and the true length measured along the central axis of the bar or wire.

SHAPE CODE (CONTINUED)

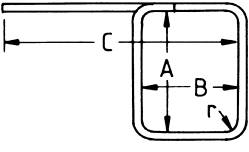
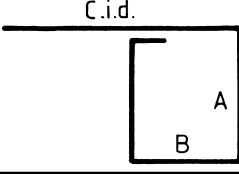
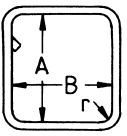
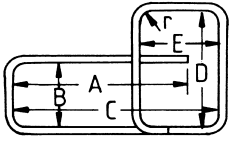
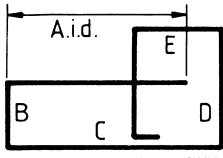
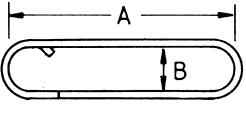
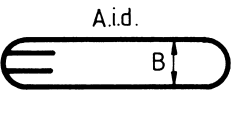
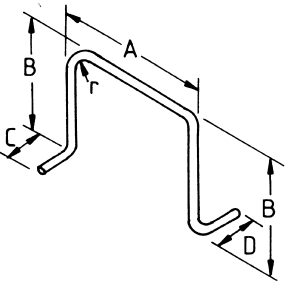
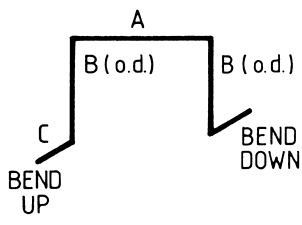
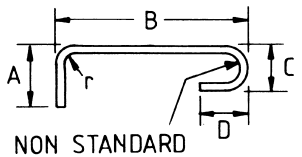
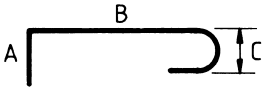
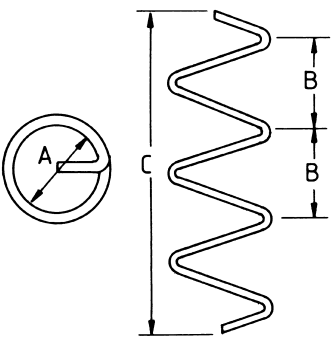
SHEET NO. I.18

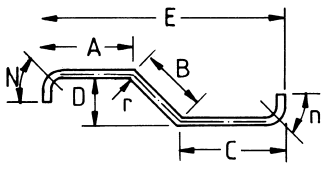
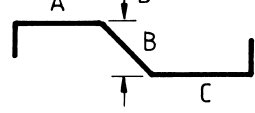
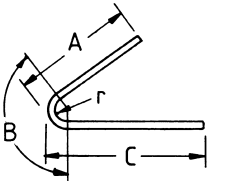
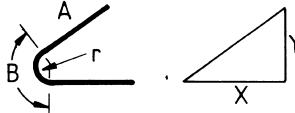
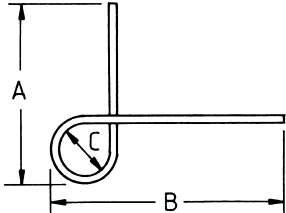
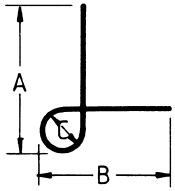
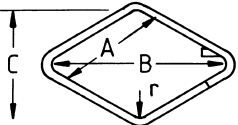
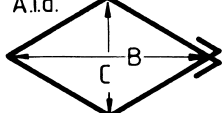
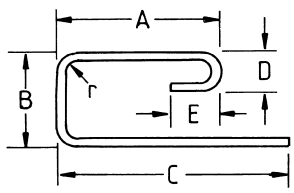
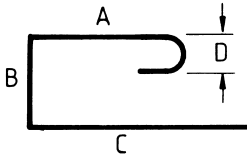
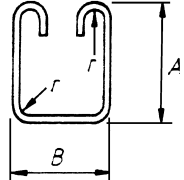
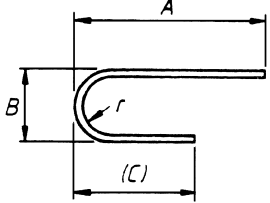
SHAPE CODE	METHOD OF MEASUREMENT OF BENDING DIMENSIONS	TOTAL LENGTH OF BAR (L) MEASURED ALONG CENTRE LINE	SKETCH & DIMENSIONS TO BE GIVEN IN SCHEDULE
42		IF ANGLE WITH HORIZONTAL IS 45° OR LESS $A + B + C + n$ IF THE ANGLE > 45° $A + 2B + C + E - 2r - 4\phi$	
43		IF ANGLE WITH HORIZONTAL IS 45° OR LESS $A + (B) - \frac{1}{2} R - d$ See note 4	
45		IF ANGLE WITH HORIZONTAL IS 45° OR LESS $A + B + (C) - \frac{1}{2} r - d$ See note 4	
48		$A + B + C$	
49		IF ANGLE WITH HORIZONTAL IS 45° OR LESS $A + B + C$	
51*		$A + (B) - \frac{1}{2} R - d$ This formula is approximate If R is minimum, use shape code 37 If R is greater than 200 mm, see note 2 to Clause 10	
52		$A + B + C + (D) - 1\frac{1}{2}r - 3d$ or 	

SHAPE CODE	METHOD OF MEASUREMENT OF BENDING DIMENSIONS	TOTAL LENGTH OF BAR (L) MEASURED ALONG CENTRE LINE	SKETCH & DIMENSIONS TO BE GIVEN IN SCHEDULE
53		$A + B + C + D + (E) - 2r - 4d$	
54		$A + B + (C) - r - 2d$	
55		$A + B + C + D + (E) - 2r - 4d$	
OR		$A + B + C + D + E - 2r - 4\phi$	
60*		$2(A + B) + 20\phi$	
62*		IF ANGLE WITH HORIZONTAL IS 45° OR LESS $A + C$	
65		These bars will be supplied straight when the radius is greater than that given in Table 5 in Sheet No. I.25 A	
72		$2A + B + 25\phi$	

SHAPE CODE (CONTINUED)

SHEET NO. I.20

SHAPE CODE.	METHOD OF MEASUREMENT OF BENDING DIMENSIONS	TOTAL LENGTH OF BAR (L) MEASURED ALONG CENTRE LINE	SKETCH & DIMENSIONS TO BE GIVEN IN SCHEDULE
78		$2A + B + C + 3d$ See note 3	
74		$2A + 3B + 10d$	Neither <i>A</i> nor <i>B</i> are to be less than $12d$ or 150 mm, whichever is greater, for grade 460 in sizes not exceeding 20 mm nor less than $14d$ for sizes of 25 mm and over. Neither <i>A</i> nor <i>B</i> are to be less than $10d$ for grade 250 with a minimum of <i>A</i> and <i>B</i> of 100 mm.
75		$2A + 3B + 18d$ If <i>B</i> is greater than $400 + 2d$, Clause 10 See note 3	
82		$2A + 3B + 22\phi$ $B \geq 2r + 2d$	
83 *		$A + 2B + C + D - 2r - 4\phi$	
85		$A + B + 0.57C + (D) - \frac{1}{2}r - 2.57d$ If <i>C</i> is greater than $400 + 2d$, see note 2 to Clause 10	
86		WHERE <i>B</i> IS NOT GREATER THAN $A/5$ $\frac{C}{B} \pi (A - d) \quad (L \leq 12 \text{ m})$	Where: <i>A</i> is the external diameter (in mm) <i>B</i> is the pitch of helix (in mm) <i>C</i> is the overall height of helix (in mm) Where <i>B</i> is greater than $A/5$ the formula does apply. There shall be at least two full turns in the helix. Note: Unless <i>A</i> is small relative to <i>d</i> , this shape is fabricated in a closed form and pulled to shape on site.

99	ALL OTHER SHAPES - TYPICAL DETAILS SHOWN BELOW.		
99		<p>IF ANGLE WITH HORIZONTAL IS 45° OR LESS & r IS 12φ OR LESS. $A + B + C + 2n$ OR $E + 2n + B - \sqrt{B^2 - D^2}$</p>	
99		<p>IF ANGLE WITH HORIZONTAL IS 45° OR LESS $A + B + C - 2(r + \phi)$ IF ANGLE IS GREATER THAN 45° & r EXCEEDS 12φ LENGTH TO BE CALCULATED.</p>	 (SEE NOTE 3)
99		$A + B + 1\frac{1}{3}C$	
99		$4A + 20\phi$	A.i.d. 
99		CALCULATE	
77		mm $2A + B + 20d$ See note 3	
88	 $B \geq 2r + 2d$	mm $A + 0.57B + (C) - 1.57d$ If B is greater than 400 + 2d, see note 2 to Clause 10	

1.9.1.1. Notations

The type and grade of reinforcement shall be abbreviated using the following letters.

- R grade 250 reinforcement complying with BS 4449
- T grade 460 type 2 reinforcement complying with BS 4449 or BS 4482
- S stainless reinforcement complying with the grade and type selected from BS 6744
- W grade 460 plain reinforcement complying with BS 4482
- D grade 460 type 1 reinforcement complying with BS 4482
- X reinforcement of a type not included in the above list having material properties that are defined in the design or contract specification.

1.9.1.2. Form of schedule

For bar reinforcement, the form of schedule shown on Sheet No. I.22 shall be used.

Note: The schedule should be referred to as a 'bar schedule' since it is customary for the reinforcement fabricator to prepare separate cutting and bending lists for fabrication. The bar schedule is usually completed in sequence of structural units, whereas the cutting and bending lists are usually sorted into type and size of bar.

For cutting and bending purposes, schedules shall be provided on separate sheets of paper of size A4 of BS 4000 and not as part of the detailed reinforcement drawings.

The schedule reference shall appear at the top right-hand corner of the schedule form and shall comprise consecutive numbers, which include a cross-reference to the drawing. Such terms as 'sheet number' or 'page number' shall not be used. The styles '1 (of 6)' and '6 (and last)' may be used on manually prepared schedules but the words in parentheses shall not form part of the schedule reference.

The first three characters of the schedule reference shall be the last three characters of the drawing number, starting at, for example, drawing number 001. The schedule number shall occupy the fourth and fifth spaces, starting at 01 and not exceeding 99 for any one drawing. The sixth space shall be used for schedule revision letters.

1.9.1.3. Radii, bend and hook allowances, couplers and lap length

Introduction

Both cases of mats and reinforcement are assembled from individual bars of manageable lengths and weights. In order to maintain continuity of the reinforcement for large components, reinforcing bars are coupled by using bar couplers. The joints can be in tension or compression. A simple tension joint is formed with a single sleeve which is compressed onto the bar using a hydraulic press. Couplers can be developed using a combination of a threaded sleeve and a stud. The object is that the tensile strength of such an arrangement must at least be equal to the strength of the bar. Sheet No. I.23 gives couplers with specifications and construction joints where they can be used.

Radii, bend and hook allowances

The total length shall be given and, unless one bending dimension, preferably an end dimension, shall be indicated in parentheses as the free dimension to allow for the permissible deviations. The r , n and h values shall be given on the schedule if they differ from the values given in Table 3 in Sheet No. I.24. The tolerances given in Table 4 shall also apply to shape code 99. A reference is made to Sheet No. I.24.

If the angle between two portions of the shape meeting at a bend is not a right angle, it shall be given and shall be defined by coordinates and not by degrees of arc.

Any shape including an acute angle shall be classified as a 99-shape code and drawn out in full with construction lines.

Note: the shape codes given do not include an acute angle.

When dimensioning an acute angle the tangential lines shall be used. Bars bent in two planes shall be sketched isometrically or shown in two elevations, using first angle projection in accordance with BS 308: Part 1. The words 'bent in two planes' or 'isometric view' shall appear on the schedule. The overall off-set dimension of a crank shall be not less than twice the size of the bar or wire. The angled length as shown shall be not less than $10d$ for grade 250 nor less than $12d$ for grade 460 in sizes of less than 20 mm nor less than $14d$ for grade 460 in sizes of 25 mm and over.

For all shapes with two or more bends in the same or opposite directions (whether in the same plane or not), the overall dimension given on the schedule shall always include a minimum straight of $4d$ between the curved portion of the bends, as shown on Sheet No. I.24. The value of x shall be not less than the following:

- (a) $10d$ for grade 250 material
- (b) $12d$ for grade 460 material not exceeding sizes of 20 mm
- (c) $14d$ for grade 460 material in sizes of 25 mm and over.

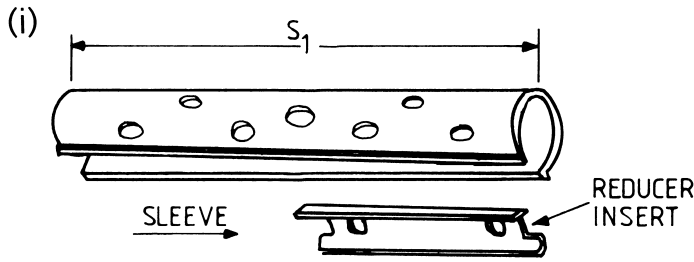
Note: the minimum values of x are expressed in terms of the nominal size of the reinforcement. In practice, rolling and bending tolerances, and the fact that the circumscribing diameter of deformed reinforcement may be up to 10% greater than the nominal size, need to be considered. For example, the actual overall dimension of a hook bent in accordance with Table 1 is greater than $2r + 2d$ and similarly two bends including a $4d$ straight have an actual overall x value greater than $2r + 6d$.

The minimum length of material to be given on the schedule to form a bend or hook shall be as given for n or h respectively in Sheet No. I.24.

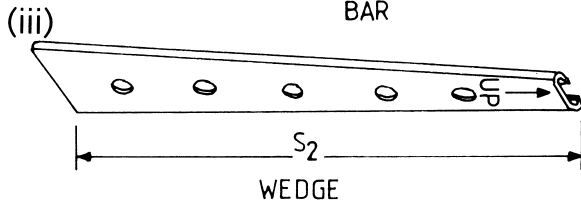
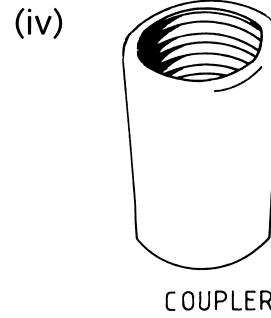
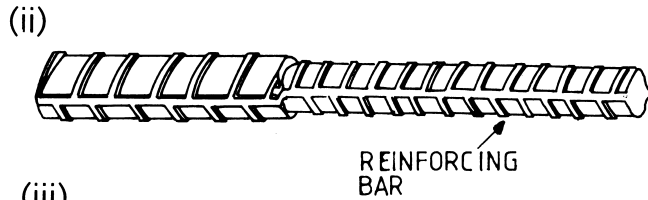
Note: the reason for this is that existing bending equipment requires such a minimum length for the rotating pin to engage with the bar and bend it round the standard former. In giving this length on the schedule, due consideration should be given to the possibility of negative cutting tolerances (up to 25 mm) reducing the actual length of material. The smaller the bar size the more

BARS, COUPLERS AND CONSTRUCTION JOINTS

SHEET NO. I.23



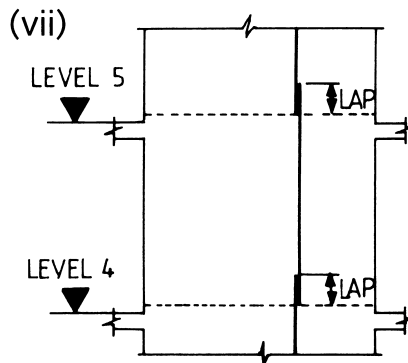
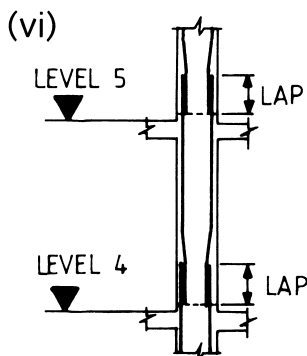
ϕ BAR DIAMETER
 ϕ_o OUTSIDE DIAMETER
 L LENGTH
 P PITCH
 L_t THREADED LENGTH OF EACH BAR



(v)

ϕ (mm)	ϕ_o (mm)	L (mm)	P (mm)	L_t (mm)	SLEEVE S_1 (mm)	WEDGE S_2 (mm)
20	35	55	1.581	40	140	200
25	43	75	1.814	45	150	210
32	50	100	2.117		192	292
40	60	125	2.540	65	240	340

Construction joints



RADII, BEND, HOOKS AND TOLERANCES

SHEET NO. I.24

Table 3. Minimum former radii, bend and hook allowances

Bend (bob)

Hook

Bar size	Type and grade R and type and grade S			Type and grade T and type and grade S			Fabric complying with BS 4483			
	<i>d</i>	<i>r</i>	<i>n</i>	<i>r</i>	<i>n</i>	<i>h</i>	<i>d</i>	<i>r</i>	<i>n</i>	<i>h</i>
6*	12	100	100	18	100	100	5	15	100	100
8	16	100	100	24	100	100	6	18	100	100
10	20	100	100	30	100	110	7	21	100	100
12	24	100	110	36	100	140	8	24	100	100
16	32	100	150	48	100	180	9	27	120	135
20	40	100	180	60	110	220	10	30	120	135
25	50	130	230	100	180	350	12	36	130	145
32	64	160	290	128	230	450	—	—	—	—
40	80	200	360	160	280	560	—	—	—	—
50*	100	250	450	200	350	700	—	—	—	—

NOTE 1. The use of hooks as end anchorages is not normally necessary with deformed bars.
 NOTE 2. See clause 8 for definition of *y*.
 * These are non-preferred sizes.

Table 4. Cutting and bending tolerances

		Tolerance
Cutting of straight lengths (including reinforcement for subsequent bending)		mm +25 -25
Bending	≤ 1000 mm	+5 -5
	> 1000 mm to ≤ 2000 mm	+5 -10
	> 2000 mm	+5 -25

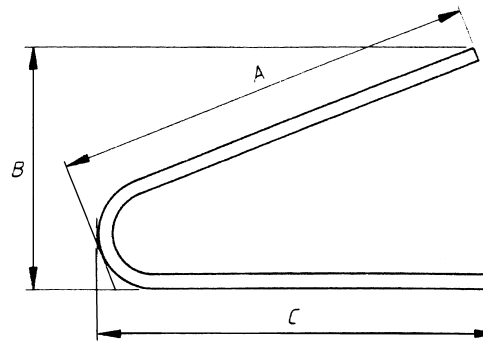


Figure 5. Dimensioning of an acute angle

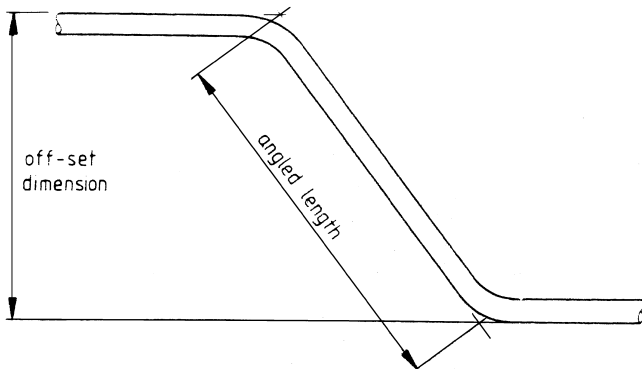


Figure 6. Dimensioning of cranked bars

RADI, BEND, HOOKS AND TOLERANCES

SHEET NO. I.24 (contd)

Table 5. Radius bending: maximum values requiring bending												
Bar size (mm)	–	6	–	8	–	10	12	16	20	25	32	40
Wire size (mm)	5	6	7	8	9	10	12	–	–	–	–	–
Radius (m)	2.4	2.5	2.6	2.75	3.0	3.5	4.25	7.5	14.0	30.0	43.0	58.0

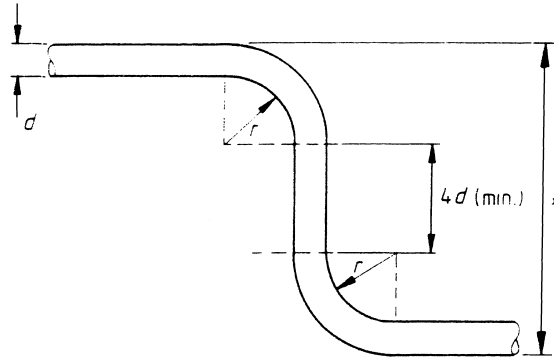


Figure 7. Example of bar with more than one bend

critical is the effect of the negative cutting tolerance, and this fact was considered when deciding on the length.

Bends and hooks

Note 1: Minimum former radii.

Note 2: The overall dimension of a bend may vary from the design dimension by up to the sum of the cutting deviations (± 25 mm) and the cumulative bending deviations.

Before taking into account the cumulative cutting tolerances, the nominal value for y in Sheet No. I.25 shall be calculated as follows:

- (a) for a bend, $n - 0.57r + 0.21d$
- (b) for a hook, $h - 2.14r - 0.57d$.

Tolerances on cutting and bending dimensions

The tolerances given shall apply for cutting and/or bending dimensions and shall be taken into account when completing the schedule. The end anchorage or the dimension in parentheses in the shape codes given in Sheet No. I.17 to I.21 shall be used to allow for any permissible deviations resulting from cutting and bending.

Radius of bending

Reinforcement to be formed to a radius exceeding that given in Sheet No. I.24 shall be supplied straight.

Note 1: The required curvature may be obtained during placing.

Note 2: For shapes with straight and curved lengths (e.g. shape codes 39, 51, 82 and 85) the largest practical radius for the production of a continuous curve is 200 mm, and for larger radii the curve may be produced by a series of short straight sections.

Bending of fabric reinforcement

Note: the schedule for fabric reinforcement includes a column headed 'Bending instruction' for the additional information that is required when specifying bent fabric. The three-dimensional characteristic of fabric reinforcement can give rise to ambiguities that are best overcome by means of a simple sketch in the 'Bending instruction' column.

Couplers and lap length

Sheet No. I.25 gives bar couplers and lap lengths in construction joint.

I.9.2. European practice and Eurocode 2

I.9.2.1. Shape codes methodology

The shape codes given in Section I.9.1 are very similar to the ones adopted under Eurocode 2. They are relevant and shall be adopted by the designer/detailer using Eurocode 2. In addition, the material given in this section, shall also be considered.

I.9.2.2. Detailing provisions

Notation

A_{cl}	Maximum area corresponding geometrically to A_{co} , and having the same centre of gravity.
A_{co}	Loaded area.
$A_{ct,ext}$	Area of concrete external to stirrups.
$A_{s,min}$	Minimum area of longitudinal tensile reinforcement.
$A_{s,prov}$	Area of steel provided.
$A_{s,req}$	Area of steel required.
$A_{s,surf}$	Area of surface reinforcement.
A_{st}	Area of additional transverse reinforcement parallel to the lower face.
A_{sv}	Area of additional transverse reinforcement perpendicular to the lower face.
F_s	Force in the tensile longitudinal reinforcement at a critical section at the ULS.
F_{Rdu}	Concentrated resistance force.
a	Horizontal clear distance between two parallel laps.
a_1	Horizontal displacement of the envelope line of the tensile force (shift rule).
b	Lateral concrete cover in the plane of a lap.
b_t	Mean width of a beam in tension zone.
c	Minimum concrete cover.
d_g	Largest nominal maximum aggregate size.
f_{bd}	Design value for ultimate bond stress.
l_b	Basic anchorage length for reinforcement.
$l_{b,min}$	Minimum anchorage length.
$l_{b,net}$	Required anchorage length.
l_s	Necessary lap length.
$l_{s,min}$	Minimum lap length.
n	Number of transverse bars along anchorage length.
n_1	Number of layers with bars anchored at the same point.
n_2	Number of bars anchored in each layer.
n_b	Number of bars in a bundle.
p	Mean transverse pressure (N/mm^2) over the anchorage length.
s_1	Spacing of longitudinal wires in a welded mesh fabric, or in surface reinforcement.
s_{max}	Maximum longitudinal spacing of successive series of stirrups.
s_t	Spacing of transverse wires in a welded mesh fabric or in surface reinforcement.
u_k	Circumference of area A_k .
α	Angle of the shear reinforcement with the longitudinal reinforcement (main steel).
α_a	A coefficient for determining the effectiveness of anchorages.
α_1	Coefficients for effectiveness of laps.
α_2	Coefficient for the calculation of the lap length of welded mesh fabrics.
θ	Angle between the concrete struts and the longitudinal axis.

1.9.2.3. Steel for reinforced concrete— general detailing arrangements

Spacing of bars

The spacing of bars shall be such that the concrete can be placed and compacted satisfactorily and that the development of adequate bond is assured.

1. The spacing of bars shall be such that the concrete can be placed and compacted satisfactorily and that the development of adequate bond is assured.
2. The maximum aggregate size, d_g , should be chosen to permit adequate compaction of the concrete round the bars.
3. The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be not less than the *maximum bar diameter or 20 mm*. In addition, where $d > 32$ mm, these distances should be not less than $d_g + 5$ mm.
4. Where bars are positioned in separate horizontal layers, the bars in each layer should be located vertically above each other and the space between the resulting columns of bars should permit the passage of an internal vibrator.
5. Lapped bars may touch one another within the lap length.

Permissible curvatures

1. The minimum diameter to which a bar is bent shall be such as to avoid crushing or splitting of the concrete inside the bend of the bar, and to avoid bending cracks in the bar.
2. For bars or wires, the minimum diameter of the mandrel used should be not less than the values given in Sheet No. I.25.
3. For welded reinforcement and mesh bent after welding the minimum diameters of mandrels are given in Sheet No. I.25.

Bond

Bond conditions

1. The quality of the bond depends on the deformation pattern of the bar, on the dimension of the member and on the position and inclination of the reinforcement during concreting.
2. For normal weight concrete, the bond conditions are considered to be good for:
 - (a) all bars, with an inclination of 45° to 90° to the horizontal during concreting
 - (b) all bars which have an inclination of 0° to 45° to the horizontal during concreting and are:
 - (i) either placed in members whose depth in the direction of concreting does not exceed 250 mm (Sheet No. I.26)
 - (ii) or embedded in members with a depth greater than 250 mm and when concreting is completed, are either in the lower half of the member (Sheet No. I.25) or at least 300 mm from its top surface (Sheet No. I.25).
3. All other conditions are considered poor.

Ultimate bond stress

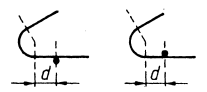
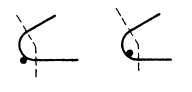
1. The ultimate bond stress shall be such that no significant relative displacement between the steel and concrete occurs under service loads, and that there is an adequate safety margin against bond failure.

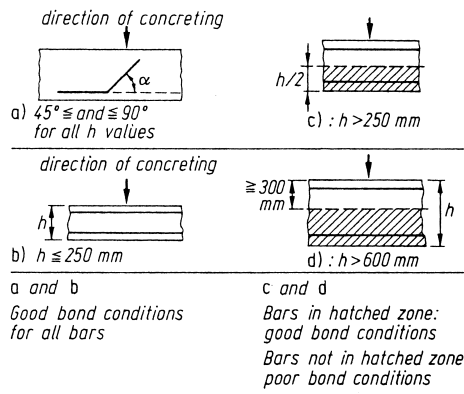
BARS AND LAPS (BASED ON EC2)

Minimum diameters of mandrels

	Hooks, bends, loops (see Figure 5.2)		Bent-up bars or other curved bars		
	Bar diameter		Value of minimum concrete cover, perpendicular to plane of curvature		
	$\varnothing < 20 \text{ mm}$	$\varnothing \geq 20 \text{ mm}$	$> 100 \text{ mm}$ and $> 7 \varnothing$	$> 50 \text{ mm}$ and $> 3 \varnothing$	$\leq 50 \text{ mm}$ and $\leq 3 \varnothing$
Plain bars S220	2.5 \varnothing	5 \varnothing	10 \varnothing	10 \varnothing	15 \varnothing
High bond bars S400, S500	4 \varnothing	7 \varnothing	10 \varnothing	15 \varnothing	20 \varnothing

Minimum diameters of mandrels for welded bent reinforcement

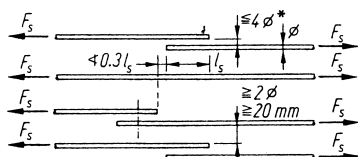
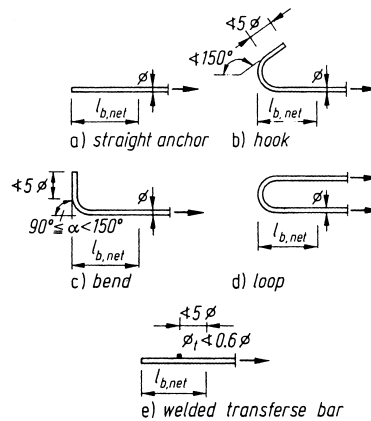
Minimum diameter of the mandrel	
Welds outside bends	Welds inside bends
	
$d < 4 \varnothing$: 20 \varnothing $d \geq 4 \varnothing$: Table 5.1 applies	20 \varnothing



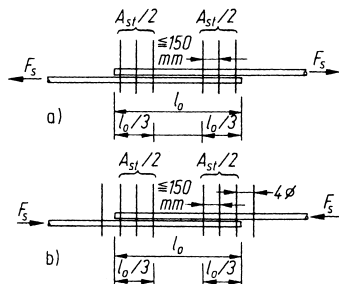
Design values f_{bd} (N/mm²) for good bond conditions (these values incorporate a γ_c value equal to 1.5)

f_{ck}	12	16	20	25	30	35	40	45	50
Plain bars	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7
High bond bars where $\varnothing \leq 32 \text{ mm}$ or welded mesh fabrics made of ribbed wires	1.6	2.0	2.3	2.7	3.0	3.4	3.7	4.0	4.3

Definition of bond conditions

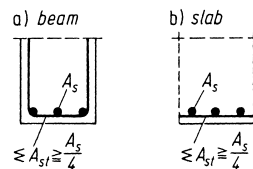


Adjacent laps

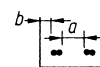


Transverse reinforcement for lapped splices
a) bars in tension b) bars in compression

Required anchorage length



Transverse reinforcement in the region of anchored bars



Evaluation of α_1

2. In conditions of good bond, the design values for the ultimate bond stress f_{bd} are given in Sheet No. I.25. In all other cases, the values in the table on Sheet No. I.25 should be multiplied by a coefficient 0.7.

These values are derived from the following formulae (with $\gamma_c = 1.5$):

- plain bars. $f_{bd} = (0.36 \cdot \sqrt{f_{ck}}) / \gamma_c$
- high bond bars $f_{bd} = (2.25 f_{ctk 0.05}) / \gamma_c$

where f_{ck} and $f_{ctk 0.05}$ are as defined.

3. In the case of transverse pressure p in N/mm^2 (transverse to the possible plane of splitting) the values of Table 5.3 of the code should be multiplied by $1/(1 - 0.04p) \geq 1.4$, where p is the mean transverse pressure.

Basic anchorage length

1. The basic anchorage length is the straight length required for anchoring the force $A_s \cdot f_{yd}$ in a bar, assuming constant bond stress equal to f_{bd} , in setting the basic anchorage length, the type of the steel and the bond properties of the bars shall be taken into consideration.
2. The basic anchorage length required for the anchorage of a bar of diameter \emptyset is:

$$l_b = (\emptyset/4)(f_{yd}/f_{bd})$$

Values for f_{bd} are given in Table 5.3 of the code.

3. For double bar welded fabrics the diameter \emptyset in Equation (5.3) should be replaced by the equivalent diameter $\emptyset_n = \emptyset\sqrt{2}$.

Anchorage

General:

1. The reinforcing bars, wires or welded mesh fabrics shall be so anchored that the internal forces to which they are subjected are transmitted to the concrete and that longitudinal cracking or spalling of the concrete is avoided. If necessary transverse reinforcement shall be provided.
2. Where mechanical devices are used, their effectiveness shall be proven by tests and their capacity to transmit the concentrated force at the anchorage shall be examined with special care.

Anchorage methods

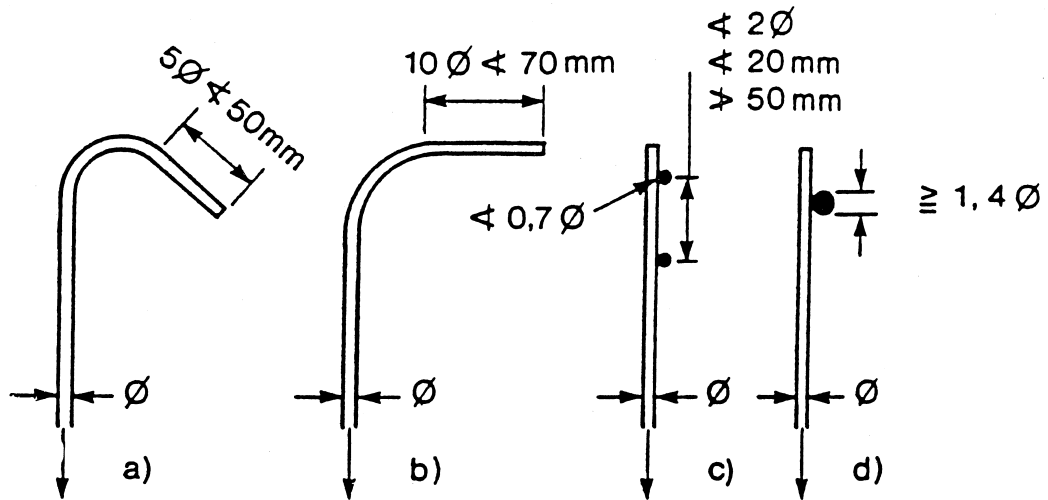
1. The usual methods of anchorage are shown in Sheet No. I.26.
2. Straight anchorages or bends (Figures a or c in Sheet No. I.26) should not be used to anchor smooth bars of more than 8 mm diameter.
3. Bends, hooks or loops are not recommended for use in compression except for plain bars which may be subjected to tensile forces in the anchorage zones, for certain load cases.
4. Spalling or splitting of the concrete may be prevented by complying with Table 5.1 of the code and avoiding concentrations of anchorages.

Transverse reinforcement parallel to the concrete surface

1. In beams transverse reinforcement should be provided:
 - (a) for anchorages in tension, if there is no transverse compression due to the support reaction (as is the case for indirect supports, for example)
 - (b) for all anchorages in compression.

ANCHORAGES (BASED ON EC2)

SHEET NO. I.26

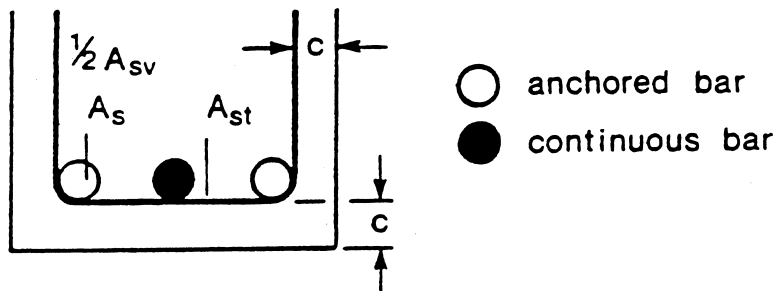


Anchorage of links

Recommended lap lengths in the transverse direction

	Diameter of bars (mm)		
	$\phi \leq 6$	$6 < \phi \leq 8.5$	$8.5 < \phi \leq 12$
High bond wires	$\leq \frac{s_1}{150}$ mm	$\leq \frac{s_1}{250}$ mm	$\leq \frac{s_1}{350}$ mm

s_1 = spacing of longitudinal wires



Additional reinforcement in an anchorage zone where the bar diameter is greater than [32 mm] and there is no transverse compression

- The minimum total area of the transverse reinforcement (legs parallel to the layer of the longitudinal reinforcement) is 25% of the area of one anchored bar (see Sheet Nos I.25 and 26):

$$\Sigma A_{st} = n \times A_{st}$$

where:

n = number of bars along anchorage length

A_{st} = area of one bar of the transverse reinforcement.

- The transverse reinforcement should be evenly distributed along the anchorage length. At least one bar should be placed in the region of the hook, bend or loop of curved bar anchorages.
- For bars in compression, the transverse reinforcement should surround the bars, being concentrated at the end of the anchorage, and extend beyond it to a distance of at least four times the diameter of the anchored bar.

Required anchorage length

Bars and wires:

- The required anchorage length $l_{b,net}$ may be calculated from:

$$l_{b,net} = \alpha_a \frac{A_{s,req}}{A_{s,prov}} \leq l_{b,min}$$

where:

l_b is given by Equation (5.3). Sheet No. I.27.

$A_{s,req}$ and $A_{s,prov}$, respectively, denote the area of reinforcement required by design—and actually provided

$l_{b,min}$ denotes the minimum anchorage length:

- for anchorages in tension $l_{b,min} = 0.3 l_b (\leq 10\emptyset)$ or
- for anchorages in compression $l_{b,min} = 0.6 l_b (\leq 100 \text{ mm})$

α_a is a coefficient which takes the following values:

$\alpha_a = 1$ for straight bars,

$\alpha_a = 0.7$ for curved bars in tension if the concrete cover perpendicular to the plane of curvature is at least $3\emptyset$ in the region of the hook, bend or loop.

Welded meshes made of high bond wires:

- The relevant equation may be applied.
- If welded transverse bars are present in the anchorage, a coefficient 0.7 should be applied to the values given.

Welded meshes made of smooth wires:

- These may be used, subject to relevant standards.

Anchorage by mechanical devices

- The suitability of mechanical anchorage devices should be demonstrated by an Agrément certificate.
- For the transmission of the concentrated anchorage forces to the concrete.

Splices

1. The detailing of splices between bars shall be such that:
 - (a) the transmission of the forces from one bar to the next is assured
 - (b) spalling of the concrete in the neighbourhood of the joints does not occur
 - (c) the width of cracks at the end of the splice does not significantly exceed the values.

Lap splices for bars or wires

Arrangement of lapped joints:

1. As far as possible:
 - (a) laps between bars should be staggered and should not be located in areas of high stress
 - (b) laps at any one section should be arranged symmetrically and parallel to the outer face of the member.
2. Clauses 5.2.3.2(1) to (4) are also applicable to lap splices in the EC2.
3. The clear space between the two lapped bars in a joint should comply with the values indicated in Sheet No. I.25.

Transverse reinforcement

1. If the diameter \emptyset of the lapped bars is less than $\boxed{16 \text{ mm}}$, or if the percentage of lapped bars in any one section is less than 20%, then the minimum transverse reinforcement provided for other reasons (e.g. shear reinforcement, distribution bars) is considered as sufficient.
2. If $\emptyset \geq \boxed{16 \text{ mm}}$, then the transverse reinforcement should:
 - (a) have a total area (sum of all legs parallel to the layer of the spliced reinforcement, see Sheet No. I.25) of not less than the area, A_s , of one spliced bar ($\sum A_{st} \geq 1.0 A_s$)
 - (b) be formed as links if $a \leq \boxed{10\emptyset}$ (see Sheet No. I.25) and be straight in other cases
 - (c) the transverse reinforcement should be placed between the longitudinal reinforcement and the concrete surface.
3. For the distribution of the transverse reinforcement, Clauses 5.2.3.3(3) and (4) apply.

Lap length

1. The necessary lap length is:

$$l_s = l_{b,\text{net}} \times \alpha_1 \leq l_{s,\text{min}}$$

with:

$$l_{s,\text{min}} \leq 0.3 \times \alpha_a \times \alpha_1 \times l_b \leq 15\phi \leq 200 \text{ mm}$$

Values of α_a are given in Clause 5.2.3.4.1.

The coefficient α_1 takes the following values:

$\alpha_1 = 1$ for lap lengths of bars in compression and of lap lengths in tension where less than 30% of the bars in the section are lapped where $a \leq \boxed{10\emptyset}$ and $b \leq \boxed{5\emptyset}$.

$\alpha_1 = 1.4$ for tension lap lengths where either

- (a) 30% or more of the bars at a section are lapped or (b) according to $a \leq \boxed{10\emptyset}$ and $b \leq \boxed{5\emptyset}$ but not both.

$\alpha_1 = 2$ for tension lap lengths if both (a) and (b) above apply simultaneously.

Laps for welded mesh fabrics made of high bond wires

Laps of the main reinforcement:

1. The following rules relate only to the most common case where laps are made by layering of the sheets. Rules for laps with intermeshed sheets are given separately from this code.
2. The laps should generally be situated in zones where the effects of actions under the rare combinations of loads are not more than 180% of the design strength of the section.
3. Where condition (2) is not fulfilled, the effective depth of the steel taken into account in the calculations in accordance with the EC2 code should apply to the layer furthest from the tension face.
4. The permissible percentage of the main reinforcement which may be lapped in any one section, referred to the total steel cross-section is:
 - (a) 100% if the specific cross-sectional area of the mesh, denoted by A_s/s , is such that $A_s/s \leq 1200 \text{ mm}^2/\text{m}$
 - (b) 60% if $A_s/s > 1200 \text{ mm}^2/\text{m}$ and if this wire mesh is an interior mesh.

The joints of the multiple layers should be staggered at $1.3 l_o$.

5. The lap length is defined by:

$$l_s = a_2 l_b \frac{A_{s,\text{req}}}{A_{s,\text{prov}}} \leq l_{s,\text{min}}$$

$$a_2 = 0.4 + \frac{A_s/S}{800} \leq 1.0$$

$$\geq 2.0$$

l_b from Equation (5.3) using f_{bd} for high bond bars $A_{s,\text{req}}$ and $A_{s,\text{prov}}$ are as defined in the code EC2:

A_s/s in mm^2/m

$$l_{s,\text{min}} = 0.3 a_s l_b \left\{ \begin{array}{l} \leq 200 \text{ mm} \\ \leq S_t \end{array} \right.$$

where:

S_t denotes the spacing of transverse welded wires.

6. Additional transverse reinforcement is not necessary in the zone of lapping.

Laps of the transverse distribution reinforcement:

1. All transverse reinforcement may be lapped at the same location. The minimum values of the lap length l_s are given on Sheet No. I.25; at least two transverse bars should be within the lap length (one mesh).

Anchorage of links and shear reinforcement

1. The anchorage of links and shear reinforcement shall normally be effected by means of hooks, or by welded transverse reinforcement. High bond bars or wires can also be anchored by bends. A bar should be provided inside a hook or bend.

2. For the permissible curvature of hooks and bends, see Clause 5.2.1.2(2).
3. The anchorage as a whole is considered to be satisfactory:
 - (a) where the curve of a hook or bend is extended by a straight length which is not less than:
 - (i) 4ϕ or 50 mm if it is a continuation of an arc of 135° or more (Sheet No. I.25).
 - (ii) 10ϕ or 70 mm if it is a continuation of an arc of 90°
 - (b) where they are near the end of a straight bar:
 - (i) either two welded transverse bars (Sheet No. I.25)
 - (ii) or a single welded transverse bar, the diameter of which is not less than 1.4 times the diameter of the link (Sheet No. I.25).

Additional rules for high bond bars exceeding 32 mm in diameter

Construction details:

1. Bars of $\phi > 32\text{ mm}$ shall be used only in elements whose minimum depth is not less than 15ϕ .
2. When large bars are used, adequate crack control shall be ensured either by using surface reinforcement or by calculation.
3. The minimum concrete cover should be $c \geq \phi$.
4. The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be not less than the maximum bar diameter or $d_g + 5\text{ mm}$ where d_g is the maximum aggregate size.

Bond:

1. For bar diameter $\phi > 32\text{ mm}$ the values f_{bd} in Sheet No. I.25 should be multiplied by the coefficient $(\frac{132}{\phi} - \phi)/100$ (ϕ in mm).

Anchorage and joints

1. Large diameter bars shall be anchored as straight bars or by means of mechanical devices. They shall not be anchored in tension zones.
2. Lapped joints shall not be used either for tension or compression bars.
3. The rules given below are complementary to those given in Clause 5.2.3.
4. In the absence of transverse compression, additional transverse reinforcement is needed in the anchorage zone in beams and slabs, additional to the shear reinforcement.
5. For straight anchorages (see the Sheet No. I.25) the additional reinforcement in (4) above should not be less than the following:
 - (a) in the direction parallel to the lower face:

$$A_{st} = n_1 \cdot 0.25 A_s$$

- (b) in the direction perpendicular to the lower face:

$$A_{sv} = n_2 \cdot 0.25 A_s$$

where:

A_s denotes the cross-sectional area of an anchored bar

n_1 is the number of layers with bars anchored at the same point in the member

n_2 is the number of bars anchored in each layer

6. The additional transverse reinforcement should be uniformly distributed in the anchorage zone with spacings, which should not exceed approximately five times the diameter of the longitudinal reinforcement.

7. For surface reinforcement, Clause 5.4.2.4 of the code applies, but the area of surface reinforcement should not be less than $0.01 A_{ct,ext}$ in the direction perpendicular to large diameter bars, and $0.02 A_{ct,ext}$ parallel to those bars.

Bundled high bond bars

General:

1. Unless otherwise stated, the rules for individual bars also apply for bundles of bars. In a bundle, all the bars shall be of the same diameter and characteristics (type and grade).
2. In design, the bundle is replaced by a notional bar having the same sectional area and the same centre of gravity as the bundle.

The 'equivalent diameter' \emptyset of this bar is such that:

$$\emptyset_n = \emptyset \sqrt{n_b} \geq 55 \text{ mm}$$

where n_b is the number of bars in the bundle, which is limited to:

$$n_b \leq 4 \text{ for vertical bars in compression and for bars in a lapped joint}$$

$$n_b \leq 3 \text{ for all other cases.}$$

3. For a bundle, 5.2.1.1(2), of the code applies, while using the equivalent diameter \emptyset_n , but measuring the clear distance from the actual external contour of the bundle of bars. The concrete cover measured from the actual external contour of the bundles should be $c > \emptyset_n$.

Anchorage and joints

1. Anchorage or lapping of a bundle of bars shall be achieved by anchorage or lapping of the individual bars. Only straight bar anchorages are permitted; they shall be staggered.
2. For bundles of 2, 3 or 4 bars, the staggering distance of the anchorages should be 1.2, 1.3 and 1.4 times the anchorage length of the individual bars, respectively.
3. The bars should be lapped one by one. In any case not more than 4 bars should be present in any one section. The lapped joints of the individual bars should be staggered as given in (2) above.

I.9.3. American standards: ACI and ASTM and states' practices

I.9.3.1. Shape codes methodology

I.9.3.2. Detailing provision

The shape codes given by the ACI and ASTM are given in this section. The reader should compare them with those adopted by the British codes (Section I.9.1). Methods of comparison are self-evident.

Notation

- A_c = area of core of spirally reinforced compression member measured to outside diameter of spiral, in.² (mm²).
- A_{cv} = net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, in.² (mm²).
- A_g = gross area of section, in.² (mm²).
- A_s = area of non-prestressed tension reinforcement, in.² (mm²).
- b_w = web width, in. (mm).
- c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, in. (mm).

- d = distance from extreme compression fibre to centroid of tension reinforcement, in. (mm).
 d_b = bar diameter, in. (mm).
 f'_c = specified compressive strength of concrete, psi (MPa).
 f_y = specified yield strength of non-prestressed reinforcement, psi (MPa).
 h = overall thickness of member, in. (mm).
 l_d = development length, in. (mm).
 l_{dh} = development length for a bar with a standard hook, in. (mm).
 l_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, in (mm).
 M_u = factored moment at section.
 s = spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, in. (mm).
 s_o = maximum spacing of transverse reinforcement, in. (mm).
 ρ = ratio of non-prestressed tension reinforcement.
 $\rho_v = A_{sv}/A_{cv}$; where A_{sv} is the projection on A_{cv} of area of distributed shear reinforcement crossing the plane of A_{cv} .

1.9.3.3. Referenced standards

The documents of the various organizations referred to in this standard are listed below with their serial designation, including year of adoption or revision. The documents listed shall be the latest edition because some of these documents are revised frequently, generally in minor detail only, the user of this book should check directly with the sponsoring group if it is desired to refer to the latest revision.

American Association of State Highway and Transportation Officials

AASHTO Standard Specifications for Highway Bridges, 16th Edition 1996

American Concrete Institute

- | | |
|---------|--|
| 117-90 | Standard Tolerances for Concrete Construction and Materials |
| 318-95 | Building Code Requirements for Structural Concrete |
| 318M-95 | Building Code Requirements for Structural Concrete (Metric) |
| 343R-95 | Analysis and Design of Reinforced Concrete Bridge Structures |
| 349-97 | Code Requirements for Nuclear Safety Related Concrete Structures |
| 359-92 | Code for Concrete Reactor Vessels and Containments |

American Railway Engineering and Maintenance-of-Way Association

Manual for Railway Engineering, Chapter 8, Concrete Structures and Foundations, 1996

American Society/or Testing and Materials

- | | |
|------------------|--|
| A 82-97a | Standard Specification for Steel Wire, Plain, for Concrete Reinforcement |
| A 185-97 | Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement |
| A 496-97a | Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement |
| A 497-97 | Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement |
| A 615/A 615M-96a | Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement |

A 616/616M-96a	Standard Specification for Rail-Steel Deformed and Plain A Bars for Concrete Reinforcement
A 617/A 617M-96a	Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
A 706/A 706M-96b	Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A 767/A 767M-97	Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A 775/A 775M-97	Standard Specification for Epoxy-Coated Reinforcing Steel Bars

American Society of Civil Engineers

ASCE 7-95 Minimum Design Loads for Buildings and Other Structures

American Welding Society

DI.4-98 Structural Welding Code—Reinforcing Steel

Association for Information and Image Management

Modern Drafting Techniques for Quality Microreproductions

Building Seismic Safety Council

NEHRP-97 NEHRP Recommended Provisions for Seismic Regulations for New Buildings

Concrete Reinforcing Steel Institute

Manual of Standard Practice, 26th Edition, 2nd Printing, 1998

Reinforcement Anchorages and Splices, 4th Edition 1997

1.9.3.4. Bending

To avoid creating excessive stresses during bending, bars must not be bent too sharply. Controls are established by specifying the minimum inside radius or inside diameter of bend that can be made for each size of bar. The radius or diameter of the bend is usually expressed as a multiple of the nominal diameter of the bar d_b . The ratio of diameter of bend to diameter of bar is not a constant because it has been found by experience that this ratio must be larger as the bar size increases.

The minimum diameters of bend specified by ACI 318 (318M) for reinforcing bars, measured on the inside of the bar, are as shown in Table I.10.

Table I.10.

Bar sizes no.	Other than ties/stirrups	Ties or stirrups
3, 4, 5 (10, 13, 16)	$6d_b$	$4d_b$
6, 7, 8 (19, 22, 25)	$6d_b$	$6d_b$
9, 10, 11 (29, 32, 36)	$8d_b$	—
14, 18 (43, 57)	$10d_b$	—

The inside diameter of bends of welded-wire fabric (plain or deformed) for stirrups and ties, as specified by ACI 318 (318M), shall not be less than $4d_b$ for deformed wire larger than D6 (MD38.7) and $2d_b$ for all other wires. Bends with inside diameter of less than $8d_b$ shall not be less than $4d_b$ from the nearest welded intersection.

1.9.3.5. Hooks

ACI 318 (318M), Section 7.2, specifies minimum bend diameters for reinforcing bars. It also defines standard hook (Section 7.1) to mean the following:

- (a) A 180° bend plus an extension of at least $4d_b$, but not less than $2\frac{1}{2}$ in. (60 mm), at the free end of the bar, or
- (b) A 90° bend plus an extension of at least $12d_b$ at the free end of the bar, or
- (c) For stirrup and tie hooks only, either a 90° bend plus $6d_b$ extension for No. 3, 4, 5 (No. 10, 13, 16), and $12d_b$ extension for No. 6, 7, and 8 (No. 19, 22 and 25), or a 135° bend plus an extension of at least $6d_b$ at the free end of the bar. For closed ties, defined as hoops in Chapter 21 of ACI 318 (318M), a 135° bend plus an extension of at least $6d_b$ but not less than 3 in. (75 mm).

The minimum bend diameter of hooks shall meet the foregoing provisions. The standard hooks (Sheet No. I.27) were developed such that the minimum requirements were met, but at the same time the need to allow for springback in fabrication and maintaining a policy of production fabrication pin size no smaller than the ASTM A615/A615M bend test pin size was recognized as well. On Sheet No. I.27, the extra length of bar allowed for the hook is designated as A or G and shown to the nearest 1 in. (25 mm) for end hooks and to the nearest 1/4 in. (5 mm) for stirrup and tie hooks.

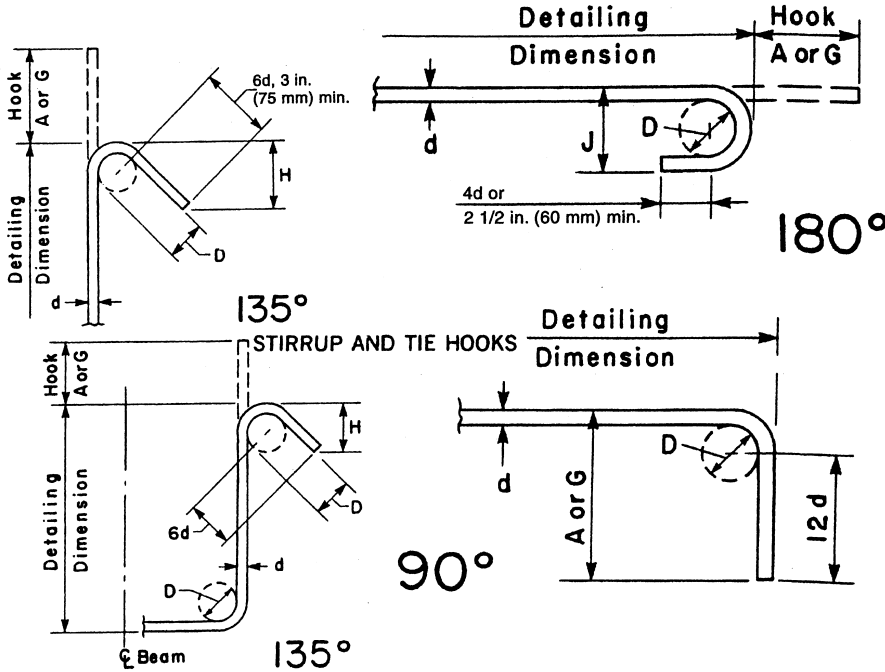
Where the physical conditions of the job are such that either J, A, G or H of the hook is a controlling dimension, it must be so noted on the drawings, schedules and bar lists.

**SHAPE CODE HOOKS AND STIRRUPS
(BASED ON ACI CODES)**

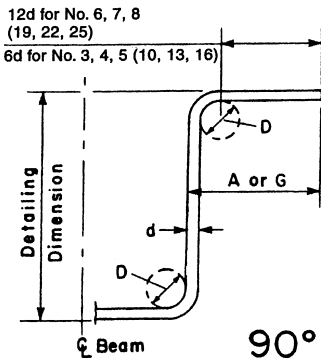
SHEET NO. I.27

–Standard hooks: All specific sizes recommended meet minimum requirements of ACI 318

135° SEISMIC STIRRUP/TIE HOOKS



**RECOMMENDED END HOOKS
All grades
D = Finished bend diameters**



Bar size, No.	D, in. (mm)	180 degree hook		90 degree hook
		A or G, ft-in. (mm)	J, ft-in. (mm)	A or G, ft-in. (mm)
3 (10)	2 1/4 (60)	5 (125)	3 (80)	6 (155)
4 (13)	3 (80)	6 (155)	4 (105)	8 (200)
5 (16)	3 3/4 (95)	7 (180)	5 (130)	10 (250)
6 (19)	4 1/2 (115)	8 (205)	6 (155)	1-0 (300)
7 (22)	5 1/4 (135)	10 (250)	7 (175)	1-2 (375)
8 (25)	6 (155)	11 (275)	8 (205)	1-4 (425)
9 (29)	9 1/2 (240)	1-3 (375)	11 3/4 (300)	1-7 (475)
10 (32)	10 3/4 (275)	1-5 (425)	1-1 1/4 (335)	1-10 (550)
11 (36)	12 (305)	1-7 (475)	1-2 3/4 (375)	2-0 (600)
14 (43)	18 1/4 (465)	2-3 (675)	1-9 3/4 (550)	2-7 (775)
18 (57)	24 (610)	3-0 (925)	2-4 1/2 (725)	3-5 (1050)

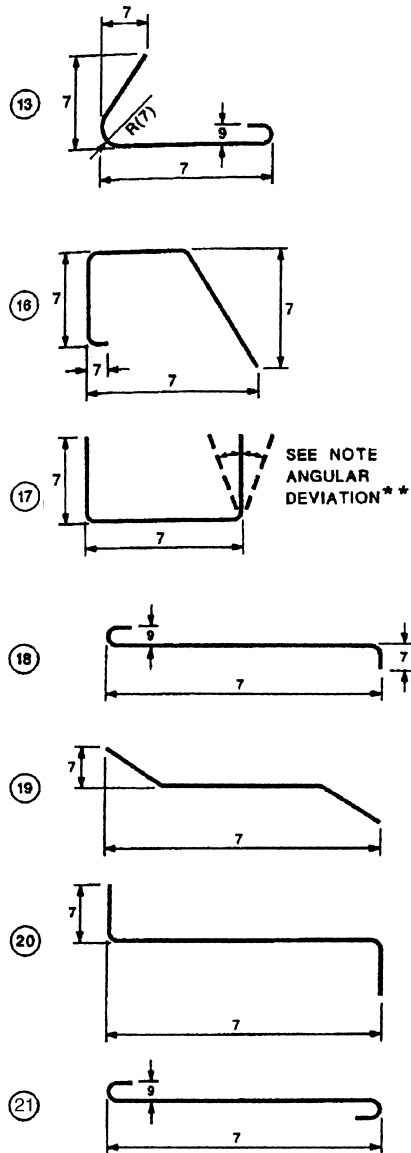
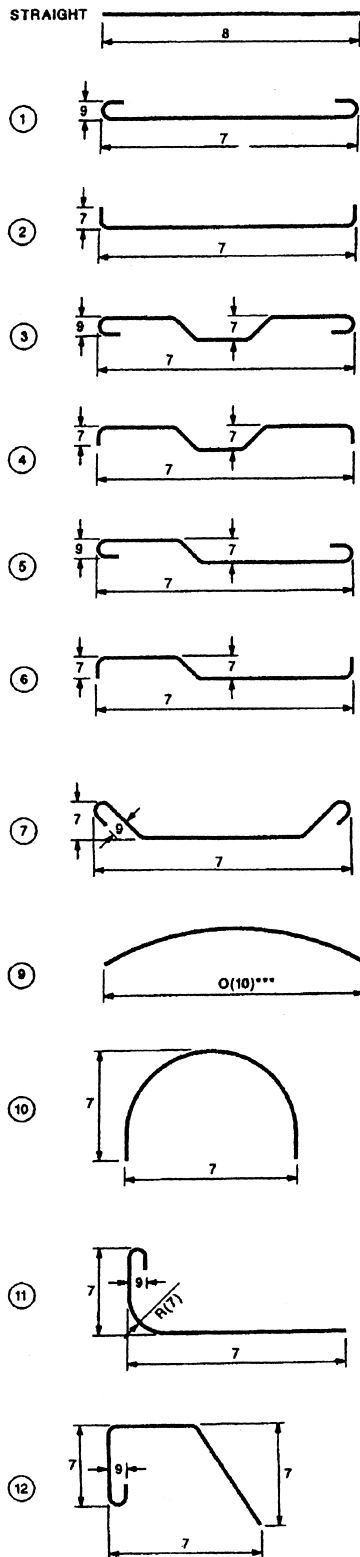
*Finished bend diameters include "spring back" effect when bars straighten out slightly after being bent and are slightly larger than minimum bend diameters in 3.7.2.

**STIRRUP
(TIES SIMILAR)
STIRRUP AND TIE HOOK DIMENSIONS
ALL GRADES**

Bar size, No.	D, in. (mm)	90 degree hook		135 degree hook	
		Hook A or G, ft-in. (mm)	Hook A or G, ft-in. (mm)	Hook A or G, ft-in. (mm)	H approx., ft-in. (mm)
3 (10)	1 1/2 (40)	4 (105)	4 (105)	4 (105)	2 1/2 (65)
4 (13)	2 (50)	4 1/2 (115)	4 1/2 (115)	4 1/2 (115)	3 (80)
5 (16)	2 1/2 (65)	6 (155)	5 1/2 (140)	5 1/2 (140)	3 3/4 (95)
6 (19)	4 1/2 (115)	1-0 (305)	8 (205)	8 (205)	4 1/2 (115)
7 (22)	5 1/4 (135)	1-2 (355)	9 (230)	9 (230)	5 1/4 (135)
8 (25)	6 (155)	1-4 (410)	10 1/2 (270)	10 1/2 (270)	6 (155)
			4 1/4 (110)	4 1/4 (110)	3 (80)
			4 1/2 (115)	4 1/2 (115)	3 (80)
			5 1/2 (140)	5 1/2 (140)	3 3/4 (95)
			8 (205)	8 (205)	4 1/2 (115)
			9 (230)	9 (230)	5 1/4 (135)
			10 1/2 (270)	10 1/2 (270)	6 (155)

**135 DEGREE SEISMIC STIRRUP/TIE
HOOK DIMENSIONS
ALL GRADES**

**SHAPE CODE HOOKS AND STIRRUPS
(BASED ON ACI CODES)**



TOLERANCE SYMBOLS

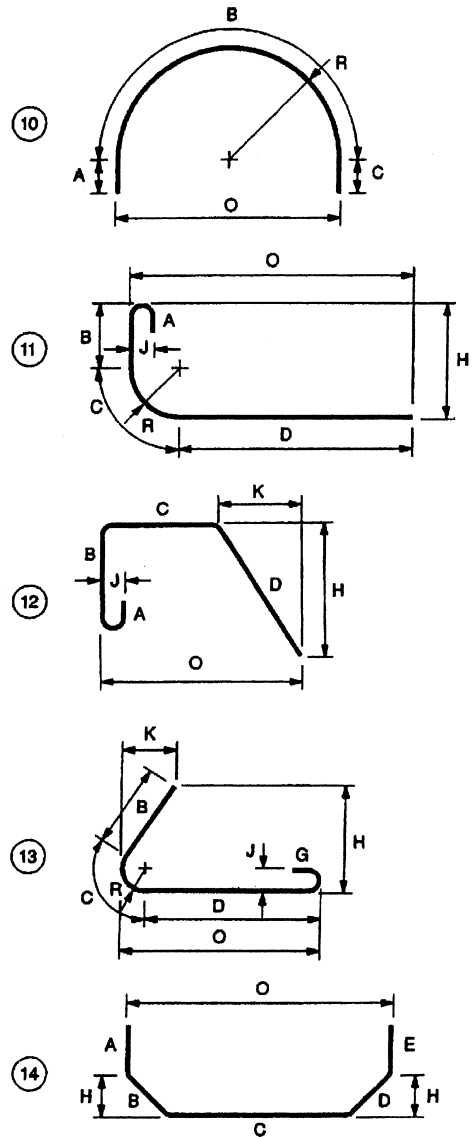
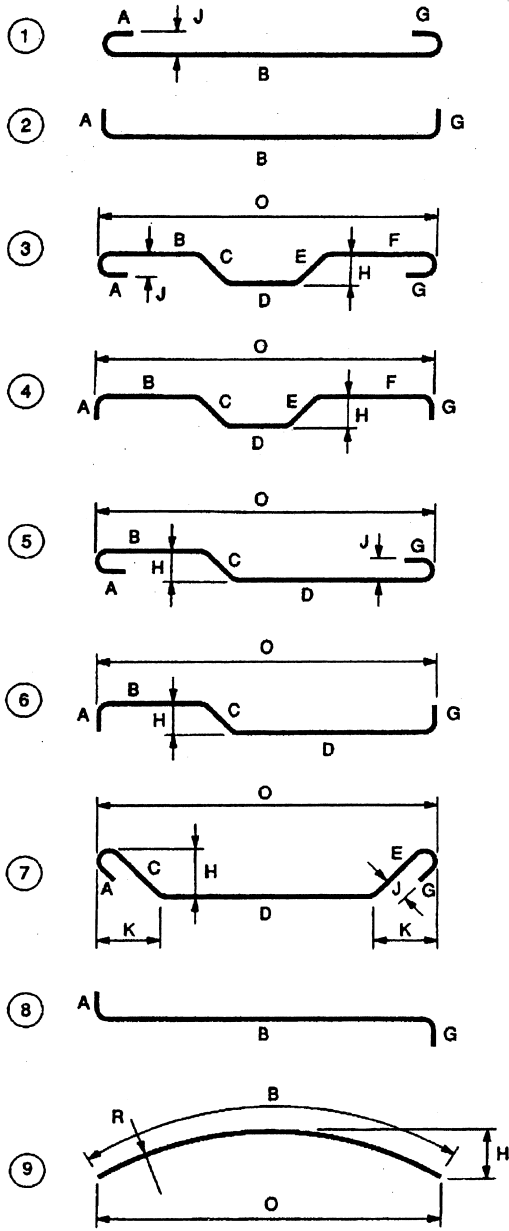
Symbol	No. 14 (No. 43)	No. 18 (No. 57)
7	= 2 1/2 in. (65 mm)	± 3 1/2 in. (90 mm)
8	± 2 in. (50 mm)	± 2 in. (50 mm)
9	± 1 1/2 in. (40 mm)	± 2 in. (50 mm)
10 = 2% × "O" dimension, ≥	± 2 1/2 in. (65 mm) min.	± 3 1/2 in. (90 mm) min.

Note: All tolerances single plane as shown.
 *Saw-cut both ends—Overall length ± 1/2 in. (15 mm).
 **Angular deviation—Maximum ± 2 1/2 degrees or ± 1/2 in./ft (40 mm/m) on all 90 degree hooks and bends.
 ***If application of positive tolerance to Type 9 results in a chord length ≥ the arc or bar length, the bar may be shipped straight.

Standard fabricating tolerances for bar sizes No. 14 and 18 (No. 43 and 57).

**SHAPE CODE HOOKS AND STIRRUPS
(BASED ON ACI CODES)**

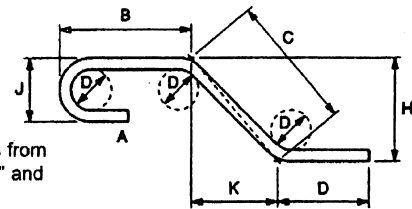
SHEET NO. I.27 (contd)



- Notes:
1. All dimensions are out-to-out of bar except "A" and "G" on standard 180 and 135 degree hooks.
 2. "J" dimensions on 180 degree hooks to be shown only where necessary to restrict hook size, otherwise ACI standard hooks are to be used.
 3. Where "J" is not shown, "J" will be kept equal or less than "H" on Types 3, 5, and 22. Where "J" can exceed "H," it should be shown.
 4. "H" dimension stirrups to be shown where necessary to fit within concrete.
 5. Where bars are to be bent more accurately than standard fabricating tolerances, bending dimensions that require closer fabrication should have limits indicated.
 6. Figures in circles show types.
 7. For recommended diameter "D" of bends and hooks, see Section 3.7.1; for recommended hook dimensions, see Table 1.
 8. Unless otherwise noted, diameter "D" is the same for all bends and hooks on a bar (except for Types 11 and 13).

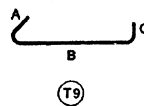
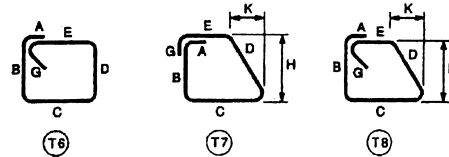
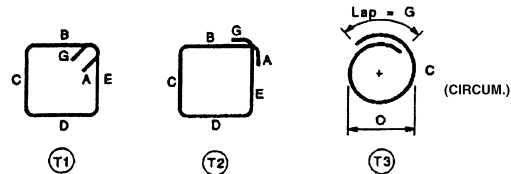
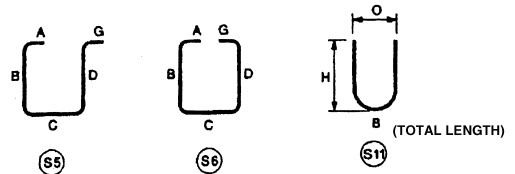
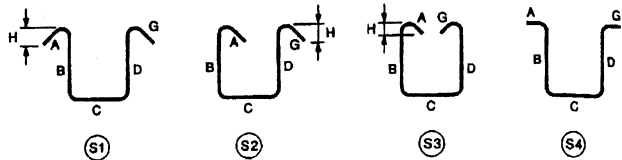
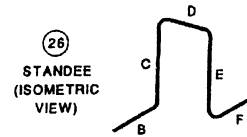
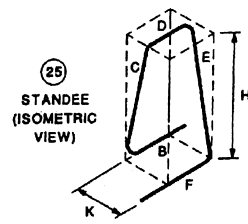
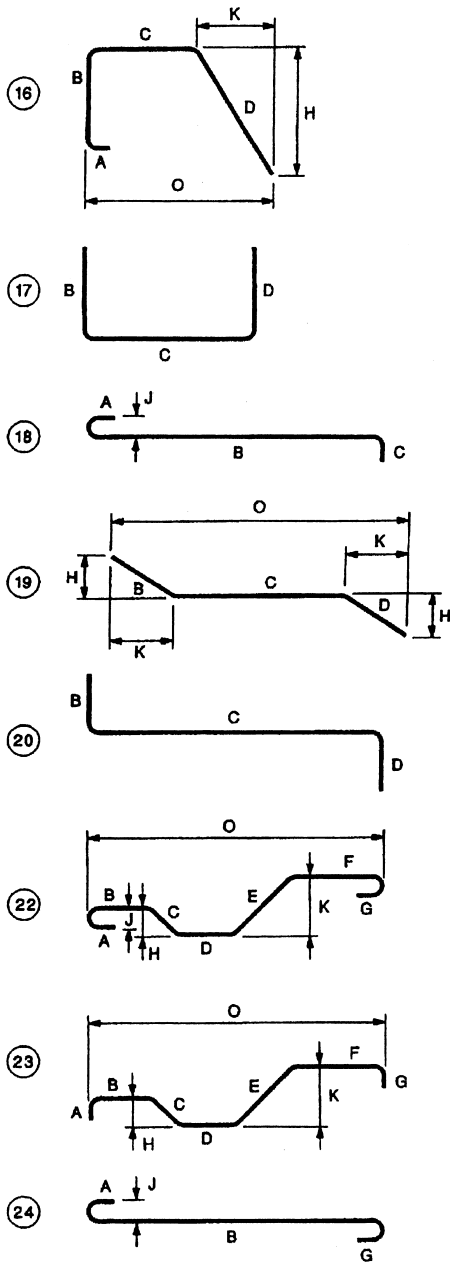
Where slope differs from 45° dimensions, "H" and "K" must be shown.

**ENLARGED VIEW SHOWING
BAR BENDING DETAILS**

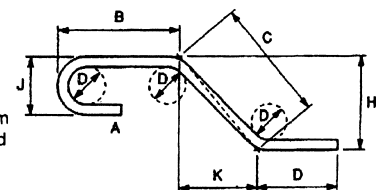


**SHAPE CODE HOOKS AND STIRRUPS
(BASED ON ACI CODES)**

SHEET NO. I.27 (contd)



Where slope differs from 45° dimensions, "H" and "K" must be shown.



**ENLARGED VIEW SHOWING
BAR BENDING DETAILS**

- Notes:
1. All dimensions are out-to-out of bar except "A" and "G" on standard 180 and 135 degree hooks.
 2. "J" dimensions on 180 degree hooks to be shown only where necessary to restrict hook size, otherwise ACI standard hooks are to be used.
 3. Where "J" is not shown, "J" will be kept equal or less than "H" on Types 3, 5, and 22. Where "J" can exceed "H," it should be shown.
 4. "H" dimension stirrups to be shown where necessary to fit within concrete.
 5. Where bars are to be bent more accurately than standard fabricating toler-

6. Figures in circles show types.
7. For recommended diameter "D" of bends and hooks, see Section 3.7.1; for recommended hook dimensions, see Table 1.
8. Type S1 through S6, S11, T1 through T3, T6 through T9: apply to bar sizes No. 3 through 8 (No. 10 through 25).
9. Unless otherwise noted, diameter "D" is the same for all bends and hooks on a bar (except for Types 11 and 13).

1.9.3.6. Stirrup anchorage

There are several permissible methods for stirrup anchorage. The most common is to use one of the hooks shown in Sheet No. I.27. Types S1 to S6 illustrate not only the uses of the two types of hooks, but also the directions in which the hooks can be turned. In detailing the anchorage, care must be taken that the ends of stirrup hooks that are turned outward into shallow slabs have adequate cover. If not, the hooks should be turned inward and this change brought to the A/E's attention.

Where the free ends of stirrups cannot be tied to longitudinal bars, or where there are no longitudinal bars, stirrup support bars should be specified by the A/E.

1.9.3.7. Standard bar bends

To list the various types of bent bars in a schedule it is necessary to have diagrams of the bars with the lengths of the portions of the bars designated by letters. A chart of such standard bar bends is shown in Sheet No. I.27.

Dimensions given for Hooks A and G are the additional length of bar allowed for the hook as shown in Sheet No. I.27. For straight portions of the bar, the distance is measured to the theoretical intersection of the outside edge line extended to the outside edge line of the adjacent straight portion, or to the point of tangency to a curve, from which point the length of the latter is tabulated, as in Types 10 and 11. Truss bar dimensioning is special and is shown in large-scale detail.

1.9.3.8. Radius bending

When reinforcing bars are used around curved surfaces, such as domes or tanks, and no special requirement is established in the contract documents, bars prefabricated to a radius equal or less than those in Table I.11 are prefabricated by the reinforcing bar fabricator. In the smaller sizes, the bars are sprung to fit varying job conditions, such as location of splices, vertical bars, jack rods, window openings and other blocked out areas in the forms. The larger size bars, which are more difficult to spring into desired position, are

Table I.11. When radial prefabrication is required

Bars are to be prefabricated when either radius or bar length is less than tabulated value

Bar size no.	Radius: ft (mm)	Bar length: ft (mm)
3 (10)	5 (1500)	10 (3000)
4 (13)	10 (3000)	10 (3000)
5 (16)	15 (4500)	10 (3000)
6 (19)	40 (12 000)	10 (3000)
7 (22)	40 (12 000)	10 (3000)
8 (25)	60 (18 000)	30 (9000)
9 (29)	90 (27 000)	30 (9000)
10 (32)	110 (33 000)	30 (9000)
11 (36)	110 (33 000)	60 (18 000)
14 (43)	180 (54 000)	60 (18 000)
18 (57)	300 (90 000)	60 (18 000)

ordinarily employed in massive structures where placing tolerances are correspondingly larger. Table I.11 shows parameters for radial fabrication.

Radially prefabricated bars of any size tend to relax the radius originally prefabricated as a result of time and normal handling. The last few feet involved in the lap splice area often appear as a tangent rather than a pure arc, due to limitations of standard bending equipment. For these reasons, final adjustments are a field placing problem to suit conditions and tolerance requirements of a particular job. See Figures 8 and 9 for radial tolerances and Section 4.2(c)3 of the code. Bars requiring a larger radius or length than shown in Table I.11 are sprung in the field without prefabrication.

The presence of the tangent end does not create any problem on bar sizes No. 3 through 11 (No. 10 through 36) as they are generally lap spliced and tangent ends are acceptable. No. 14 and 18 (No. 43 and 57) bars cannot be lap spliced, however, and are usually spliced using a proprietary mechanical splice or a butt weld. It is a problem to place a radially bent bar when using a mechanical splice sleeve because of the tangent ends on bars bent to small radii. To avoid this problem, all No. 14 and 18 (No. 43 and 57) bars bent to a radius of 20 ft (6000 mm) or less should be furnished with an additional 18 in. (450 mm) added to each end. This 18 in. (450 mm) tangent end is to be removed in the field by flame cutting. Bars bent to radii greater than 20 ft (6000 mm) will be furnished to the detailed length with no consideration given to the tangent end. The ends of these bars generally are saw cut.

Shop removal of tangent ends can be made by special arrangement with the reinforcing bar supplier.

1.9.3.9. Slants

To determine the length of the straight bar necessary to form a truss bar, the length of the slant portion of the bar must be known. The standard angle is 45° for truss bars, with any other angles being special. Slants and increments are calculated to the closest 1/2 in. (10 mm) so that for truss bars with two slants, the total increment will be in full inches (25 mm). This makes the computation easier and is within the tolerances permitted. It is important to note that when the height of the truss is too small, 45° bends become impossible. This condition requires bending at a lesser angle and lengthens the slant portion.

1.9.3.10. Tolerances

There are established, standard industry fabricating tolerances that apply unless otherwise shown in the project specifications or structural drawings. Sheet No. I.27 define these tolerances for the standard bar bends shown in Sheet No. I.27. Note that tolerances more restrictive than these may be subject to an extra charge. For further tolerance information, see ACI 117.

II. Reinforced concrete beams and slabs

II.1. Reinforced concrete beams

Beams are structural elements carrying external loads that cause bending moments, shear forces and torsional moments along their length. The beams can be singly or doubly reinforced and can be simply supported, fixed or continuous. The structural details of such beams must resist bending, diagonal tension, shear and torsion and must be such as to transmit forces through a bond without causing internal cracking. The detailer must be able to optimize the behaviour of the beams under load. He must liaise with the structural engineer on the choice of structural details needed for particular conditions.

The shapes of the beam can be square, rectangular, flanged or tee (T). Although it is more economical to use concrete in compression, it is not always possible to obtain an adequate sectional area of concrete owing to restrictions imposed on the size of the beam (such as restrictive head room). The flexural capacity of the beam is increased by providing compression reinforcement in the compression zone of the beam which acts with tensile reinforcement. It is then called a doubly reinforced concrete beam. As beams usually support slabs, it is possible to make use of the slab as part of a T-beam. In this case the slab is generally not doubly reinforced.

Where beams are carried over a series of supports, they are called continuous beams. A simple beam bends under a load and a maximum positive bending moment exists at the centre of the beam. The bottom of the beam which is in tension is reinforced. The bars are cut off where bending moments and shear forces allow it. This aspect was discussed in Section I. In a continuous beam the sag at the centre of the beam is coupled with the hog at the support. A negative bending moment exists at the support. Where a positive moment changes to a negative moment, a point of contraflexure or inflection occurs at which the bending moment is zero. An adequate structural detailing is required to cater for these changes. Again this aspect is discussed in Section I. The reinforcement bars and their cut-off must follow the final shape of the final bending moment diagram.

Where beams, either straight or curved, are subjected to inplane loading, they are subjected to torsional moments in addition to flexural bending and shear. The shape of such a moment must be carefully studied prior to detailing of reinforcement. The codes including BS 8110 give a comprehensive treatment on the provision of shear reinforcement, namely links and bent bars. Again, whether the beams are simply supported, rigid or continuous the shear force diagram will give a proper assessment of the number and spacing of such bars.

In circumstances where the bars are given lap lengths, they must be in line with the provisions of a code. As discussed in Section I, all bars are checked for bond using standard formulae, so that it should be possible to transfer stresses from one material to the other. The structural detailing of reinforcing bars must prevent relative movement or slip between them and the concrete.

As discussed earlier, the increased compressive area of concrete obtained by using a T-beam is not available at the support. Over the support, the

compression zone lies below the neutral axis. In order to strengthen the beam at the support a greater depth with a haunch is provided. The beam will have a different section at the support from that at the centre. Special care is needed to design and detail such a beam.

II.1.1. Detailing based on British codes and practices

Since beams are reinforced in the longitudinal direction against bending, Sheet No. II.1(a) shows structural detailing of simply supported reinforced concrete beams for light loading (II.1(a)(i)) and for heavy loading (II.1(a)(iii)) together with an isometric view (II.1(a)(ii)) indicating how main bars and links are placed. The reinforcement layouts are self-explanatory, for example under II.1(a)(i), 28R110-03-175 means 28 numbers of round 10 mm diameter mild steel bars of identification number 3 are placed at 175 mm centre to centre. All such bar sizes and spacings are determined from the loading and secondary conditions such as fire and corrosion. Where down-stand and upstand beams in construction become necessary, an optimum reinforcement layout should be devised. One such layout is shown in Sheet No. II.1(b). A typical doubly reinforced concrete beam layout is given in II.1(c)(i) and II.1(c)(ii).

Sheet No. II.2 demonstrates how links and bent bars are placed in relation to main reinforcement. The examples chosen are for rectangular, L- and T-beams with a single system of links. A double system of links is specifically included in II.2(b)(iv). Straight and inclined bars for resisting shear are detailed under II.2(c).

Sheet No. II.3(a) gives reinforcement layouts for both inverted and upright T-beams. A composite bending moment diagram is given in II.3(b)(i) with cut-off positions along with two types of reinforcement layouts. A single continuous beam is shown in II.3(b)(ii). A continuous beam with bent bars in a frame is detailed with several cross-sections in II.3(c). Continuous beams with slabs and columns are detailed separately in Section IV.

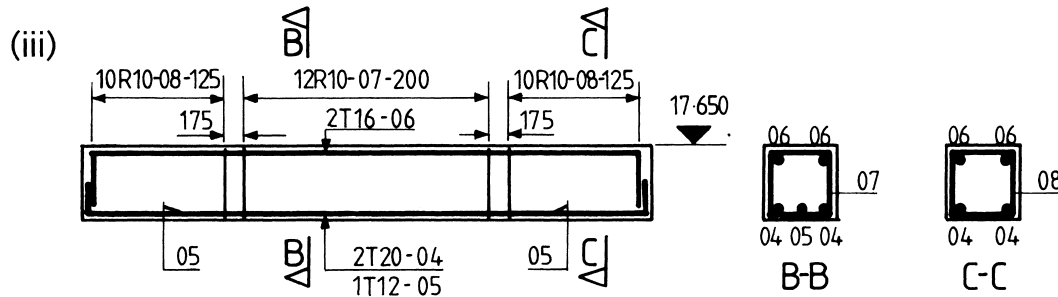
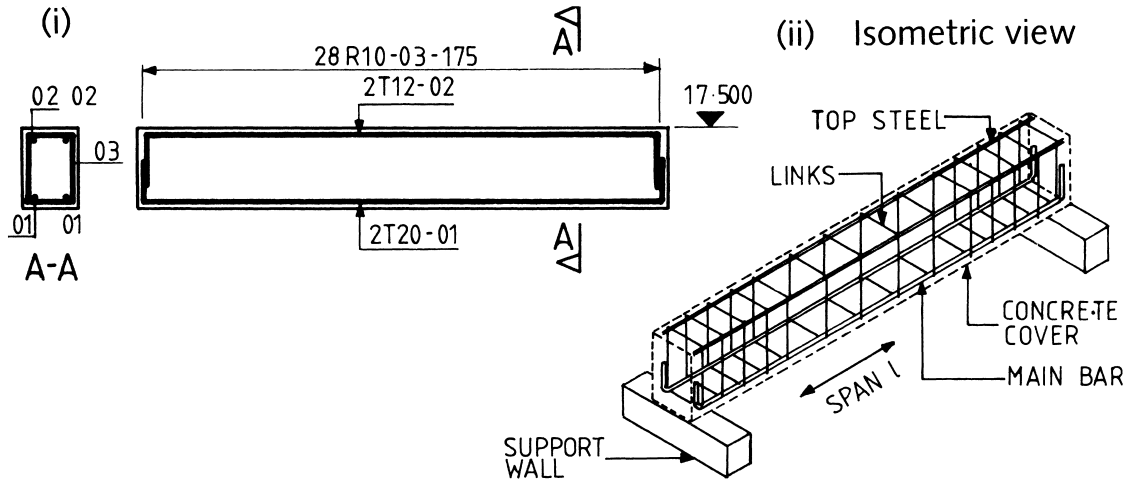
Sheet No. II.4 shows some details of interconnected beams with and without holes and shear bars.

- (a) Beam grid (Sheet No. II.4). Cases (i) and (ii) show the layout and a typical detailing of primary and secondary beams. Main reinforcement, shear links or stirrups and connecting U-bars are clearly indicated.
- (b) Beam monolithic with a wall. When a beam is monolithic with a wall, the minimum lap or bond length of a hook shall be 0.1 times beam length or 45 times the diameter of the bar. The total steel area of the top bars with hooks shall not be less than half of the total area of main steel. This is shown in case (b) on Sheet No. II.4.
- (c) Cantilever beams. A cantilever beam shall be reinforced in a manner shown in case (c) on Sheet No. II.4. Again, the top hooked bars of total steel area A_s shall have a bond length not less than half of the effective span length. Where bars are extended beyond 0.5 times length or 45 times diameter, the area of steel shall not be less than half the steel area A_s .
- (d) Holes in a beam. There are several ways of reinforcing holes in a beam. The most well known are the square and the orthogonal layouts which are shown in case (d) on Sheet No. II.4. In all cases the bond length beyond the hole shall not be less than 45 times the diameter of the bar.
- (e) Bent-up bars. Sometimes shear links and their shear amount cannot resist enormous shear forces. The bent-up bars are introduced to resist these shear forces. A typical layout is shown in case (e) on Sheet No. II.4.

BEAMS

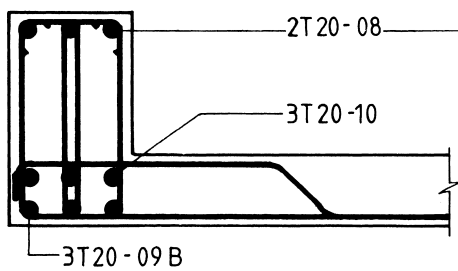
SHEET NO. II.1

(a) Simply supported rectangular beams

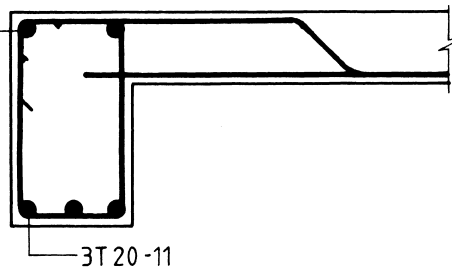


(b) Simply supported L-Beams

(i) Upright

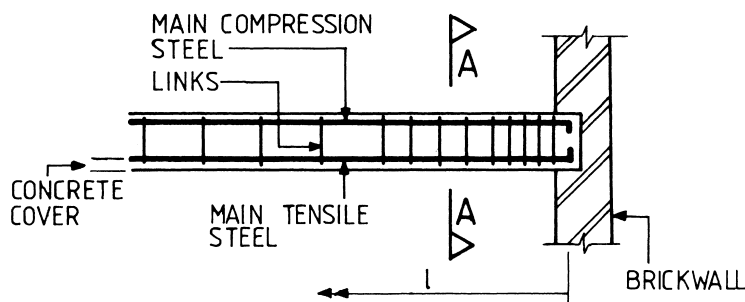


(ii) Downstand

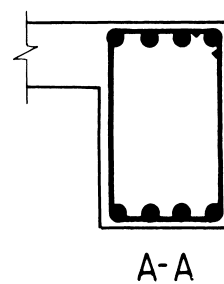


(c) Simply supported doubly reinforced beams

(i)

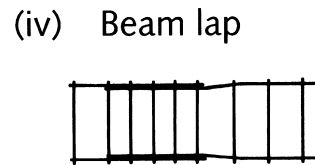
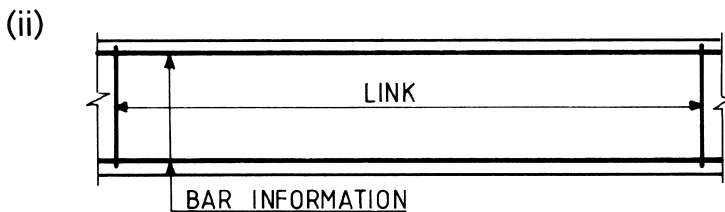
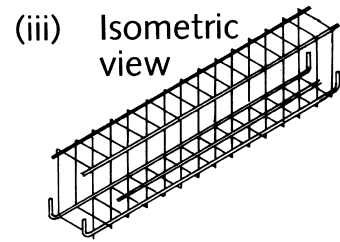
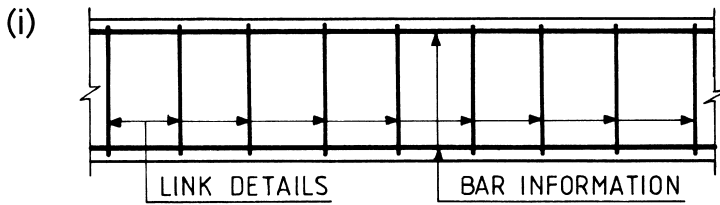


(ii)

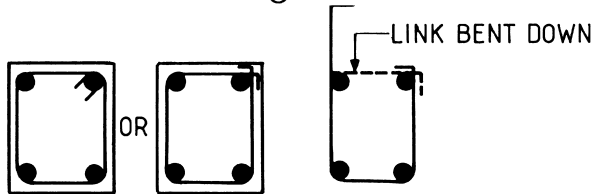


LINKS AND BENT BARS

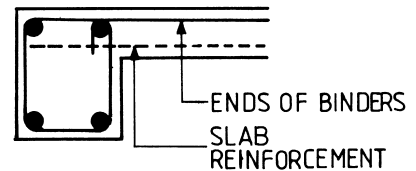
(a) Links in beams



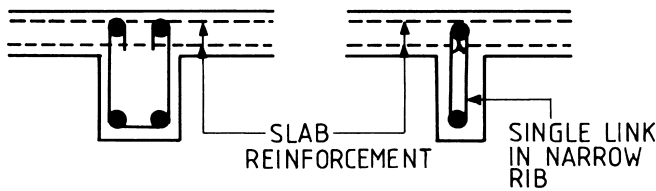
(b) (i) Links in rectangular beams



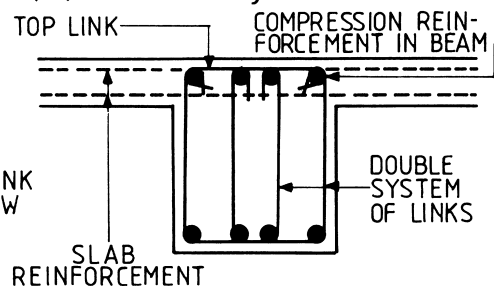
(ii) Links in L-Beams



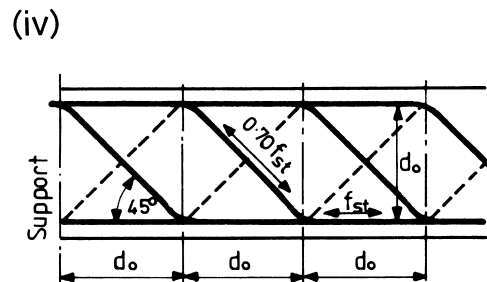
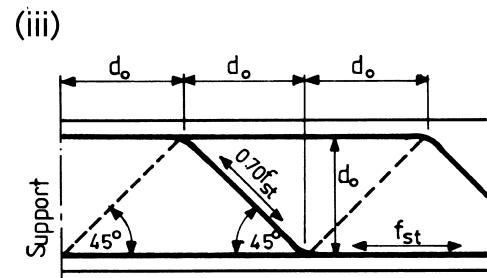
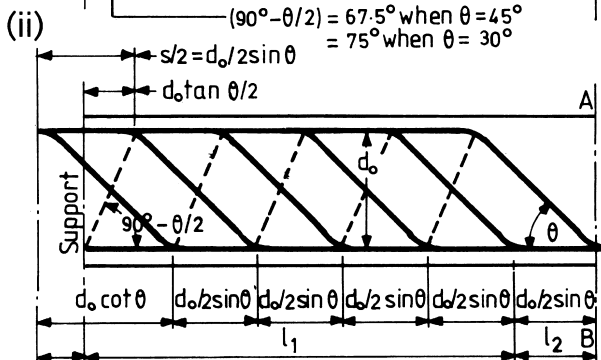
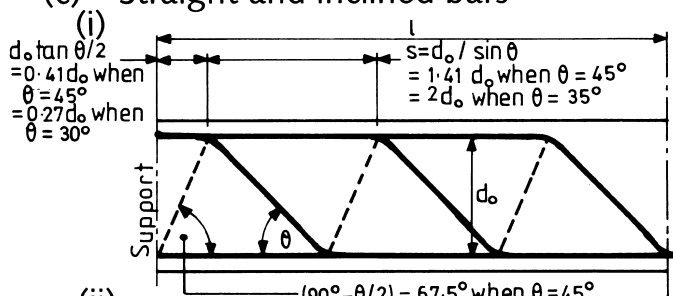
(iii) Links in T-Beams



(iv) Double system of links



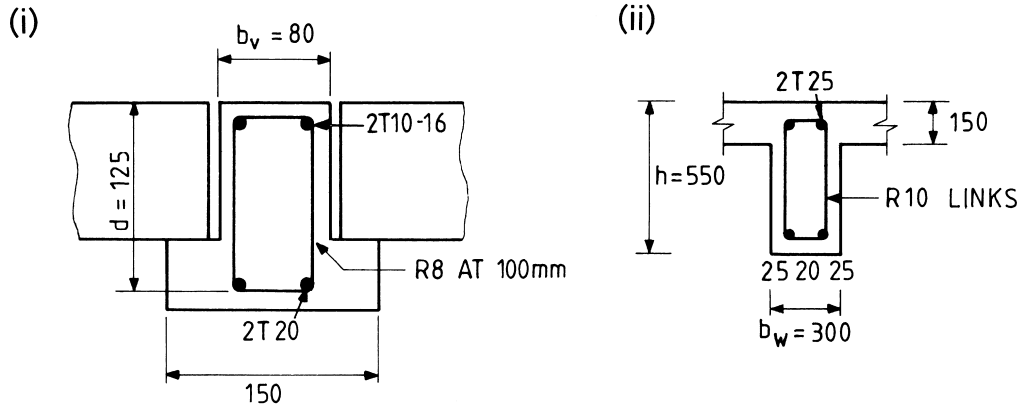
(c) Straight and inclined bars



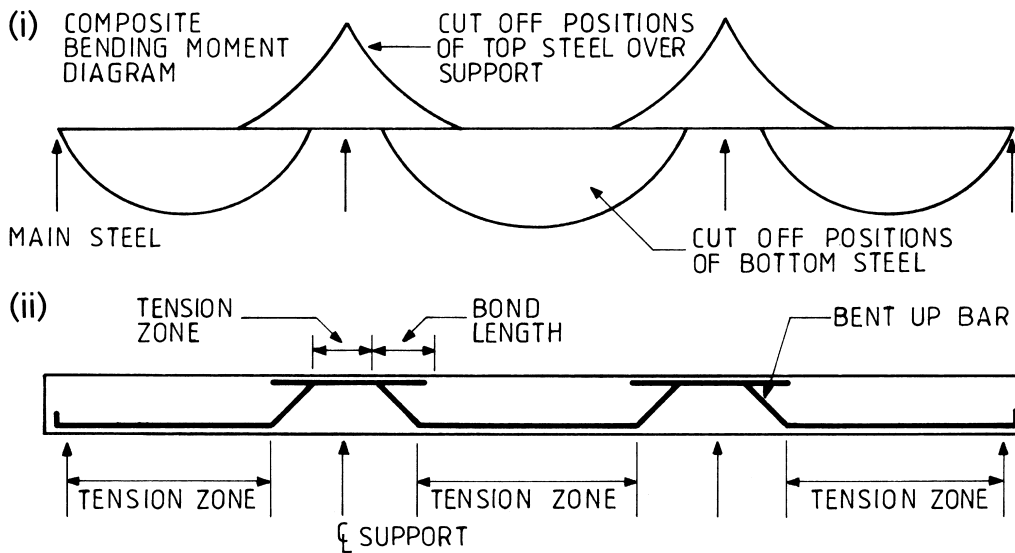
REINFORCEMENT LAYOUTS

SHEET NO. II.3

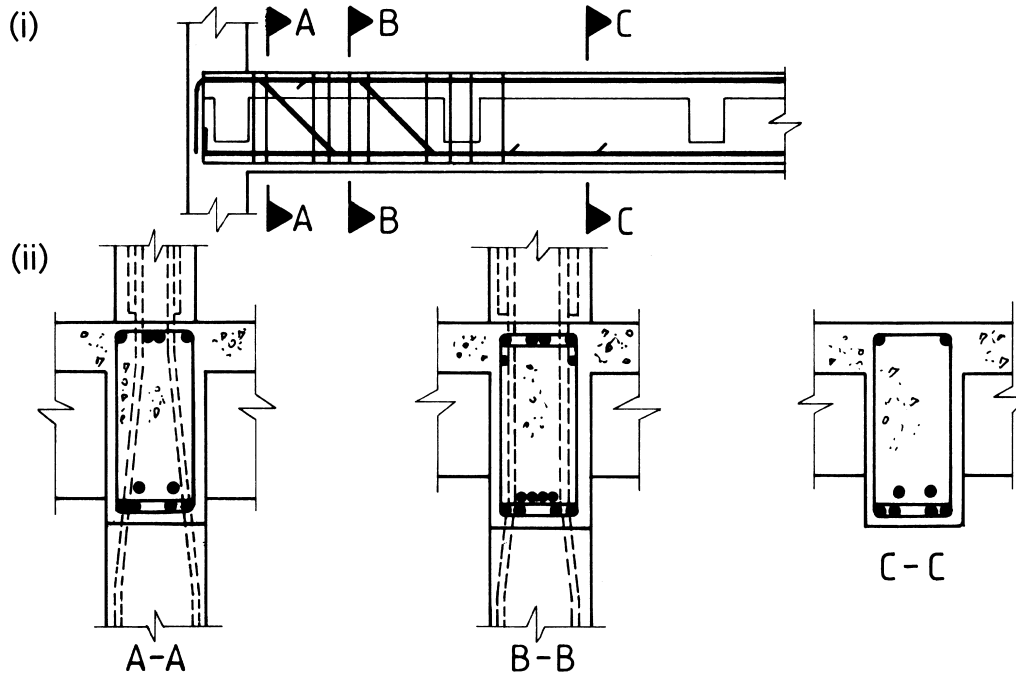
(a) Simply supported T-beam



(b) Continuous beams

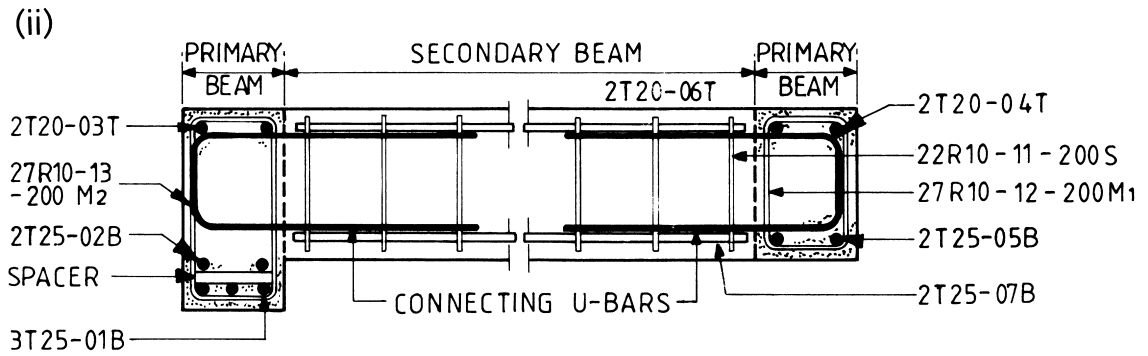
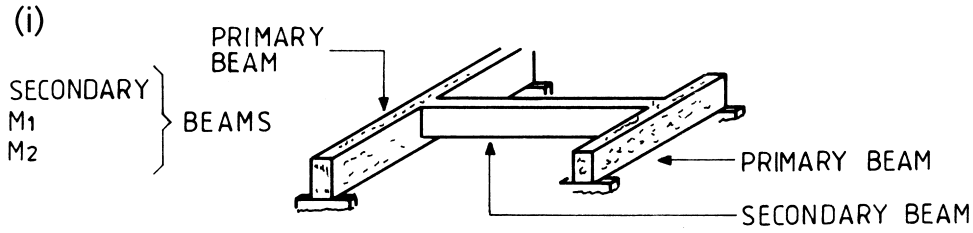


(c) Continuous beam in a frame with bent bars

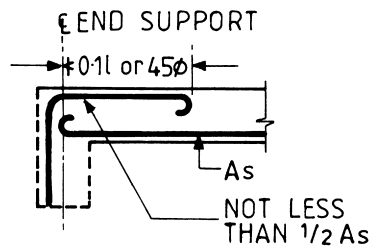


INTERCONNECTED BEAMS

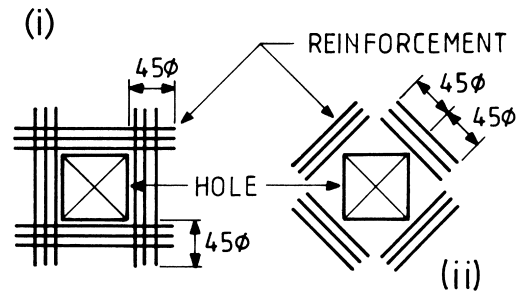
(a) Beam grid



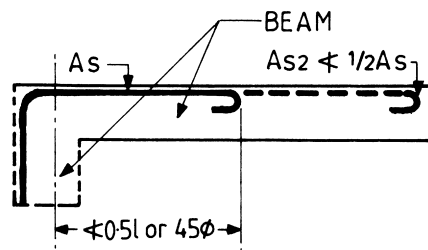
(b) Beam monolithic with walls



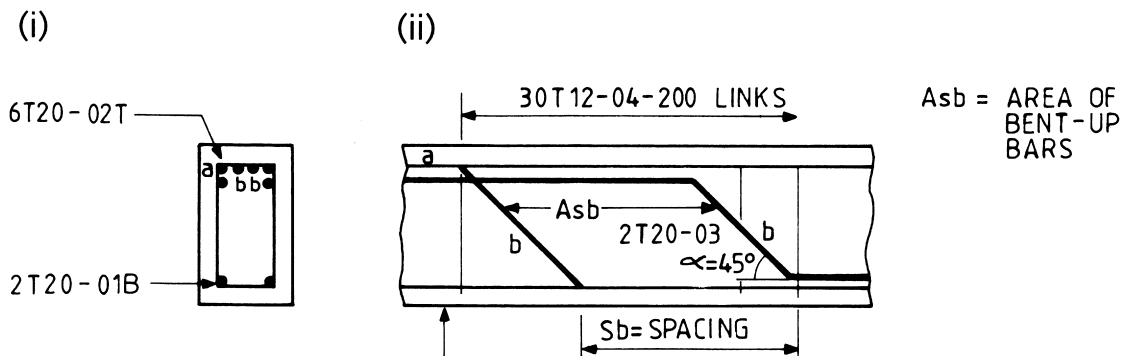
(d) Holes in a beam



(c) Cantilever beams



(e) Shear resistance of bent-up bars



II.1.1.1. Curtailment of bars in beams

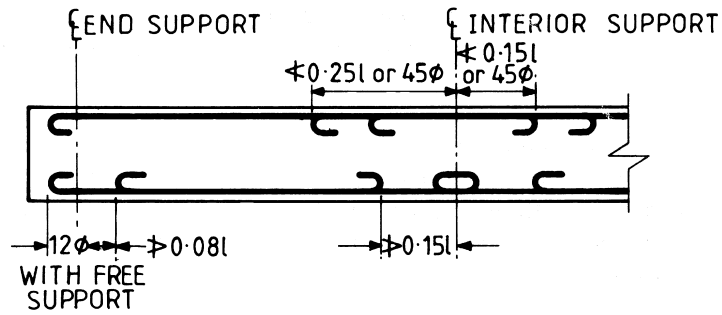
Bending moments and other loading effects vary from one section of the span to the other. Where maximum effects are achieved, a correct amount of reinforcement is provided. As the maximum effects are reduced, economy of reinforcement is achieved by stopping-off or curtailment bars (BS 8110 or any others). The codes generally give clear-cut rules for curtailment in different elements of structures. Cases given on Sheet No. II.5 are based on BS 8110. Where other codes are involved, the bibliography should be consulted and the drawings modified and prepared accordingly.

The general layout of the reinforcement is based on both bending moments in spans and bending moments due to direct loads on columns. Typical examples are shown in Sheet No. II.6.

Where the ends are restrained, the provisions for U-bars, trombone bars and L-bars are given in Sheet No. II.6(a). Where in beam areas, the slabs cannot be avoided, the general recommendations for bar curtailment are given in Sheet No. II.6(b). For a continuous beam/slab with straight bars, the lap lengths and bars curtailment shall be in accordance with part (c) on Sheet No. II.6.

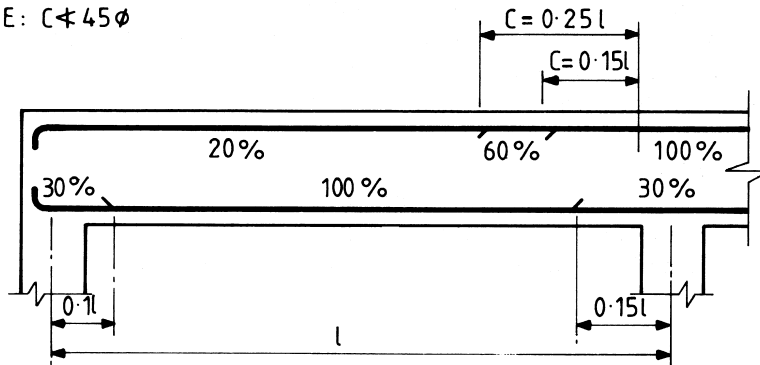
CURTAILMENT OF BARS IN BEAMS

(a) Continuous beams without bent bars with hooks

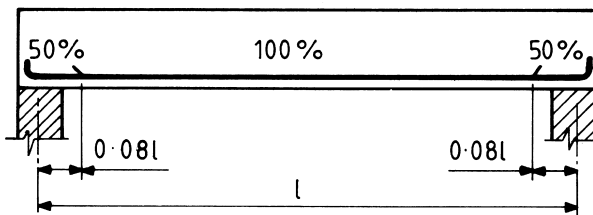


(b) Continuous beams with straight bars and end anchorage

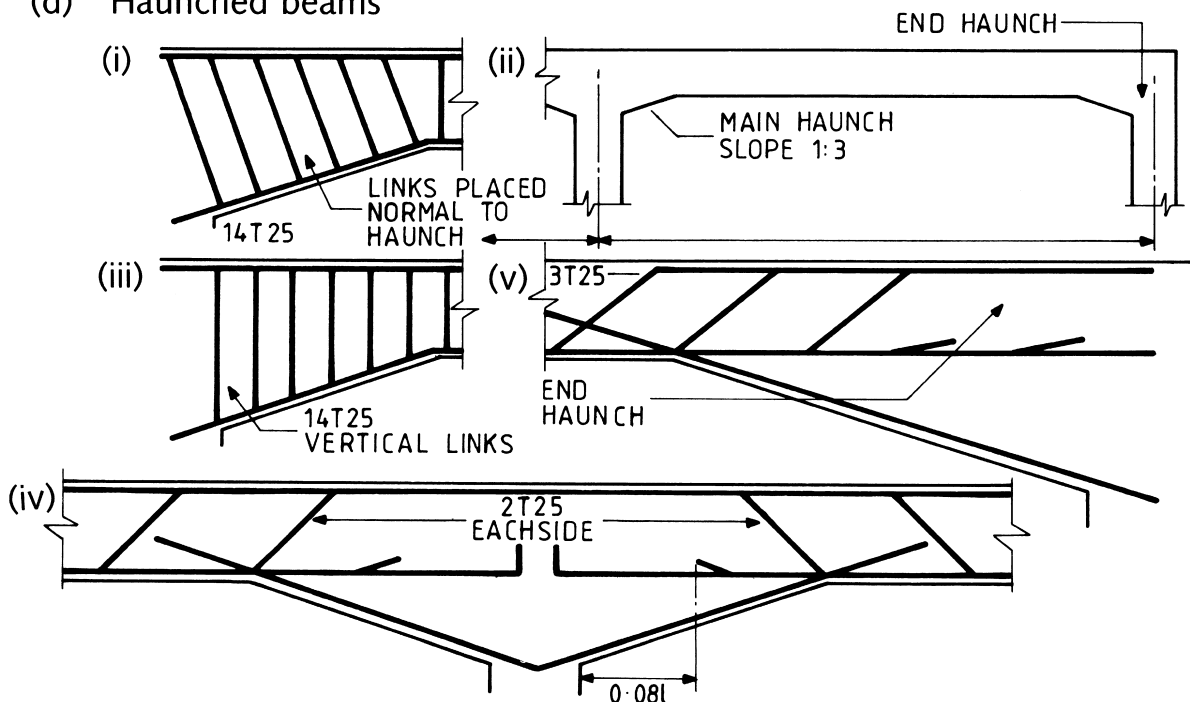
NOTE: $C \leq 45\phi$



(c) Simply supported beams



(d) Haunched beams

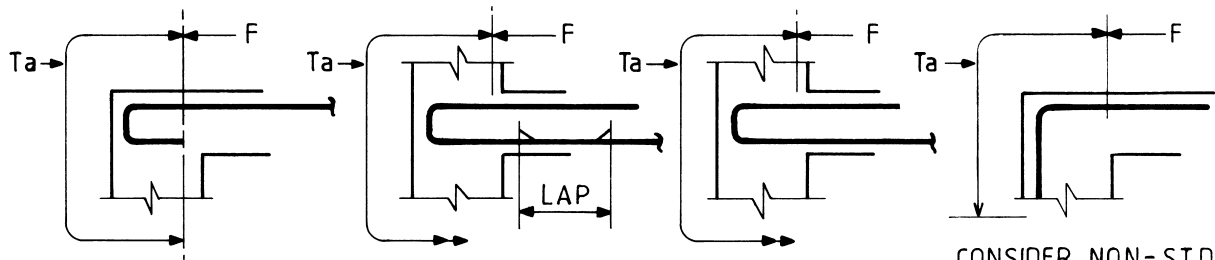


LAPS AND BAR CURTAILMENT

SHEET NO. II.6

(a) Restrained ends

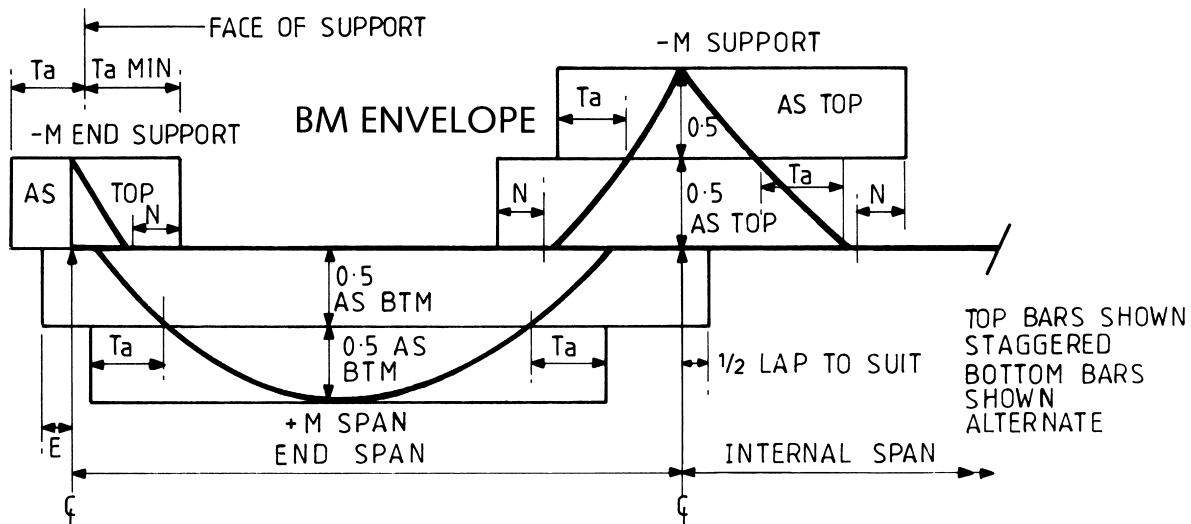
- (i) U-Bars in top (ii) U-Bars extended (iii) 'Trombone' bars (iv) L-Bars in top



KEY F= FACE OF VERTICAL BAR IN RESTRAINING MEMBER
 Ta= TENSION ANCHORAGE LENGTH

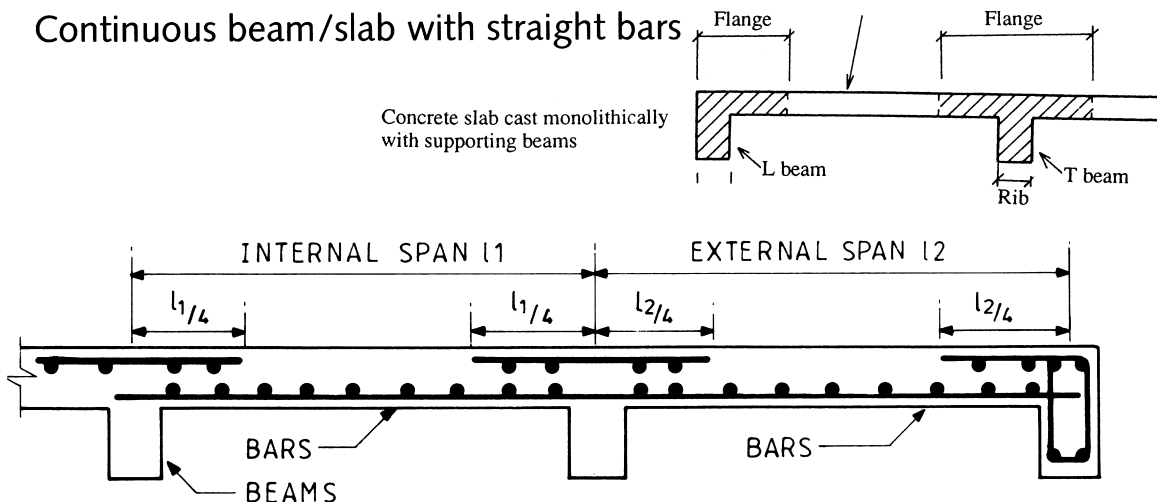
CONSIDER NON-STD RADIUS VERTICAL LEG > 8φ

(b) General recommendations for bar curtailment in slabs



Ta = TENSION ANCHORAGE LENGTH
 E = EFFECTIVE END ANCHORAGE REQUIRED IF SIMPLE SUPPORT
 N = NOMINAL 12φ or d (EFFECTIVE DEPTH) WHICHEVER IS GREATER
 φ = BAR NOMINAL SIZE
 BTM = BOTTOM
 BM,M=BENDING MOMENT

(c) Continuous beam/slab with straight bars



II.1.1.2. Requirements for beams

There are a number of dimensional requirements and limitations applicable to concrete beams which the designer needs to consider since they can affect the design:

- (a) effective span of beams
- (b) deep beams
- (c) slender beams
- (d) main reinforcement areas
- (e) minimum spacing of reinforcement
- (f) maximum spacing of reinforcement.

Certain other aspects such as bond, anchorage and, if applicable, the curtailment and lap lengths of reinforcement, require consideration at the detailing stage.

The main structural design requirements for which concrete beams should be examined are as follows:

- (a) bending ULS
- (b) cracking SLS
- (c) deflection SLS
- (d) shear ULS.

Let us now consider how each of these dimensional and structural requirements influences the design of beams.

Effective span of beams

The effective span or length of a simply supported beam may be taken as the lesser of:

- (a) the distance between the centres of bearing
- (b) the clear distance between supports plus the effective depth d .

The effective length of a cantilever should be taken as its length to the face of the support plus half its effective depth d .

Deep beams

Deep beams having a clear span of less than twice their effective depth d are outside the scope of BS 8110. Reference should therefore be made to specialist literature for the design of such beams. Refer also to the following book written by the author, *Manual of numerical methods in concrete: modelling and applications validated by experimental and site-monitoring data* (Thomas Telford, London, 2001). A typical deep beam reinforcement layout under top and bottom loading is shown on Sheet No. II.7.

Slender beams

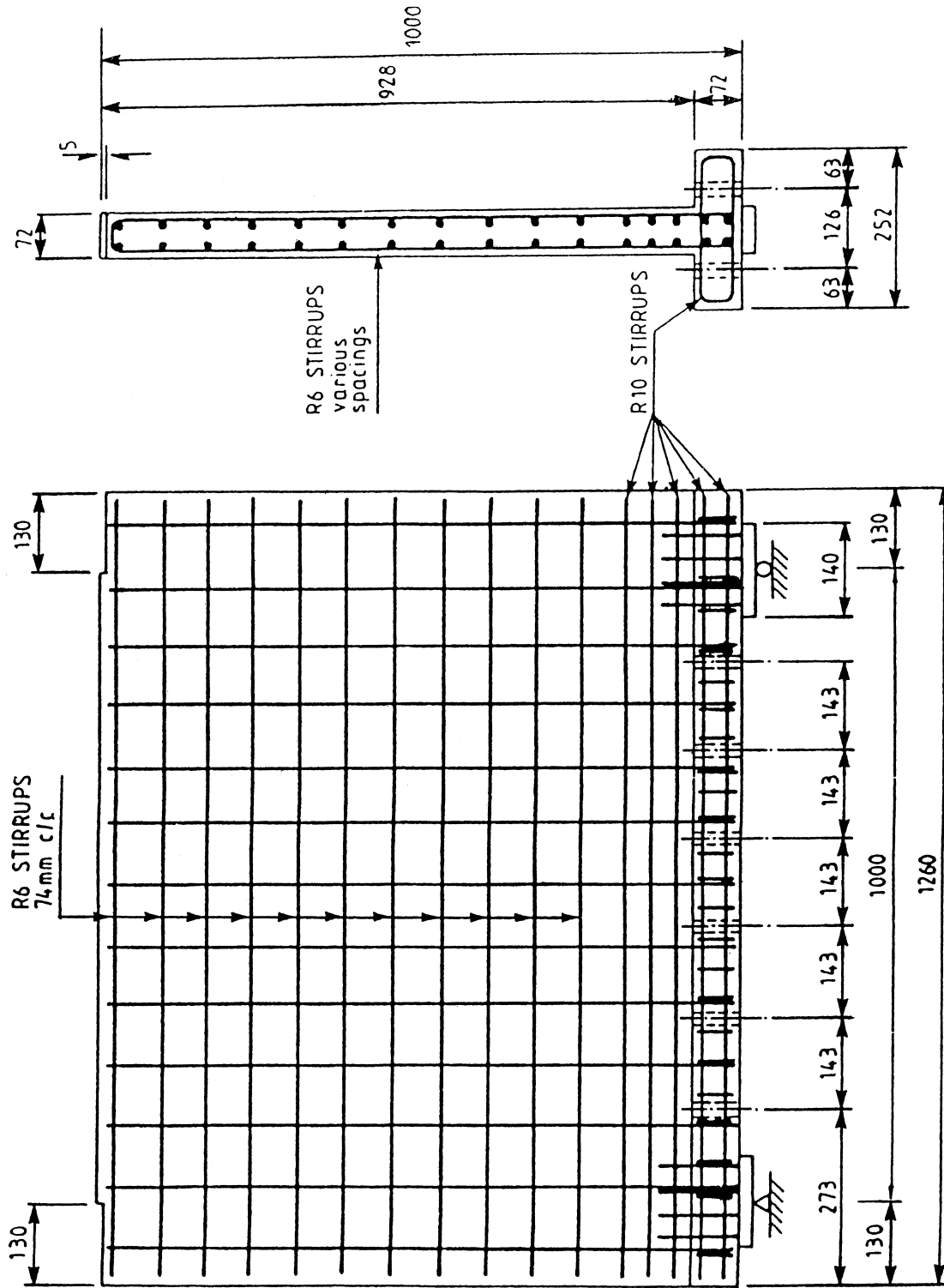
Slender beams, where the breadth of the compression face is small compared with the depth, have a tendency to fail by lateral buckling. To prevent such failure, the clear distance between lateral restraints should be limited as follows:

- (a) for simply supported beams, to the lesser of $60b_c$ or $250b_c^2/d$
- (b) for cantilevers restrained only at the support, to the lesser of $25b_c$ or $100b_c^2/d$.

These slenderness limits may be used at the start of a design to choose preliminary dimensions. Thus, by relating the effective length of a simply supported beam to $60b_c$, an initial breadth can be derived. This can then be substituted in the bending design formula, and an effective depth d determined. Finally this can be compared with the second slenderness limit of $250b_c^2/d$.

DEEP BEAMS UNDER TOP AND BOTTOM LOADING

SHEET NO. II.7



Dimensions and reinforcement details of Leeds test wall-beams (dimensions in mm).

A.R. CUSENS, University of Leeds

Main reinforcement areas

Sufficient reinforcement must be provided in order to control cracking of the concrete. Therefore the minimum area of tension reinforcement in a beam should not be less than the following amounts:

- (a) 0.24% of the total concrete area, when $f_y = 250 \text{ N/mm}^2$
- (b) 0.13% of the total concrete area, when $f_y = 460 \text{ N/mm}^2$.

To ensure proper placing and compaction of concrete around reinforcement, a maximum steel content is also specified. Thus, the maximum area of tension reinforcement in a beam should not exceed 4% of the gross cross-sectional area of the concrete.

The area needed should generally be provided by not less than two bars and not more than eight bars. If necessary, bars may be in groups of two, three or four, in contact. For the purpose of design such groups should be considered as a single bar of equivalent area. In addition the size of main bars used should normally not be less than 16 mm diameter.

Minimum spacing of reinforcement

During concreting the aggregate must be allowed to move between bars in order to achieve adequate compaction. For this reason BS 8110 Part 1 recommends a minimum bar spacing of 5 mm greater than the maximum coarse aggregate size h_{agg} . That is:

$$\text{Minimum distance between bars or group of bars} = h_{agg} + 5 \text{ mm}$$

When the diameter of the main bar or the equivalent diameter of the group is greater than $h_{agg} + 5 \text{ mm}$, the minimum spacing should not be less than the bar diameter or the equivalent diameter of the group.

A further consideration is the use of immersion type (poker) vibrators for compaction of the concrete. These are commonly 40 mm diameter, so that the space between bars to accommodate their use would have to be at least 50 mm.

Maximum spacing of reinforcement

When the limitation of crack widths to 0.3 mm is acceptable and the cover to reinforcement does not exceed 50 mm, the maximum bar spacing rules given in BS 8110 Part 1 may be adopted.

Cracking SLS

Crack widths need to be controlled for appearance and to avoid corrosion of the reinforcement.

The cracking serviceability limit state will generally be satisfied by compliance with detailing rules given in BS 8110 Part 1. These relate to minimum reinforcement areas and bar spacing limits which for beams have already been stated in Sections 3.9.4 and 3.9.6 of BS 8110. They ensure that crack widths will not exceed 0.3 mm.

Where it is necessary to limit crack widths to particular values less than 0.3 mm, perhaps for water tightness, then reference should be made to the guidance given in BS 8110 Part 2.

Deflection SLS

Reinforced concrete beams should be made sufficiently stiff so that excessive deflections, which would impair the efficiency or appearance of the structure, will not occur. The degree of deflection allowed should be commensurate with the capacity of movement of any services, finishes, partitions, glazing, cladding and so on that the member may support or influence.

In all normal situations the deflection of beams will be satisfactory if the basic span to effective depth ratios are as given in BS 8110 Part 1, Table 3.10 (reproduced here).

Table 3.10. Basic span to effective depth ratios for rectangular or flanged beams (BS 8110 Part 1: 1985)

Support conditions	Regular sections	Flanged beams with $b_w/b \leq 0.3$
Cantilever	7	5.6
Simply supported	20	16.0
Continuous	26	20.8

II.1.2. Detailing based on Eurocode 2 and European practices

II.1.2.1. Introduction

Detailing under this code is identical to the one described in Section II.1.1. When it comes to detailing practices, obviously there are some differences which are based on traditions of a particular European country. The principles behind detailing of concrete structures are technically identical. Certain clarifications are given in Section I.

II.1.2.2. Detailing practice of beams

The detailing aspect of reinforced concrete beams is very similar to the one adopted in Section II.1. Certain individual details are exceptional and they have been dealt with within this section. A reference is made to Sheet Nos II.1 to II.6 and II.8 for some noted details on beams.

Longitudinal reinforcement

Minimum area

Minimum area $A_{st,min} < (0.6b_t d / f_{yk}) < 0.0015b_t d$, where f_{yk} is the characteristic yield stress of reinforcement.

At supports in monolithic construction where simple supports are assumed in the design (Sheet No. II.8(f)), $A_{st}(\text{support}) < (1/4) A_{st}(\text{span})$.

Maximum area

Maximum area $A_{st,max}$ or $A_{sc,max} > 0.04A_c$, where A_c is the cross-sectional area of concrete.

Shear reinforcement

General

Shear reinforcement should form an angle of 90° to 45° with the mid-plane of the beam.

Shear reinforcement (Sheet No. II.8) may consist of a combination of:

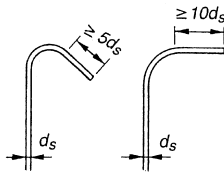
- links enclosing the longitudinal tensile reinforcement and the compression zone
- bent-up bars (figures)
- shear assemblies of cages, ladders, etc., which do not enclose the longitudinal reinforcement but are properly anchored in the compression and tension zones.

All shear reinforcement should be effectively anchored. Lap joints on the leg near the surface of the web are permitted only for high-bond bars.

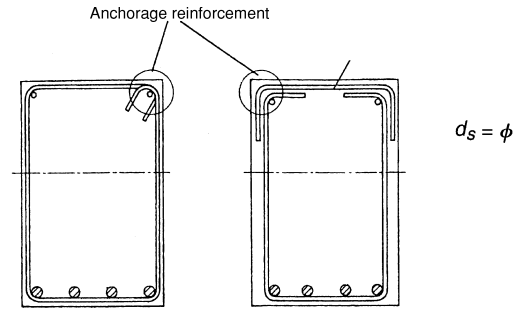
At least 50% of the necessary shear reinforcement should be in the form of links.

Minimum area, A_{sw}

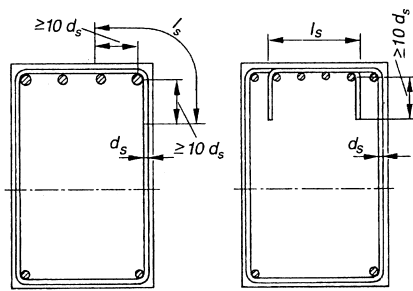
$$\rho_w = A_{sw} / s b_w \sin \alpha$$



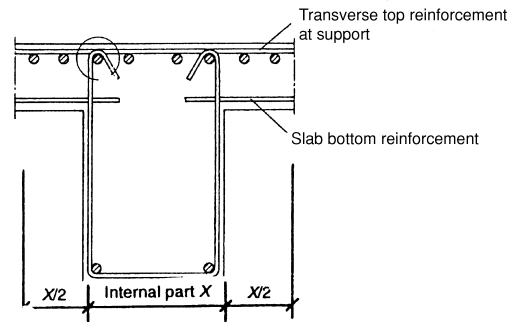
a) *Bending bars and hooks*



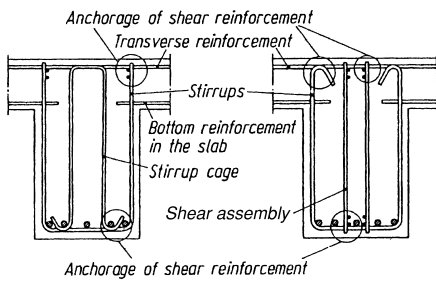
b) *Minimum area of longitudinal reinforcement*



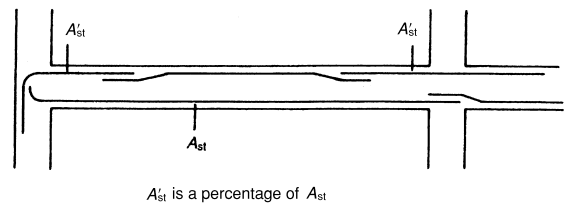
c)



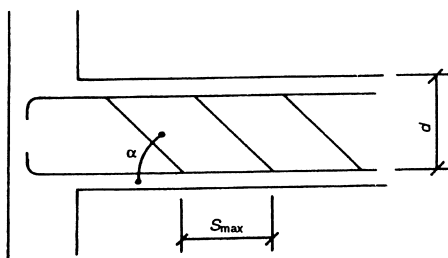
d) *Distribution of support reinforcement*



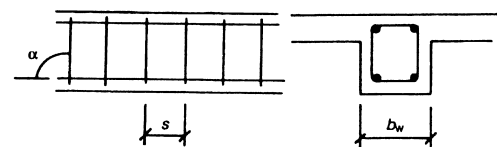
e) *Combination of links and shear assemblies: stirrup cage as shear assembly; ladders as shear assembly*



f) *Longitudinal reinforcement at supports in monolithic construction*



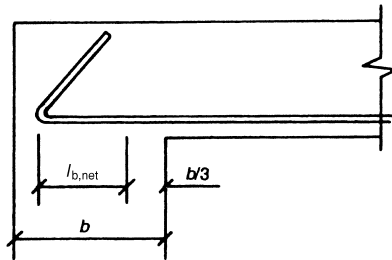
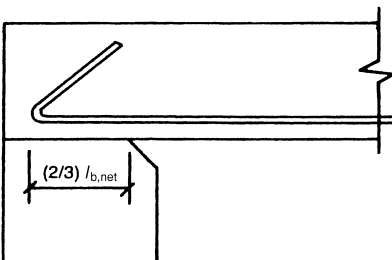
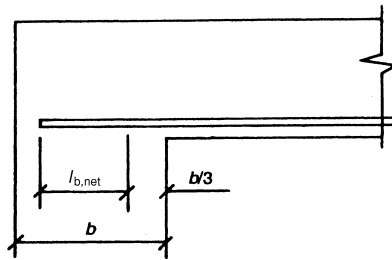
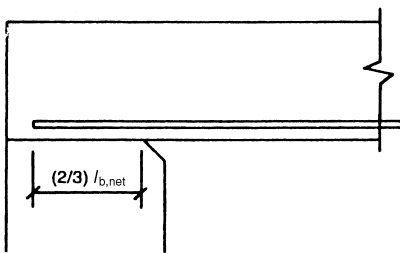
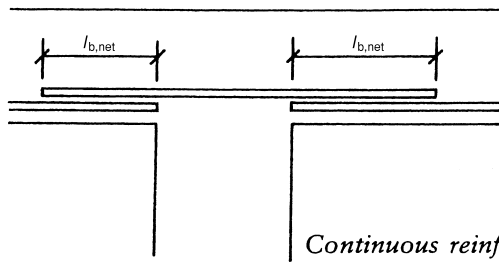
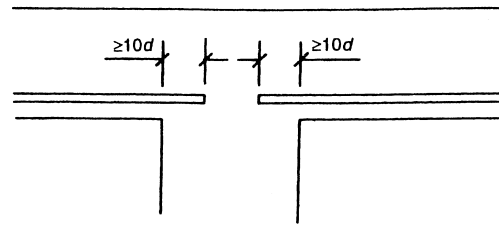
g) *Maximum longitudinal spacing of bent-up bars*



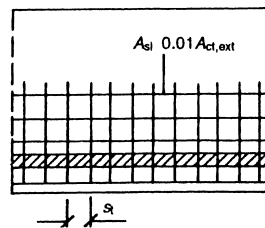
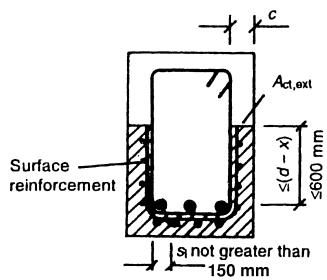
h) *Minimum area of shear reinforcement*

ANCHORAGE REQUIREMENTS (BASED ON EC2)

SHEET NO. II.8 (contd)



Anchorage requirements



Arrangement and anchorage requirements

where ρ_w is the shear reinforcement ratio, A_{sw} is the area of shear reinforcement within length s , and α is the angle between the shear reinforcement and the longitudinal steel. Minimum values for ρ_w are given.

Maximum diameter:

Diameter of reinforcement should not exceed 12 mm where plain round bars are used.

Maximum spacing, S_{max}

See Figs (b) and (d) on Sheet No. II.8 for the maximum longitudinal spacing of links and shear assemblies.

$$V_{sd} \leq \frac{1}{5} V_{Rd2}: S_{max} = 0.8d \nless 300 \text{ mm}$$

$$\frac{1}{5} V_{Rd2} < V_{sd} \leq \frac{2}{3} V_{Rd2}: S_{max} = 0.6d \nless 300 \text{ mm}$$

$$V_{sd} > \frac{2}{3} V_{Rd2}: S_{max} = 0.3d \nless 200 \text{ mm}$$

where V_{sd} is the design shear force and V_{Rd2} is the maximum shear force that can be carried by concrete.

The maximum longitudinal spacing of bent-up bars is given as:

$$S_{max} = 0.6d (1 + \cot \alpha)$$

For the maximum transverse spacing of shear link legs:

$$V_{sd} \leq \frac{1}{5} V_{Rd2}: S_{max} = d \text{ or } 800 \text{ mm, whichever is smaller}$$

$$V_{sd} > \frac{1}{5} V_{Rd2}: \text{as for longitudinal spacing}$$

Curtailment of longitudinal reinforcement

Any curtailed reinforcement should be provided with an anchorage length $l_{b,net}$, but not less than d from the point where it is no longer needed. This should be determined taking into account the tension caused by the bending moment and that implied in the truss analogy used for shear design. This can be done by shifting the point of the theoretical cut-off based on the bending moment by a_1 (see below for definition) in the direction of decreasing moment. This procedure is also referred to as the 'shift rule'.

If the shear reinforcement is calculated according to the standard method:

$$a_1 = z(1 - \cot \alpha)/2 < 0$$

where α is the angle of the shear reinforcement to the longitudinal axis. If the shear reinforcement is calculated according to the variable strut method:

$$a_1 = z(\cot \theta - \cot \alpha)/2 < 0$$

where θ is the angle of the concrete struts to the longitudinal axis. Normally z can be taken as $0.9d$.

For reinforcement in the flange, placed outside the web, a_1 should be increased by the distance of the bar from the web.

Anchorage at supports

End support

When there is little or no fixity at an end support, at least a quarter of the span reinforcement should be carried through to the support. EC2 recommends that the bottom reinforcement should be anchored to resist force of $(V_{sd} a_1/d) + N_{sd}$ where V_{sd} is the shear force at the end, a_1 is as defined in Section 10.2.10.3 of the code for the shift rule and N_{sd} is the axial force, if any, in the member.

EC2 goes on to illustrate the anchorage requirement in Figure 5.12 of the code, which arbitrarily reduces the anchorage requirement to $0.67l_{bnet}$ for direct supports. Clearly there is a presumption of adequate lateral pressure. It may be safer to use the formula in Section 10.2.4.2 of the code and arrive at the anchorage requirements. Figure 5.12 in the code is reproduced in Sheet No. II.8(a), but it must be realized that l_{bnet} for curved bars is 70% of that for straight bars. The anchorages' length should be measured as in Sheet No. II.8(a) and should be l_{bnet} .

Intermediate supports — general requirements

At intermediate supports, $\geq 25\%$ of the mid-span bottom reinforcement should be carried to the support.

If no facer bars are provided, bottom reinforcement should be anchored at a minimum of $10\emptyset$ beyond the face of the support. This does not mean that the support must be greater than $20\emptyset$ wide, as the bars from each side of the support can be lapped. However, it is recommended that continuous reinforcement be provided to resist accidental forces.

Skin reinforcement

Skin reinforcement to control cracking should normally be provided in beams over 1.0 m in depth where the reinforcement is concentrated in a small portion of the depth. This reinforcement should be evenly distributed between the level of the tension steel and the neutral axis, and be located within the links.

Surface reinforcement

Surface reinforcement may be required to resist spalling of the cover, for example arising from fire or where bundled bars or bars greater than $32\downarrow$ are used.

This reinforcement should consist of small-diameter high-bond bars or wire mesh placed in the tension zone outside the links.

The area of surface reinforcement parallel to the beam tension reinforcement should not be less than $0.01A_{ct,ext}$, where $A_{ct,ext}$ is the area of concrete in tension external to the links.

The longitudinal bars of the surface reinforcement may be taken into account as longitudinal bending reinforcement and the transverse bars as shear reinforcement, provided they meet the arrangement and anchorage requirements of these types of reinforcement.

Anchorage length required l_{bnet}

$$l_{b,net} = \alpha_a l_b (A_{s,req} / A_{s,prov}) \leq l_{b,min}$$

where α_a equals 1.0 for straight bars, and equals 0.7 for curved bars in tension if the concrete cover perpendicular to the plane of curvature is at least $3\emptyset$; l_b is the basic anchorage length; $l_{b,min}$ is the minimum anchorage length = $0.6l_b$ for tension, = $0.3l_b$ for compression, $\nless 10\emptyset$ or $\nless 100$ mm).

If welded transverse bars are *present* in the anchorage, the above *expression* for $l_{b,net}$ may be multiplied by 0.7.

Transverse reinforcement

At anchorage, tensile stresses are induced in concrete which tend to split the concrete cover. Lateral reinforcement should be provided to cater for these lateral tensile stresses.

Transverse reinforcement should be provided for:

- (a) anchorage in tension, if no compression is caused by support reactions
- (b) all anchorages in compression.

In tension anchorages, the transverse reinforcement should be evenly distributed along the anchorage length, with at least one bar placed in the region of a hook, bend or loop.

In compression anchorages, the transverse reinforcement should surround the bars and be concentrated at the end of the anchorage, as some of the forces will be transferred by the end of the bar (pin effect) and this in turn will result in bursting forces.

Anchorage of links

Links and shear reinforcement may be anchored using one of the methods shown in Sheet No. II.8. However, in any case the transverse bars are welded.

Spaces between adjacent laps

Laps between bars should be staggered and should not be located at sections of high stress. Spaces between lapped bars should comply with the requirements shown.

Lap lengths, l_s

$$l_s = \alpha_1 l_{b,net} \nless l_{s,min}$$

where:

$\alpha_1 = 1.0$ for compression laps and for tension laps where:

- (a) less than 30% of the bars at a section are lapped
- (b) the clear distance between adjacent lapped bars $>10\emptyset$ and the cover $>5\emptyset$ when \emptyset is the diameter of the bar.

$\alpha_1 = 1.4$ for tension laps where either:

- (a) 30% or more of the bars at a section are lapped, or
- (b) the clear distance between adjacent lapped bars $<10\emptyset$ or the cover $<5\emptyset$.

$\alpha_1 = 2.0$ for tension laps where both (a) and (b) for $\alpha_1 = 1.4$ above are satisfied.

$$l_{s,min} = 0.3 \alpha_a \alpha_1 l_b \nless 15\emptyset \nless 200 \text{ mm}$$

Table II.1. Anchorage and lap lengths as multiples of bar size: deformed bars type $f_{yk} = 460 \text{ N/mm}^2$

Concrete strength:	f_{ck}	20	25	30	35	40
N/mm ²	f_{cu}	25	30	37	45	50
Anchorage: straight bars, compression and tension		44	37	34	30	27
Anchorage: curved bars,* tension		31	26	24	21	19
Laps: compression, tension†		44	37	34	30	27
Laps: tension‡		62	52	48	42	38
Laps; tension§		88	74	68	60	54

The values in the table apply to (a) good bond conditions and (b) bar size ≤ 32 . For poor bond conditions the table values should be divided by 0.7. For bar size > 32 the values should be divided by $(132\downarrow)/100$, where \downarrow is the bar diameter in mm.

* In the anchorage region, cover perpendicular to the plane of curvature should be at least $3\emptyset$.

† The percentage of bars lapped at the section $< 30\%$, clear spacing between bars $\geq 10\emptyset$ and side cover to the outer bar $\geq 5\emptyset$.

‡ The percentage of bars lapped at the section $> 30\%$, or clear spacing between bars $< 10\emptyset$ or side cover to the outer bar $< 5\emptyset$.

§ The percentage of bars lapped at the section $> 30\%$ and clear spacing between bars $< 10\emptyset$ or side cover to the outer bar $< 5\emptyset$.

Table II.2. Anchorage and lap lengths as multiples of bar size: plain bars, $f_{yk} = 460 \text{ N/mm}^2$

Concrete strength:	f_{ck}	20	25	30	35	40
N/mm ²	f_{cu}	25	30	37	45	50
Anchorage: straight bars, compression and tension (not applicable to bar diameter $> 8 \text{ mm}$)		50	46	41	39	37
Anchorage: curved bars,* tension		35	32	29	27	26
Laps: compression, tension†		50	46	41	39	37
Laps: tension‡		70	64	60	56	52
Laps: tension§		100	92	84	78	74

The values in the table apply to good bond conditions.

For poor bond conditions the table values should be divided by 0.7.

* In the anchorage region, cover perpendicular to the plane of curvature should be at least $3\emptyset$.

† The bars lapped at the section $< 30\%$, clear spacing between bars $\geq 10\emptyset$ and side cover to the outer bar $\geq 5\emptyset$ (from NAD).

‡ The bars lapped at the section $> 30\%$, or clear spacing between bars $< 10\emptyset$, or side cover to the outer bar $< 5\emptyset$.

§ The bars lapped at the section $> 30\%$ and clear spacing between bars $< 10\emptyset$, or side cover to the outer bar $< 5\emptyset$.

Transverse reinforcement at lapped joints

As at anchorages, tensile stresses are induced in concrete at lapped joints and these stresses tend to split the concrete cover. Lateral reinforcement should be provided to resist these stresses. Failure of splices without transverse reinforcement is sudden and complete, whereas those with transverse reinforcement tend to exhibit a less brittle failure and also possess residual strength beyond the maximum load.

No special reinforcement is required when the diameter of the lapped bars is less than 16 mm, or the lapped bars in any section are less than 20%. Under these conditions the minimum reinforcement is considered adequate to cope with the tensile stresses generated at laps.

If the diameter of the lapped bars is greater than 16 mm, transverse reinforcement should be placed between the longitudinal reinforcement and the concrete surface. Where the clear distance between adjacent lapped bars $\leq 10\emptyset$, the transverse reinforcement should be in the form of links *in* beams.

Bars with $\emptyset.32$ mm**General**

The minimum depth of the element should not be less $15\emptyset$.

For crack control, surface reinforcement may be used or crack width should be calculated and justified.

Concrete cover should be greater than \emptyset . The clear distance (horizontal and vertical) between bars should not be less than \emptyset or the maximum aggregate size + 5 mm.

Bond

The values of ultimate bond stress should be multiplied by $((132 - \emptyset)/100) \emptyset$ (in mm).

Anchorage

- (a) Bars should be anchored as straight bars or by means of mechanical devices. They should not be anchored in tension zones.
- (b) Lapped joints should not be used and mechanical devices (e.g. couplers) should be considered.
- (c) In the absence of transverse compression, additional transverse reinforcement should be provided:

$$A_{st} = n_1 \cdot 0.25A_s$$

$$A_{sv} = n_2 \cdot 0.25A_s$$

where A is the cross-sectional area of the anchored bar, n_1 is the number of layers with anchored bars in the same section, and n_2 is the number of bars anchored in each layer.

- (d) The additional transverse bars should be distributed evenly in the anchorage zone with their spacing not exceeding $5\emptyset$.

Welded mesh**Minimum diameters of mandrels**

The diameter depends on whether the welded cross wires are inside or outside the bends and on their location with respect to the tangent point of the bend.

Laps for welded mesh fabrics made of high-bond wires**General**

Mesh reinforcement may be lapped by (a) intermeshing (the lapped wires occurring in one plane) or (b) layering (the lapped wires occurring in two planes separated by the cross-wires).

When intermeshing is used in one direction, the wires at right angles will automatically be layered.

Table II.3. Amount of main reinforcement that may be lapped

A_s/s	Interior mesh	Exterior mesh
$\leq 1200 \text{ mm}^2/\text{m}$	100%	100%
$> 1200 \text{ mm}^2/\text{m}$	60%	Laps not allowed

EC2 does not provide guidance for lapping by intermeshing, which is the most efficient method. There is no technical reason not to use the EC2 provisions for intermeshing. A reference is made to Sheet No. II.9 and Sheet No. II.10.

Location of laps (main reinforcement)

Laps should be in zones where the effects of actions under the rare combination of loads are not more than 80% of the design strength of the section.

The amount of main reinforcement that may be lapped in any one section depends on the specific section area of the mesh, denoted by A_s/s (i.e. area of reinforcement per unit width), and whether the mesh is an interior or exterior mesh in a multiple layer mesh.

Lap length

$$\text{Lap length } l_0 = \alpha_2 l_b (A_{s,\text{req}}/A_{s,\text{prov}}) \leq l_{0,\text{min}} \quad 15$$

where:

$$\alpha_2 = 0.4 + [(A_s/s)/800] \leq 1 \text{ and } \geq 2$$

l_b is the basic anchorage length

$$l_{0,\text{min}} = 0.3 \alpha_2 l_b \leq 200 \text{ mm}$$

$$\leq S_t, \text{ the spacing of transverse welded bars.}$$

The lap lengths required may be expressed as multiples of the diameter of the main reinforcement bars, as in Table II.4.

Table II.4. Lengths l_b for weld mesh made of high-bond wires ($f_{yk} = 460 \text{ N/mm}^2$) as multiples of main wire size

Concrete strength, f_{ck} : N/mm^2	20	25	30	35	40
Basic lap length*	50	43	38	34	31

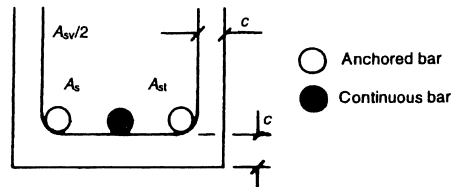
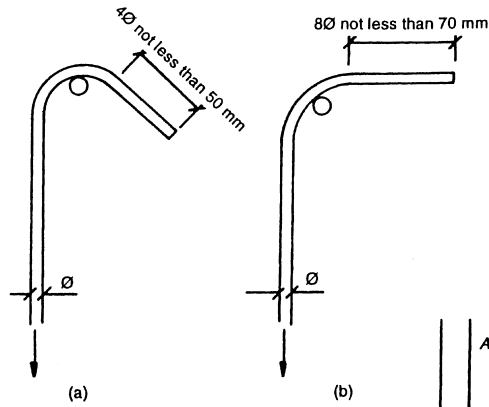
* The basic lap length applies to mesh with A_s/s up to $480 \text{ mm}^2/\text{m}$. For mesh with A_s/s between 480 and $1280 \text{ mm}^2/\text{m}$, the basic lap length should be multiplied by α_2 , obtained by linear interpolation between the following values: for $A_s/s = 480 \text{ mm}^2/\text{m}$, $\alpha_2 = 1.00$; for $A_s/s \geq 1280 \text{ mm}^2/\text{m}$, $\alpha_2 = 2.00$.

The values in Table II.4 apply to (a) good bond conditions and (b) bar size ≤ 32 .

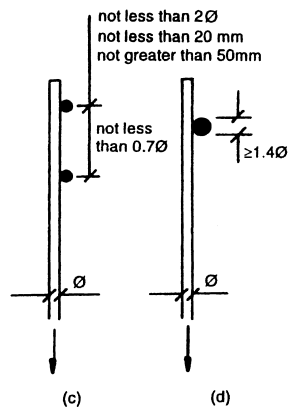
For poor bond conditions, the table values should be divided by 0.7.

For bar size > 32 the values should be divided by $[(132 - \emptyset)/100]$, where \emptyset is the bar diameter in mm.

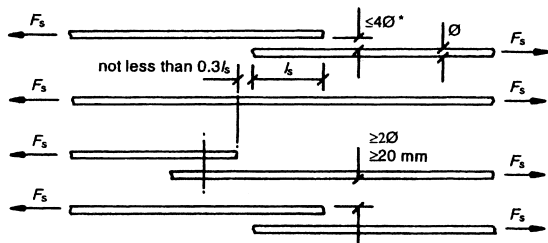
BENDS HOOKS AND LAPPING OF BARS



Additional transverse reinforcement

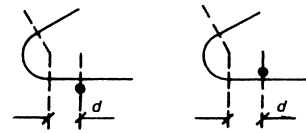


Anchorage of links



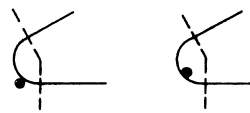
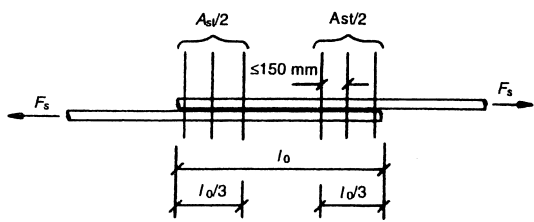
* If $> 4\phi$, the lap length shall be increased by the amount by which the clear gap exceeds 4ϕ

Spaces between lapped bars



$d < 4\phi : 20\phi$
 $d \geq 4\phi - \text{Table 10.2}$

(a)

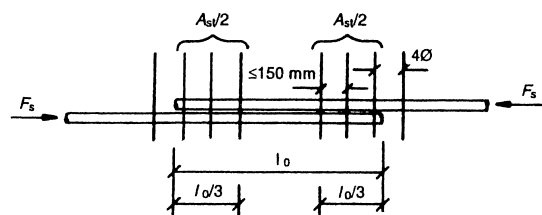


20Ø

(b)

Minimum diameter of the mandrel:
 inside bends

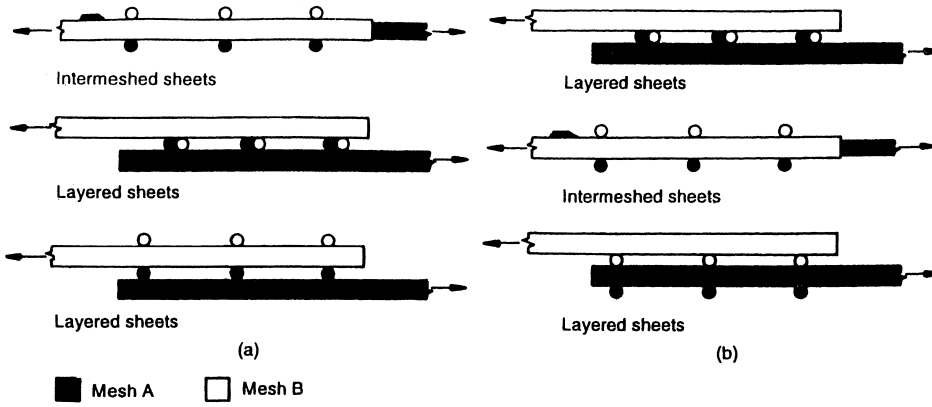
(a) welds outside bends; (b) welds inside bends



Placing of transverse reinforcement

MESHES

SHEET NO. II.10



Layering of wires: (a) main reinforcement; (b) transverse reinforcement

Laps for transverse distribution reinforcement

All transverse bars may be lapped at the same location.

The lap length should be at least equal to S_1 (the spacing of the longitudinal wires) or the values, given in Table II.5.

Table II.5. Minimum lap length requirements

Diameter of transverse bars	Minimum lap length
$\emptyset \leq 6$ mm	150 mm
$6 \text{ mm} < \emptyset < 8.5$ mm	250 mm

Welded mesh using smooth wires

EC2 does not provide direct guidance on this, but refers to national codes. In the UK, BS 8110 provides guidance for such a mesh. Table II.6 may be used to determine the lap length.

Table II.6. Anchorage and lap lengths as multiples of bar size: smooth wire fabric, $f_{yk} = 460$ kN/mm²

Concrete strength:	f_{ck}	20	25	30	35	40
N/mm ²	f_{cu}	25	30	37	45	50
Straight anchorage: compression		26	24	22	20	19
Straight anchorage: tension		33	30	27	25	23
Laps: compression, tension*		33	30	27	25	23
Laps: tension†		46	42	38	34	33
Laps: tension‡		66	60	54	49	47

The values in the table apply to (a) good bond.

For poor bond conditions the table values should be divided by 0.7.

The values apply provided: the fabric is welded in a shear-resistant manner complying with BS 4483, and the number of welded intersections within the anchorage is at least equal to $4 \times (A_{sreq}/A_{sprov})$. If the latter condition is not satisfied, values appropriate to the individual bars/wires should be used.

* The bars lapped at the section $< 30\%$, clear spacing between bars $\geq 10\emptyset$ and side cover to the outer bar $\geq 5\emptyset$ (from NAD).

† The bars lapped at the section $> 30\%$, or clear spacing between bars $> 10\emptyset$, or side cover to the outer bar $> 5\emptyset$.

‡ The bars lapped at the section $> 30\%$, or clear spacing between bars $> 10\emptyset$, or side cover to the outer bar $> 5\emptyset$.

II.1.3. Detailing based on American practices

II.1.3.1. Reinforced concrete beam detailing

A reference is made to Sections II.1.1 and II.1.2 for the detailing philosophy of reinforced concrete beams. Mostly these details are based on ACI codes and ASCE codes. Some variations do exist which are directly related to individual state regulations. In some cases detailing needs to cater for the interstate construction activities. The designs are based on the working stress design and strength reduction approach. All beams are designed and detailed to ensure the moments shears and deflections produced by factored load do not exceed the available flexural design strength of the cross section at any point along the length of the beam. If the flexural design strength (ϕ) M_n just equals to the required flexural strength M_u , the criterion for the design is established. Where M_n is the nominal moment capacity of the cross section and ϕ is the strength reduction of (generally = 0.9) the section using ACI code. Sometimes

distribution of shear stresses created by torsion need to be checked. The interaction equations for shear and torsional strengths of concrete must be involved in order to assess the strength capacity of the beam. Various tables and charts are available to aid the design and detailing.

II.1.3.2. Beams and girders

Beam widths

To permit satisfactory placing of concrete and to protect bars from corrosion, the engineer must provide for adequate clear distance between parallel bars and between bars and forms.

The engineer must specify the required concrete protection for the reinforcement.

The engineer must also specify the distance between bars for bond development and concrete placing. For buildings, the clear space is at least one bar diameter, $1\frac{1}{3}$ times the maximum size of coarse aggregate to be used, but not less than 1 in. For cast-in-place bridges, required clear space is not less than 1.5 bar diameters, 1.5 times maximum size aggregate, nor 1.5 in.

A wide range of beam widths and the maximum number of bars permitted in a single layer for $\frac{3}{4}$ in. and 1 in. maximum aggregate size, respectively, as provided by ACI 318-83 (revised 1986). Similarly this gives the same information for beams designed under the provisions of the AASHTO 1983 bridge specification. These tables are provided for the use of the engineer; the detailer is not in a position to determine whether bars should be permitted to be placed in more than a single layer.

Beams and girders

Schedules for beams and girders must contain: the beam mark; size of member; number and size of straight and bent bars; special notes on bending; number, size, grade, and spacing of stirrups or stirrup-ties; location of top bars; and any special information, such as the requirement of two layers of steel. Show sections for beam-column joints, where necessary.

In continuous beams the number and spacing of top bars to be placed in T-beam flanges (slabs) for crack control must be shown, if so required by the design.

Beams and joists

For beams, joists, and girders, reinforcement is usually shown in schedules. Bending details may be separate or incorporated in the schedule. Show number, mark, and size of members; number, size, and length of straight bars; number, size, mark, and length of bent bars and stirrups; spacing of stirrups; offsets of bars; lap splices; bar supports; and any other special information necessary for the proper fabrication and placement of the reinforcement. For typical layout a reference is made to Sheet No. II.11. Among the special items that must be noted are:

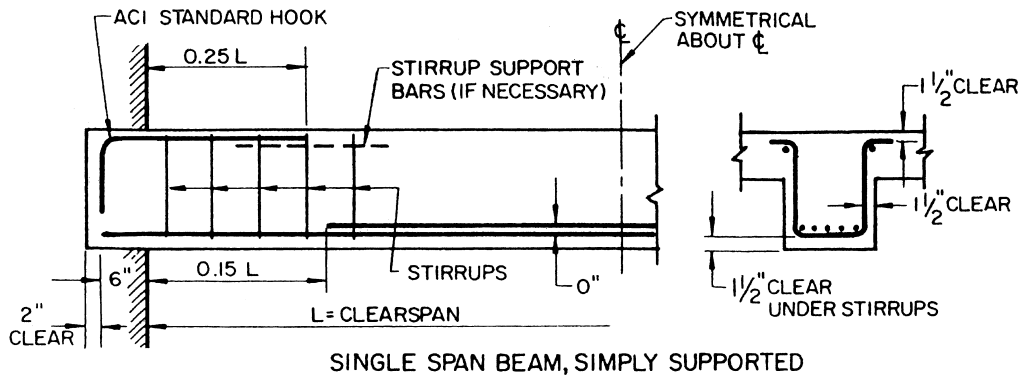
1. overall length of bar
2. height of hook where such dimensions are controlling
3. lap splice lengths
4. offset dimensions, if any, and
5. location of bar with respect to supporting members where the bar is not dimensioned symmetrically on each side of the support.

For one-way joists, a reference is made to Sheet No. II.12.

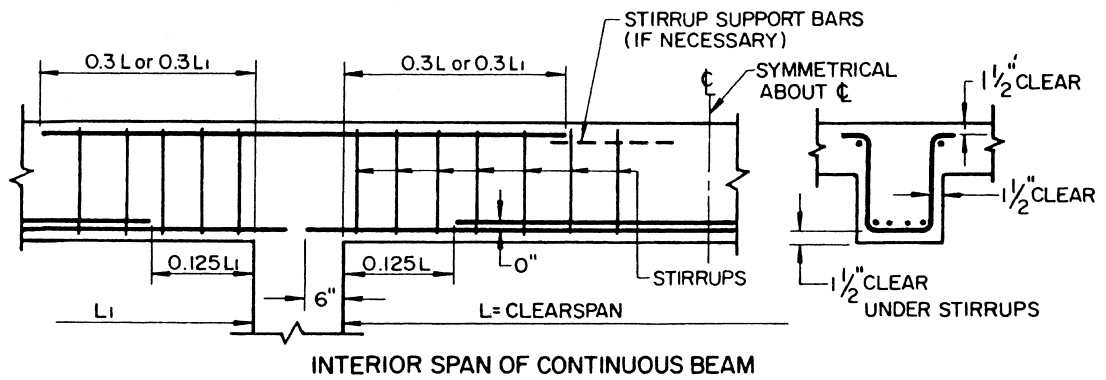
Reinforcement

Drawings must show the grade, size, spacing, splices, and location of the coated and uncoated bars in the structure. The bar schedule (combined

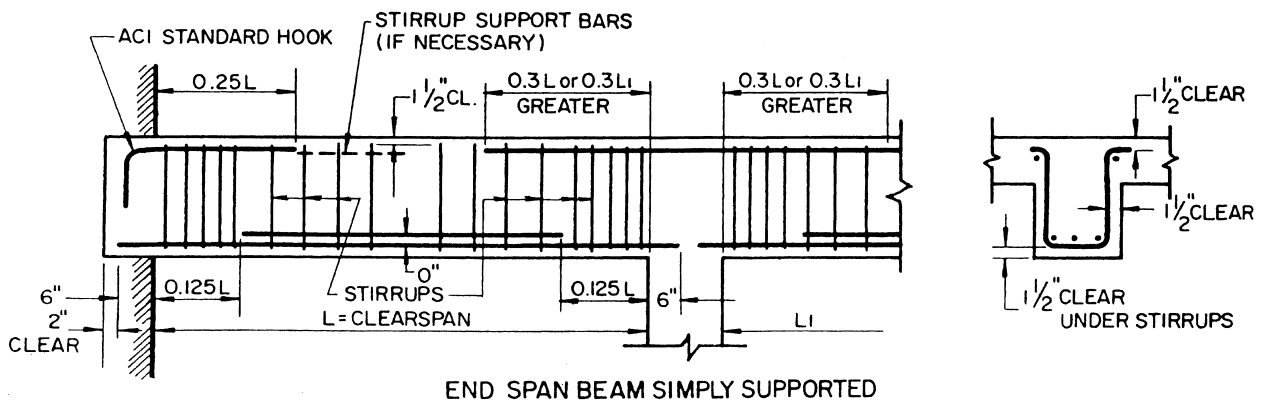
**TYPICAL DETAILS OF RC BEAMS
(ACI, ASCE, AND OTHER PRACTICES IN STATES)**



SINGLE SPAN BEAM, SIMPLY SUPPORTED



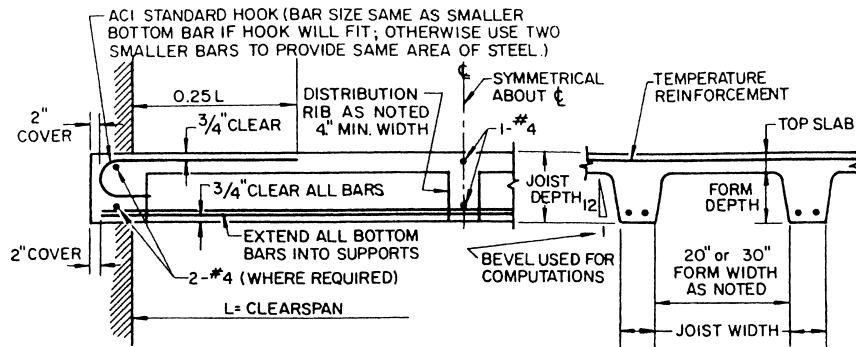
INTERIOR SPAN OF CONTINUOUS BEAM



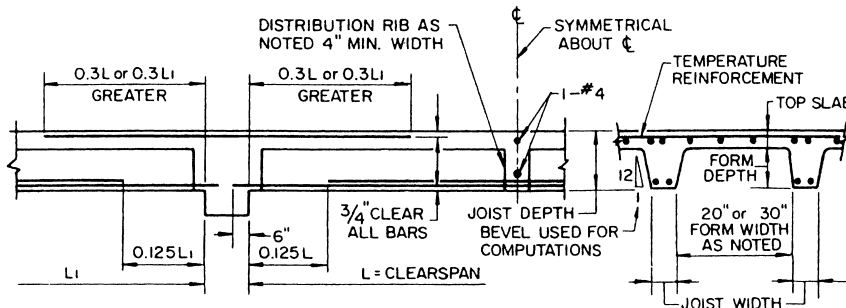
END SPAN BEAM SIMPLY SUPPORTED

**TYPICAL DETAILS OF ONE-WAY JOIST CONSTRUCTION
(ASCE, ACI AND OTHER PRACTICES IN STATES)**

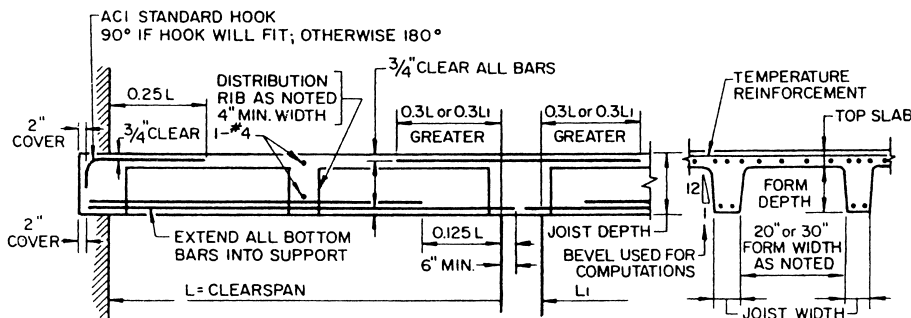
SHEET NO. II.12



SINGLE SPAN JOIST CONSTRUCTION



INTERIOR SPAN JOIST CONSTRUCTION



END SPAN JOIST CONSTRUCTION

KEY: 1 in. = 25.4 mm
1 in. = 0.304 m

drawing) must show the number of pieces, size, length, mark of bars, and bending details of all bent bars.

Reinforcement for larger structures is usually detailed, fabricated, and delivered by units for the convenience of both contractor and fabricator; for example, footings, abutments, piers and girders. The bar list is then similarly subdivided. If the structure is sufficiently large, a separate drawing and bar schedule is made for each unit.

Reinforcing bars for foundations, piers, abutments, wing walls, and slabs are usually shown on the plan, section or elevation. Reinforcement may be shown in the simplest and clearest manner; however, the bar schedule must be a complete summary.

To be certain that all of the reinforcement is properly placed or positioned in a unit, a cross section is frequently required in addition to the plan and elevation of the unit whereon the bars are shown.

Reinforcement supports

Plain metal supports are widely used as a means of securely holding reinforcement in proper position while concrete is being placed. Plastic coated or stainless steel legs can be specified to avoid possible rusting at points of exposure. Precast concrete blocks are used in some states, particularly in western US. Other types of proprietary supports are available and may be suitable. Support bars, when required, should be clearly shown and identified.

Where exposed concrete surface is to receive special finishing treatments such as sandblasting, bush-hammering, or any other removal of surface mortar, special consideration must be given to selecting bottom bar supports, side-form spacers, etc., which will not rust or otherwise impair the finished surface appearance.

The class of bar support, blocks, or other proprietary supports, and locations where each is to be employed, should be specified or shown in the contract documents. The detailer must identify the specified types and show locations where each is to be used.

Bending

To avoid creating excessive stresses during bending, bars must not be bent too sharply. Controls are established by specifying the minimum inside radius or inside diameter of the bend which can be made for each size of bar. The radius or diameter of the completed bend is usually expressed as a multiple of the nominal diameter of the bar d_b . The ratio of diameter of bend to diameter of bar is not a constant because it has been found by experience that this ratio must be larger as the bar size increases.

The minimum diameter of bend specified by ACI 318-83 (revised 2000) for reinforcing bars, measured on the inside of the bar, is as follows:

#3 through #8	$6d_b$
#9, #10, #11	$8d_b$
#14, #18	$10d_b$

and, for stirrups and ties only,

#3, #4, #5	$4d_b$
------------	--------

The inside diameter of bends of welded wire fabric (smooth or deformed) for stirrups and ties, as specified by ACI 318-83 (revised 2000), shall not be less than $4d_b$ for deformed wire larger than D6 and $2d_b$ for all other wires. Bends with inside diameter of less than $8d_b$ shall not be less than $4d_b$ from the nearest welded intersection.

Hooks

ACI 318-83 (revised 2000) specifies minimum bend diameters for reinforcing bars (Section 3-7.2). It also defines 'standard hook' (Section 7.1) to mean the following:

- (a) a 180° bend plus an extension of at least $4d_b$ but not less than $2\frac{1}{2}$ in. at the free end of the bar, or
- (b) a 90° bend plus an extension of at least $12d_b$ at the free end of the bar, or
- (c) for stirrup and tie hooks only, either a 90° bend plus $6d_b$ extension for #3, #4, #5, and $12d_b$ extension for #6, #7 and #8 or a 135° bend plus an extension of at least $6d_b$ but not less than $2\frac{1}{2}$ in. at the free end of the bar. For closed ties defined as hoops in Appendix A of ACI 318-83, a 135° bend plus an extension of at least $10d_b$.

The minimum bend diameter of hooks must meet the foregoing provisions. The standard hooks (Table 1 of the code ACI 318-83 (revised 2000)) were developed such that the minimum requirements were met but at the same time recognizing the need to allow for 'springback' in fabrication, and maintaining a policy of production fabrication pin size no smaller than the ASTM A 615-85 bend test.

Stirrup anchorage

There are several permissible methods to stirrup anchorage. The most common is to use one of the hooks shown in Table 1 of the code ACI 318-83 (revised 2000). Types S1 to S6 in Fi. illustrate not only the uses of the two types of hooks but also the directions in which the hooks may be turned. In detailing the anchorage, care must be taken that the ends of stirrup hooks turned outward in shallow slabs have adequate cover. If not, the hook should be turned inward and this change brought to the engineer's attention.

Where the free ends of stirrups cannot be wired to longitudinal bars, or where there are longitudinal bars, stirrup support bars should be specified by the engineer.

Standard bar bends

To list the various types of bent bars in the schedule, it is necessary to have diagrams of the bars with the lengths of the portions of the bars designated by letters. A chart of such standard bar bends is shown in Figure 6 of the code.

Dimensions given for Hooks A and G are the additional length of bar allowed for the hook as shown in Table 1. For straight portions of the bar, the distance is measured to the theoretical intersection of the outside edge line extended to the outside edge line of the adjacent straight portion, or to the point of tangency to a curve, from which point the length of the latter is tabulated.

Radius bending

When reinforcing bars are used around curved surfaces, such as domes, tanks, etc., and when no special requirement is established in the contract, bars prefabricated to a radius equal or less than those in Table II.7 are prefabricated by the reinforcing bar fabricator. In the smaller sizes, the bars are sprung to fit varying job conditions such as location of splices, vertical bars, jack rods, window openings, and other blocked out areas in the forms. The larger size bars which are more difficult to spring into desired position are ordinarily employed in massive structures where placing tolerances are correspondingly larger. Radially prefabricated bars of any size tend to relax the radius originally prefabricated as a result of time and normal handling. The last few

feet involved in the lap splice area often appear as a tangent rather than a pure arc due to limitations of standard bending equipment. For these reasons, final adjustments are left as a field placing problem to suit conditions and tolerance requirements of a particular job of the ACI code. See Figures 4 and 5 for radial tolerances and Section 4.2(c)3 of the ACI code. Bars requiring a larger radius or length than shown in Table II.7 are sprung in the field without pre-fabrication.

Table II.7. When radial prefabrication is required

Bar size	Bars are to be prefabricated when either radius or bar length is less than the ACI requirements given in the code	
	Radius: ft	Bar length: ft
#3	5	10
#4	10	10
#5	15	10
#6	40	10
#7	40	10
#8	60	10
#9	90	30
#10	110	30
#11	110	60
#14	180	60
#18	300	60

The presence of the tangent end does not create any problem on bar sizes #3 through #11 since they are generally lap spliced and tangent ends are acceptable. However, #14 and #18 bars cannot be lap spliced and are usually spliced using a mechanical device or by butt-welding. It is a problem to place a radially bent bar when using a mechanical splice sleeve due to the tangent ends on bars bent to small radii. To avoid this problem, all #14 and #18 bars bent to a radius of 20 ft or less are to be furnished with an additional 1 ft 6 in. added to each end. This 1 ft 6 in. tangent end is to be removed in the field by flame cutting. Bars bent to radii greater than 20 ft will be furnished to the detailed length with no consideration given to the tangent end. The ends of these bars generally are saw cut.

Shop removal of tangent ends may be made by special arrangement with the reinforcing bar supplier.

Slants

To determine the length of straight bar necessary to form a truss bar, the length of the slant portion of the bar must be known. The standard angle is 45° for truss bars, with any other angles being special. Slants and increments are calculated to the closest V_z in. so that for truss bars with two slants, the total increment will be full inches. This makes the computation easier and is within the tolerances permitted. It is important to note that when the height of the truss is too small 45° bends become impossible. This condition requires bending at a lesser angle and lengthens the slant portion.

Splices

In beams or girders that require bars longer than can be carried in stock, splices must be specified. The engineer must show or specify by notes how the splicing is to be realized; viz, lapping, welding, or mechanical connections. For #14 and #18 bars, ACI 318-83 (revised 1986) does not permit lap splices except to smaller bars in compression.

The engineer must also show by details on engineering drawings the location of all splices. In beams or girders splices should preferably be made where the stress in the bar is minimum, i.e. at the point of inflection. Splices where the critical design stress is tensile should be avoided by the engineer wherever possible. Lapped bars may be either in contact or separated. The engineer should show or note on the drawings whether splices are to be staggered or made at the same location. Bars to be spliced by non-contact lapped splices in flexural members shall not be spaced transversely more than one-fifth the length of lap nor 6 in. (150 mm).

Lap splices

Since the strength of a lap splice varies with bar diameter, concrete strength, position of the bar, distance from other bars, and type of stress (compressive or tensile), it is necessary for the engineer to show location of all splices, and to indicate by 'C' or 'T' whether compression or tension controls. If tension controls, the engineer should indicate class of splice required and whether it is 'top' or 'other'. Preferably the engineer should dimension each splice. Where bars of two sizes are lap spliced, the detailer will use the appropriate tensile lap splice for the smaller bar, unless otherwise noted.

Tables are provided principally for the convenience of the engineer. The detailer may use these tables to dimension the spliced bars and submit for final approval to the engineer.

Schedules

Highway structure engineering drawings most often show details of the various elements directly on the plan or elevation. Schedules are sometimes used for piers, small structures, and even retaining walls. Highway engineering drawings usually include, when completely detailed, a type of schedule that is really a bill of material, sometimes segregated by elements of a structure. These drawings are used by the reinforcing bar fabricator to prepare shop bar lists.

Stirrup anchorage

The engineer must show or specify by notes the sizes, spacings, location, and types of all stirrups. These types include open stirrups and closed stirrups (or stirrup-ties).

There are various permissible methods of anchorage, but the most common is to use one of the standard stirrup-tie types as shown in Section I. Types S1 through S6, T1 and T2, using standard hooks.

II.2. Reinforced concrete slabs

II.2.1. Slab reinforcement and method of detailing based on British Standard Code BS 8110

Sheet No. II.13(a)(i) gives an isometric view of the main steel and distribution steel in a simply-supported concrete slab. A specification based on BS 8110 is given when the wall support is given to this slab. For different end restraints cases (b) to (d) on Sheet No. II.13 show the reinforcement arrangement and anchorages. The specifications indicated are based on the requirements of BS 8110. This sheet can be modified for other codes.

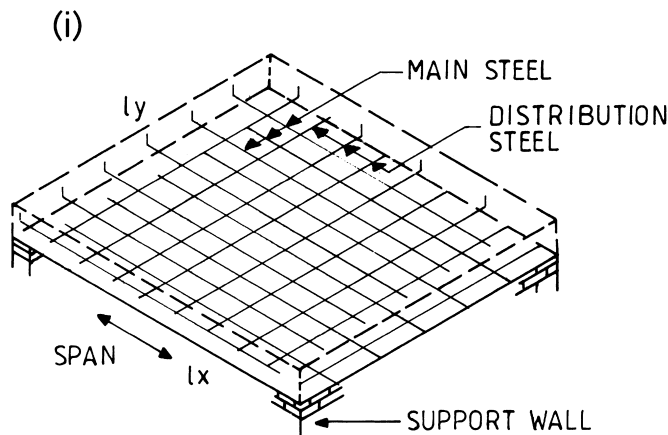
Sheet No. II.13(a) shows various types of restrained ends which can be adopted for slabs. Case (b) on Sheet No. II.13 indicates the procedure for bar curtailment in a slab recommended by BS 8110. A typical bar arrangement is shown in cases (c) and (e) on Sheet No. II.13.

Slabs are divided into suspended slabs and supported slabs. Suspended slabs may lie divided into two groups: (1) slabs supported on edges of beams and walls and (2) slabs supported directly on columns without beams and known flat slabs. Supported slabs may be *one-way slabs* (slabs supported on

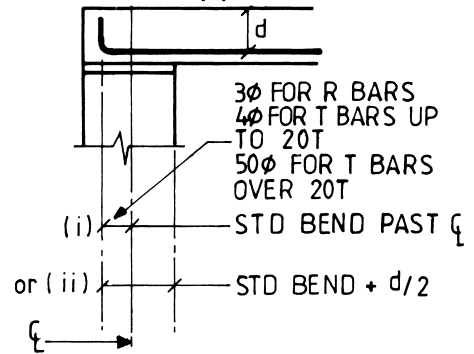
SLAB REINFORCEMENT

SHEET NO. II.13

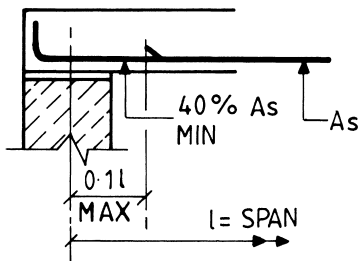
(a) Isometric view



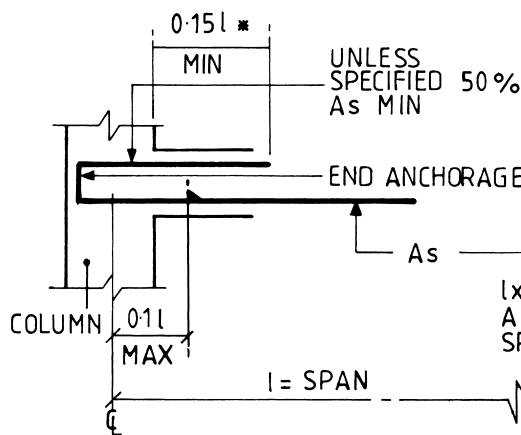
(ii) Wall support



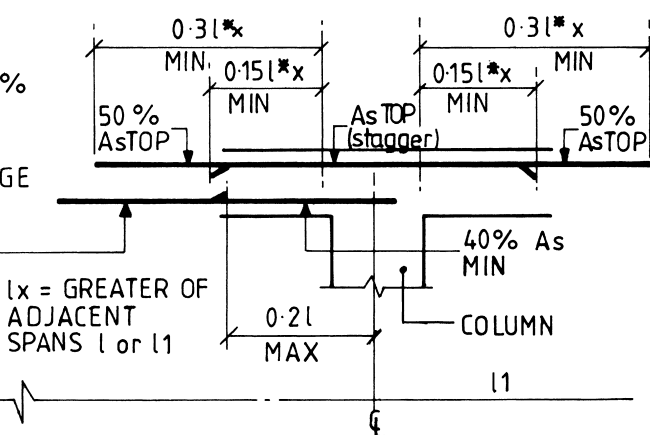
(b) Simple end



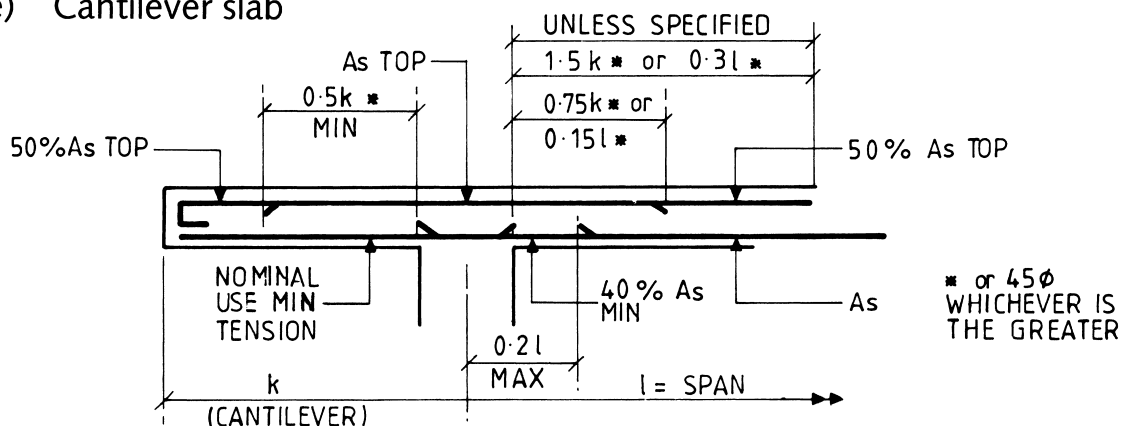
(c) Restrained end support



(d) Continuous internal support



(e) Cantilever slab



two sides and with main reinforcement in one direction only) and *two-way slabs* (slabs supported on four sides and reinforced in two directions). In one-way slabs, as shown on Sheet No. II.14(a), the main reinforcement *is* provided along the shorter span. In order to distribute a load, a distribution steel is necessary and it is placed on the longer side. One-way slabs generally consist of a series of shallow beams of unit width and depth equal to the slab thickness, placed side by side. Such simple slabs can be supported on brick walls and can be supported on reinforced concrete beams in which case lacer bars are used to connect slabs to beams, a typical detailing of this is shown on Sheet No. II.14(b) and anchorage details will be the same as for simple beams.

Where the reinforcement is very complicated, especially, the use of fabric, top and bottom reinforcement is separated for clarity and drawn onto two identical outlines, preferably on the same drawing. Abbreviations for top outer layer and second layer are identified as T_1 and T_2 . Similarly for the bottom outer and second layer respectively shall be designated as B_1 and B_2 . Bars detailed elsewhere are shown as a thick dashed line. Where bars of varying lengths exist, each bar in the zone is given the same bar mark but a different suffix, beginning with 'a'. The bar schedule will allocate different bar lengths to each suffix where needed. In a long panel, the bars of convenient length can be lapped from end to end of the panel. State minimum lap. Sometimes cranked and bent bars are drawn on plan as though laid flat. Sections and notes are provided to clarify where bars are required to be fixed flat and some upright.

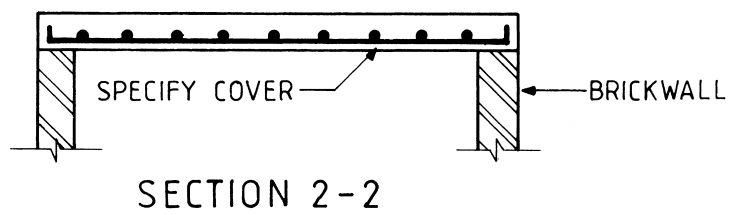
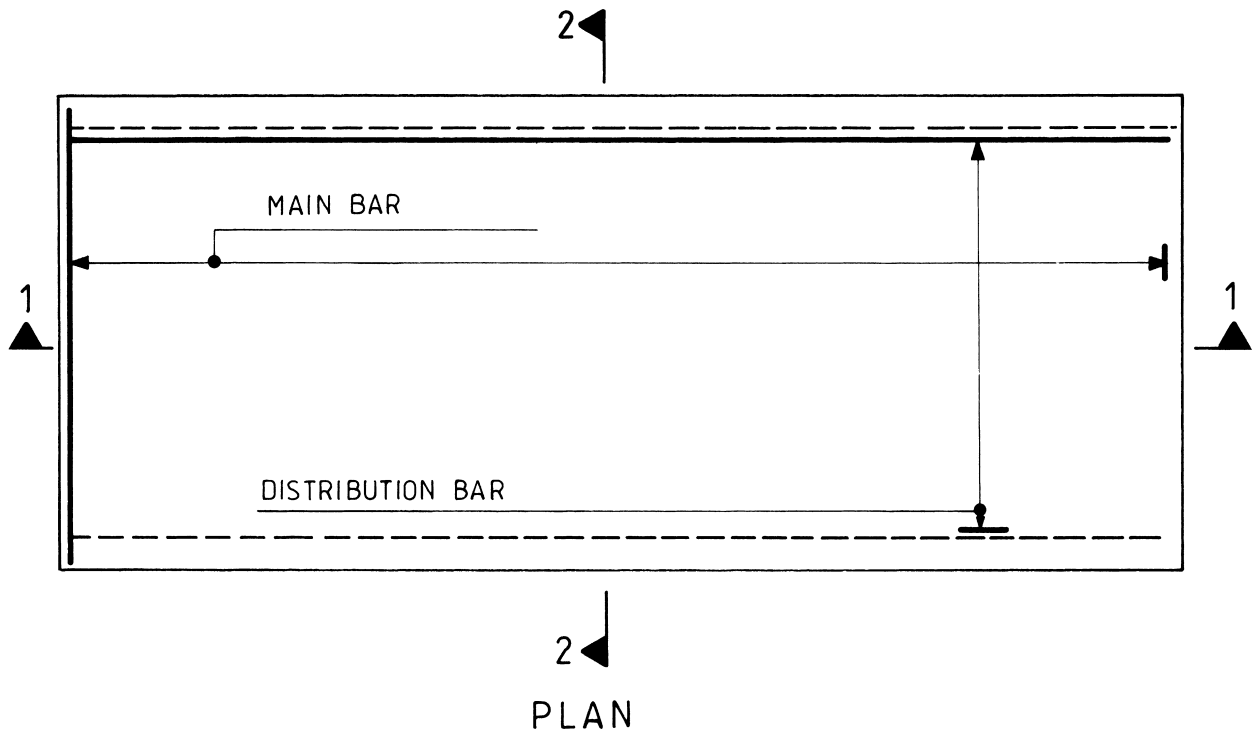
For the trimming of holes in slabs, the design data should specify any special reinforcement. Section I gives a preliminary arrangement for holes with significant structural applications.

Slabs with reinforcement in two directions or two-way slabs bent in two directions. The principal values of bending moments determine the size and number of reinforcement bars in each direction. Most codes give formulae and tables of coefficients for computing bending moments in both directions. A typical layout for a two-way simply supported slab is shown on Sheet No. II.15.

ONE-WAY SLABS ON WALLS AND BEAMS

SHEET NO. II.14

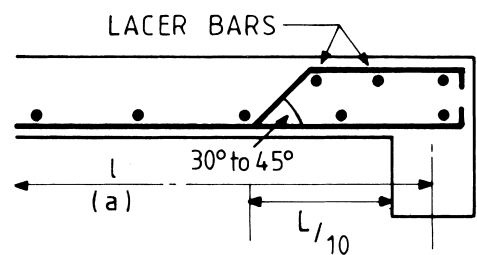
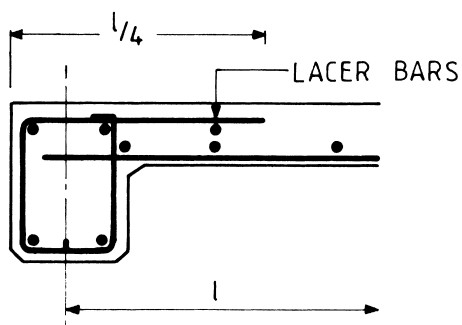
(a) One-way simple slab



(b) Simple slabs supported on concrete beams

(i)

(ii)

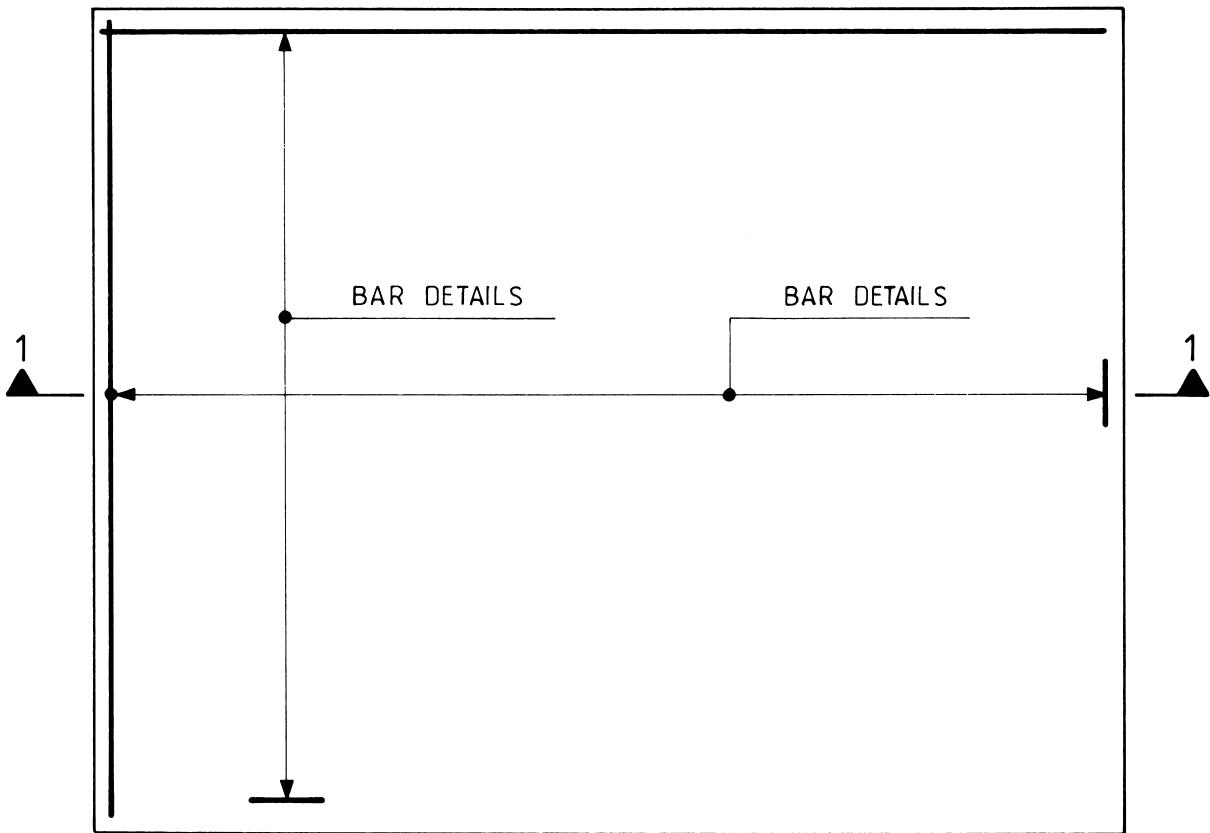


TWO WAY SLAB

SHEET NO. II.15

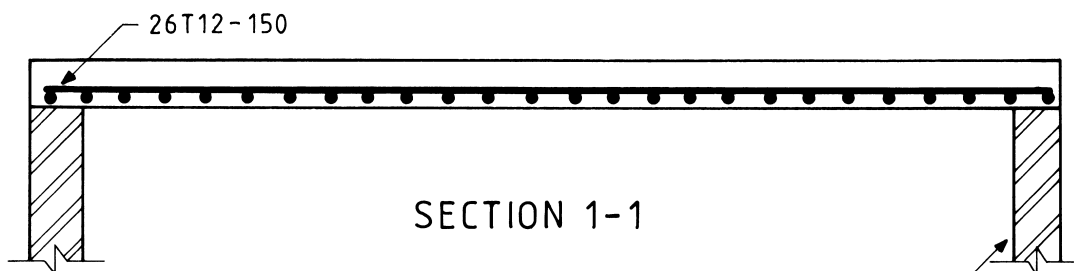
(a) Simply supported two-way slab

(i)

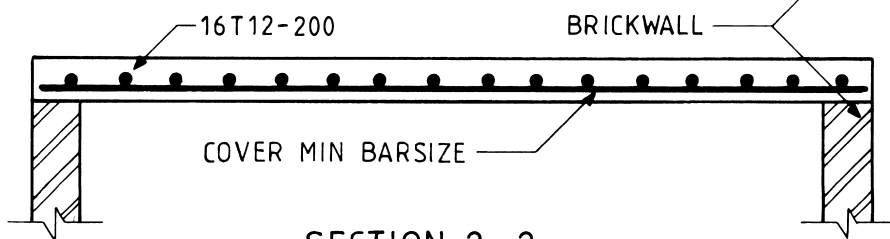


PLAN

(ii)



(iii)



SECTION 2-2

II.2.1.1. Flat slab

A flat slab is a reinforced concrete, slab supported directly on and built monolithically with the columns. As shown on Sheet No. II.16 the flat slab is divided into middle strips and column strips. The size of each strip is defined using specific rules. The slab may be of uniform thickness supported on simple columns. It is more economical to thicken the slab around the columns and to provide columns with flared heads. They are called *drops* and stiffen the slab over the columns and, in turn, reduce the shear stress and reinforcement. Flat slabs become economical where a number of panels of equal or nearly equal dimensions are required or where, for a limited headroom, large clear floor spaces are required. These flat slabs may be designed as continuous frames. However, they are normally designed using an empirical method governed by specified coefficients for bending moments and other requirements which include the following:

- (a) there should be not less than three rectangular bays in both longitudinal and transverse directions
- (b) the length of the bay $\frac{l_5}{4}$ etc. shall not be greater than $\frac{l_1}{3} \times \text{width}$ $\frac{l_3}{3}$
- (c) the length of the adjacent bays should not vary by more than 10%.

Cases (i) and (ii) on Sheet No. II.16 give a full picture of the panel division system and the reinforcement layout.

The panel with drops is 1.25 to 1.50 times thicker than the slab beyond the drop. The minimum slab thickness is 125 mm or $l/36$ for interior continuous panels without drops and end panels with drops or $l/32$ for end panels without drops or $l/40$ for interior continuous panels with drops. The length l is the average length and width of the panel. For some unknown reason, when the last edge of the slab sits on a column, the details of such an edge shall be carried out as shown in Sheet No. II.14.

For column shear heads, the following criteria shall be adopted.

1. A minimum of 2 shear perimeters are spaced at $0.75d$ from face of column.
2. Vertical shear legs are Shape Code 81 (2 legs) or Shape Code 85 (1 leg) spaced at a maximum of $1.5d$ around each perimeter.
3. Links can be threaded onto say T12 lacer bars to form convenient 'ladders' which are fixed alongside the B_2 then T_2 layers of slab reinforcement. This detail also ensures that adequate cover to links is achieved.

Column drops:

- (a) main slab reinforcement carries through
- (b) nominal mat: T12 at 300 each way. Design data to specify other.

Torsion reinforcement in restrained slabs

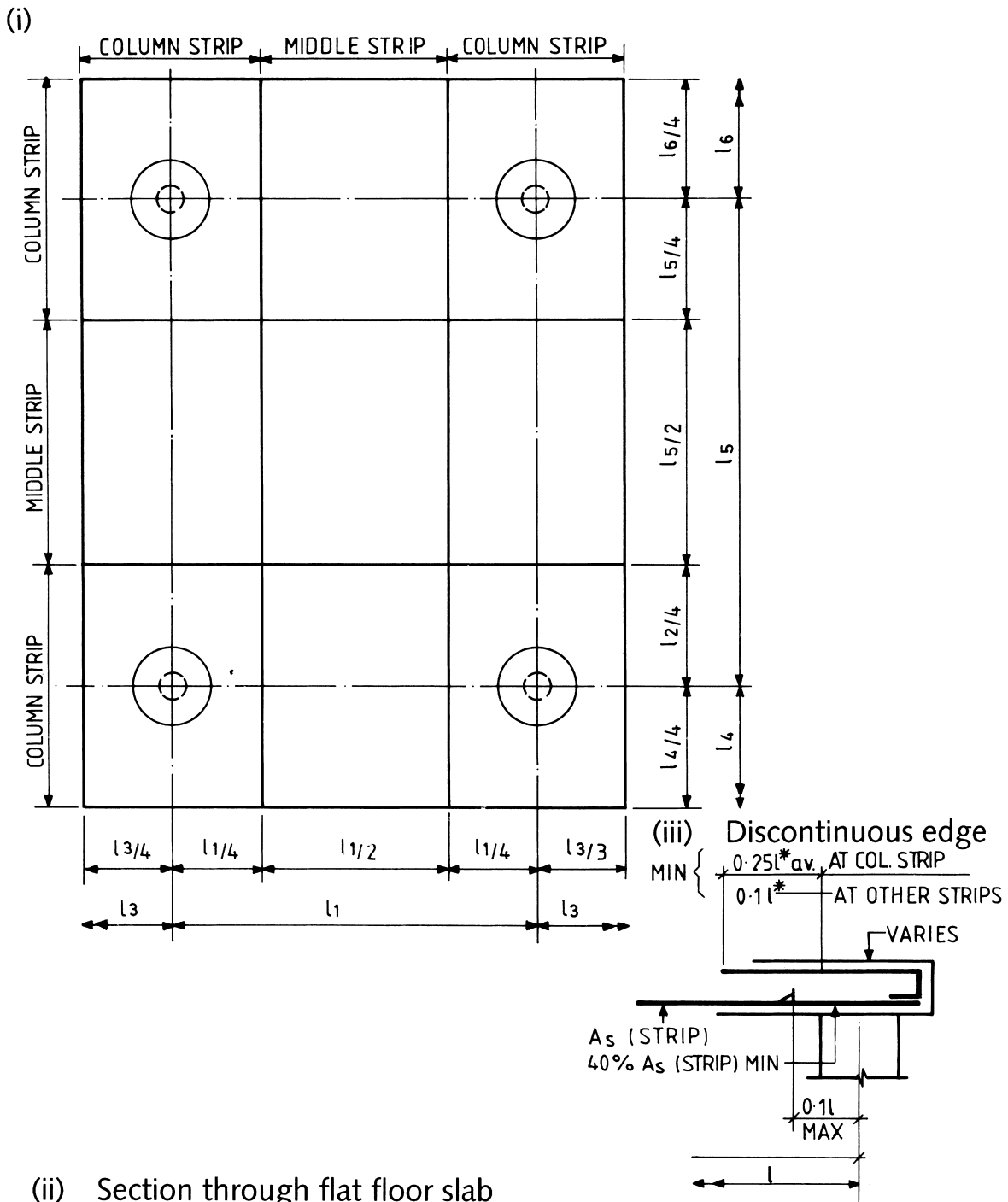
At corners (two discontinuous edges, both simple supports):

- (a) torsion reinforcement required top and bottom
- (b) gross area required = $0.75 \times \text{maximum } A_s \text{ span bottom each way in both top and bottom}$
- (c) extent of torsion bars = $0.2 \times \text{shorter span}$.

FLAT SLABS WITH COLUMN DROPS

SHEET NO. II.16

Slab division



Fabric reinforcement in slabs

- (a) General. Two-directional reinforcement can be factory welded and fabricated into sheets to help speed fixing and achieve economy in construction costs. BS 4466: 1981 defines three types of fabric:
- (i) designated (standard mesh) fabric section—stock sheet sizes are 4.8×2.4 m; these can be reduced by cutting to suit. Wire sizes range up to 12 mm with standard 100/200 mm meshes. Peripheral wires are welded at $\frac{1}{2}$ pitch from the edge of the sheet
 - (ii) scheduled (non-standard) fabric—wire sizes (maximum 12 mm) and sheet sizes can be varied. Wire pitches must remain constant but may be non-standard. Wire projections at edges may vary.
 - (iii) detailed (purpose-made) fabric—these sheets can be specified using standard reinforcing bars. These bars can be set at varying pitches and edge projections. Sheet sizes can vary with due consideration given to handling and transportation.
- (b) Suspended solid floor construction. For clarity on plan it is recommended that the top sheets of fabric be drawn separately from the bottom sheets, preferably on the same drawing. Fabric is identified as a chain double-dashed line. Fabric detailing on plan. Each individual sheet is given a mark number and related on the plan to the concrete outline. Indicate the direction of the main reinforcement and its layer notation. Wherever multiple sheets of identical marks occur they can be combined as shown.

Areas of reinforcement can be increased by double 'layering'. Structural mesh type 'B' is often specified for suspended slabs, possibly with the addition of loose bars. With reasonable production runs, consideration should be given to specifying 'purpose made' fabric. For each fabric mark indicate its reinforcement in a table alongside the plan. Minimum reinforcement requirements are shown in laps in fabric. The need for laps should be kept to a minimum and, where required, should be located away from regions of high tensile force. Allow sufficient clearance to accommodate any 'multilayering' of sheets at laps, reducing these occurrences where possible by 'staggering' sheets.

Voided-slab construction

A nominal designated fabric is normally placed within the topping of trough and waffle-type floors. The extent of the fabric is shown by a diagonal on the plan of the reinforcement drawing and the fabric type scheduled as gross area in m^2 by adding a suitable percentage to the net area of the floor to allow for laps. For ordering purposes, the contractor should translate this gross area into the quantity of sheets required to suit this method of working.

Ground-slab construction

The presence of fabric reinforcement can be indicated by a sketch and a prominent note on the drawing. This can be the general-arrangement drawing (in straightforward cases). The note should include the type of fabric, location within the depth of slab and minimum lap requirements. A typical section to clarify this construction should be included. The fabric type is scheduled as a gross area by adding a suitable percentage to the net area of slab to allow for laps. For ordering purposes, the contractor should translate this gross area into the quantity of sheets required to suit his or her method of working.

Beam and slab arrangement

In typical steel beam-slab composite constructions, connectors are used between the concrete slab and steel beams. Several types of connectors are used for this type of construction. Sometimes steel beams are encased in concrete and the bressumers, as they are known, are monolithic with concrete slabs. A brief summary is shown on Sheet No. II.17.

In reinforced concrete building construction, every floor generally has a beam/slab arrangement and consists of fixed or continuous one-way or two-way slabs supported by main and secondary beams. Sheet No. II.18(a) shows such an arrangement. The usual arrangement of a slab and beam floor consists of slabs supported on cross-beams or secondary beams parallel to the longer side and with main reinforcement parallel to the shorter side. The secondary beams in turn are supported on main beams or girders extending from column to column. Part of the reinforcement in the continuous slabs is bent up over the support, or straight bars with bond lengths are placed over the support to give negative bending moments. In large slabs, separate reinforcement over the support may be necessary. This is also demonstrated in Section I. A typical one-way continuous slab/beam arrangement is given in Sheet No. II.18(b) for the general arrangement given in II.18(a).

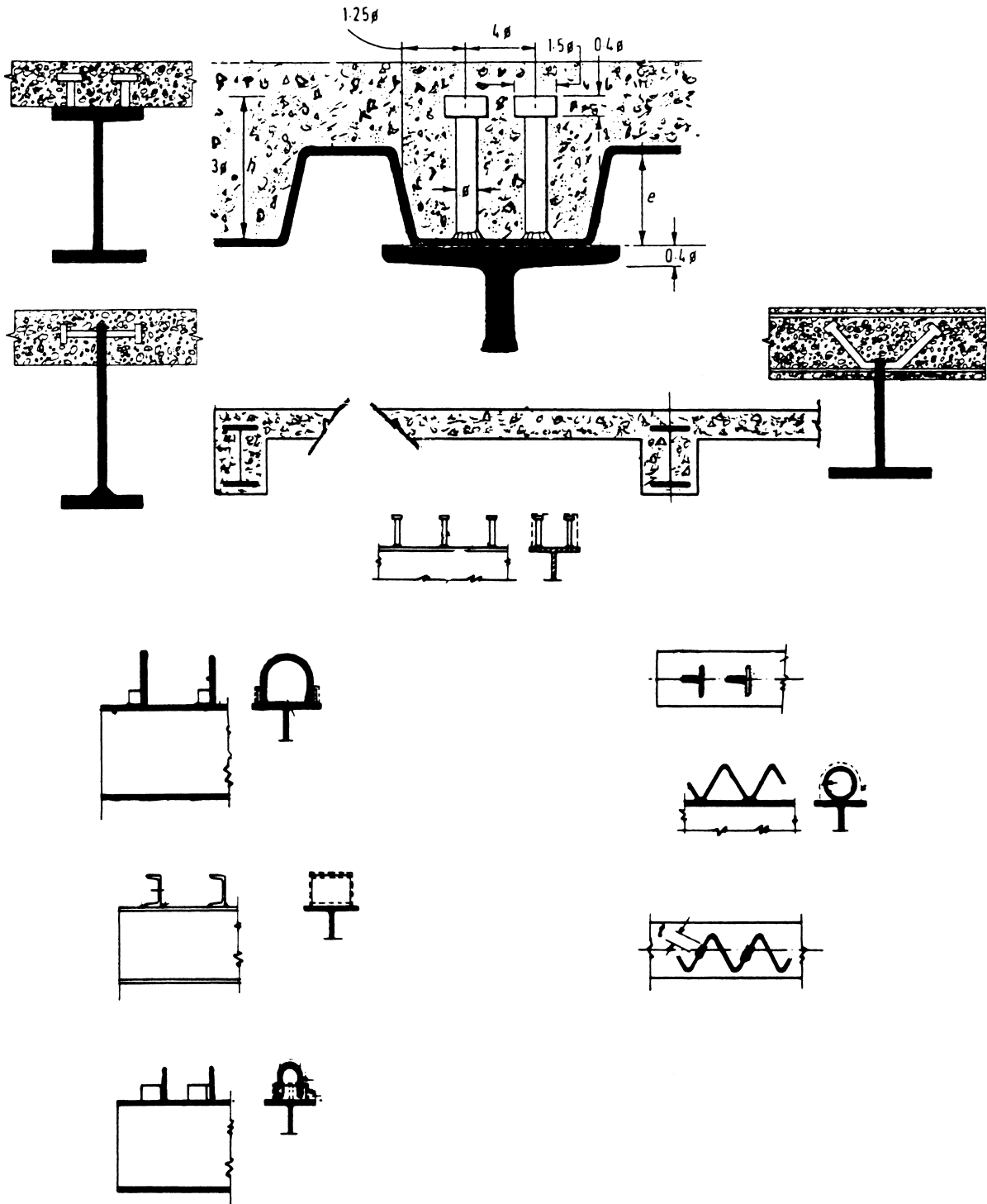
A *flat slab*, as discussed earlier, if supported directly on and built monolithically with columns, may differ from a two-way slab in that it is not supported on beams. The slab may be of uniform thickness supported on simple columns. Generally the slab around the columns is thickened in order to provide columns with *flared heads*, known as *drops*. The drop stiffens the slab over the column and reduces the shear stress and the reinforcement. Codes also recommend the distribution of bending moments between column strips and middle strips as shown in Section I. A great deal of research has been carried out on flat slabs without drops. Flat slabs without column drops and with drops are respectively detailed on Sheet Nos II.19 and II.20.

Continuous slabs with mesh fabric are given on Sheet No. II.21. Ribbed slab panel with reinforcement details are given in II.21(iii).

Sheet No. II.22 gives a reinforcement layout for a simple panel under missile impact. In Section IV, additional structural detailing is demonstrated for beam/slab column arrangements.

COMPOSITE SECTIONS AND CONNECTOR TYPES

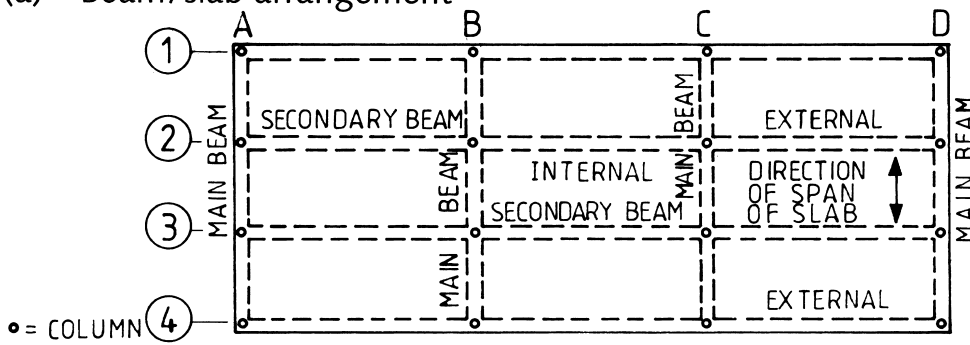
SHEET NO. II.17



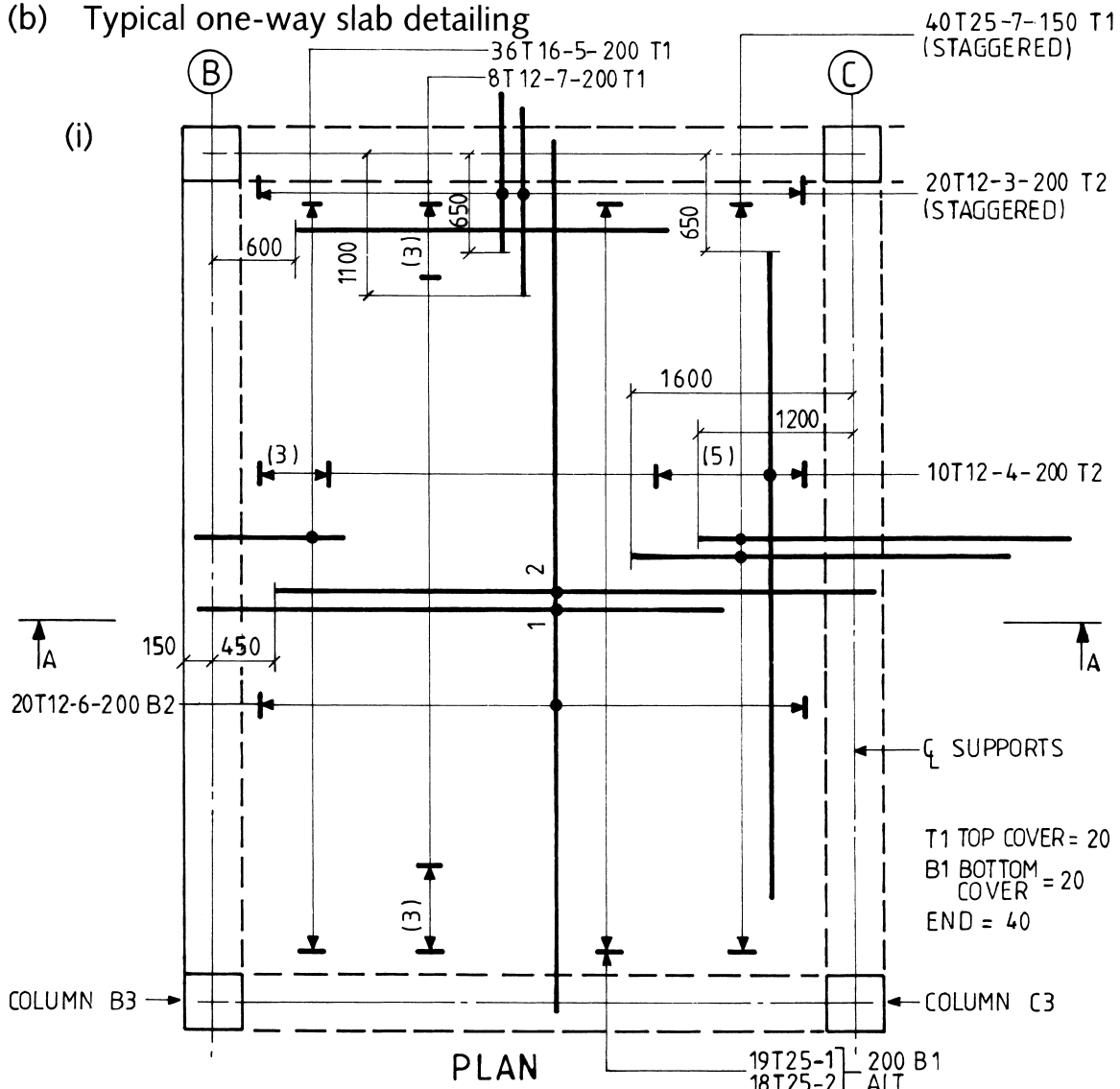
BEAM AND SLAB ARRANGEMENT

SHEET NO. II.18

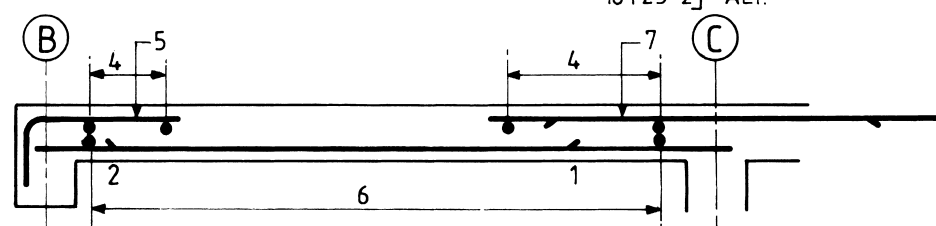
(a) Beam/slab arrangement



(b) Typical one-way slab detailing

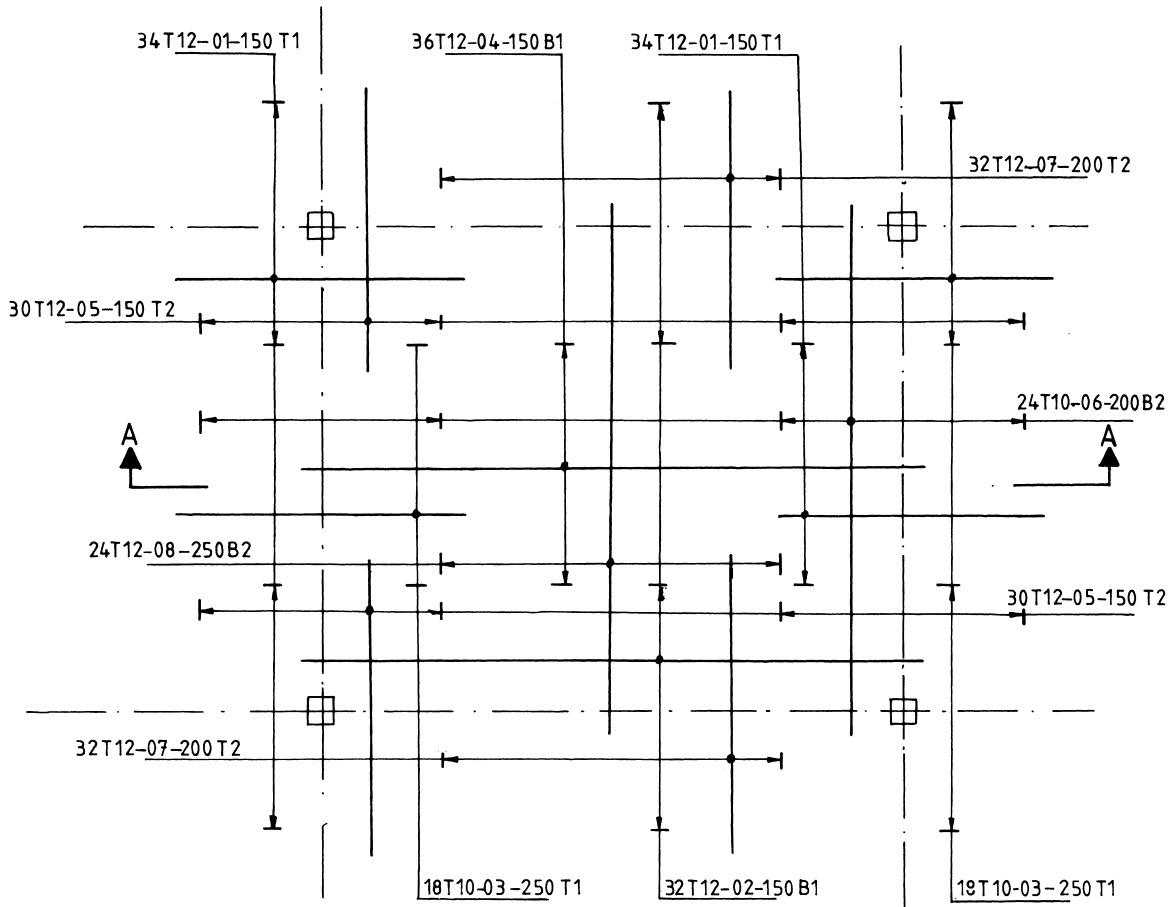


(ii)

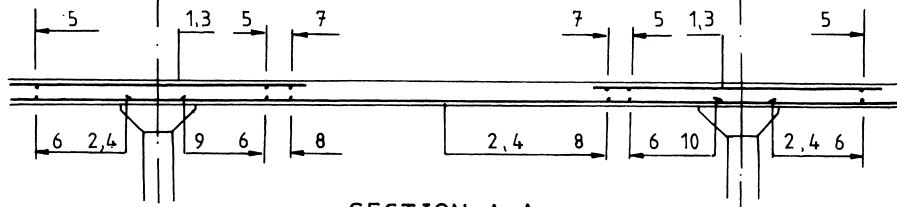


FLOOR SLAB WITH DROPS

SHEET NO. II.20



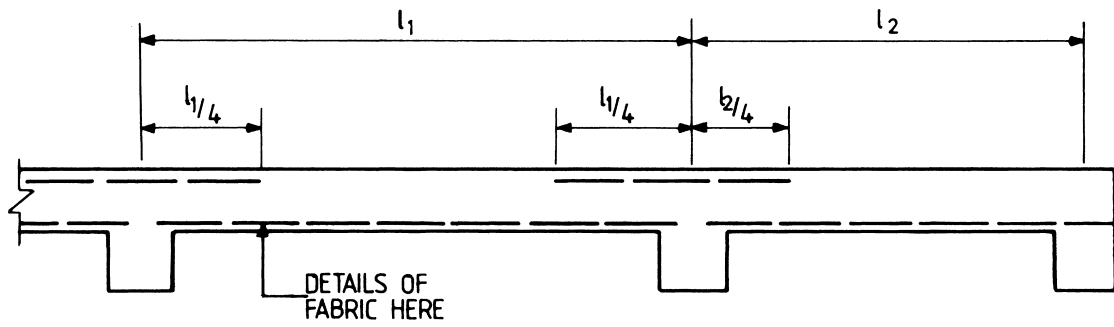
FLOOR SLAB PLAN
SCALE: 1:50



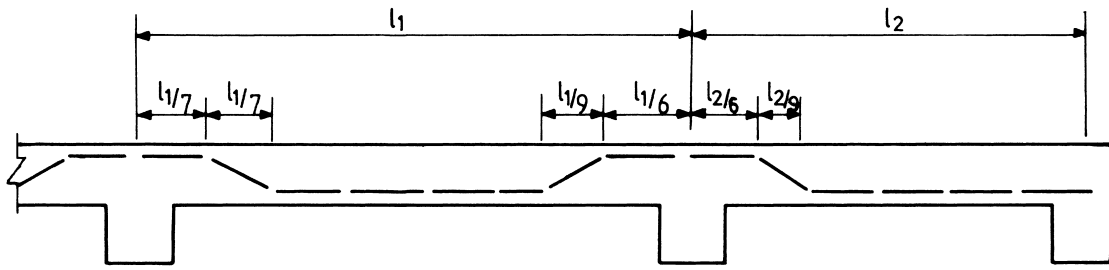
SECTION A-A
SCALE: 1:50

CONTINUOUS SLAB REINFORCED WITH MESH FABRIC AND RIBBED SLAB PANEL

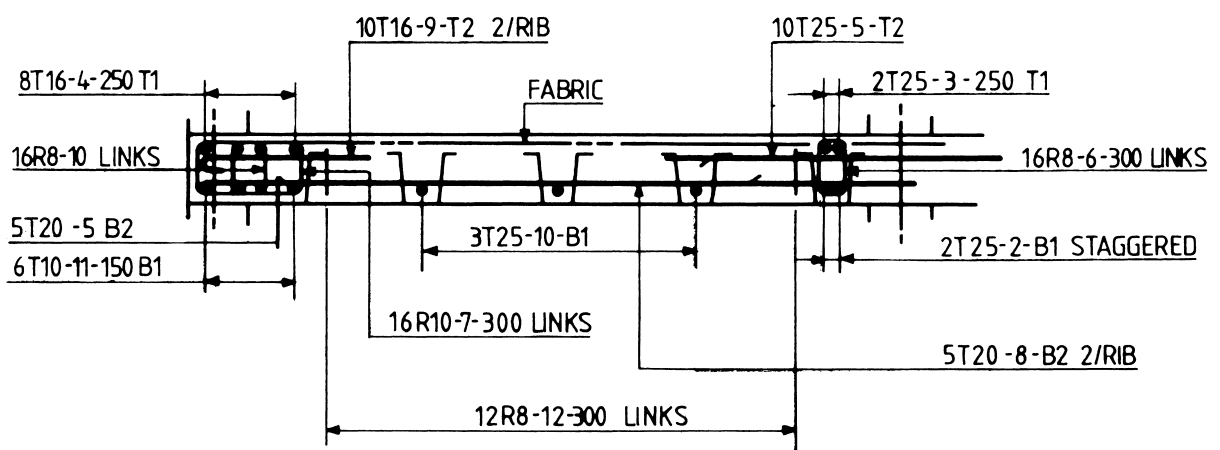
SHEET NO. II.21



(i) Flat slab



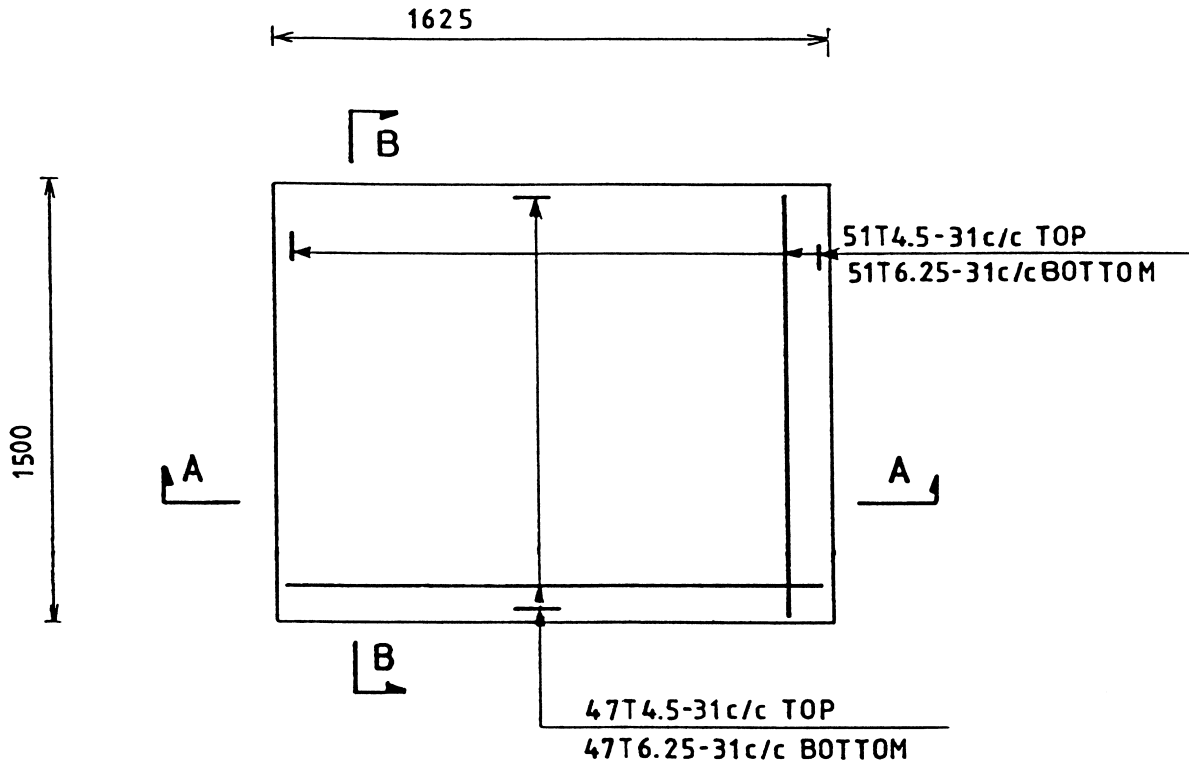
(ii) Waved fabric



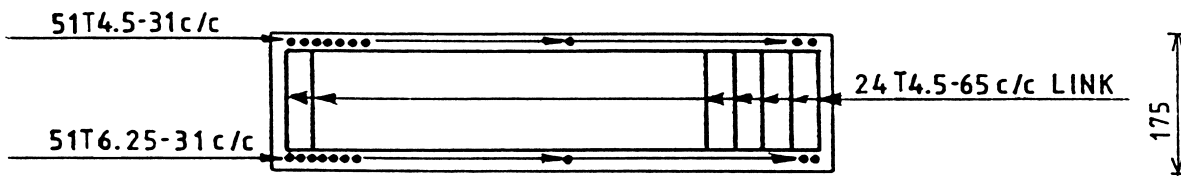
(iii) Ribbed slab panel

RC DETAIL OF TARGET SLAB

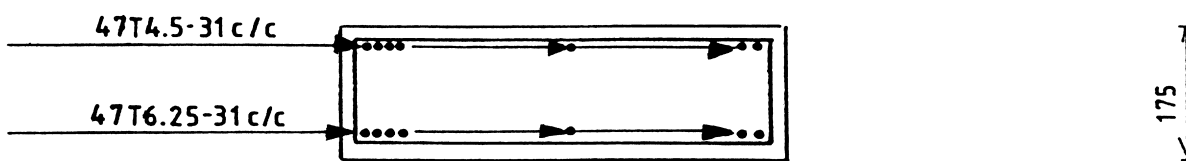
SHEET NO. II.22



PLAN



SECTION A A



SECTION B B

Reinforcement designation

In this section a comparative study is given for reinforcement designation. Drawings are modified to replace British Reinforcement Designation and others are noted below (see Table II.8).

Table II.8. Reinforcement designation and tabular method of detailing—a comparative study

Country	Reinforcement designation
Britain	4T25-05-25tl or T, B (4 number of 25 mm diameter high tensile bar of No. 5 at 250 mm centres top (t, T) or bottom (B) 4R8-06-300 Links
Sweden	4Ø25 C 250 4Ø8 C 300 stirrups
Pakistan/India	4 No. 25 mm Ø 250 mm C/C 4 No. 8 mm Ø 300 mm stirrup spacings
Germany	4Ø25 (bars) $S_{bu} = 25$ cm
Soviet Union (now CIS)	25 Ø (5) 4 NL 25 l ($L =$ length of the bar)
France	4Ø25 C 250 4Ø8 C 300 stirrups
USA	4#8 250 crs # 4 stirrups No. legs 300 mm crs (written in Imperial units)

All bars in slabs and other structures are designated using examples based on the Tabular Method of Detailing, which is shown on Sheet No. II.23.

Note: where drawings are produced by computer graphics, the method of the preparation and presentation should be adhered to standard principles. Typical reinforcement details are given on Sheet No. II.22 for the impactor against a typical target slab.

Composite sections

This book gives a number of cases for detailing composite sections later. The following are the main types:

- (a) steel sections encased in concrete beams/slabs
- (b) steel beam flanges embedded in concrete beams/slabs
- (c) steel studs in concrete welded to flanges of steel beams or any other sections

Sheet No. II.17 gives some composite sections, as discussed earlier.

II.2.2. Slab reinforcement and method of detailing based on Eurocode 2

II.2.2.1. Introduction and basic detailing requirements

The slabs can be simply supported or fixed on some or all edges and can be continuous. In all circumstances, the following terms and conditions are recommended by the Eurocode 2.

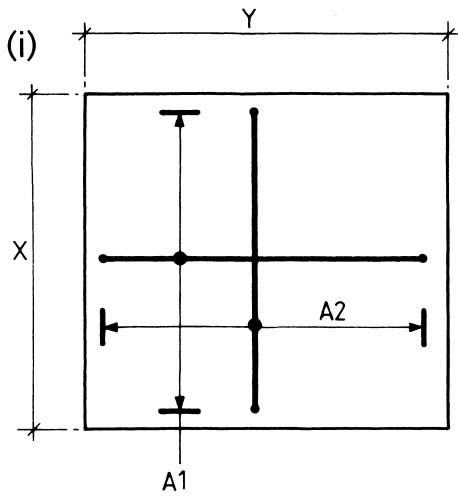
Minimum dimension

Minimum, overall depth = 50 mm

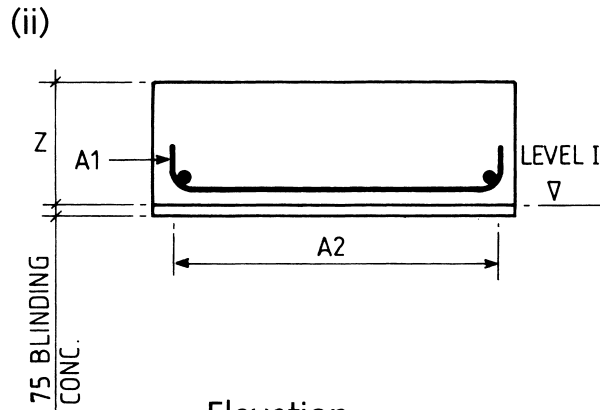
EXAMPLES OF TABULAR METHOD OF DETAILING

SHEET NO. II.23

NOTATION
 COL - COLUMN
 SECT - SECTIONAL
 ELEV - ELEVATION
 DIMS - DIMENSION



Plan

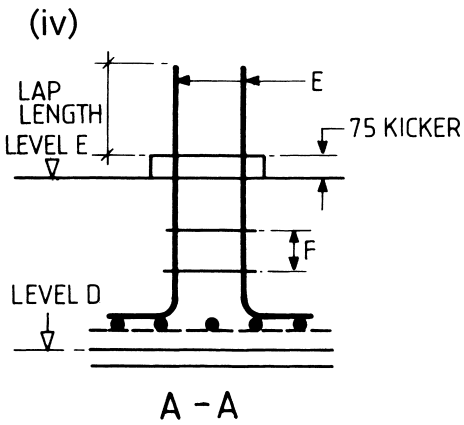


Elevation

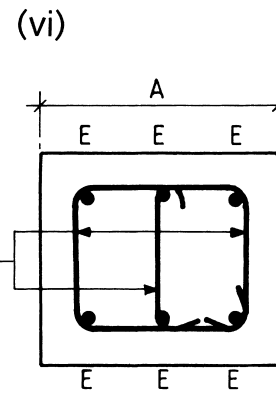
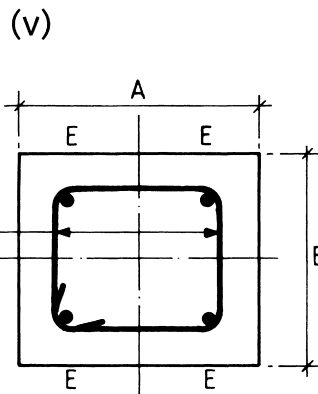
(iii)

COLUMN BASES

BASE	No. OFF	X	Y	Z	REINFORCEMENT		LEVEL D
					B1	B2	



A - A



(vii)

COL	No. OFF	LEVEL		REINFORCEMENT		SECT	ELEV	COLUMN DIMS	
		C	D	E	F			A	B

Longitudinal reinforcement*Minimum area* $A_{st,min}$

$$A_{st,min} \leq \frac{0.6b_c d}{f_{yk}} \leq 0.0015b_c d$$

where f_{yk} is the characteristic yield stress of reinforcement (Sheet No. II.24).

Maximum area $A_{st,max}$

$$A_{st,max} \geq 0.04 A_c$$

where A_c is the cross-sectional area of concrete.

Maximum spacing S_{max}

$$S_{max} \geq 1.5h \geq 350 \text{ mm}$$

Reinforcement near supports

Span reinforcement: minimum 50% of the reinforcement in the span should be anchored at supports (Sheet No. II.22). End supports with partial fixity, but simple support is assumed in design (Sheet No. II.22).

Curtailmnt rules for slabs

These are similar to those for beams.

Transverse reinforcement*Minimum area* A_s

See Sheet No. II.24.

Maximum spacing

$$S_{max} \geq 3h \geq 400 \text{ mm}$$

See Sheet No. II.24.

See Sheet No. II.25.

Corner reinforcement

Suitable reinforcement is required where slab corners are restrained against lifting. See Sheet Nos II.24 and 25.

U-bars in each direction extend 0.21 into span.

Reinforcement at free edges

See Sheet No. II.24.

See Sheet No. II.25.

Shear reinforcement

Minimum slab depth $h \leq 200$ mm where shear reinforcement is to be provided.

General

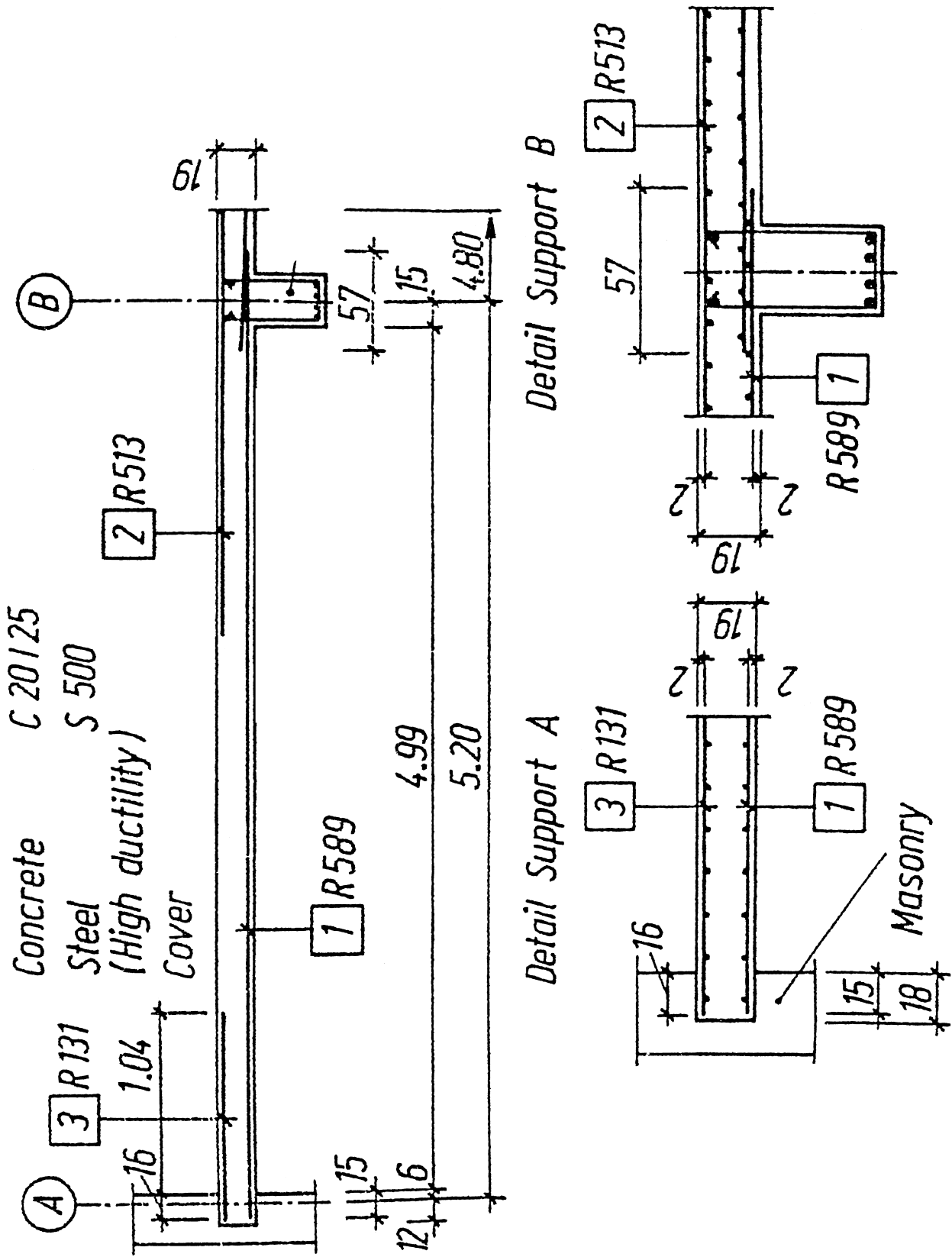
The requirements given in Section 10.2.10.2 of the code for beams apply generally to slabs, with the following modifications.

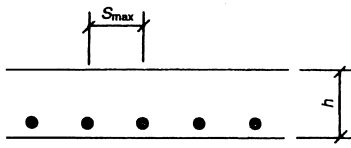
Form of shear reinforcement: shear reinforcement may consist entirely of bent-up bars or shear assemblies where:

$$V_{sd} > 1/3 V_{Rd2}$$

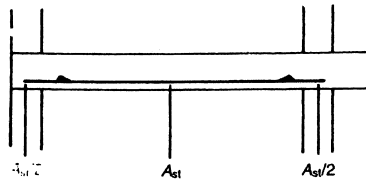
CONTINUOUS REINFORCED CONCRETE SLAB (EC2)

SHEET NO. II.24





Maximum spacing of longitudinal reinforcement



Span reinforcement

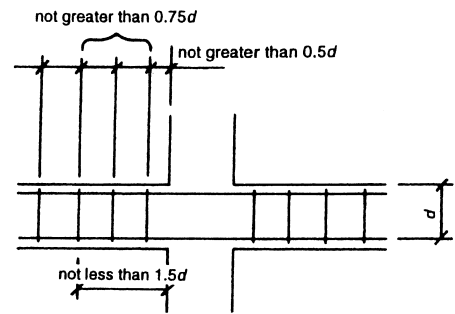
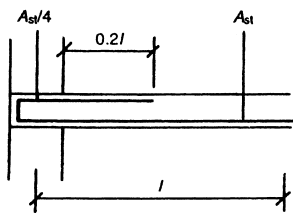
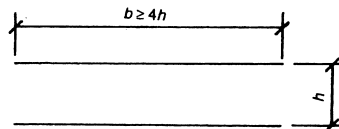
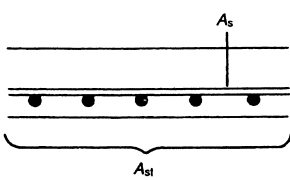


Fig. 10.34. Shear reinforcement near supports for links



End supports with partial fixity



Minimum overall depth for solid slabs

Minimum area of transverse reinforcement

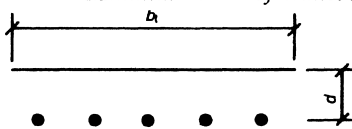
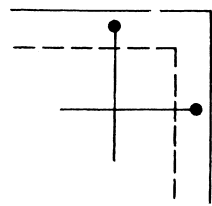
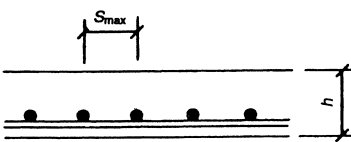
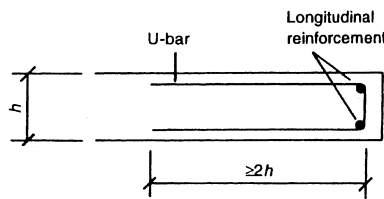


Fig. 10.24. Minimum area of longitudinal reinforcement

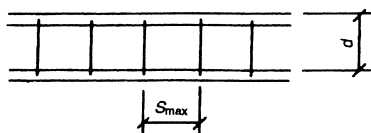
Maximum spacing of transverse reinforcement



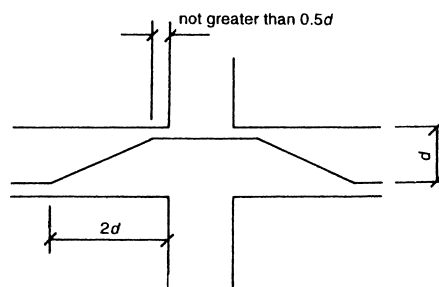
Corner reinforcement



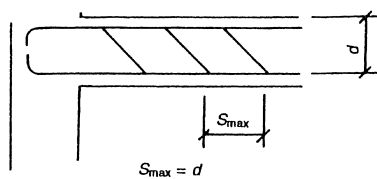
Reinforcement at free edges



Maximum spacing for links



Shear reinforcement near supports for bent-up bars



Maximum spacing for bent-up bars

Maximum spacing for links:

$$V_{Sd} \leq \frac{1}{5} V_{Rd2}: S_{max} = 0.8d$$

$$\frac{1}{5} V_{Rd2} < V_{Sd} \leq \frac{2}{3} V_{Rd2}: S_{max} = 0.6d$$

II.2.3. Slab reinforcement and method of detailing based on ACI, ASCE and other states' practices

II.2.3.1. Introduction

One-way slabs in concrete are defined in the codes as large plates that are supported by reinforced concrete beams, walls, columns and by ground. They are supported on two sides only. A reference is made to Section II.2.1 where the one-way slab has been described. There are differences and they have been highlighted on Sheet No. II.26 where reinforcement detailing for single span and end span simply supported are given. A one-way slab is assumed to be a rectangular beam with a large ratio of width to depth.

A two-way slab is supported by beams or walls and columns on all four edges and bending occurs in both directions. A continuous one-way slab is a slab continuous over beams or columns with end span edges simply supported or fixed or partially restrained. A reference is made to Sheet No. II.27. Similarly to Section II.2.1, when the slabs are supported by columns arranged generally in rows so that the slabs can deflect in two directions, they are also usually referred to as two-way slabs. Two-way slabs may be strengthened by addition of beams by thickening the slabs around the columns (drop panels) and by flaring the columns under the slabs (column capitals). The ACI and ASCE codes give methods for designing two-way slabs either by direct design method or by equivalent frame method. The discussion on these two methods are beyond the scope of this text. The reader is referred to various texts on these methods.

Sheet No. II.26 shows the maximum bend point locations and extensions for reinforcement in slabs without beams with and without drop panels. Detailing of a two-way slab with small detailing differences is identical to British or European codes.

II.2.3.2. Two-way slabs without beams—moderate seismic risk

Reinforcement for the fraction of M_s to be transferred by moment (Eq. (13-1), ACI 318-83 (revision 1990)), but not less than half the total reinforcement required for the column strip, must be placed in the width of slab between lines 1.5 times slab or direct panel thickness on opposite faces of the column (width equals $3h + c_2$ for edge and interiors column, $1.5h + c_2$ for corner column). The engineer must show the reinforcement to be concentrated in the critical width.

A minimum of one-fourth of the column strip with reinforcement must be continuous throughout the span.

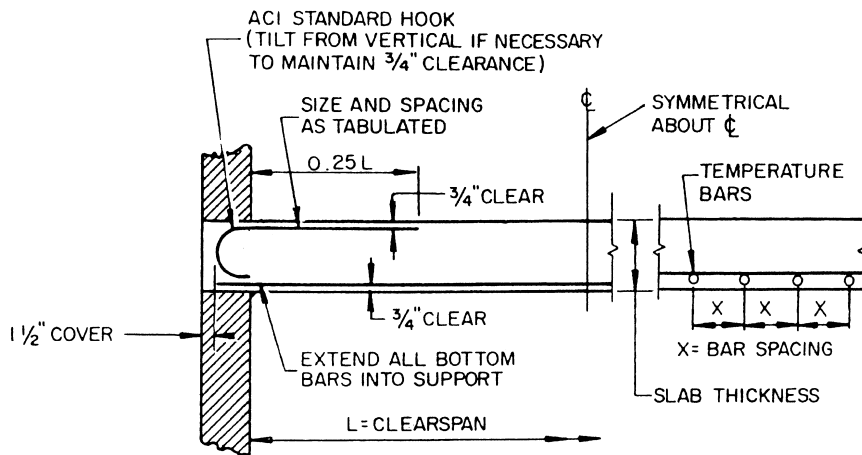
Continuous column strip bottom reinforcement must be not less than one-third of the total column strip top reinforcement at the support. A minimum of one-half of all bottom reinforcement at midspan must be continuous and developed at faces of supports. All top and bottom reinforcement must be developed.

II.2.3.3. Slabs details in seismic zone

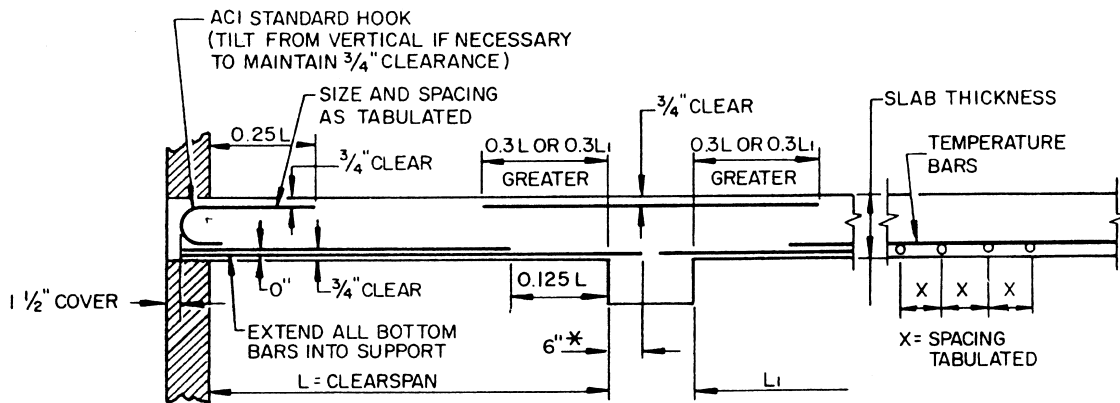
A reference is made to the author's text on seismic design: *Prototype building structures—analysis and design* (Thomas Telford, London, 1999).

The incorporation of seismic design procedures in building design was adopted first in the 1920s when the importance of inertia forces began to be appreciated and structural detailing in seismic zones became a priority. ACI

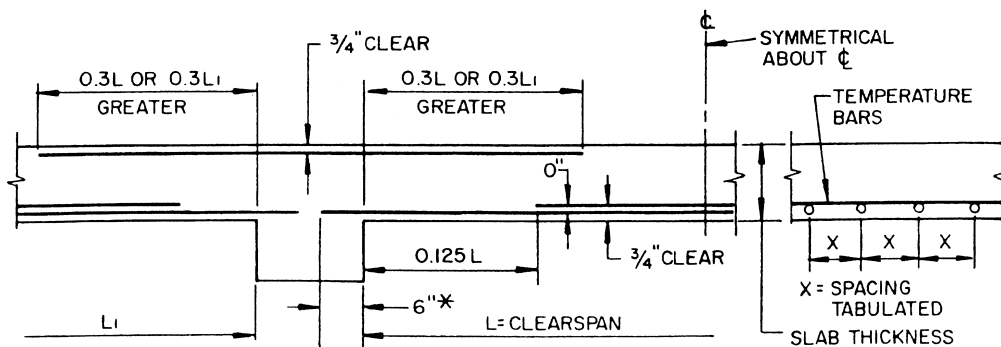
**TYPICAL DETAILS FOR ONE-WAY SOLID SLABS
(ASCE, ACI AND OTHER PRACTICES IN STATES)**



SINGLE SPAN, SIMPLY SUPPORTED

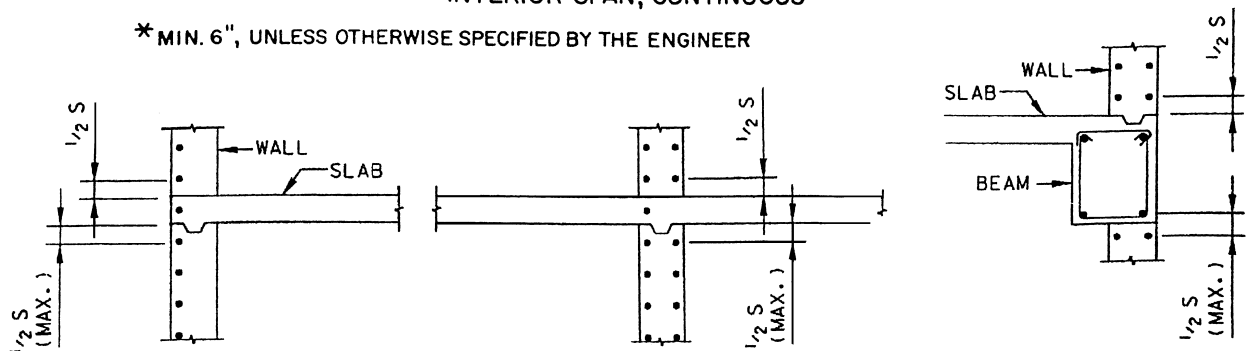


END SPAN, SIMPLY SUPPORTED



INTERIOR SPAN, CONTINUOUS

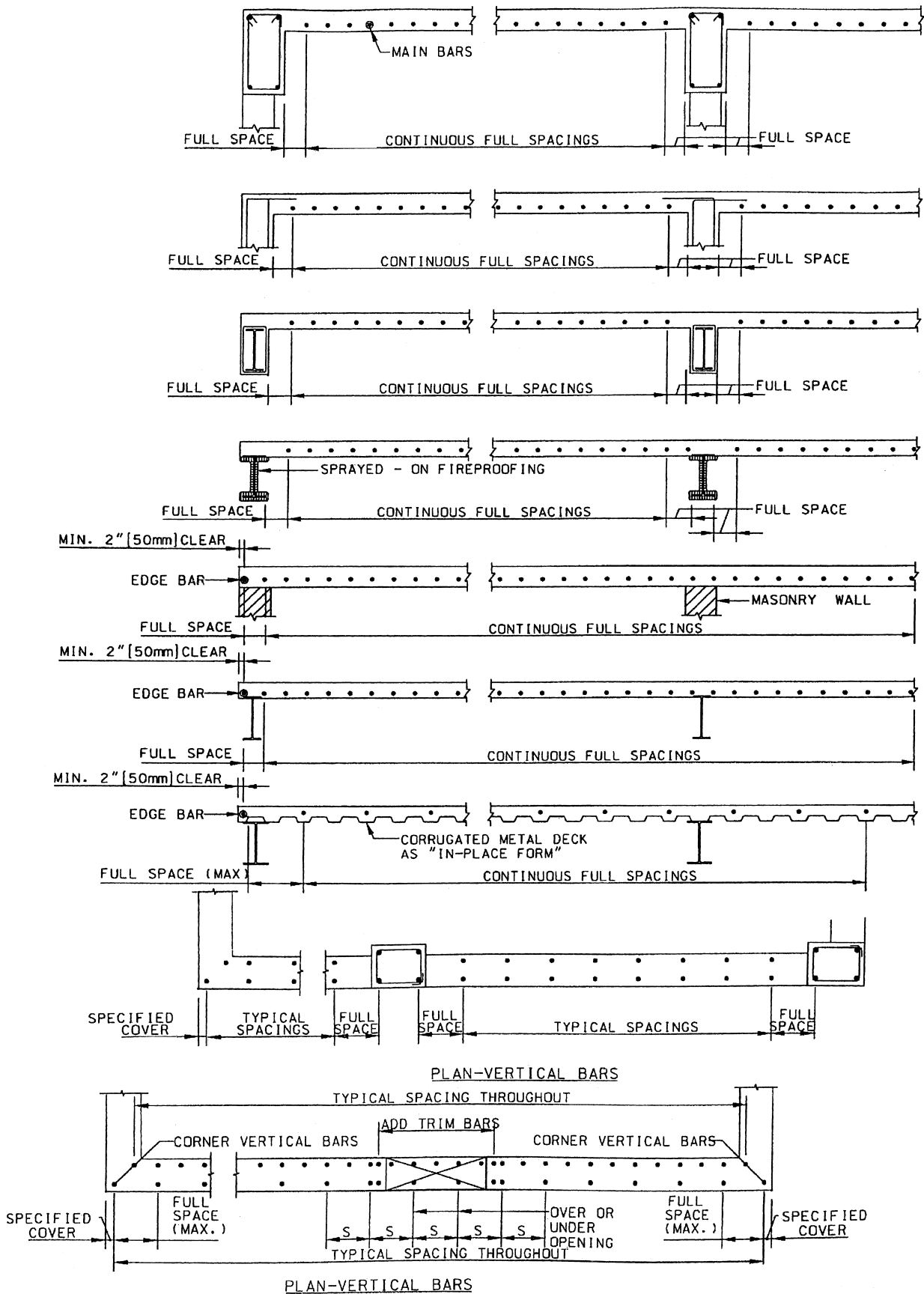
* MIN. 6", UNLESS OTHERWISE SPECIFIED BY THE ENGINEER



VERTICAL SECTIONS - WALLS AT FLOORS

ONE-WAY SUPPORTED WITH SHRINKAGE AND TEMPERATURE REINFORCING BARS

SHEET NO. II.27



codes exist on the seismic design, the details of which are out of the scope of this text.

$$V_{sd} > \frac{2}{3} V_{Rd2}: S_{max} = 0.3d$$

Maximum spacing for bent-up bars:

Shear reinforcement near supports for links:

Shear reinforcement near supports for bent-up bars.

Where a single line of bent-up bars is provided, their slope may be reduced to 30°. It may be assumed that one bent-up bar carries the shear force over a length of $2d$.

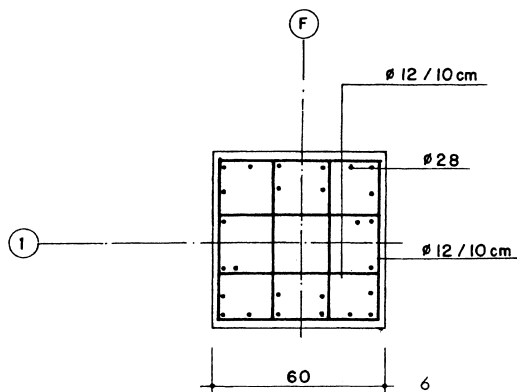
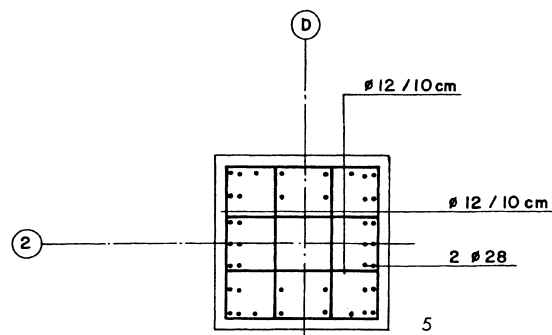
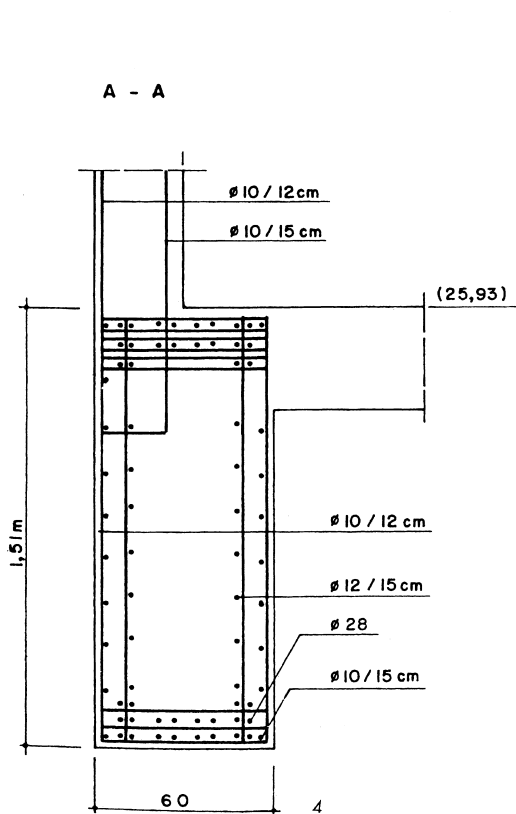
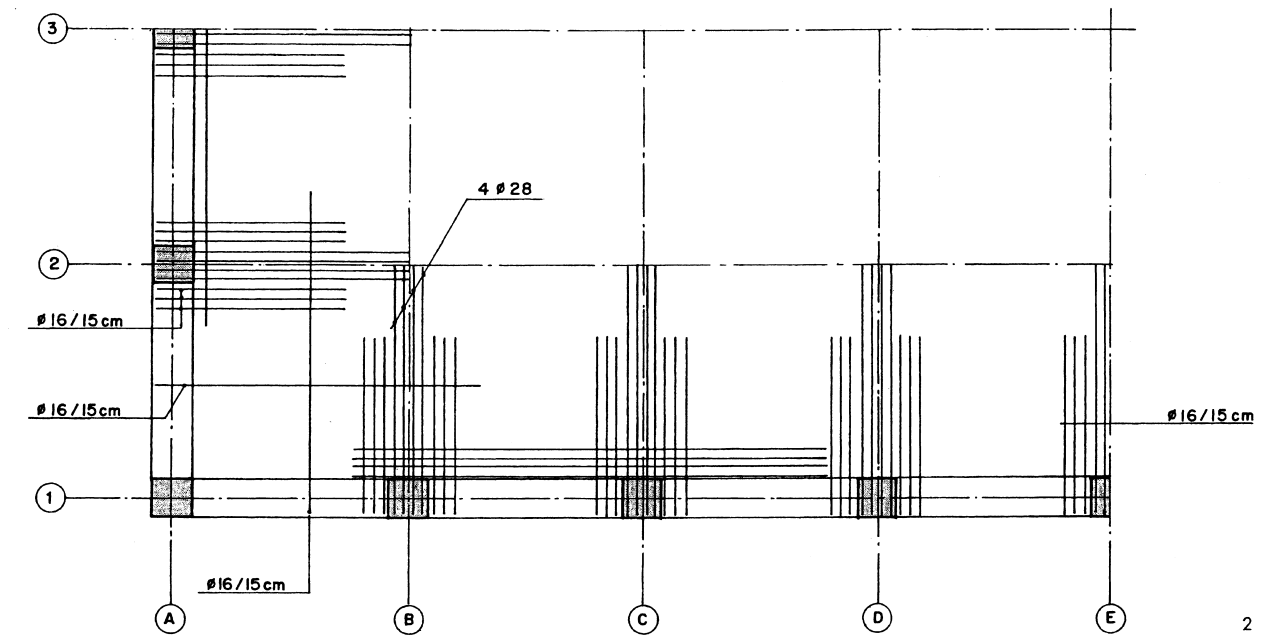
II.2.3.4. Floor slab supported by RC columns and deep beams

Sheet No. II.28 shows structural and reinforcement details of floor slab supported by reinforced concrete columns and deep beams using Eurocode 2. The details for the reinforcement are identical to the criteria given for slabs in Section II.1. A reference is also made to Sheet No. II.29 indicating vertical section of a deep beam resting on columns.

The required vertical reinforcement may be established by considering vertical strips of deep beam as columns subjected to the local intensity of vertical load and transverse moment. Where required, these columns should be designed to take account of slenderness effects in accordance with ENV 1992.

**REINFORCED CONCRETE FLOOR SLAB ON COLUMNS
AND DEEP BEAMS (EC2)**

SHEET NO. II.28

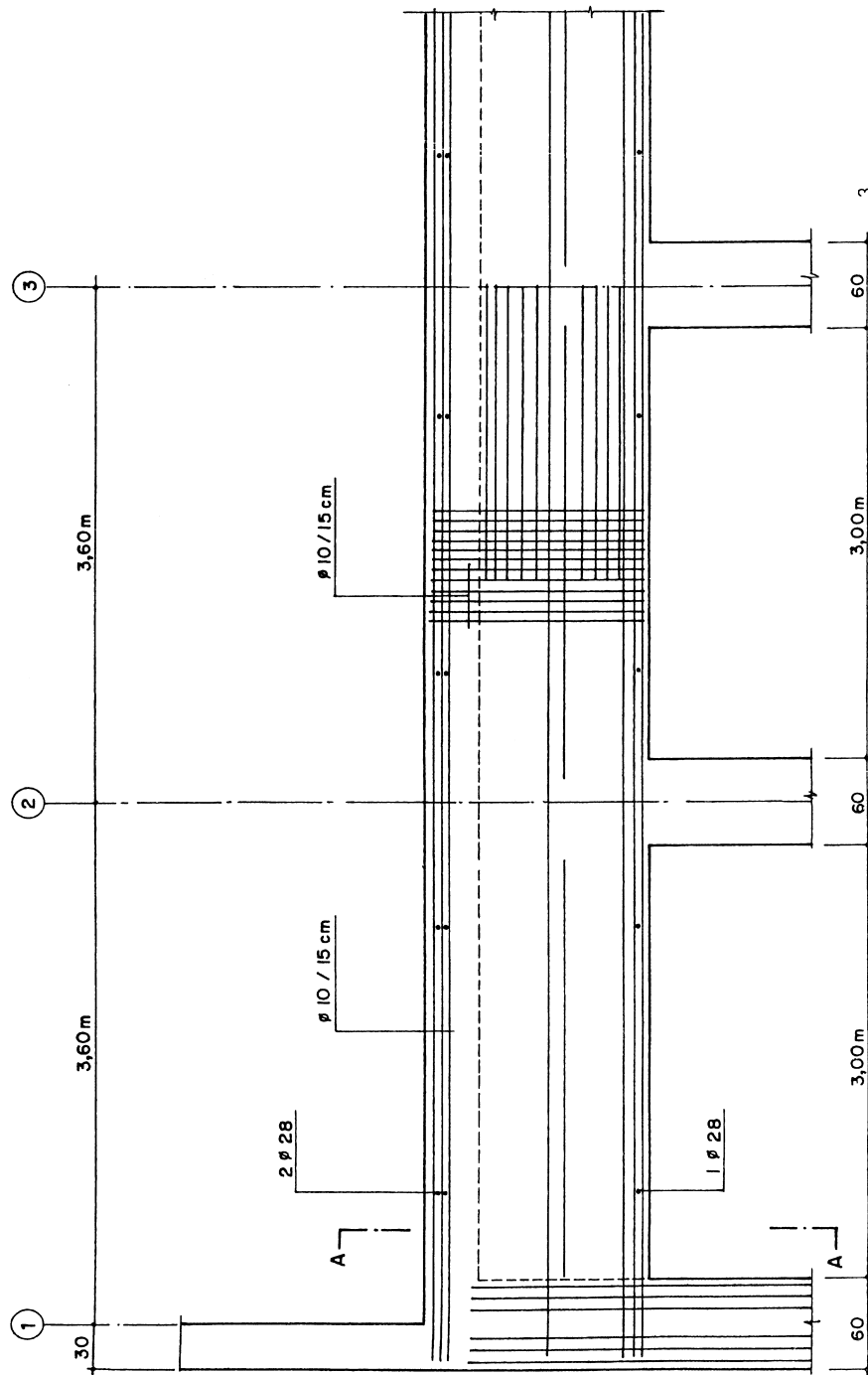


Structural elements

1. Column line;
2. Floor detail: reinforcement;
3. Typical grid line;
4. Cross-section of the beam: reinforcement;
- 5-6. Column sections: reinforcement.

DEEP BEAM (EC2)

Vertical section



III. Stairs and staircases

III.1. Stairs and their types

Stairs lead from floor to floor. They are of several types. The common ones are:

- (a) a sloping slab spanning from one floor to a landing or another floor
- (b) a sloping slab carried on sloping beams from one floor to another or to a landing.

Typical reinforcement details are given for these staircases on Sheet Nos III.1 and III.2. As seen there are several variations in the presentation of the structural detailing of staircases. In all situations the staircases have landings and waste as shown in the key diagram (i) on Sheet No. III.2. The dimensions are established in III.3. The rise of the stair does not usually exceed 150 mm and the tread 250 mm including a nosing of about 25 mm beyond the vertical surface of the rise. The load for which these staircases is designed varies with the type of building. In all circumstances, the detailer must check specifications with the structural engineer prior to carrying out reinforcement details.

III.1.1. Specifications and basic data on staircases

A stair is constructed with steps rising without a break from floor to floor, or with steps rising to a landing between floors, with a series of steps rising further from the landing to the floor above. There are three basic ways in which stairs are planned:

- (a) a *straight flight stair*, which rises from floor to floor in one direction with or without landing
- (b) a *quarter turn stair*, which rises to a landing between floors, turns through 90°, then to the floor above
- (c) a *half turn stair*, which rises to a landing between floors, turns through 180°, then rises, parallel to the lower flight, to the floor above. This type of stair is sometimes called 'dog-leg' or 'scissor-type stair'.

III.1.1.1. Geometric stairways

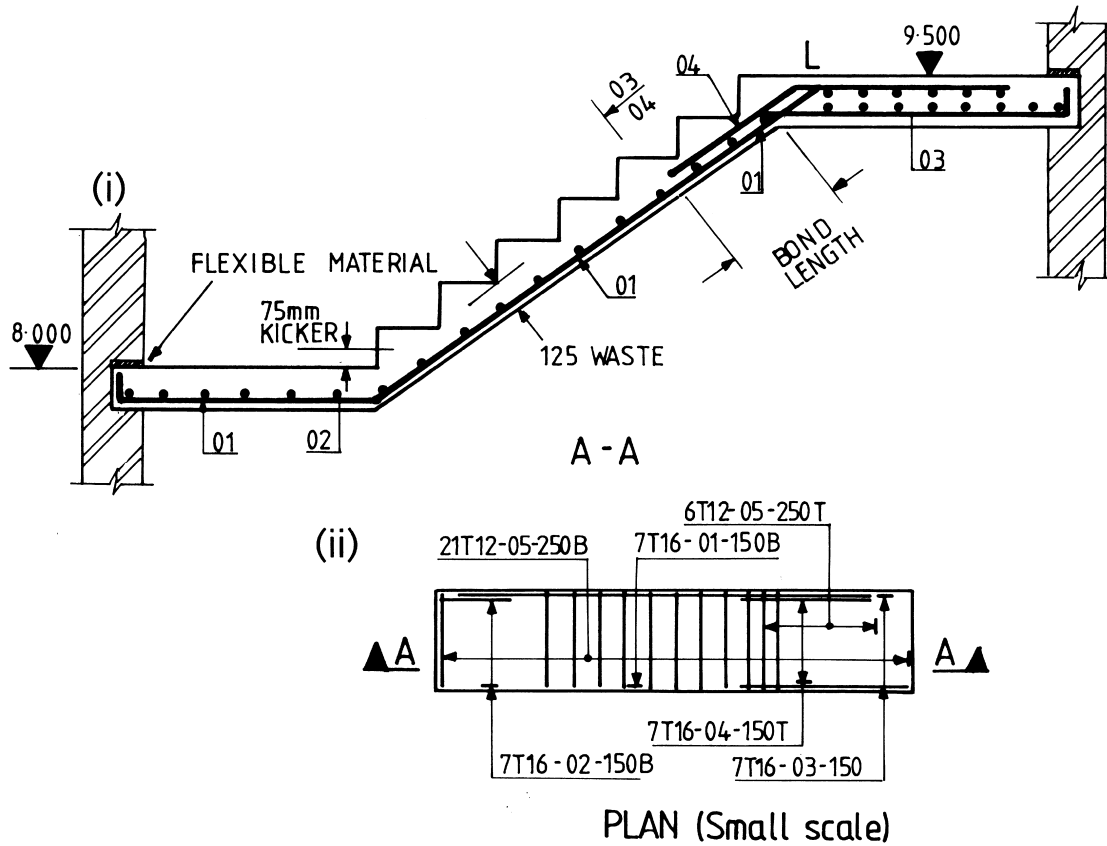
The stairs mentioned above are generally freestanding ones. In addition to these, stairs known as geometrical stairs can be designed into spiral, helical, circular, elliptical and other shapes. They can all be in concrete, steel, timber or combination. The stairs are sometimes described as open well stairs where a space or well exists between flights.

Again, in free-standing stairs the main types are:

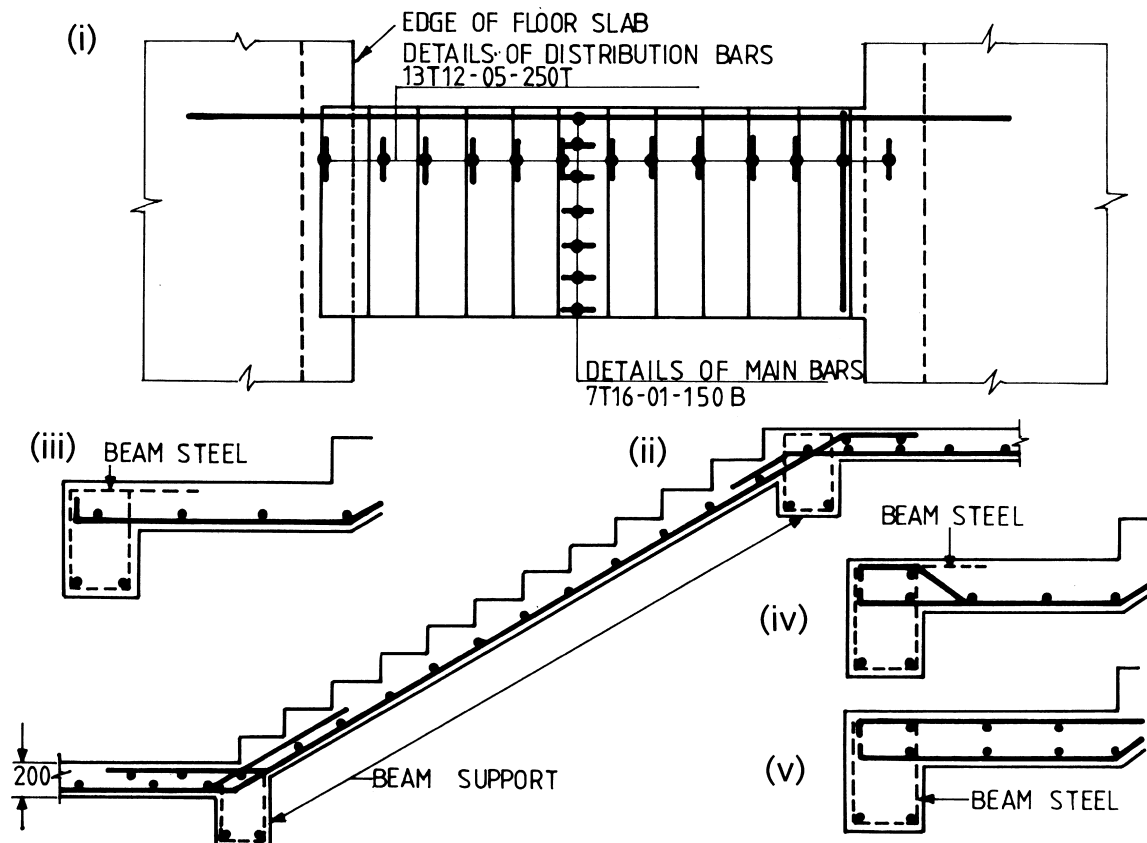
- (a) type 1: those supported transversely or across the flight—stringer beams are needed on one or both sides
- (b) type 2: those spanning longitudinally along the flight of steps either on walls or on landing beams or on wall beams
- (c) type 3: cantilever type projecting from walls or wall beams with each step acting as a cantilever

STAIRCASE REINFORCEMENT

(a) Straight stairflight supported on brickwork



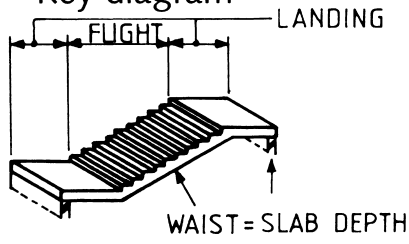
(b) Straight stairflight on beam supports



STAIRCASE WITH SLAB CONSTRUCTION

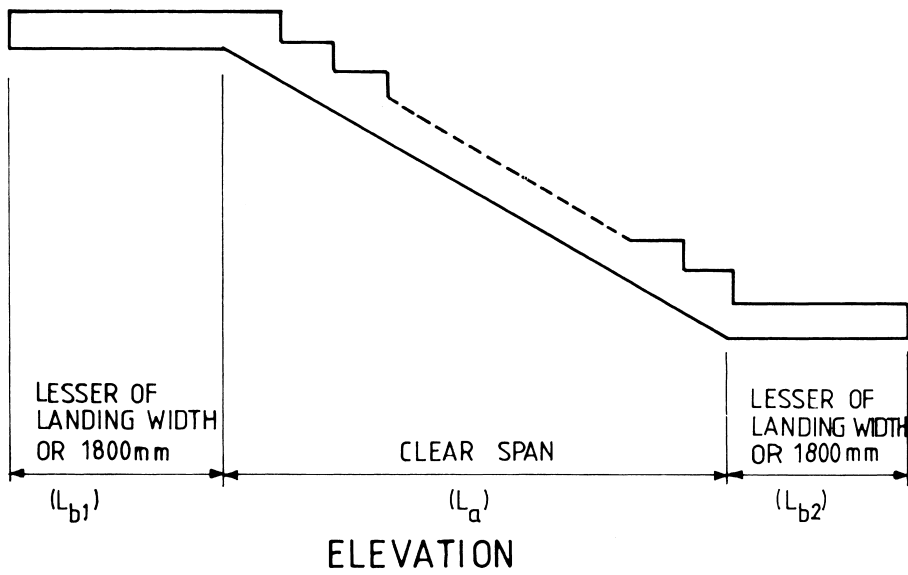
SHEET NO. III.2

(i) Key diagram

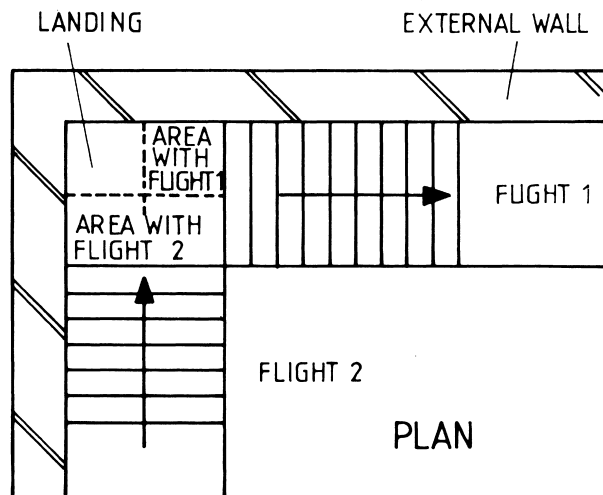


(a) Slab construction

(i)



(ii)

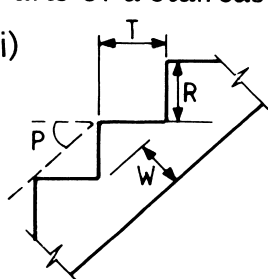


- G - GOING
- N - NOSING
- P - PITCH
- R - RISE
- T - TREAD
- W - WASTE

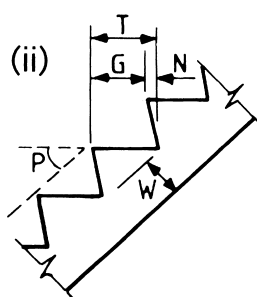
PLAN

(b) Parts of a staircase

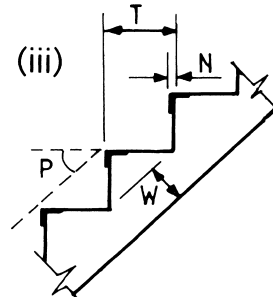
(i)



(ii)

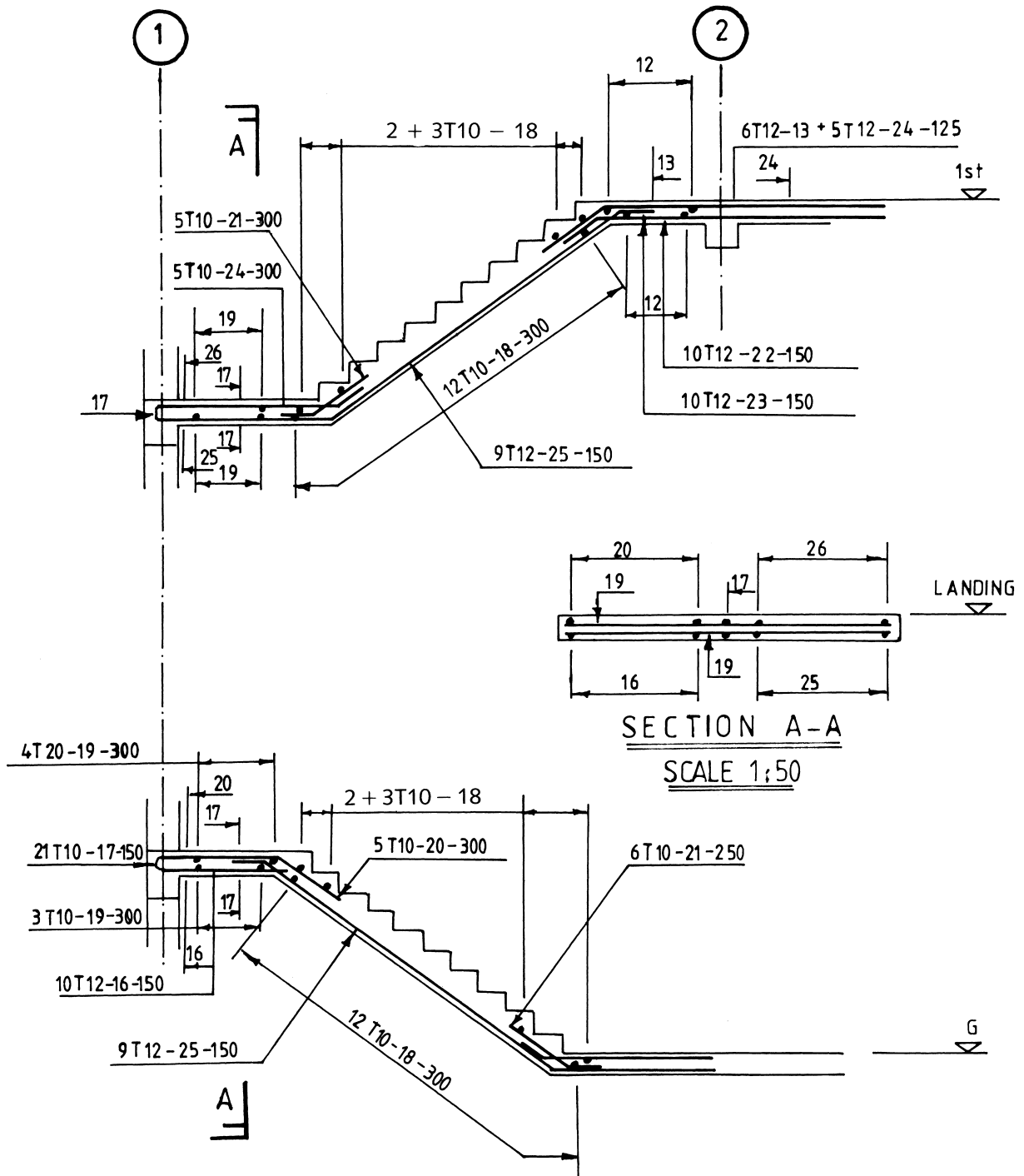


(iii)



STAIRCASE ELEVATION

SHEET NO. III.3



STAIRCASE ELEVATION

SCALE 1:50

- (d) type 4: combination of type 2 and type 3—every 4th or 5th step is cantilevered with sloped soffit with a slab continuous between two steps.

III.1.2. Stairway layouts

Stairway layouts depend on several factors including building type and its layout, choices, material, etc. Comfortable stairways should be designed in relation to the dimensions of the human figure. The British Standard on stairs BS 5395 (1977) defines some of these dimensions in Figure 1.6. The British and the European practices use the following criteria for width, length and headroom, etc.

- (a) Flats—two storey to four storey $w_F = 900$ mm; more than four storey $w_F = 1000$ mm.
- (b) Public buildings using each floor—under 200 persons $w_F = 1$ m; 200 to 400 persons $w_F = 1.5$ m; in excess of 400 persons $w_F > 3$ m. Where the width is 1.8 m or over, the width should be divided by a handrail.
- (c) The length and rise a minimum of three steps and a maximum of 16 steps. There must not be more than 36 rises in consecutive flights without a change in the direction of travel of 30° or more. The total rise must not exceed 6 m.

III.1.2.1. Landings, landing beams and flights

A quarter space landing in wood is generally supported by a newel post carried down to the floor below. In small houses quarter or half turn stairs are sometimes constructed with winders instead of quarter or half space landings. Winders are triangular shaped steps constructed at the turn from one flight to the next. The landing beams are designed as rectangular or flanged beams, for the reactions from the two flights or steps on one side and the landing on the other.

III.1.2.2. Strings or stringers

These are available in steel, concrete, timber and composite. There are two types of wood string, namely, the open (cut) and the close (closed) strings. In wood their top edges project some 50 to 60 mm above the line of nosing or tread. Wall strings are closed ones. The outer strings, particularly those made in timber, are cut to the profile of the treads and risers and are secured by wood bearers screwed to both strings and treads or risers in the underside of the flight.

III.1.3. Additional basic layouts and data

Sheet No. III.4 gives additional basic layouts and data for various parameters required for the planning and design of staircases.

The dimensions and other specifications are derived from the general layout of the building or structure where the stairs are to be used. Two typical layouts given on Sheet No. III.5 show the exact positioning of these staircases with respect to grid work and floor levels. Sheet No. III.4 gives beam/slab/column reinforcement layouts with respect to a staircase. Sheet No. III.5 gives a beam/slab/column plan on a section showing levels and grid work.

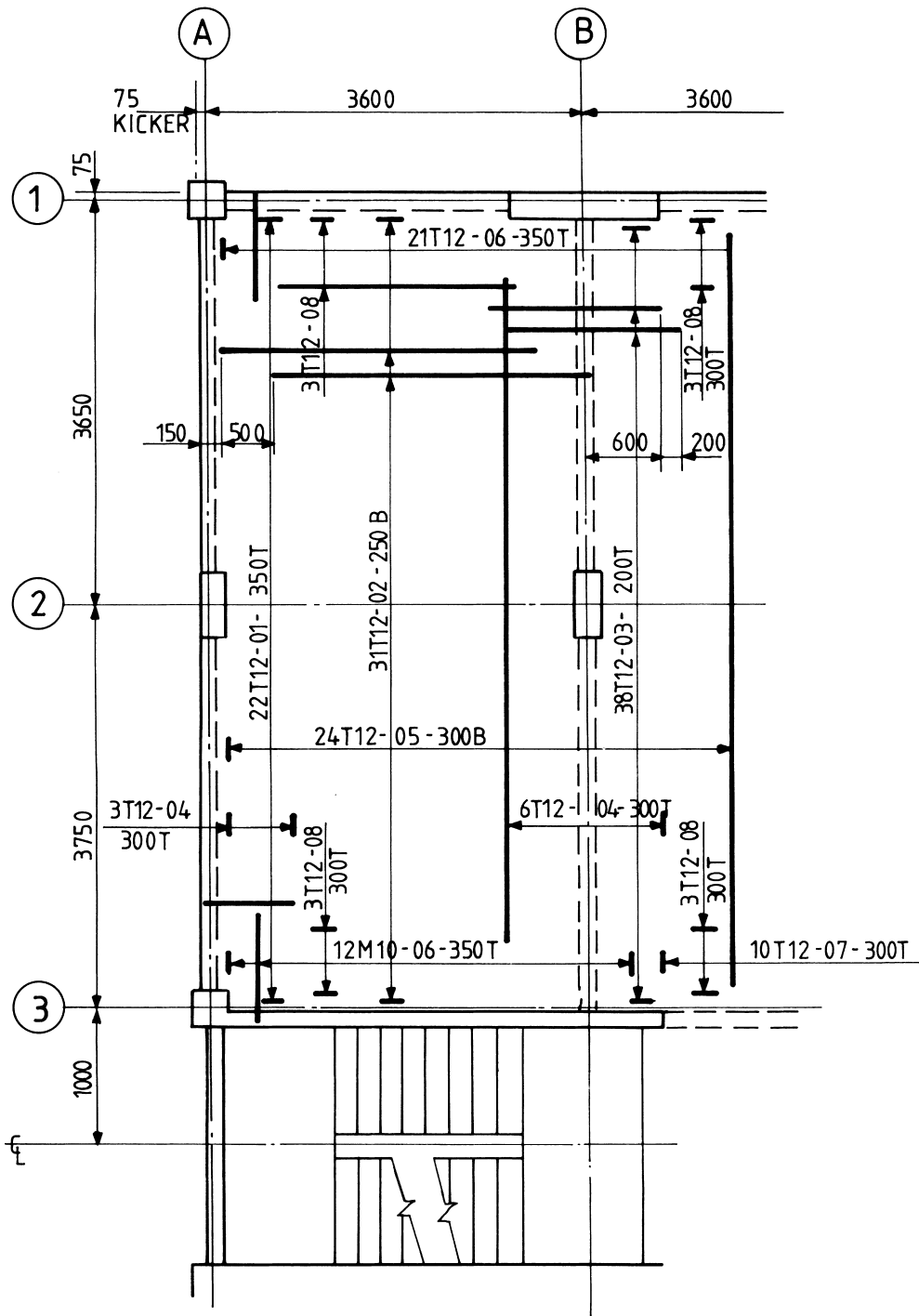
These staircases will have the reinforcement details as outlined on Sheet Nos III.1 and III.2.

There are a number of other types, such as stairs cantilevered from a side wall, spiral stairs with sides cantilevered out from a central column and free-spanning spiral stairs. They can be easily designed and detailed. Geometrical stairs are described on Sheet No. III.6.

Precast concrete staircases have recently become very popular and a number of companies are involved in producing them. In this book Birchwood

STAIRCASE WITH THE BEAM SLAB ARRANGEMENT

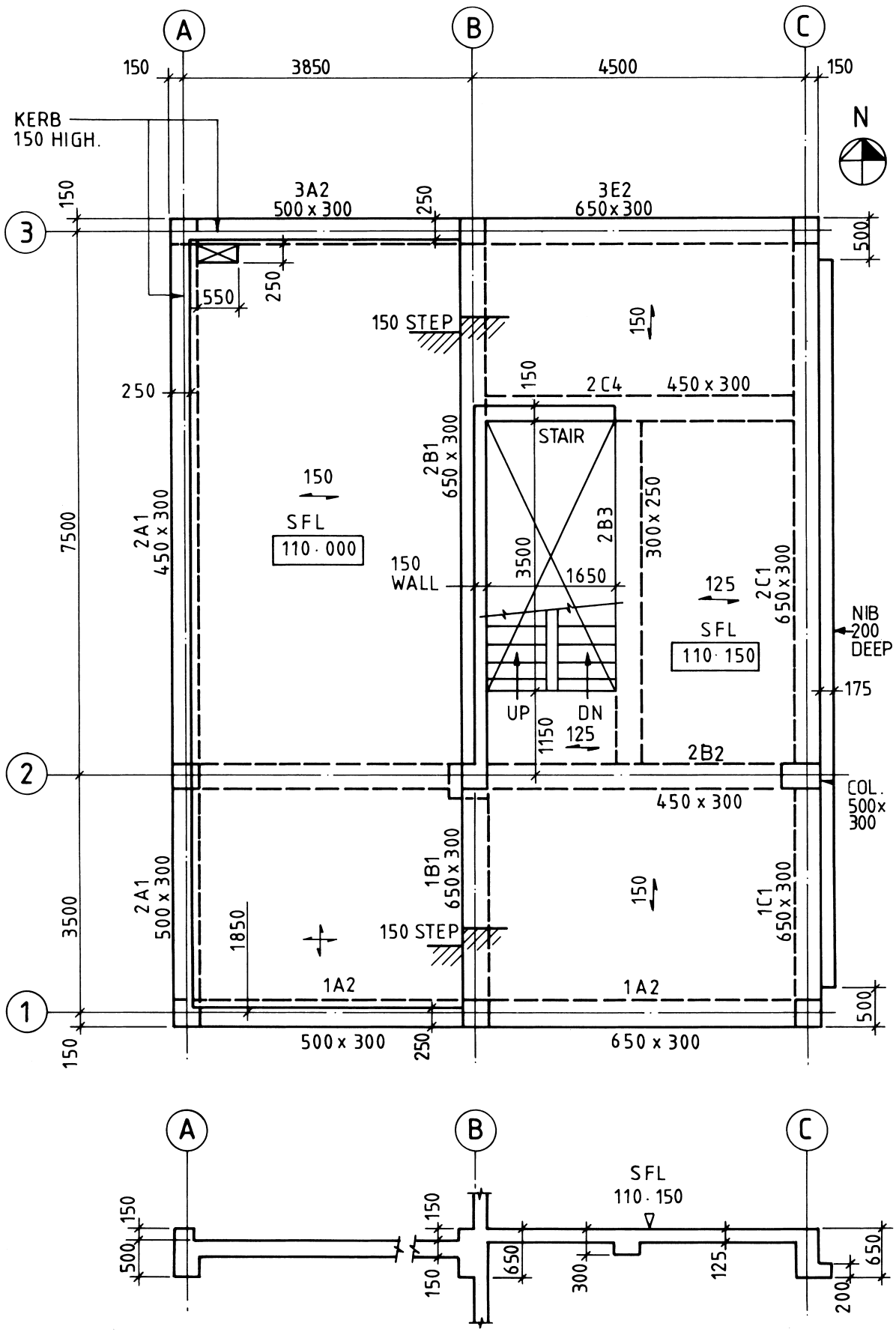
SHEET NO. III.4



PLAN

For staircase reinforcement details see Sheet Nos. III.1 and III.2.

GENERAL ARRANGEMENT PLAN FOR CONCRETE STRUCTURES SHEET NO. III.5



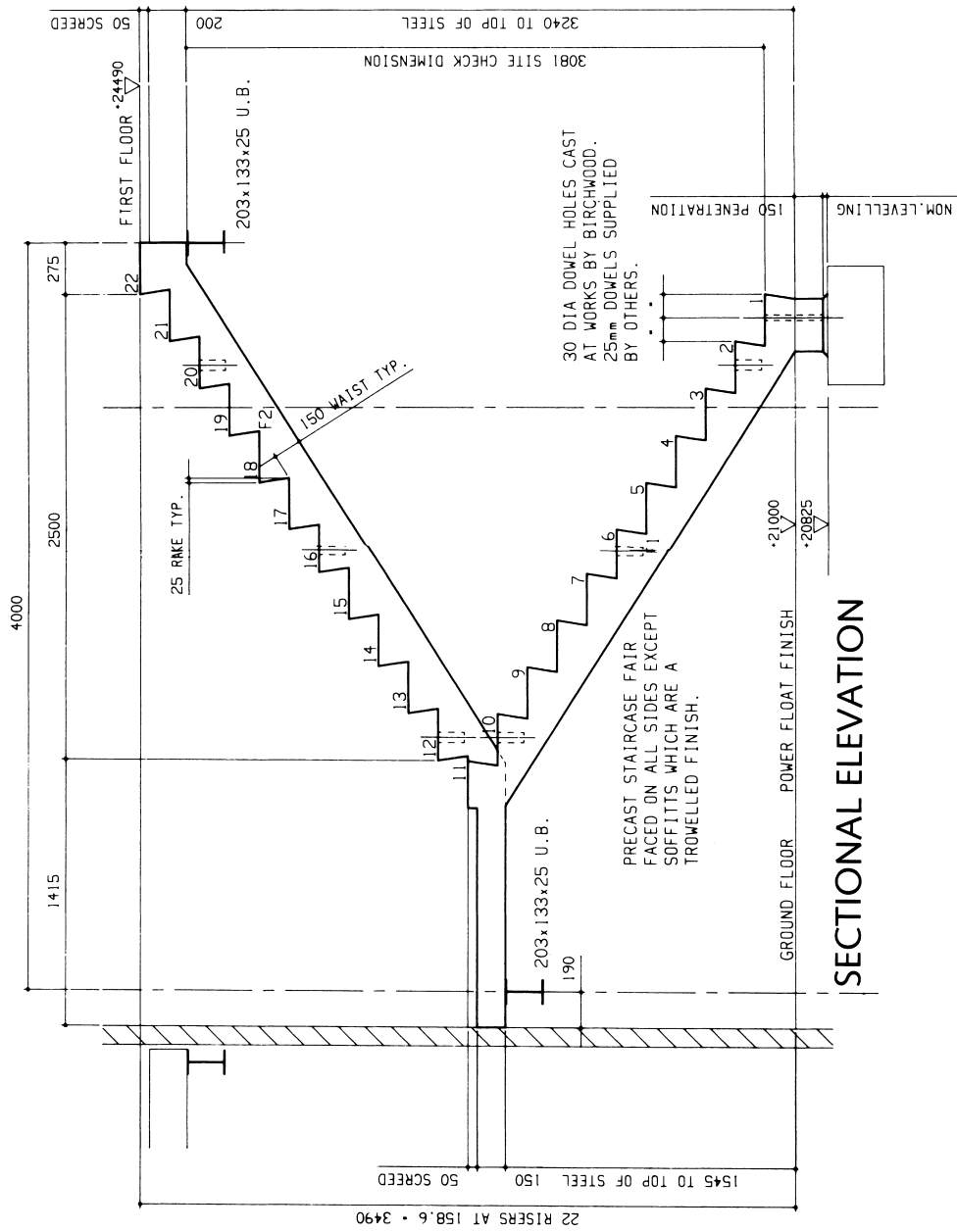
prestressed concrete staircases are shown on Sheet Nos III.7 to III.10. Sheet Nos III.7 and III.8 give sectional elevations and plans of staircase details. The stairs are connected to precast concrete floor units which themselves are connected to cross-landings and bearings. Sheet Nos III.9 and III.10 show typical reinforcement details for stairs, bearings and landings. Loading and material specifications are given on each drawing. The standard accepts vinyl tiles, sheet or carpet direct. Where any two or more members join, it is recommended to use site-applied screed to the landing. Dimensions and details for the rise and going for these stairs are given below in Table III.1.

Table III.1. Dimensions and details for the rise and goings of staircases (information abstracted from Building Regulations 2000 and International Building Code 2000)

Staircase type	Rise (max.)	Going (min.)
Private—giving single access	220	220
Common—giving joint or multi-access	190	240
Disabled	170	250
Institutional and assembly buildings	180	250
Any type not described	190	250

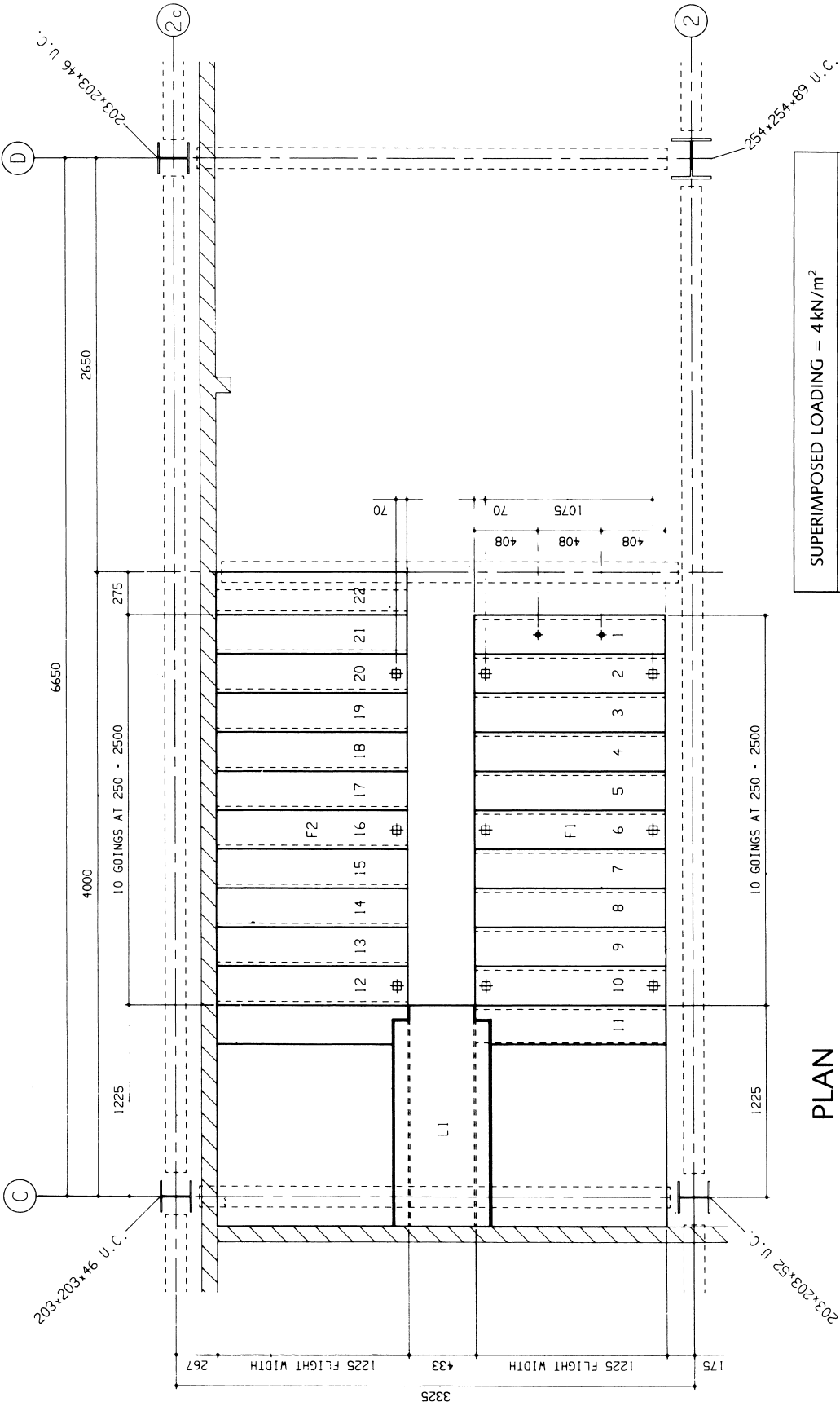
BIRCHWOOD PRECAST CONCRETE STAIRCASE

SHEET NO. III.7



BIRCHWOOD PRECAST CONCRETE STAIRCASE

SHEET NO. III.8



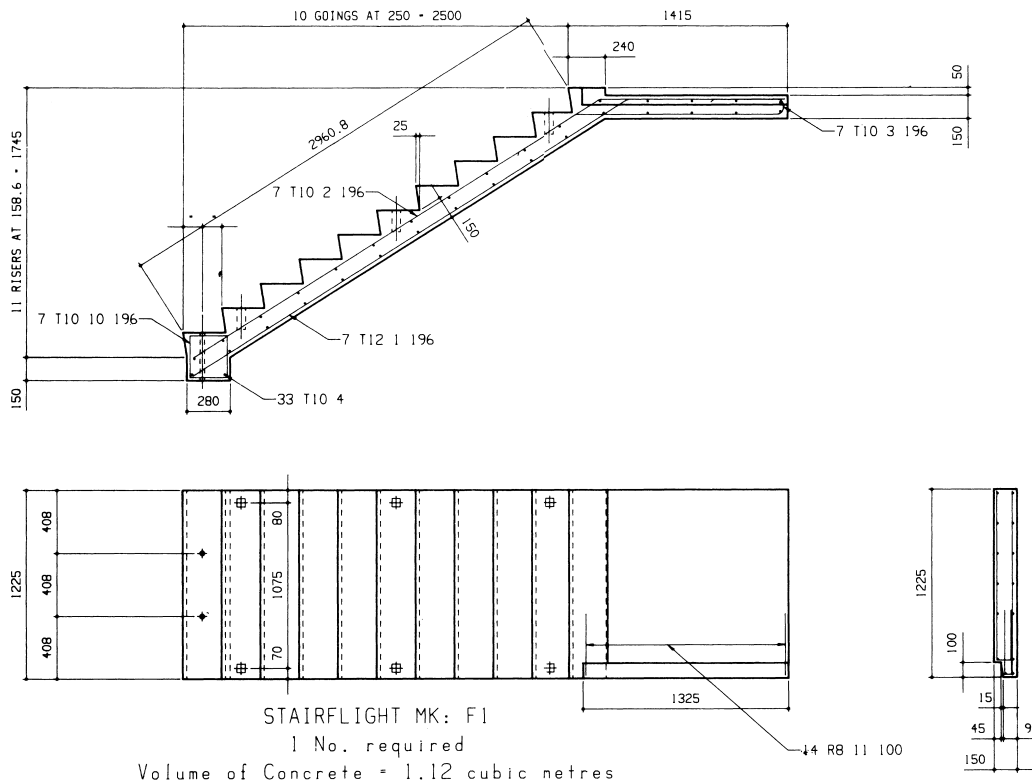
SUPERIMPOSED LOADING = 4 kN/m²

1. Treads and risers finished smooth to receive carpet or vinyl tiles direct.
2. Concrete strength 40 N/mm² at 28 days.
3. Baluster pockets 55 × 55 × 140 deep.
4. All laps to be 40d.
5. Cover = 20 mm.

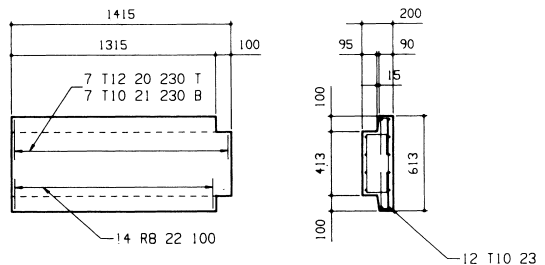
PLAN

BIRCHWOOD PRECAST CONCRETE STAIRCASE
Reinforcement Details— 1

SHEET NO. III.9



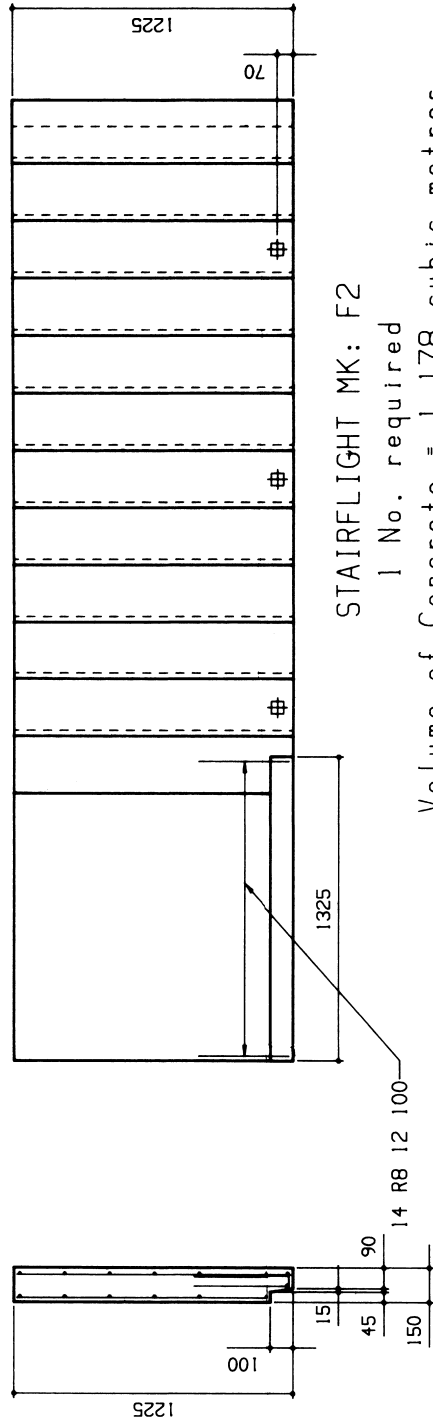
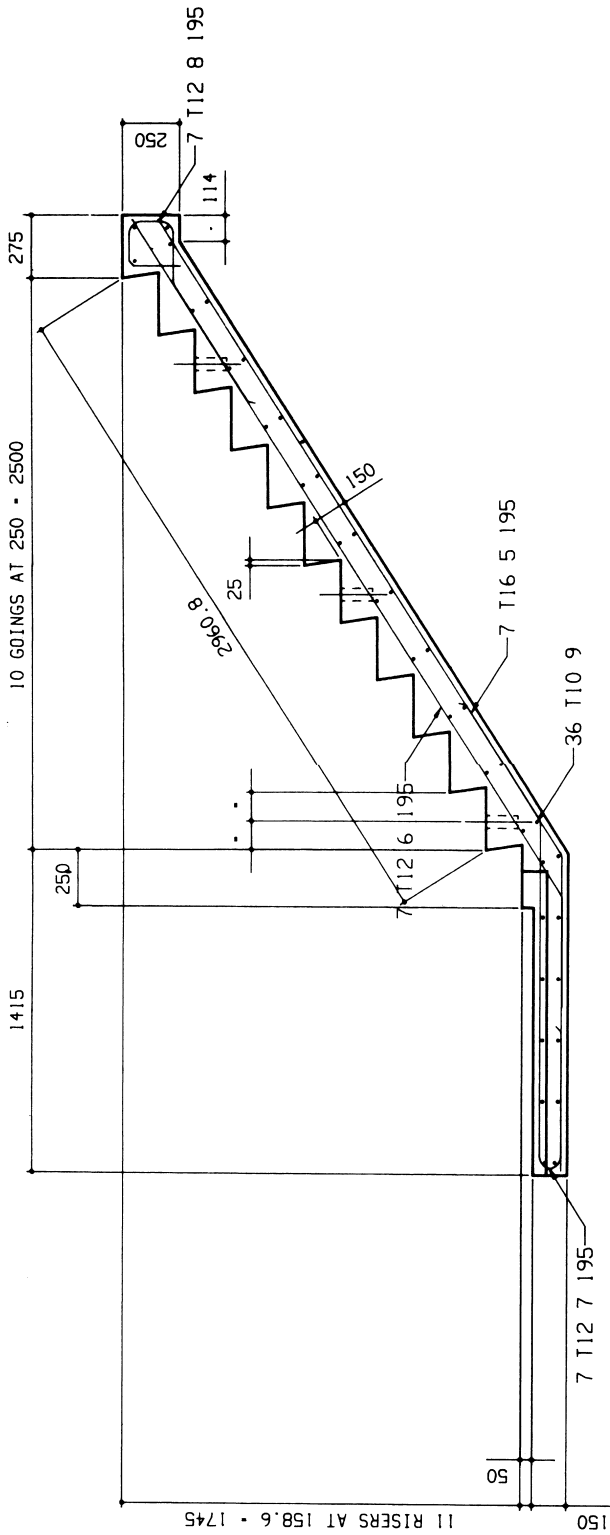
- | |
|--|
| <p>SUPERIMPOSED LOADING = 4 kN/m²</p> <ol style="list-style-type: none"> 1. Treads and risers finished smooth to receive carpet or vinyl tiles direct. 2. Concrete strength 40 N/mm² at 28 days. 3. Baluster pockets 55 × 55 × 140 deep. 4. All laps to be 40 d. 5. Cover = 20 mm. |
|--|



LANDING MK: L1
 1 No. required
 Volume of Concrete = 0.169 cubic metres
 Total Weight of Landing = 0.406 tonnes

BIRCHWOOD PRECAST CONCRETE STAIRCASE
Reinforcement Details—2

SHEET NO. III.10



STAIRFLIGHT MK: F2

1 No. required

Volume of Concrete = 1.178 cubic metres

Total Weight of Flight = 2.976 tonnes

*III.1.3.1. Data for
geometric stairways*

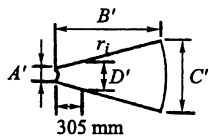
A brief introduction to these staircases is given in Section I.1. It is vital to give brief data on spiral/helical staircases. These staircases are manufactured in a variety of diameters. The most common materials for tread and platform are steel, aluminium and wood. Steel and aluminium can be smooth plate, checker plate, pan or tray type and bar. A variety of hardwoods can be used. For exterior or wet area interiors, zinc-chromated rust inhibitor, black acrylic enamel and black epoxy are usual. Platform dimensions usually are 2 in. (50 mm) larger than the stair radius. Table III.3 in Sheet No. III.11 gives specifications for spiral and helical stairs. Where horse-shoe shapes are involved, the data for helical stairs circular in plan are modified include the geometry of the inclined straight arms. The data collected are from countries such as Britain, Spain, USA, Germany, Sweden, Pakistan, India, Italy, Turkey and Japan. A reference is made to some structural details of various geometric stairways given on Sheet Nos III.12 to III.17.

SPIRAL/HELICAL STAIRS: PARAMETERS AND FRAMING DIMENSIONS

SHEET NO. III.11

Table III.2. Specifications for spiral/helical stairs parameters.

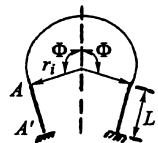
Diameter (cm)	101.1	121.9	132.0	152.4	162.6	182.9	193.0	223.5	243.8
Centre column (cm)	10.1	10.1	10.1	10.1	10.1	10.1	10.1	16.8	16.8
Weight (kg)	93.9	99.8	106.6	113.4	120.2	140.6	147.4	197.3	220.0
Tread detail A' (cm)	10.1	10.1	10.1	10.1	10.1	10.1	10.1	16.8	16.8
Tread detail B' (cm)	45.7	55.9	61.0	71.1	81.3	86.4	91.4	106.7	121.9
27 tread detail C' (cm)	23.5	28.3	30.8	32.7	37.8	42.5	44.8	52.0	56.7
27 tread detail D' (cm)	19.4	20.3	21.0	22.2	21.6	21.9	21.0	25.4	26.7
30 tread detail C' (cm)	26.7	31.9	34.6	39.7	42.5	48.0	50.5	58.4	63.8
30 tread detail D' (cm)	21.6	21.9	22.2	22.5	22.9	23.5	23.8	28.9	29.2
Landing size (cm)	55.9	66.0	71.1	81.3	86.4	96.5	101.6	116.8	132.0



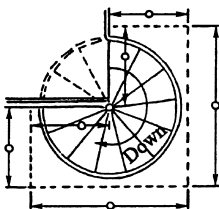
a) Sector in plan



b) Circular in plan



c) Horse-shoe in plan



d) Framing dimensions

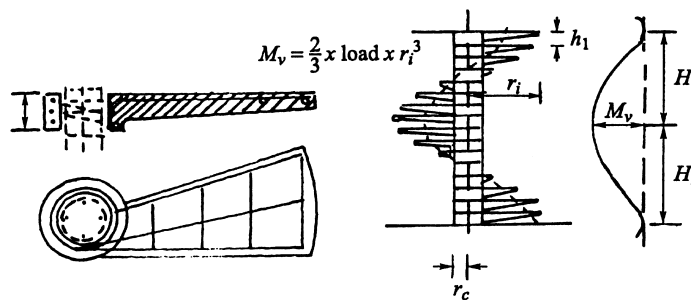
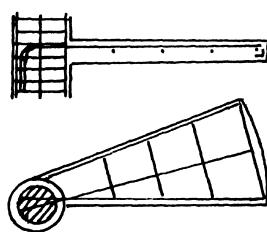
Framing dimensions (cm)

Stair diameter	2.54	5.08	7.62	10.16	12.70
40	50.8	50.8	67.0	111.8	111.8
48	67.0	67.0	71.1	132.0	132.0
52	66.0	66.0	76.2	142.2	142.2
60	76.2	76.2	68.4	162.6	162.6
64	81.3	81.3	91.4	172.7	172.7
72	91.4	91.4	101.6	193.0	193.0
76	96.5	96.5	106.7	203.2	203.2
88	111.8	111.8	121.9	233.7	233.7
96	121.9	121.9	132.0	254.0	254.0

27 riser table

30 riser table

Finish floor height (cm)	Number of steps	Circle degree	Finish floor height (cm)	Number of steps
228.6-243.8	11	297	215.9-241.3	9
246.4-264.2	12	324	243.8-264.2	10
266.7-284.5	13	351	266.7-289.6	11
287.0-304.8	14	375	292.1-312.4	12
307.3-325.1	15	405	315.0-337.8	13
327.7-345.4	16	432	340.4-360.7	14
348.0-365.8	17	459	363.2-386.1	15
368.3-386.0	18	486	388.6-408.9	16
388.6-406.4	19	513	411.5-434.3	17
408.9-426.7	20	540	436.9-457.2	18



e) Reinforcement details of treads and columns

Formulae

$$\bar{G} \text{ (outer going)} = 2(r_i - 270^\circ) \sin \theta / 2$$

Clear headroom = $r_i - r_c$, where r_c is the radius of the column or post at the centre.

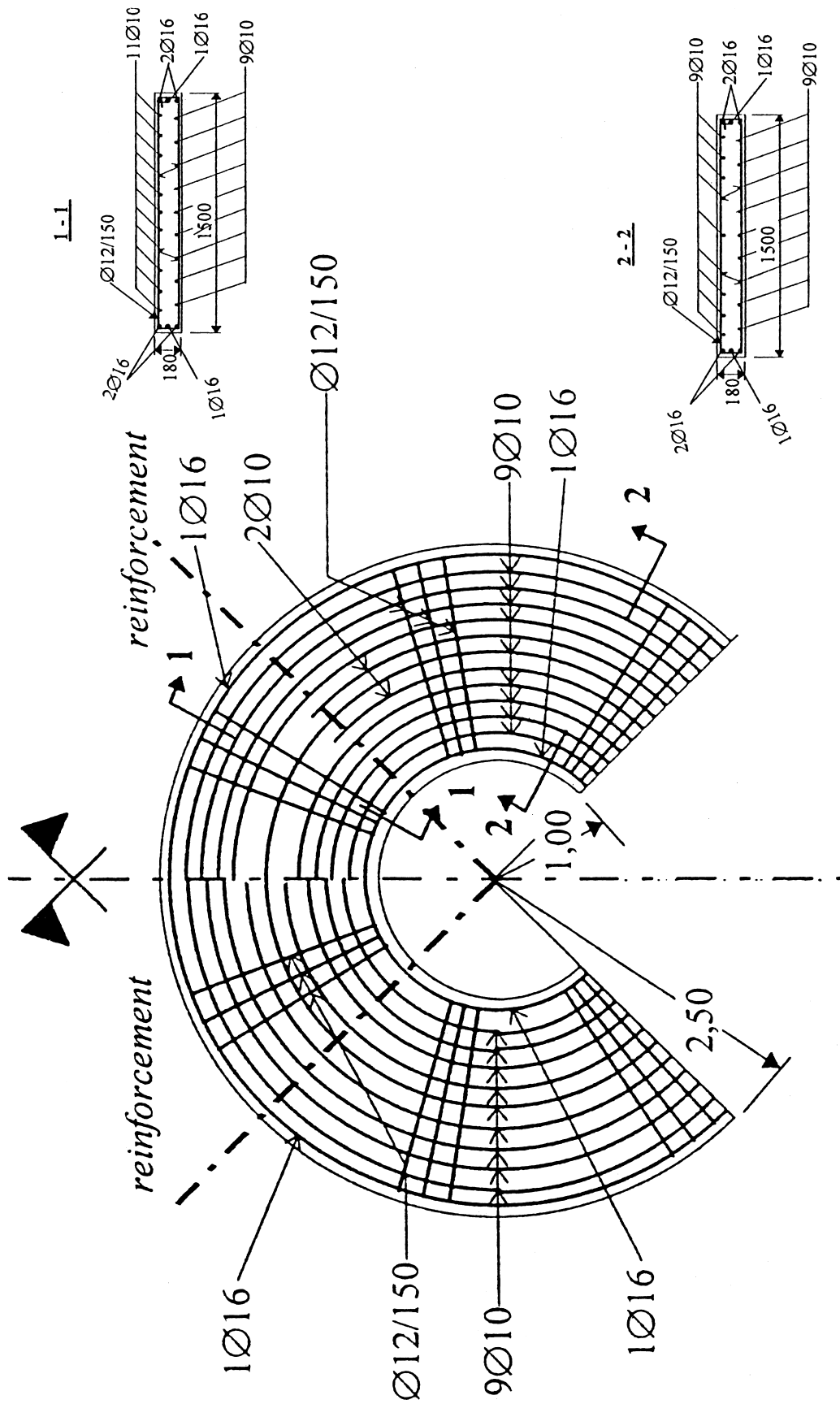
$$\bar{G} \text{ (inner going)} = 2(r_c - 270^\circ) \sin \theta / 2$$

Clear headroom: $2H_i = h_1(\phi/\theta) - t_L$, where ϕ = the angle of rotation at a distance along radius, θ = the angle of taper of tread, h_1 = the rise, t_L = the thickness of landing.

Minimum splayed straight length $L = B' + 2/3 \times \text{bearing}$.

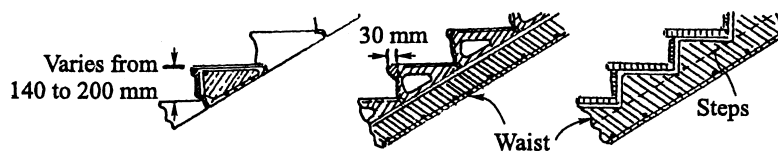
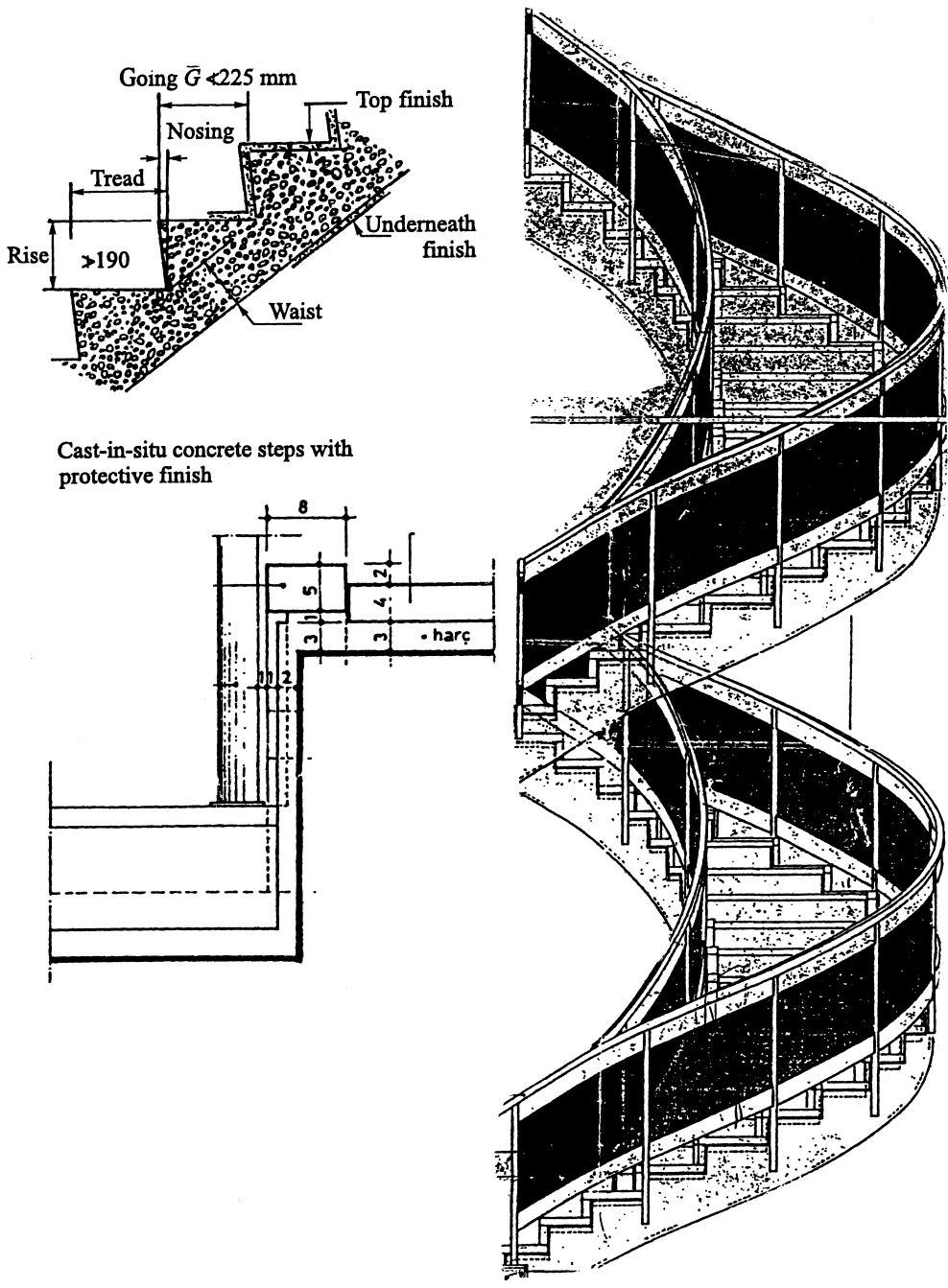
**HELICAL STAIRS—PLAN SECTIONS (BASED ON EC2)
RC DETAILING**

SHEET NO. III.12



HELICAL STAIRS (BASED ON EC2)

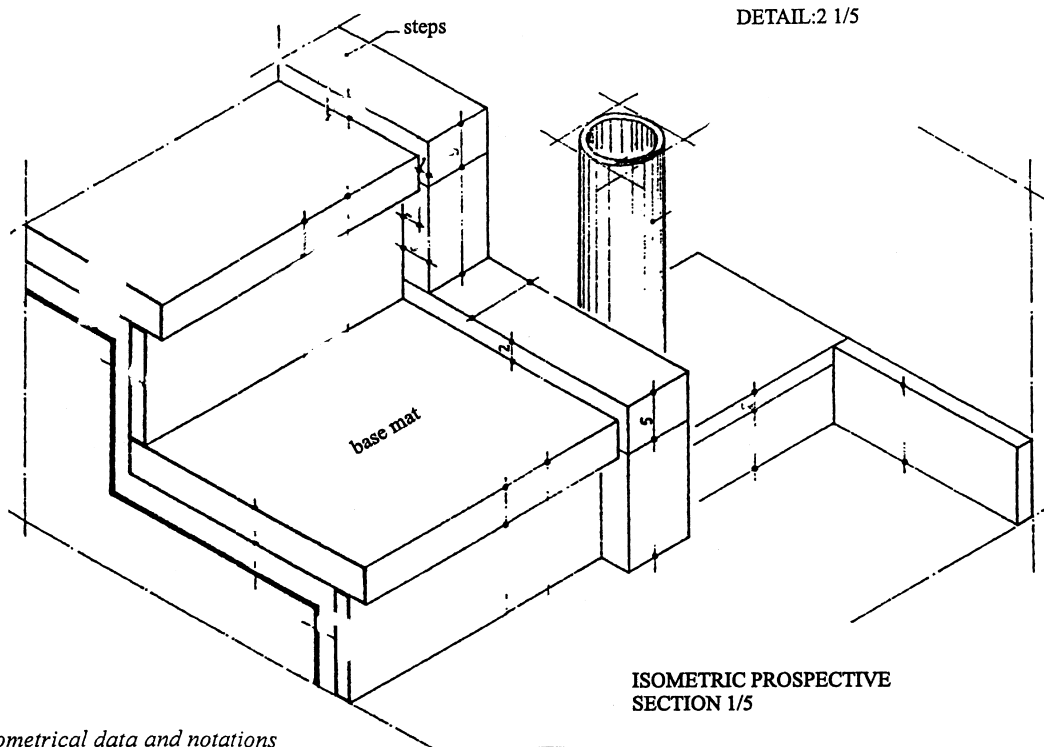
SHEET NO. III.13



Precast concrete steps

**BASE MATS AND STEPS (HELICAL STAIRS)
(BASED ON EC2)**

SHEET NO. III.13 (contd)



Geometrical data and notations

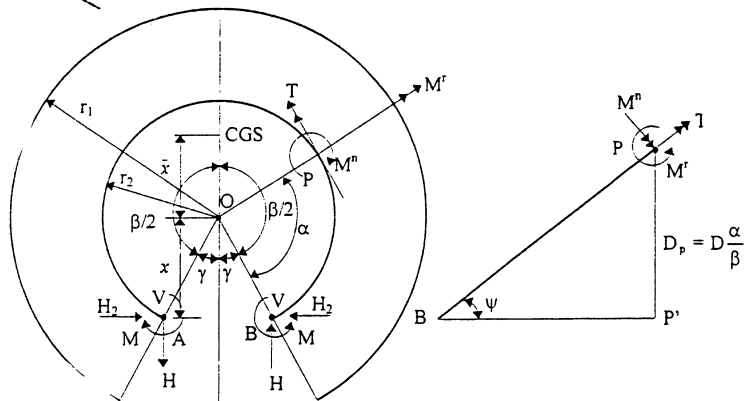
- r_i - internal radius of the slab;
- r_e - external radius of the slab
- b_w - width of the slab;
- D - the level's high;
- β - the unfolding slab's angle;
- γ - the supports angle;
- h - the slab's thickness $h=200$ mm
- h_i - the step's height; $h_i=150$ mm
- b_i - the step's width. $b_i=300$ mm

Geometrical Elements

The load's centre line:
 $r_1 = 1,85$ m

The slab's centre line:
 $r_2 = 1,75$ m

The slab's angle:
 $\text{tg } \psi = 0,5$
 $\psi = 26,6^\circ$



(Section)	α (°)	T_{Sd} (kNm)	M^r_{Sd} (kNm)	M^n_{Sd} (kNm)	V^r_{Sd} (kN)	V^n_{Sd} (kN)	N_{Sd} (kN)
0	0	-37,8	0,0	75,6	-48,3	-48,1	78,0
1	45	-19,6	-32,0	123,8	0,0	-16,0	84,3
2	90	-1,3	-9,3	93,8	48,3	-1,7	54,8
3	135	0,0	8,0	0,0	68,3	0,0	0,0

Loading Assessment

Dead load (weight of the slab and steps $h_{med}=220$ mm, $\gamma_b=25$ kN/m³):
 $G_k=5,50$ kN/m²

Live load:
 $Q_k=3,00$ kN/m²

Materials Resistance

concrete C20/25: $f_{ck}=20$

$\gamma_c=1,5$; $f_{cd} = \frac{20}{1,5} = 13,33$ MPa

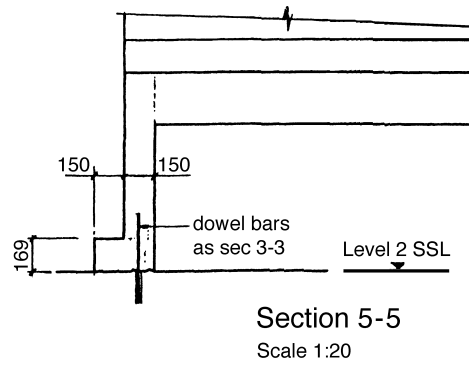
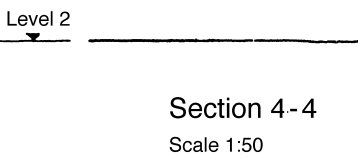
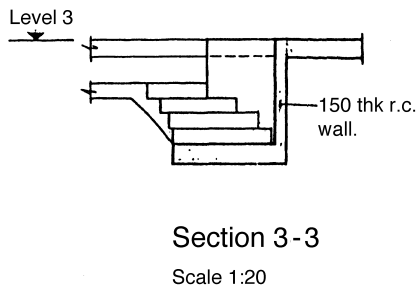
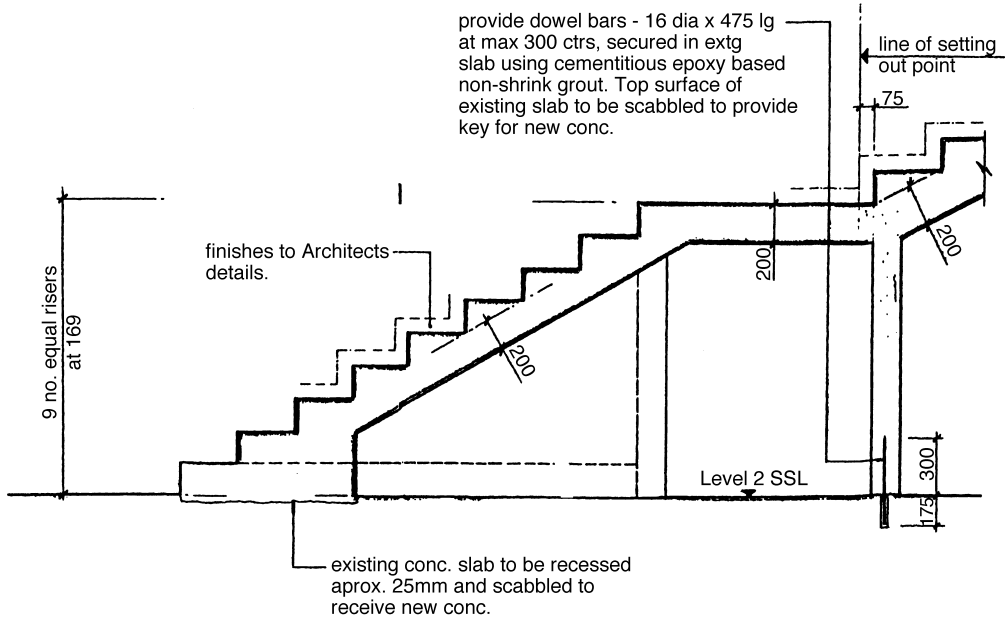
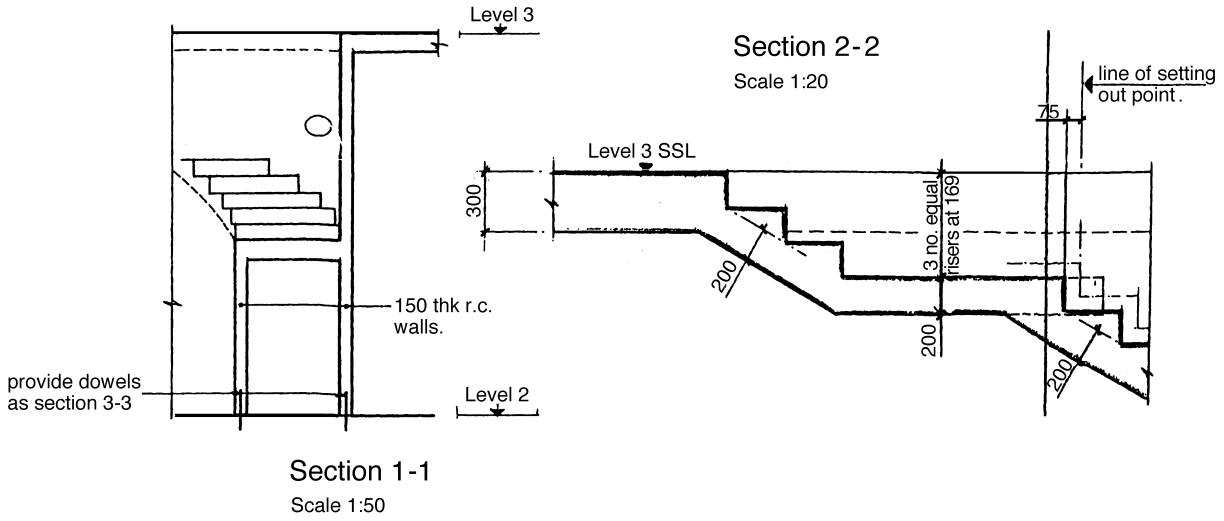
Steel longitudinal bars: on the axis of symmetry

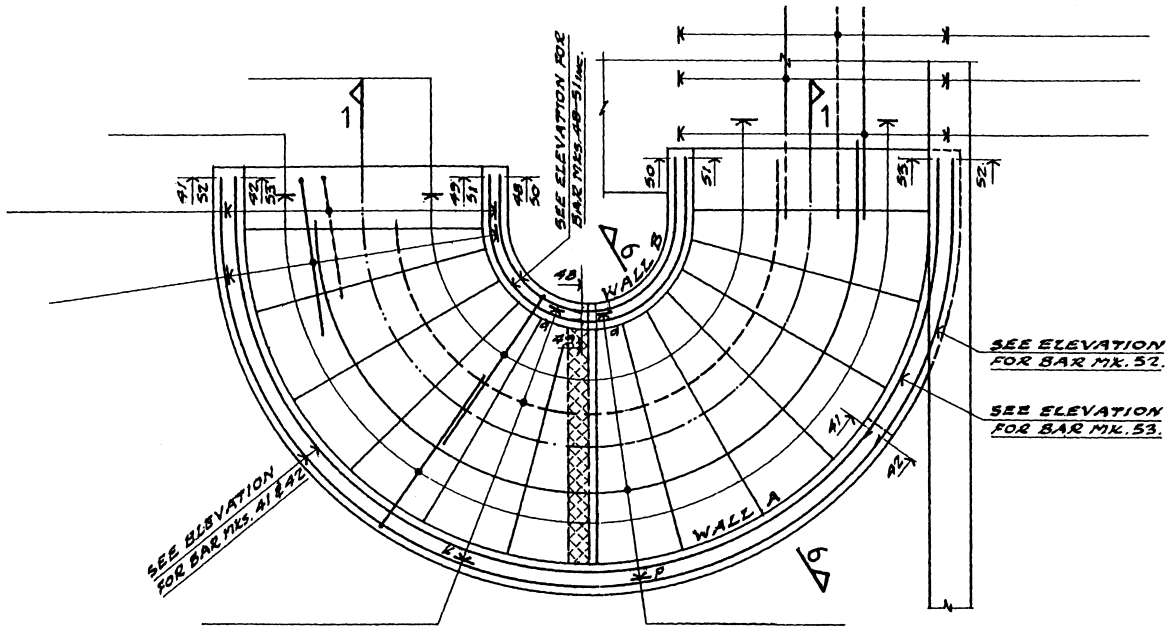
$f_{yk}=500$ MPa, $\gamma_s=1,15$; directed horizontal reaction:

$H=H_A=H_B=68,3$ kN

MIXED RC STAIRCASE (BRITISH CODES)

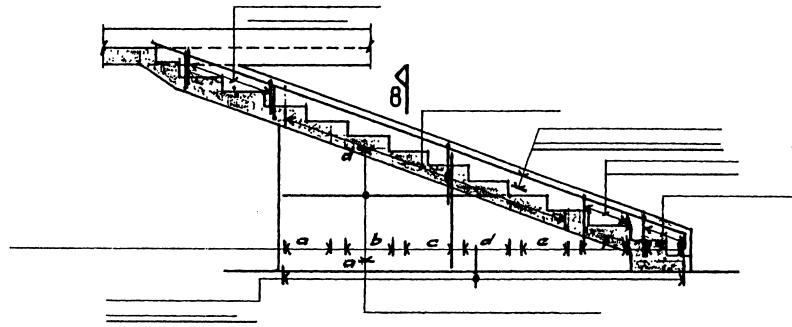
SHEET NO. III.14





NOTE
 BAR MIXES 33, 34, 50, 51
 52 & 53 TO BE CUT AND
 BENT ON SITE TO SUIT.

Plan on Flight A



Elevation of Wall A
 (Developed)
 (SCALE 1:50)

WARD + COLE
 consulting engineers

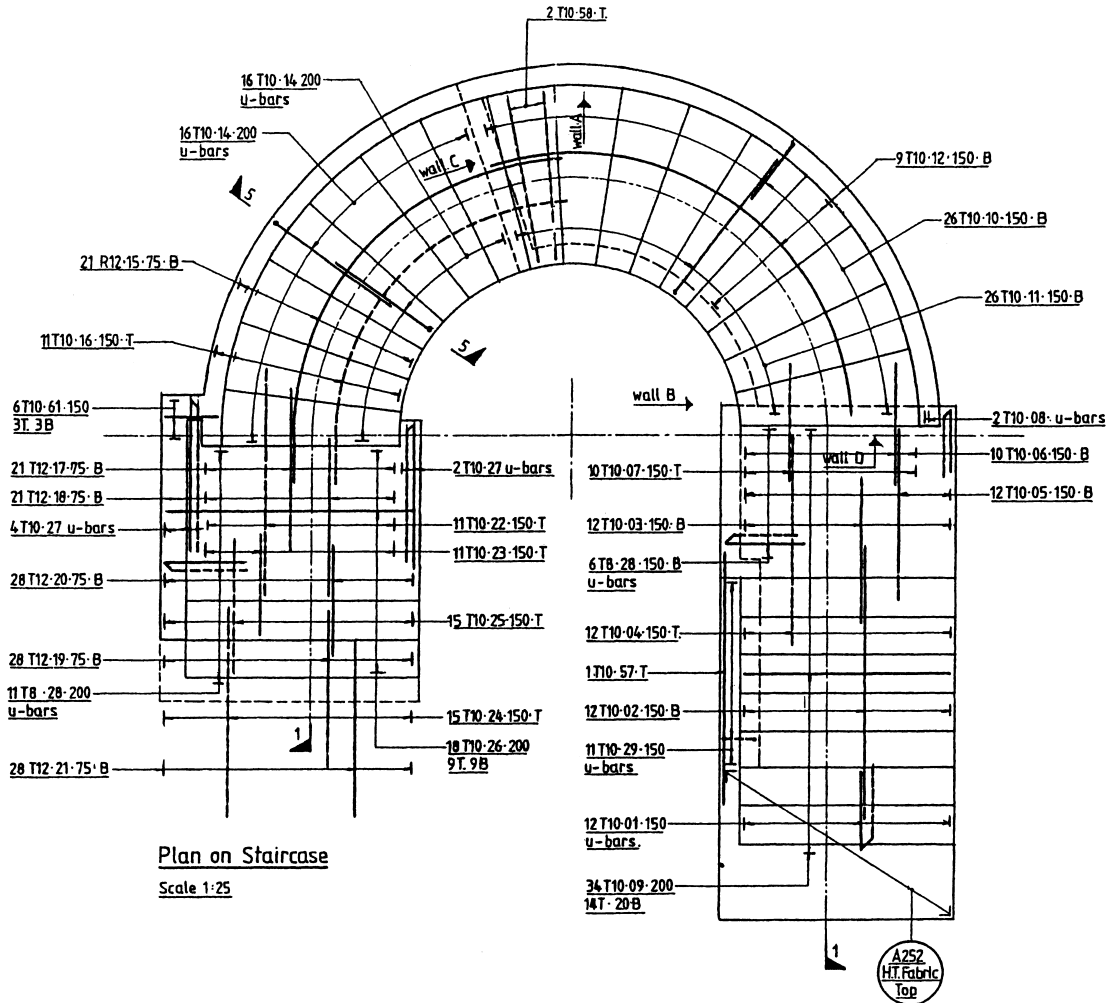
1, old lodge place,
 st. margarets, twickenham,
 middlesex ur1 1tg
 tel. 081-744-0900 fax. 081-744-0902

drawing title
Proposed Structural Work.
Tax Office Stair
R. C. Details.

drawing no. 30/127/400	rev. 0
scale 1:50, 1:20	date AUG. 93
drawn by D.P.	

Mixed staircase – Straight-cum circular/helical (Ward & Cole, London) (British practice).

SHEET NO. III.14 (contd)



Plan on Staircase
Scale 1:25

WARD & COLE
consulting engineers
1 old lodge place st margarets twickenham
middlesex tw1 1rq
tel 0181 744 0900 fax 0181 744 0902

drawing title
R.C. Details of Staircase & Walls
Central Staircore - Level 2 to 3

scale: date: drawn by checked by

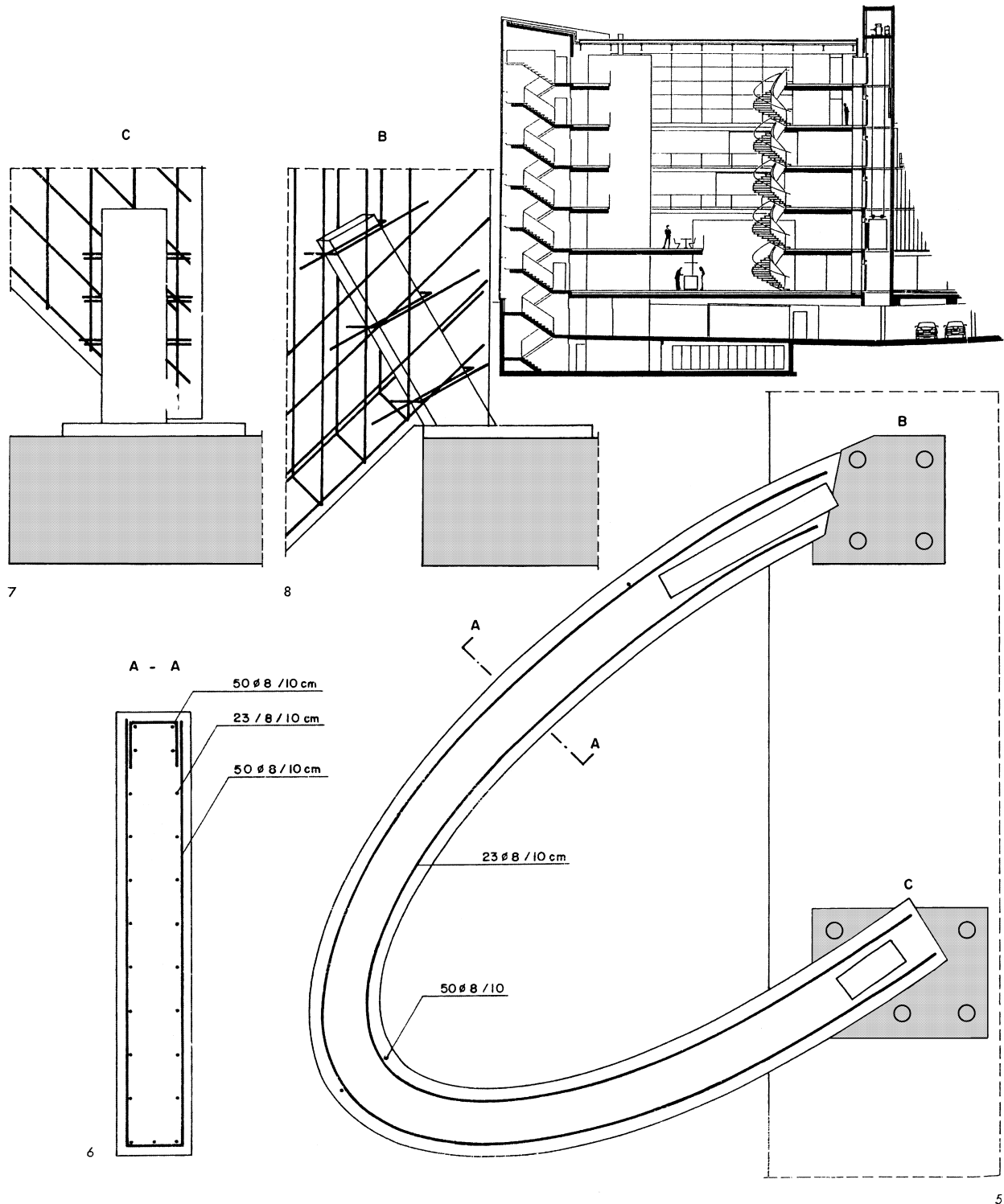
NOTES:

1. For general notes refer to Ward + Cole drawing no 30/127/313.
2. For general arrangement refer to Ward + Cole drawing no: 30/127/355.
3. Concrete to be grade C35 with 20mm max. size of aggregate.
All concrete to be in accordance with BS.8110
4. High yield steel, prefixed "T" on drawing and delivery tags, to be in accordance with BS.4449 OR BS. 4461.
Mild steel, prefixed "R" on drawing and delivery tags, to be in accordance with BS 4449
High yield fabric to be in accordance with BS 4483
5. Minimum laps to reinforcement:
T8 - 325
T10 - 350
T12 - 425
6. Concrete cover to reinforcement to be:
Staircase - 35mm U.N.O.
Walls - 25mm U.N.O.
7. The Engineer is to inspect the fixed reinforcement prior to pouring concrete. Contractor to give Ward + Cole min. 48 hours prior notice.

Mixed staircase – Straight-cum circular/helical (Ward & Cole, London) (British practice).

HELICAL STAIRCASE DETAILS (BASED ON EC2)

SHEET NO. III.16



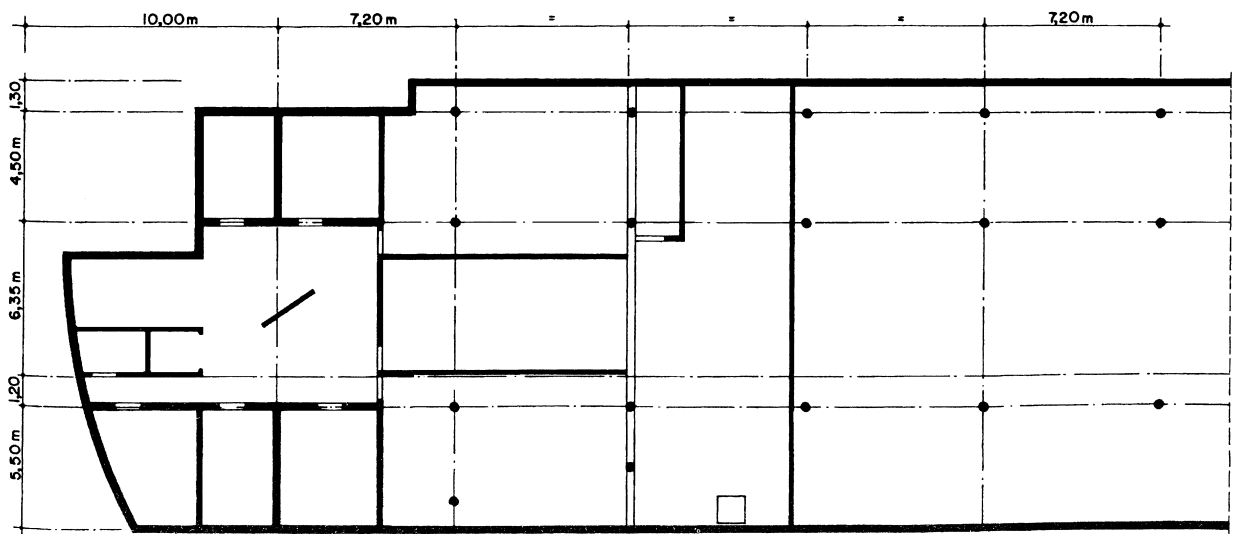
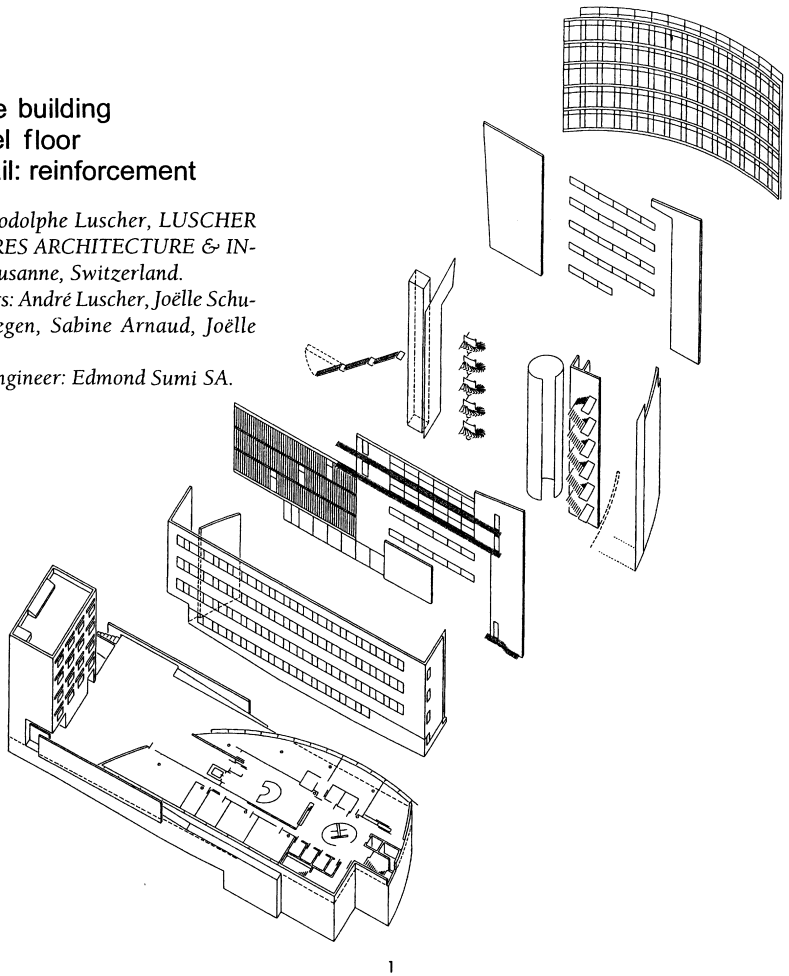
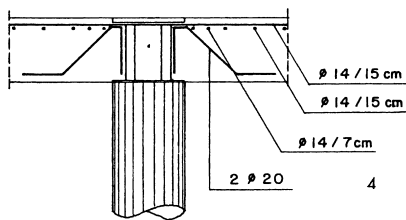
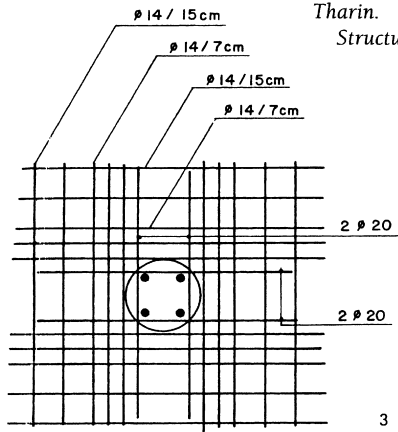
5. Horizontal section of the stair upright: reinforcement; 6. Vertical section: reinforcement; 7. Lower fitting of the upright; 8. Upper fitting of the upright.

**CANTON ADMINISTRATION BUILDING (MORGES)
SWITZERLAND (BASED ON EC2)**

SHEET NO. III.17

1. Axonometric view of the building
2. Plan of the second level floor
- 3-4. Foundation and detail: reinforcement

*Architect: Rodolphe Luscher, LUSCHER & PARTENAIRES ARCHITECTURE & INGENIERIE, Lausanne, Switzerland.
Collaborators: André Luscher, Joëlle Schumann, Jürg Degen, Sabine Arnaud, Joëlle Tharin.
Structural engineer: Edmond Sumi SA.*



III.1.3.2. Loads and load combinations

Loads and their combinations vary from one country to another. The partial safety factors associated with these loads vary as well and they largely depend on whether the stairs are analysed by the elastic, limit state, strength reduction and other concepts. In general, it is easy to compute dead loads and loads due to self weight and finishes. The disagreements are on the imposed loads (3 kN/m^2 to 5 kN/m^2) and the partial safety factors for loads and materials. Several examples in the text will indicate this dilemma. The general opinion is that steps should be loaded also with concentrated loads. The British practice is to check individual treads by placing on them two loads of 0.9 kN at 300 mm spacing and placed symmetrically about the centre line of the tread. For details, individual codes should be consulted.

III.1.3.3. Materials and stresses

For materials and their allowable stresses, individual codes are referred to.

III.1.3.4. Additional specifications for the reinforcement of concrete stairs

Reinforcement size

A standard range of bars and sizes is available for use in reinforced concrete. They may be hot-rolled (mild steel, high yield steel) or cold worked (high yield steel). Bars are made in a range of diameters from 8 to 40 mm . Special sizes of 6 and 50 mm are seldom available. The specification for steel covers chemical composition. Tensile strength, ductility, bond strength, weldability and cross-section area can be found in various codes.

Fabric

Fabric reinforcement is manufactured to BS 4483 and to ASTM 1992 requirements. There are four types of fabric made from hard drawn mild steel wire of $f_y = 485 \text{ N/mm}^2$ or from cold-worked high yield bars.

IV. Columns, frames and walls

IV.1. Columns

IV.1.1. Introduction

Columns are usually under compression and they are classified as short columns or long and slender columns. Long columns are liable to buckle under axial loads. The length of the column is the distance between supports at its end or between any two floors. The effective length of a column is governed by the condition of fixity at its ends in position and in direction. Codes give the values of the effective length for a number of cases by multiplying the actual length by a factor. Columns may also be loaded eccentrically or there may be a bending moment imposed on them in addition to concentric (axial) and eccentric loads.

Reinforcement in the form of longitudinal bars is provided both in short and long columns. These reinforcements are necessary to withstand both tension and compression loads. The most efficient location of the longitudinal reinforcement is near the faces of the columns. Such locations reduce the possibility of the reinforcement buckling with the resulting inability to take the load for which the column is supposed to be designed. Such buckling is prevented by lateral reinforcement in the form of ties or closely-spaced spirals. Again, codes give detail specifications for their design including sizes, spacings and the strength of concrete. The minimum number of longitudinal bars in a tied column should be four and the minimum diameter of bar is 12 mm. Typical details of columns and ties are given in Section I. They give locations of longitudinal reinforcement in specifically shaped columns and the method of providing ties to stop them buckling in any direction.

In any large job, it is necessary to give elevations to columns with their respective cross-sections and a column bar schedule must be provided to give column reference, column type and elevation and reinforcement details at various elevations. Sheet No. IV.1 gives, in brief, all these requirements for a specific construction. As column design is based on requirements already discussed above, sizes and reinforcement may change but the layout will be identical to the one given on Sheet No. IV.1.

The reinforcement bars, depending upon a specific construction, may require couplers, provided on the lines suggested in Section I. Care is taken to provide adequate laps where construction joint detailing is needed. Again a reference is made to specific codes and to Section I.

IV.1.2. Column detailing based on British codes

Most columns with straightforward profiles are prepared in tabular form, especially for the larger jobs. Columns can also be shown in full elevation. Normally the concrete profile dimensions are abstracted from the relevant general-arrangement drawings.

IV.1.2.1. Column schedules

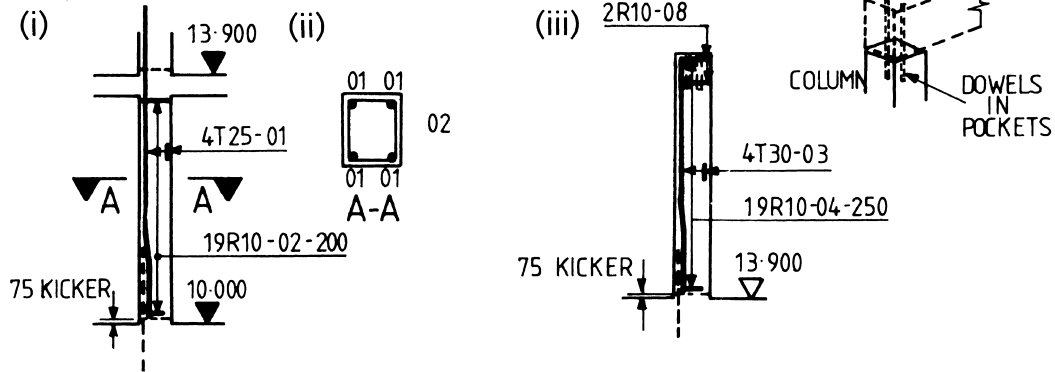
The elevation is prepared in economical tabular form, the concrete profile appearing only in the section. A typical column section is shown on Sheet No. IV.1.

Each column type is scheduled, indicating storey height, floor levels, kicker heights and depth of horizontal member. The vertical reinforcement and links are added to the schedule and bar mark location identified from a mid-storey

COLUMN DETAILS AND SCHEDULE

SHEET NO. IV.1

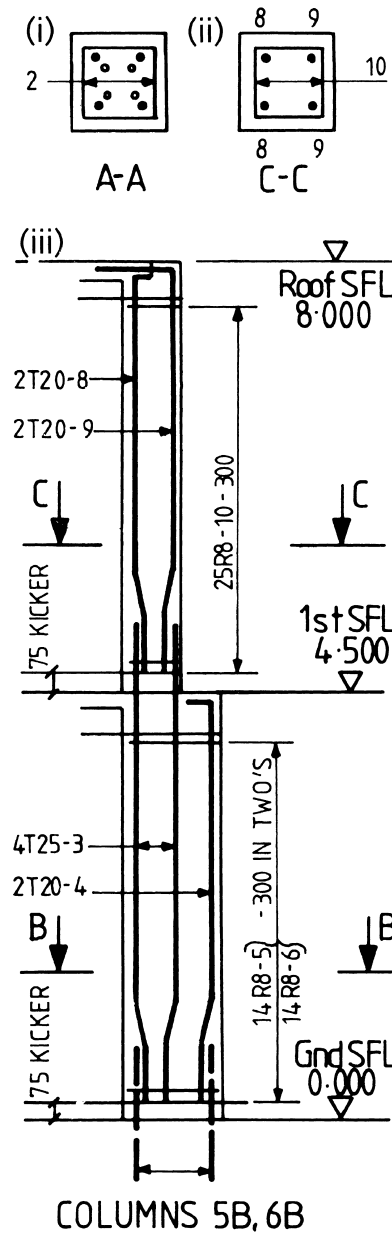
(a) Typical arrangement



(b) Column schedule

COLUMN SCHEDULE		
Column Reference	Type C-35 No	5B-6B-35No.
Roof SFL 8.500		
4.000	4T25-7 20R10-2-300 75 KICKER	A-A 2T20-8 2T20-9 25R8-10-300
	1st SFL 4.500	
4.500	4T30-1 20R10-2-300 75 KICKER	A-A 4T25-3 2T20-4 14R8-5 14R8-6
	Gnd SFL 0.000	

(c) Column elevations



COLUMNS 5B, 6B

height section. Column starter bars cast with and projecting from other members should be detailed with those members.

Often the most critical details on a job will be the column-beam junctions. Careful thought should be given at an early stage to the arrangement of bars. Preferably one detail solution should be consistently followed throughout the job. The simplest solution is to allow the column bars to run through the beam at constant cover. The transverse beam reinforcement is detailed to avoid these bars. The lower end of the vertical bar is joggled to accommodate the splice with the lower member.

Alternatively, the top end of the vertical bar is joggled to avoid beam bars and/or to accommodate the splice above. When there are large reductions in the size of the column above, the step between faces can become excessive for a bar to joggle. Usually the vertical bar is terminated within the beam and a separate starter bar provided. Where large bending moments occur at ends, it is sometimes necessary to provide separate bars fixed with the column to carry these moments *from* the framing beam.

IV.1.2.2. Bar detailing on columns

On elevation

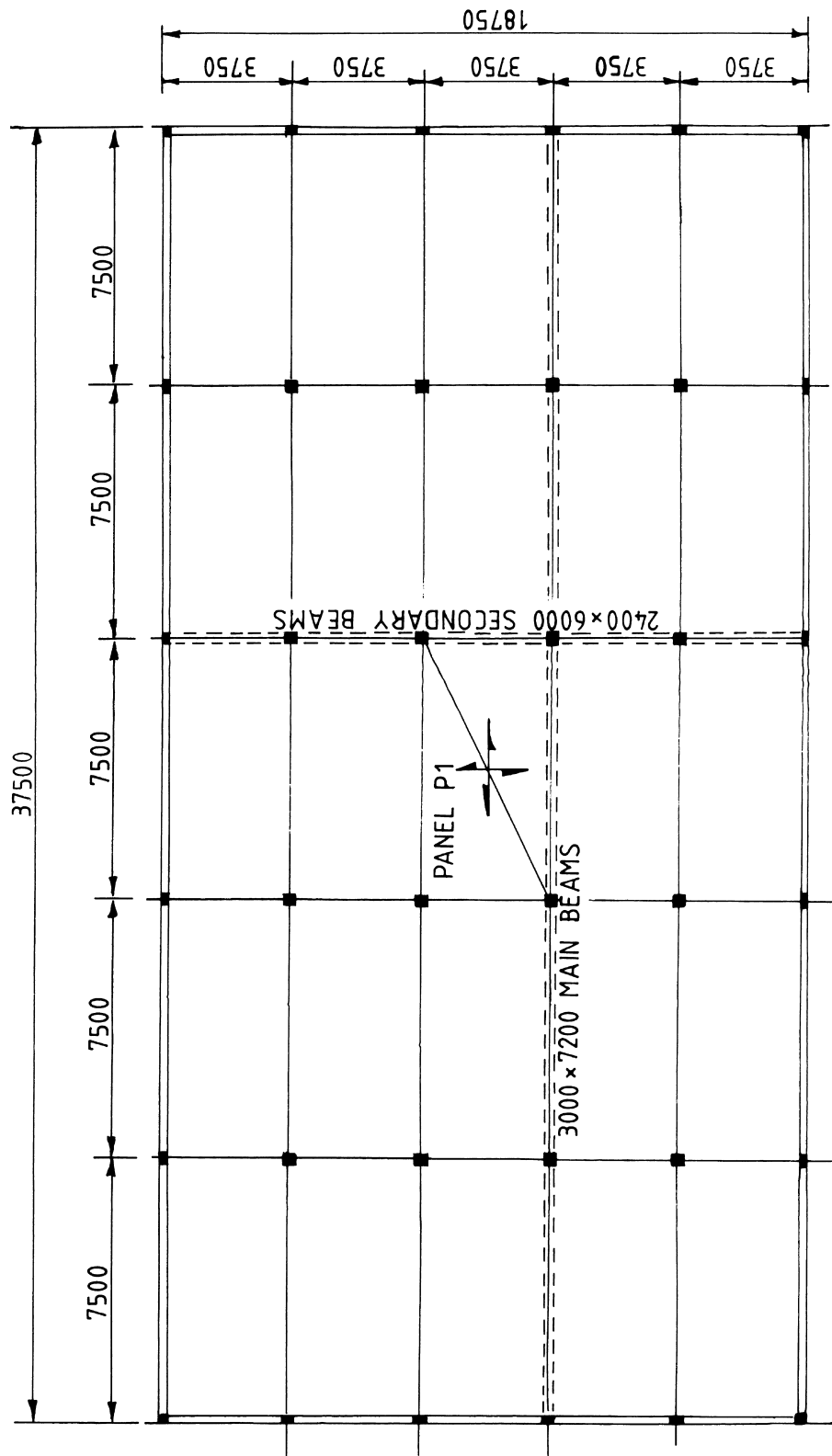
- (a) *Vertical bars.* Generally each bar mark is illustrated by a typical bar drawn as a thick line in elevation. Bars detailed elsewhere are shown as a thick dashed line.
- (b) *Links.* Generally the spread of links is indicated by an indicator line terminated by arrowheads. The links are provided to restrain the vertical bars from buckling. Generally the top link terminates at the soffit of the slab for peripheral columns, or at the soffit of the shallowest beam for internal columns.
- (c) *On section.* Generally sections are drawn at mid-storey height looking down. Sections are preferably drawn to a suitable scale to clarify the fixing of the links and to locate the vertical bars. Bars cut in section appear as black dots with the appropriate mark. Any starter bars beyond appear as open circles. Links are drawn with a thick line.
- (d) *Column heads.* Column head shear reinforcement is a special requirement specified by the designer. This can be incorporated with the column reinforcement or referenced and detailed as a separate item or with the slab.

In most constructions, carrying out integrated structural detailing in concrete is unavoidable. Generally in framed constructions, slabs, beams and columns are involved. Sheet No. IV.2 gives a general plan for a typical floor showing main and secondary beams, slabs and columns. Sheet Nos IV.3 to IV.6 give examples of detailing. Various reinforced concrete components needed for the floor are shown on Sheet No. IV.2. All structural details are self-evident and indicate how an integrated structure can be designed and detailed.

RC DETAILS OF SLAB BEAMS AND COLUMNS

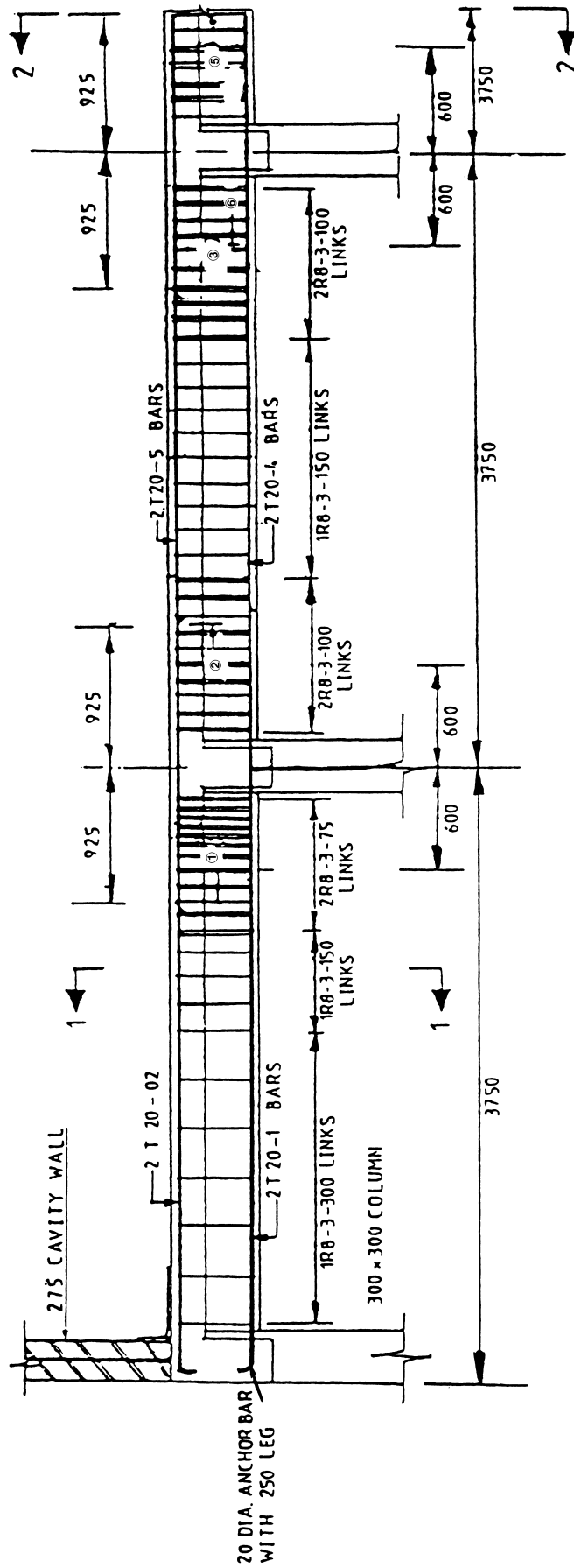
SHEET NO. IV.2

RC DETAILS OF SLAB BEAMS & COLUMNS



PLAN OF TYPICAL UPPER FLOOR

RC DETAILS OF BEAMS, COLUMNS & SLAB

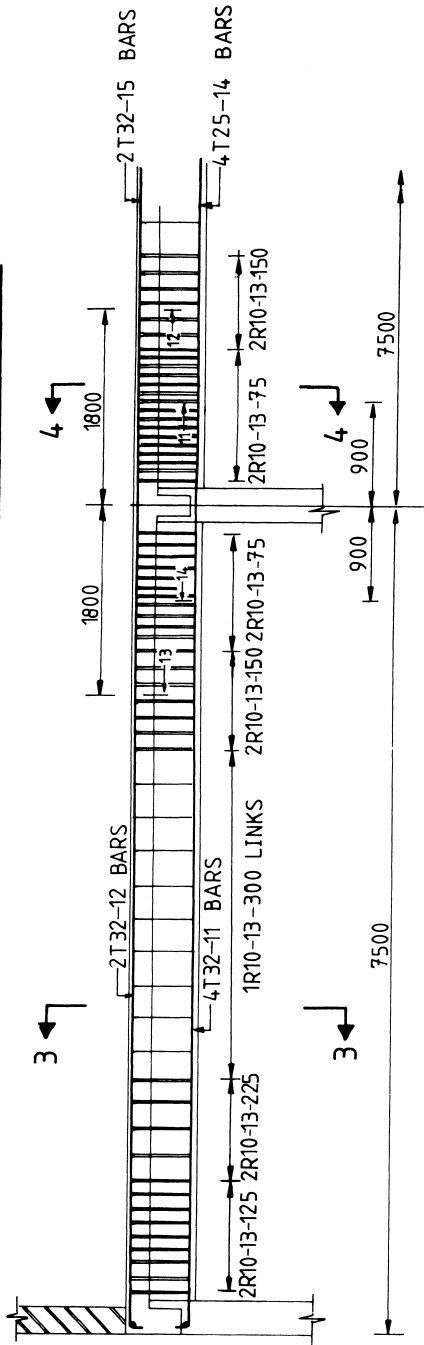


SECONDARY BEAMS

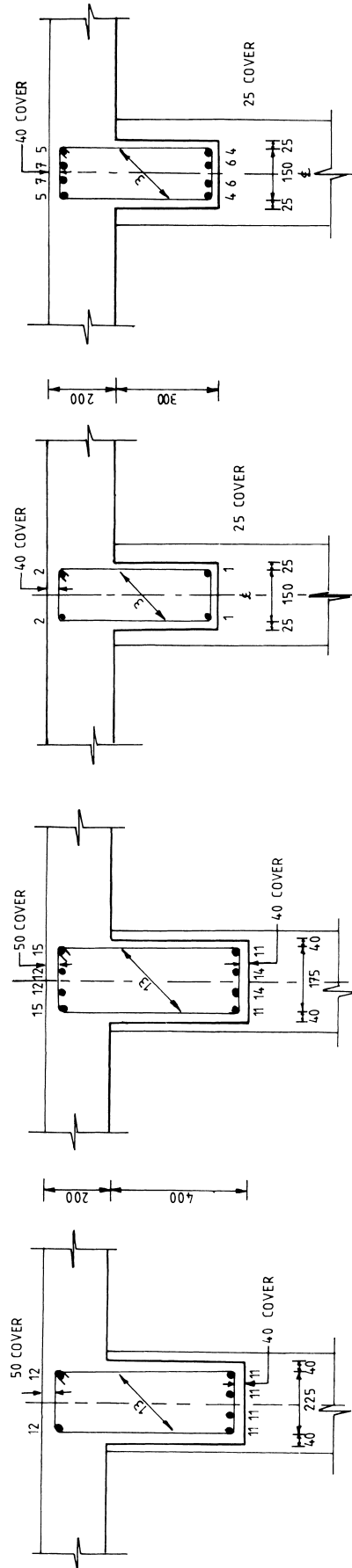
RC DETAILS OF SLAB BEAMS AND COLUMNS

SHEET NO. IV.4

RC DETAILS OF SLAB BEAMS & COLUMNS



MAIN BEAMS

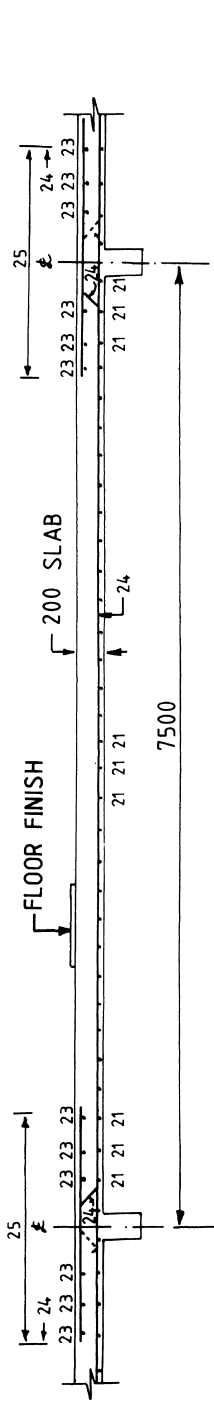


SECTION 3-3

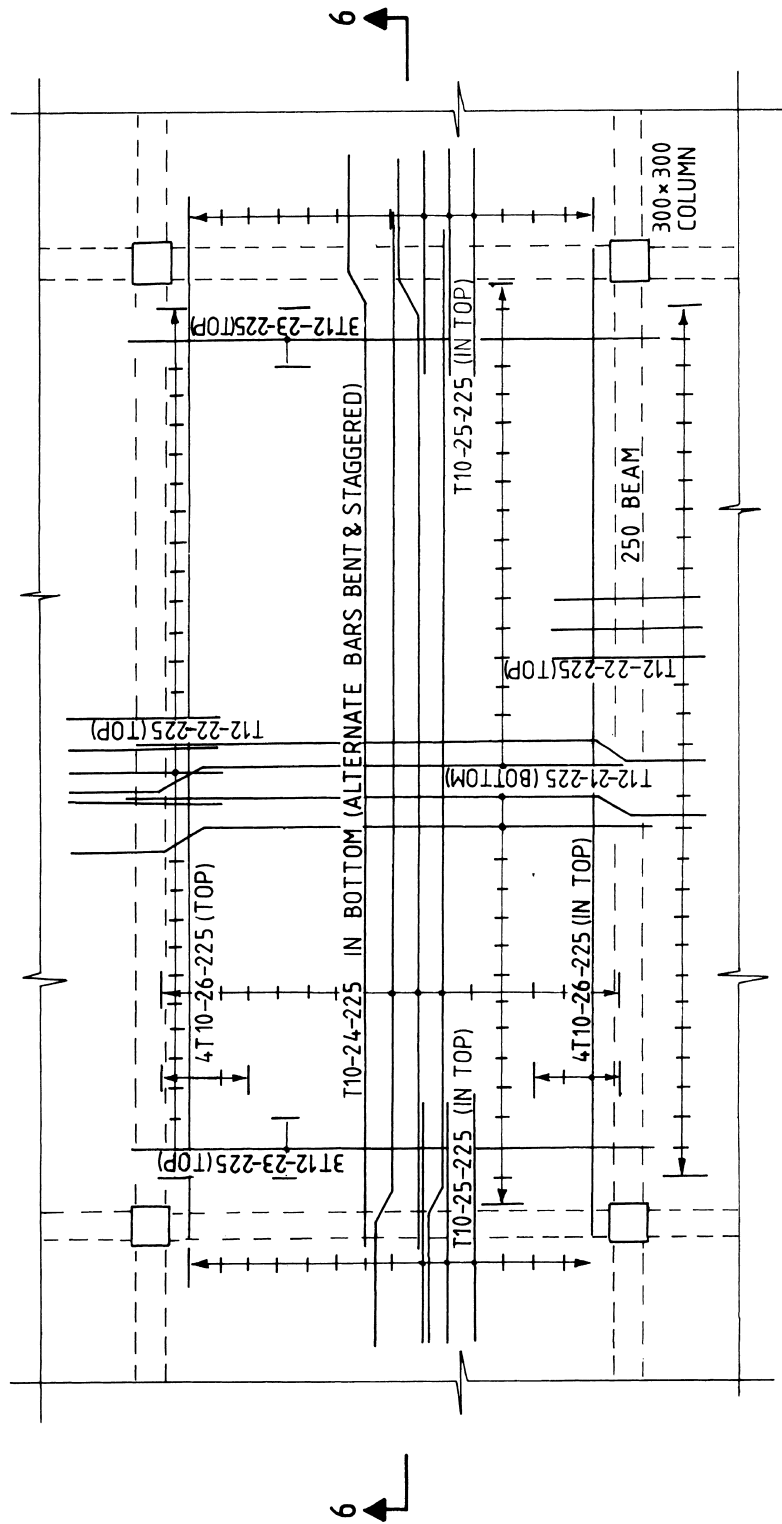
SECTION 4-4

SECTION 1-1

SECTION 2-2

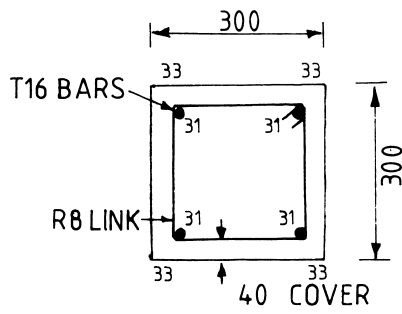
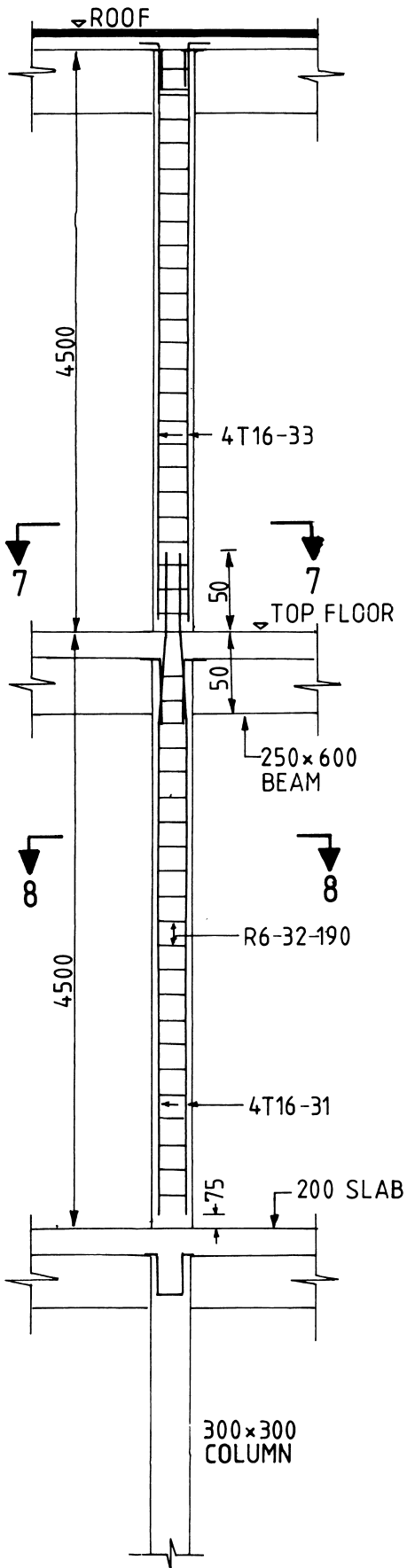


SECTION 6-6

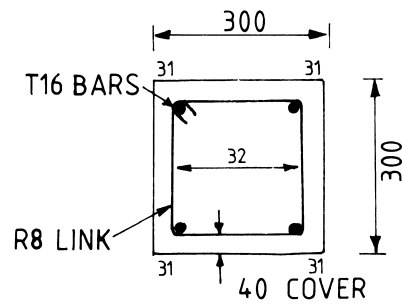


RC DETAIL OF INTERIOR COLUMN

SHEET NO. IV.6



SECTION 7-7



SECTION 8-8

IV.1.3. Wall detailing based on British codes

IV.1.3.1. Introduction—reinforced concrete walls

Reinforced concrete walls are considered to contain at least the minimum area of reinforcement expressed as a percentage of the gross concrete cross-sectional area. The wall shall have length exceeding four times the wall thickness.

Vertical reinforcement

Minimum A_{sc} not less than 0.4% (0.2% each face).

Maximum A_{sc} not to exceed 4%.

Maximum bar spacing, when A_{sc} exceeds 2%, should not exceed 16 times the vertical bar size.

Horizontal reinforcement

This reinforcement should be evenly spaced in the outer layers to minimize crack widths and contain the vertical compression bars. Where vertical bars are in tension, particularly in retaining walls, these are sometimes placed in the outer layer to facilitate fixing and to maximise the lever arm (see Table IV.1).

Table IV.1.

		Grade 250	Grade 460
Minimum horizontal reinforcement	1 face, or	0.3%	0.25%
	$\frac{1}{2}$ each face	0.15%	0.125%

The minimum size of bar should not be less than one-quarter the size of the vertical bar and preferably not less than 8 mm diameter. In plain walls with no reinforcements needed for design, it is essential to counteract possible flexural, thermal and hydration shrinkage cracks, particularly in external walls and at the junction of internal members, minimum reinforcement is required. This should be provided as a mat of small bars at relatively close spacings, with reinforcement areas expressed as a percentage of the gross concrete cross-sectional area. The horizontal bars should be placed in the outer layer (see Table IV.2).

Table IV.2.

		Grade 250	Grade 460
Minimum reinforcement in both horizontal and vertical direction	1 face, or	0.3%	0.25%
	$\frac{1}{2}$ each face	0.15%	0.125%

Plain and reinforced walls in tension

Bars should be arranged in two layers and the maximum spacing of tension bars should generally not exceed 150 mm where $f_y = 460 \text{ N/mm}^2$ or 300 mm where $f_y = 250 \text{ N/mm}^2$.

IV.1.3.2. Bar detailing on walls

On elevation

Notation for layers of reinforcement

Reinforcement is fixed in two layers at right-angles to form a mat, normally one mat at each wall face:

- (a) abbreviation for near face, outer layer N1
- (b) abbreviation for near face, second layer N2
- (c) abbreviation for far face, outer layer F1
- (d) abbreviation for far face, second layer F2.

Typical bar and indicator line

The convention for illustrating and 'calling-up' bars on walls follows closely that of slab. Note that identical bars appearing on different faces are itemized separately. To avoid congestion in thin walls less than 150 mm thick, a single mat of reinforcement may be provided, if design requirements permit.

*On section**Intermediate storeys*

Walls are normally cast in storey-height lifts, with standard 75 mm high kickers at each floor level. Kickers help to align the formwork above. The vertical reinforcement should not be less than 12 mm and is lapped above the kicker to provide structural continuity.

Top storeys

A variety of details are possible depending on design and construction requirements. Allow sufficient top cover for clearance of intersecting reinforcement.

Offset walls above

Normally offsets of up to 75 mm can be achieved by joggling the relevant vertical bars. Otherwise, the lower bar is terminated below the floor and a separate splice-bar starter provided.

External wall/slab junction

In the case where a significant bending movement is transmitted into the wall from the slab, it may be necessary to use an L-shaped bar to provide full anchorage. If the L-bar is to be cast with the wall it should be scheduled with the wall reinforcement, but this is a non-preferred detail.

Half-landings

A 20 mm deep rebate is preformed in the supporting wall to provide bearing for the slab poured later. The junction is provided with up to 12 mm. usually mild steel, U-bars.

Corner details

- (a) *Closing corners.* For corners that are closing, two simple alternatives are shown. Bars should be provided with adequate anchorage and appropriate laps.
- (b) *Opening corners.* For corners with the opening, the method of detailing is far more important, especially if the bending moment is significant.

Wall reinforcement spacers

Where links are not required to restrain vertical compression bars, reinforcement spacers can be used to stabilize the two faces during construction. Adopt say T.10 @ 1000, shape code 38. Cover to the outer bars is normally achieved by using plastic spacers.

Trimming of holes in walls

To prevent cracks springing from corners, provide nominal bars placed diagonally as shown. Additional trim requirements should be indicated in the calculations.

Fabric reinforcement in walls

Conventional wall layouts lend themselves to the advantages of fabrication as described. Nevertheless purpose-made wall sheets can be ordered to deal effectively with lapping and intersection details.

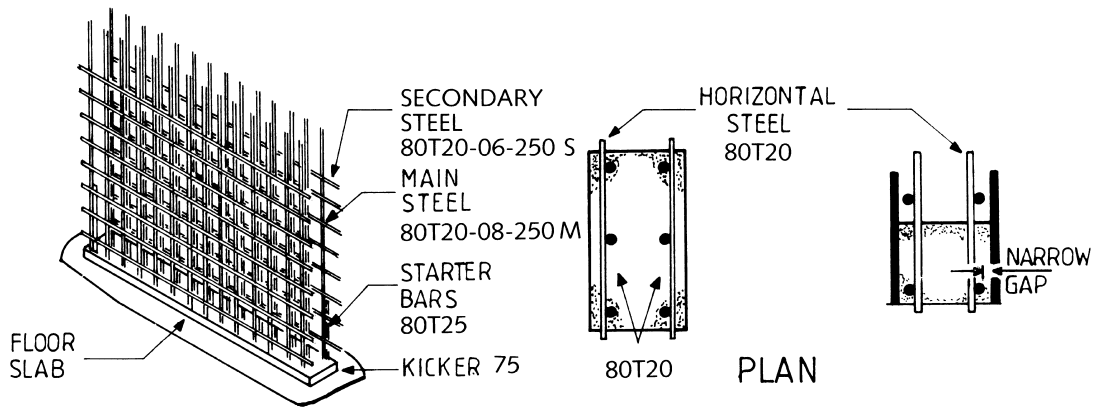
Reinforced concrete walls are generally subjected to axial loads. A typical isometric view of the reinforcement mesh is given on Sheet No. IV.7(a). A schedule is prepared for long, short and starter bars and their cut-off lengths together with opening and closing corners. The sizes of these bars depend on a specific construction and loads carried by a wall. Sheet No. IV.8 gives reinforcement details for an integrated structure, beams, slabs and walls (including shear walls and laterally loaded walls). Again codes are consulted for wall specifications and minimum bar diameters and wall thicknesses.

It is vital to introduce at this stage some sample design structural details for the lift shaft walls. On buildings where a lift shaft exists, it is important to provide guidance to the structural detailing of the walls, details to entrance core and major specifications go with them. A brief of specifications for such walls can be seen on Sheet No. IV.9. Generally, C.50 grade concrete is recommended since many lift fixtures create enormous stress concentrations in various pockets and ordinary concrete in those areas might be under severe strain and hence premature cracking might be induced due to lift (elevator) travelling up and down. Lift pit bases are clearly difficult to design, references are made to various relevant codes and analytical/numerical tools. The covers and minimum laps are clearly established. The entrance core details are shown on Sheet No. IV.10 with key plan and section shown on Sheet No. IV.11.

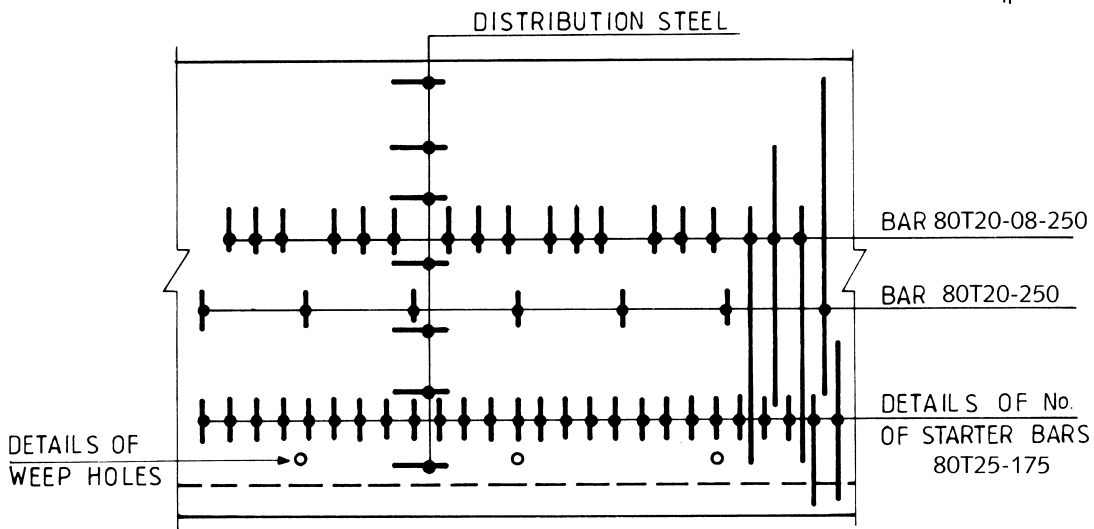
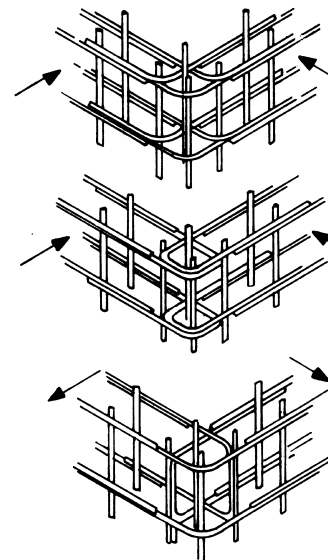
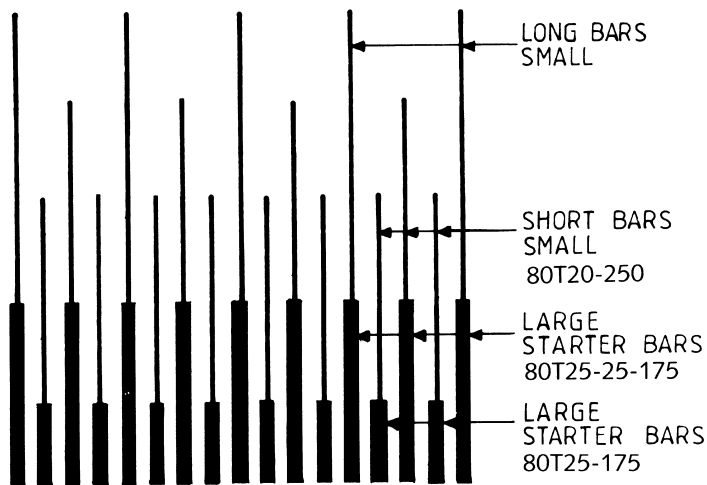
STRUCTURAL WALLS

SHEET NO. IV.7

(a) Wall reinforcement isometric view

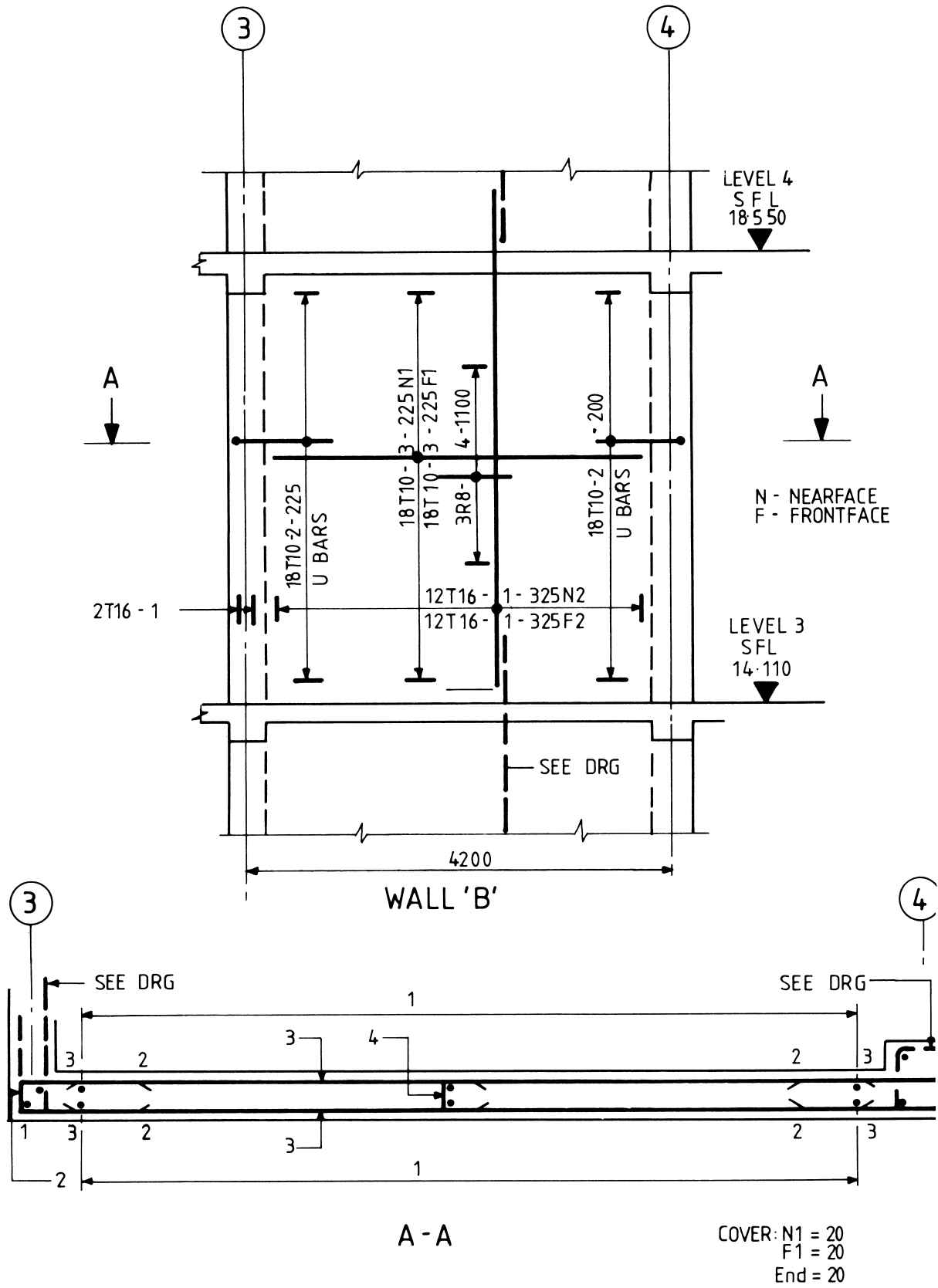


(b) Opening and closing corners



STRUCTURAL WALL

SHEET NO. IV.8



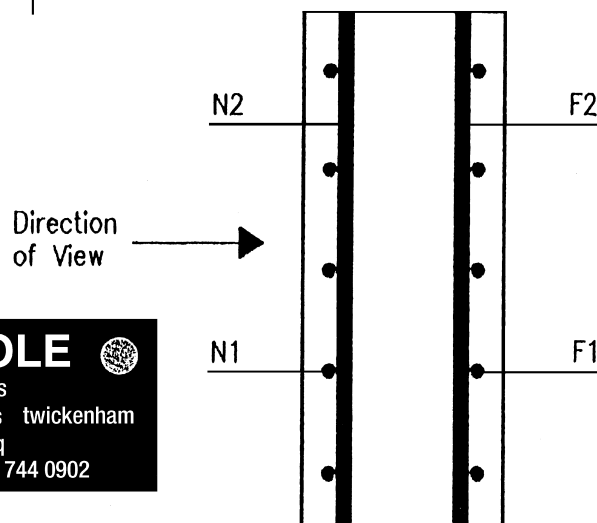
ENTRANCE CORE DETAILS

SHEET NO. IV.9

Notes:

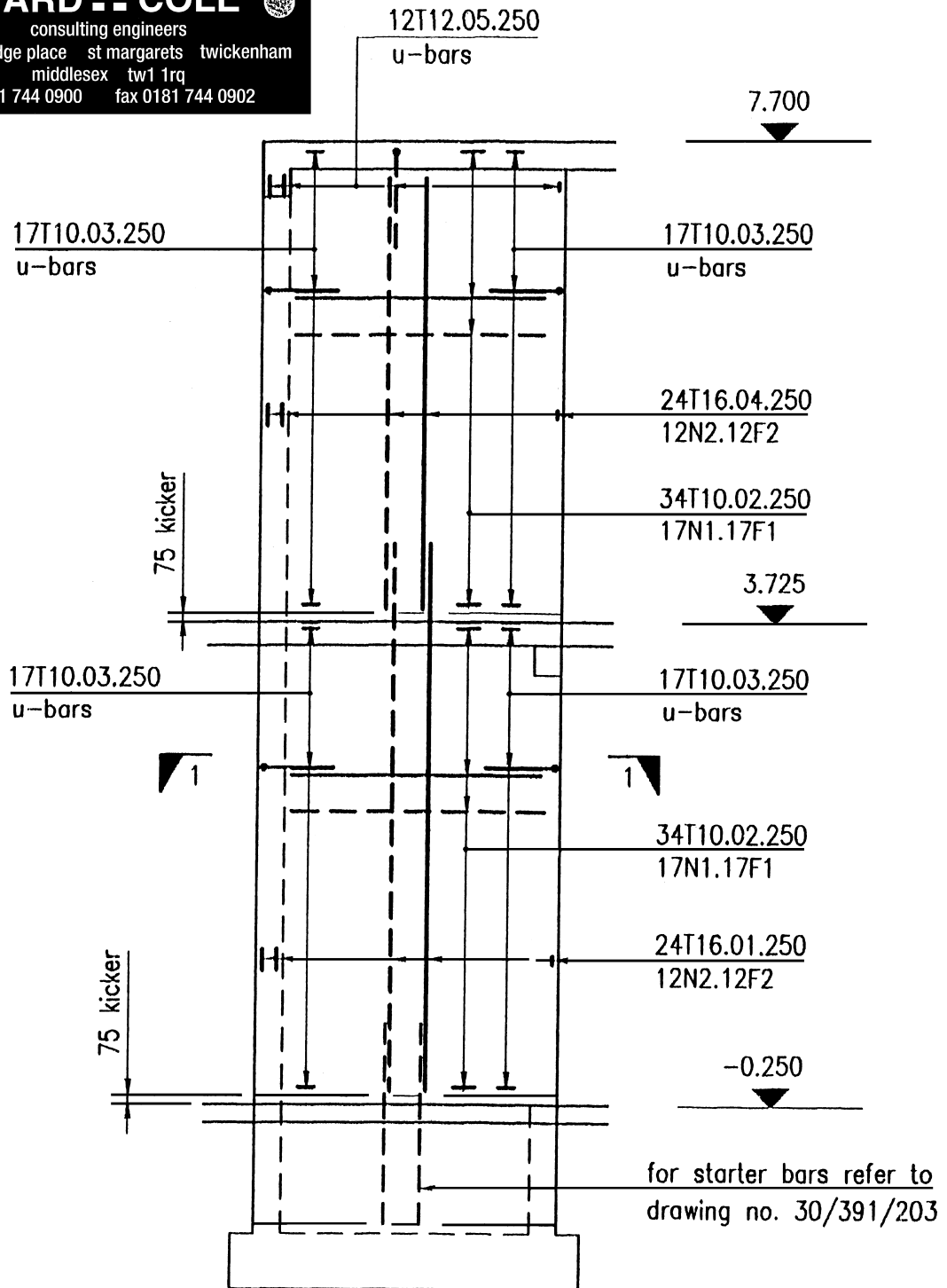
1. For general notes refer to Ward Cole drawing number 30/391/600
2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110. All 300x300 internal columns to be Grade C50 concrete up to soffit of first floor.
4. High yield steel prefixed 'T' on drawing and delivery tags, to be in accordance with BS 4449 or BS 4461. Mild steel prefixed 'R' on drawing and delivery tags to be in accordance with BS 4449.
5. Concrete cover to reinforcement to be as follows:
Lift Pit Base – 50 cover all round to all steel
Shaft Walls – 30 cover to all steel
6. Minimum laps to reinforcement to be as follows:
T16 – 575mm
T12 – 425mm
T10 – 350mm
7. This drawing must be read in conjunction with Specialist lift manufacturers details

Details of lift shaft walls

**WARD :: COLE**

consulting engineers
 1 old lodge place st margarets twickenham
 middlesex tw1 1rq
 tel 0181 744 0900 fax 0181 744 0902

WARD COLE
 consulting engineers
 1 old lodge place st margarets twickenham
 middlesex tw1 1rq
 tel 0181 744 0900 fax 0181 744 0902

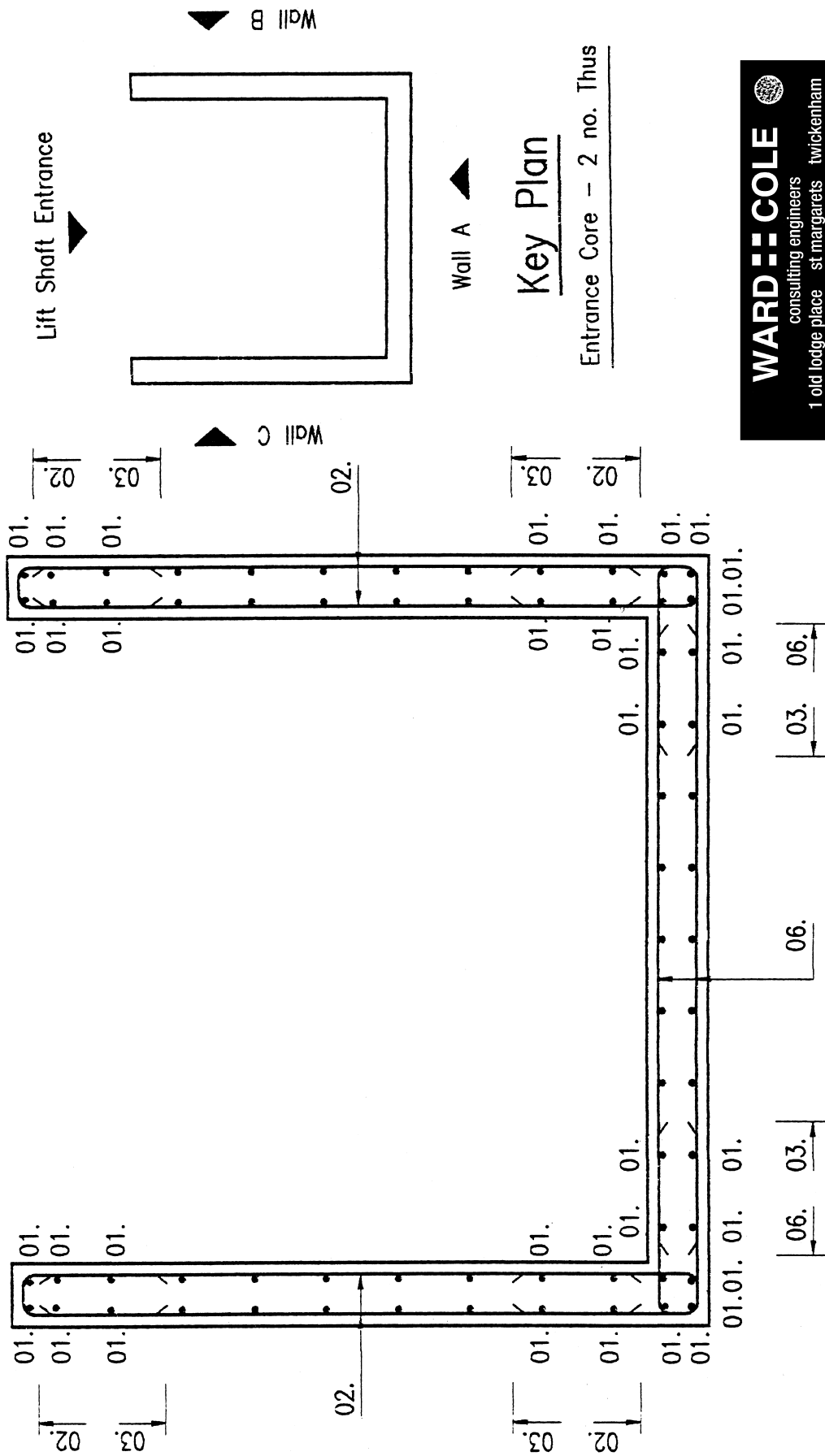


Elevation on Wall B – 2 No Thus

Wall C Similar But Handed – 2 No Off

ENTRANCE CORE SECTION

SHEET NO. IV.11



WARD :: COLE
 consulting engineers
 1 old lodge place st margarets twickenham
 middlesex tw1 1rq
 tel 0181 744 0900 fax 0181 744 0902

Section 1 - 1

IV.1.4. Portals and frames

IV.1.4.1. Introduction

A vast literature exists on portals and frames. They can be in bending with and without axial forces and with and without shear. Beam-column joints require detailed investigations; knee joints shall be investigated using properly devised experiments and numerical tools. Both static, dynamic and blast loadings where necessary shall be applied. Earthquake detailing shall conform to codes where applicable. Where frames are involved, global structural behaviour of bar frames must be investigated using experimental and numerical/analytical modelling. Where required analytical models for structural damage, a proper investigation, at least, in two dimensions would be necessary. The same procedures shall be adopted where structural detailing is needed for reinforced concrete infilled frames.

In this section some basic structural details are given for portals and frames. They should form the basis for structural detailing of more complex portals and frames under complex loadings.

Structural detailings

Sheet No. IV.12 gives reinforcement details for three different types of portals and frames. Again the thicknesses of frames and bar sizes depend on types of load. The general layout will still assume the same shape as indicated on Sheet No. IV.13. The frame corner reinforced with loops is shown on Sheet No. IV.13. It is practised in Germany and is widely recommended by the Eurocode on Concrete and by the American Concrete Institute. This is also recommended with small changes where necessary by the British codes of practices. Sheet No. IV.14 shows reinforced concrete details of frames with various sections at various levels using the following British codes of practice.

1. BS 8110 for Concrete.
2. BS 4449 or 4461 for high yield and mild steel and delivery tags.
3. BS 4483 for high yield fabric where required.

The minimum laps to reinforcement have been recommended in the specifications. Both columns and beams have been thoroughly detailed.

If welded mesh fabric is used for transverse reinforcement ϕ of wires ≤ 5 mm

Spacing. General *maximum* spacing S_{\max}

$$S_{\max} \not\geq 12 \times \text{minimum } \phi \text{ of main bars}$$

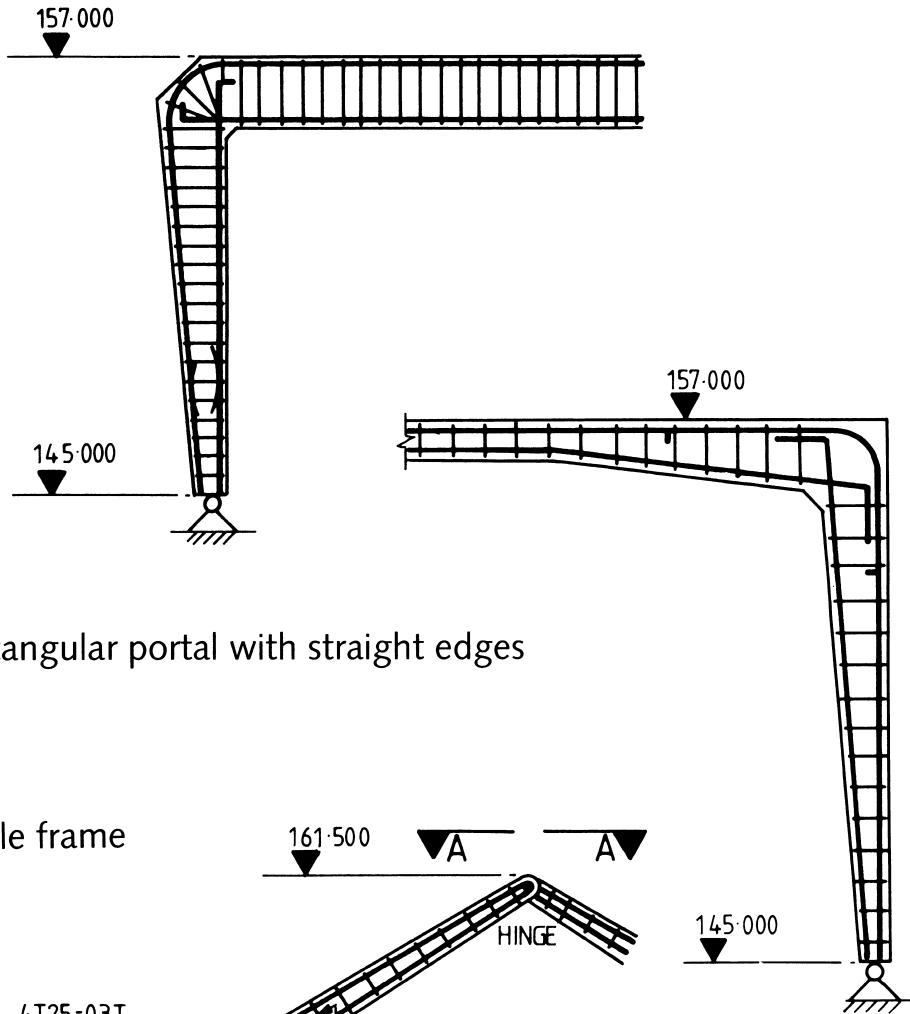
Or h

Or 300 mm

RC DETAILS OF FRAMES (BASED ON BRITISH CODES)

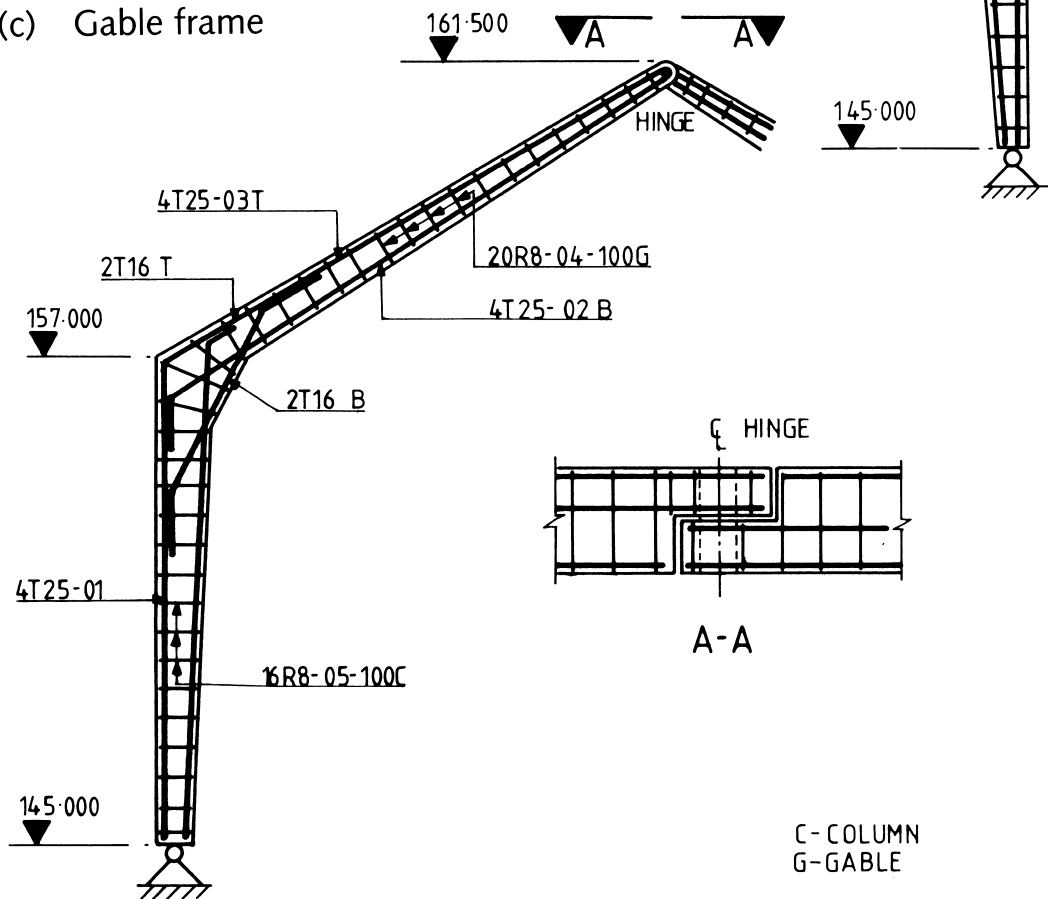
SHEET NO. IV.12

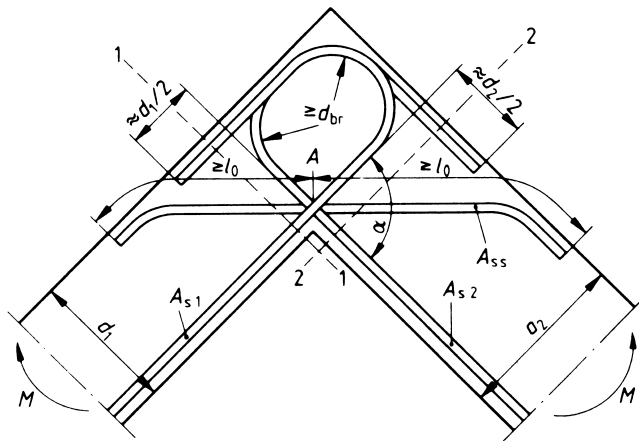
(a) Rectangular portal with chamfered edges



(b) Rectangular portal with straight edges

(c) Gable frame





(With Permission of DIN Deutsches Institut Für Normung E.V. Berlin, p. 63 DIN 1045)

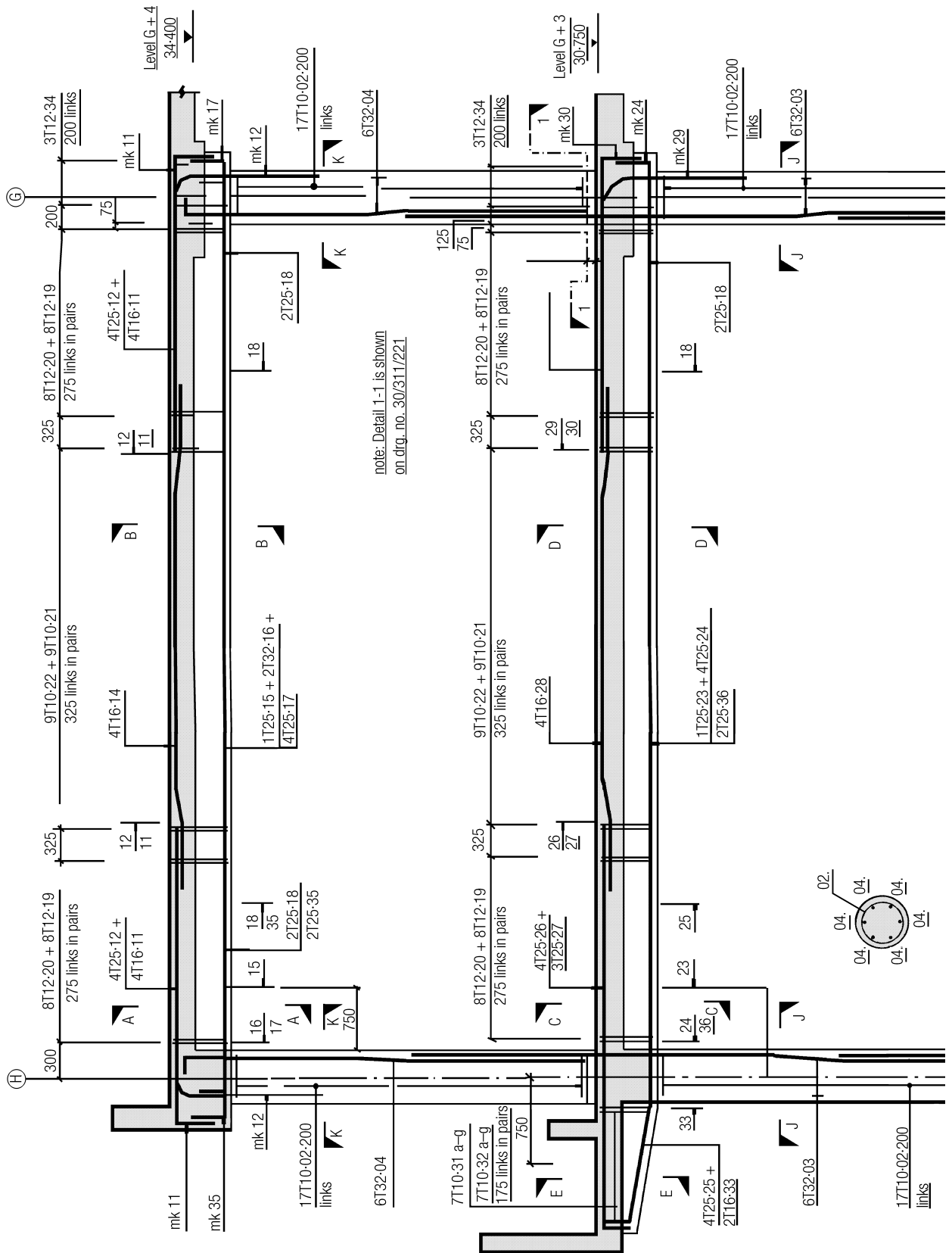
Bar diameter $d_{br} > 5 \text{ cm}$ and $> 3 d_s \leq 5 \text{ cm}$ and $\leq 3 d_s$

Hooks, bends, loops, links $15 d_s^{28}$ $20 d_s$ d_1 and $d_2 \geq 100 \text{ cm}$ $d_s = \varnothing$ bar diameter in cm

Sections 1-1 and 2-2 are required for the design.

Transverse reinforcement or links shall be the same as on Sheet Nos. IV.8 and 9.

SHEET NO. IV.14 (see over)



IV.2. Column, wall and frame detailing based on Eurocode 2

IV.2.1. Introduction

A reference is made to Section IV.1.2 for various columns detailing procedures which are also the British versions of the Eurocode 2 (EC2). Where differences exist on certain aspects of reinforced concrete detailing, they are shown on various sheets given under this section.

IV.2.2. Columns

IV.2.2.1. Minimum dimensions—longitudinal reinforcement

Minimum diameter

Minimum diameter is 12 mm.

Minimum area $A_{s,min}$

$$A_{s,min} = \frac{0.15N_{sd}}{f_{yd}} \leq 0.003A_c$$

where N_{sd} is the design axial force, f_{yd} is the yield strength of reinforcement and $A_c = bh$.

Maximum area $A_{s,max}$

$$A_{s,max} = 0.08 A_c$$

This maximum value also applies at laps. This is purely a practical consideration.

IV.2.2.2. Minimum number of bars—transverse reinforcement

All transverse reinforcement must be adequately anchored. Every longitudinal bar (or group of bars) placed in a corner should be held by transverse reinforcement. A maximum of five bars in or near each corner may be secured by any one set of transverse reinforcement. Although this is not stated in EC2, it will be advisable to limit the distance of the furthest bar from the corner to 150 mm.

Minimum diameter

$$\phi \leq \frac{\text{Maximum } \phi \text{ main bars} \leq 6 \text{ mm}}{4}$$

At lapped joints where maximum ϕ of main bars >14 mm, reduced spacing of $0.6S_{max}$ should continue for the length of the lap. Sheet No. IV.15(a)(i) shows spacing at changes in the direction of longitudinal bars.

Spacing of transverse reinforcement should be calculated taking account of the forces generated by the change of direction.

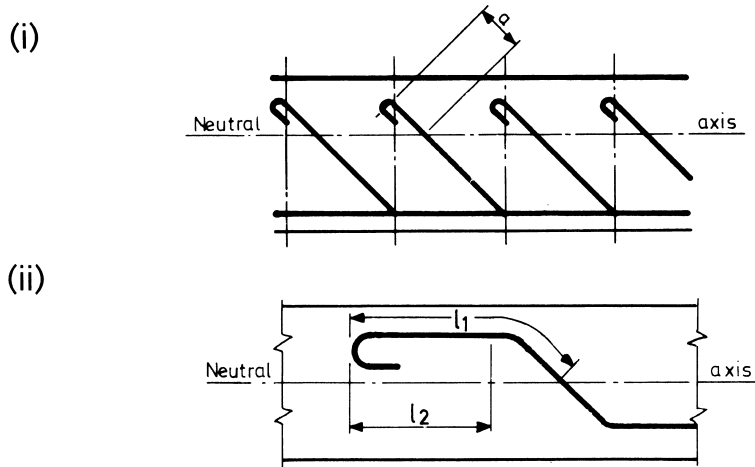
Case (a) on Sheet No. IV.15 gives bent bars as recommended by the European Code on Concrete (EC2). This case is similar to the structural detailing of bent-up bars adopted for many years in France and Germany.

Case (b) on Sheet No. IV.15 refers to column links. Longitudinal bars are the main steel, and links which contain the main steel are provided to prevent the main steel from bursting through the sides of the column. Where column lengths are such that bar splices are required they should have adequate lap lengths. The links can be of a square or a continuous helix type. A square cage is shown in the isometric view. In order to achieve continuity in the column-foundation system, starter bars of adequate length from the foundations are produced and are lapped with the main longitudinal reinforcement of the column. Details of these are fully described later on in this book. Sheet Nos IV.15 to IV.17 summarize the column detailing practice.

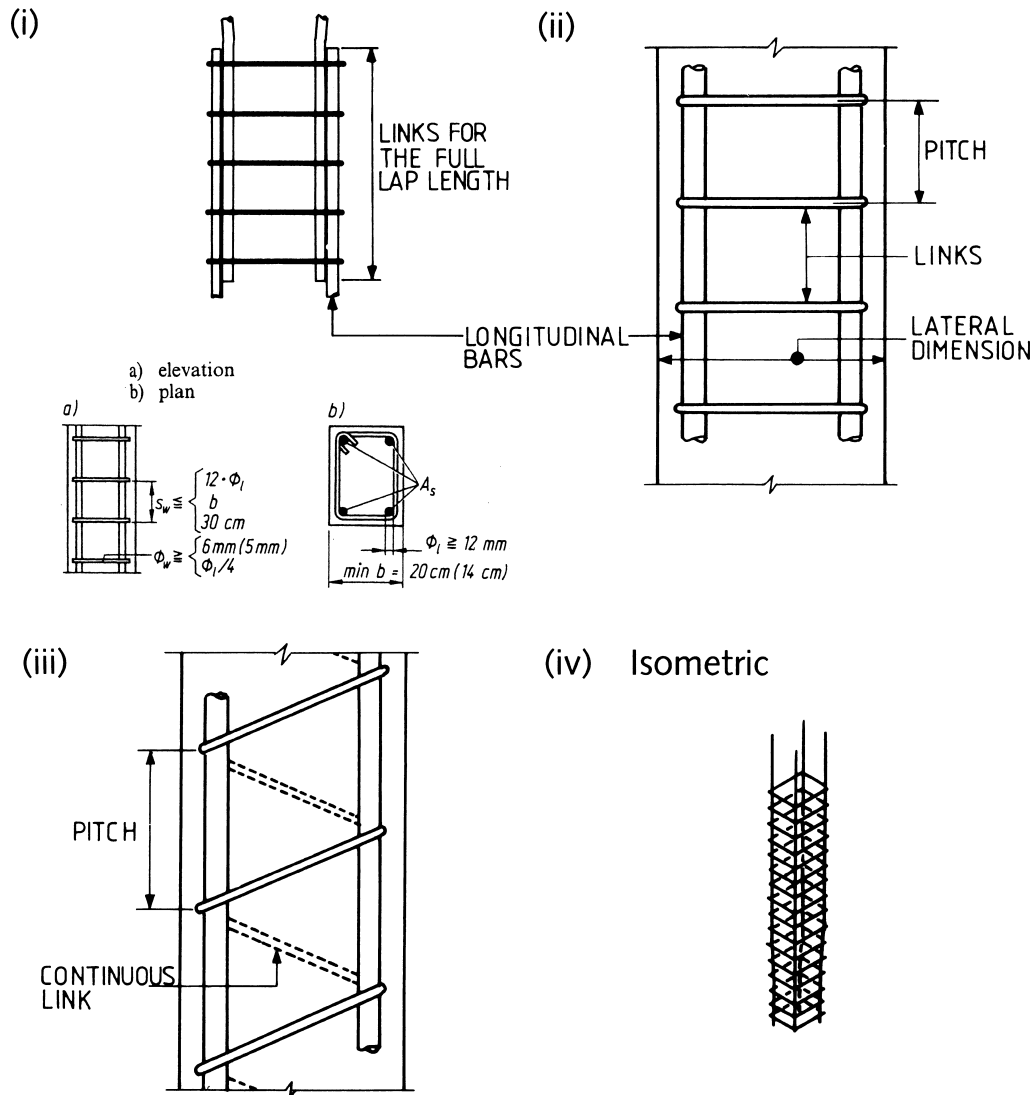
BENT BARS AND COLUMN LINKS (BASED ON EC2)

SHEET NO. IV.15

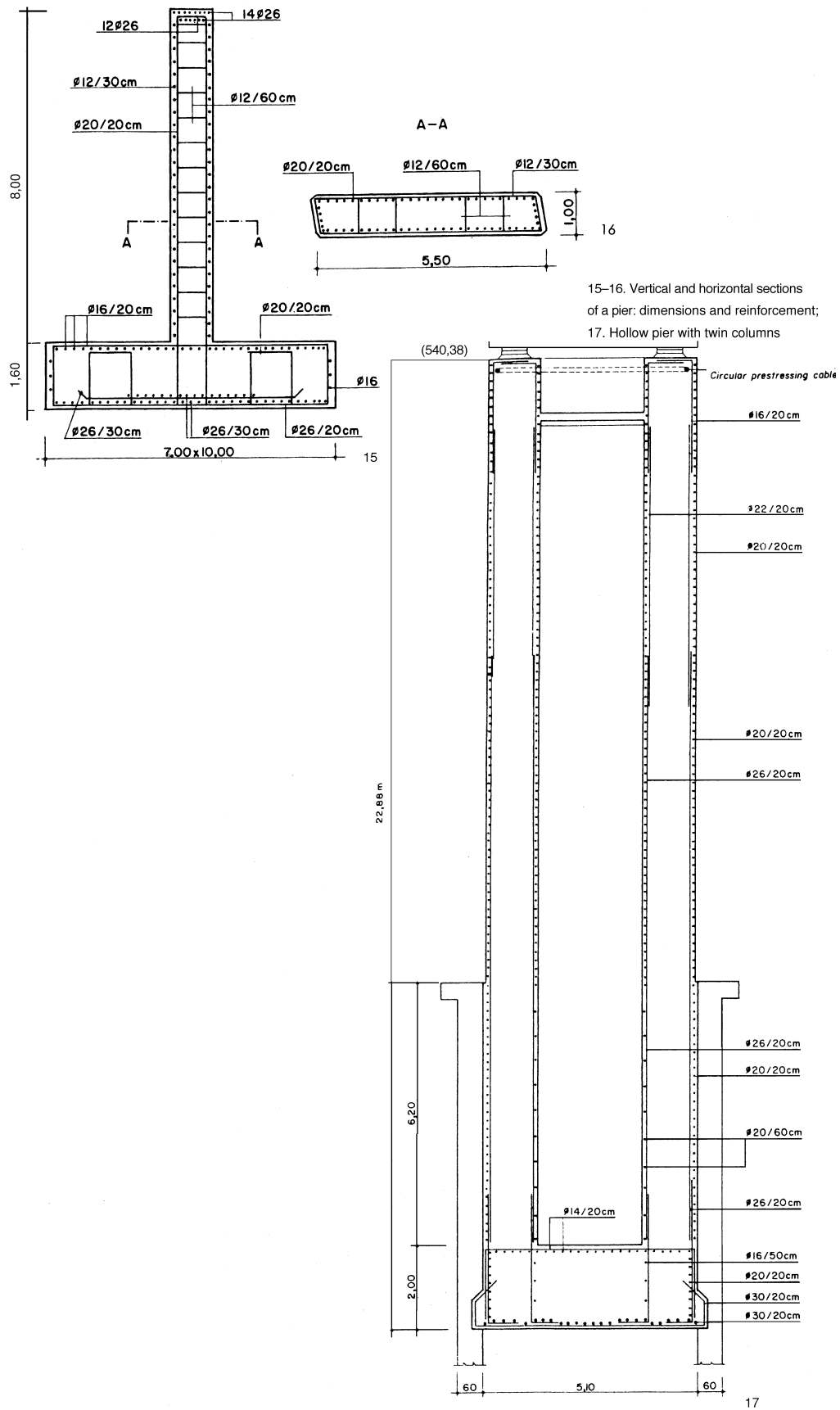
(a) Bent bars in deep beams



(b) Transverse reinforcement and vertical spacing of links



COLUMN DETAILS— 1 (BASED ON EC2)

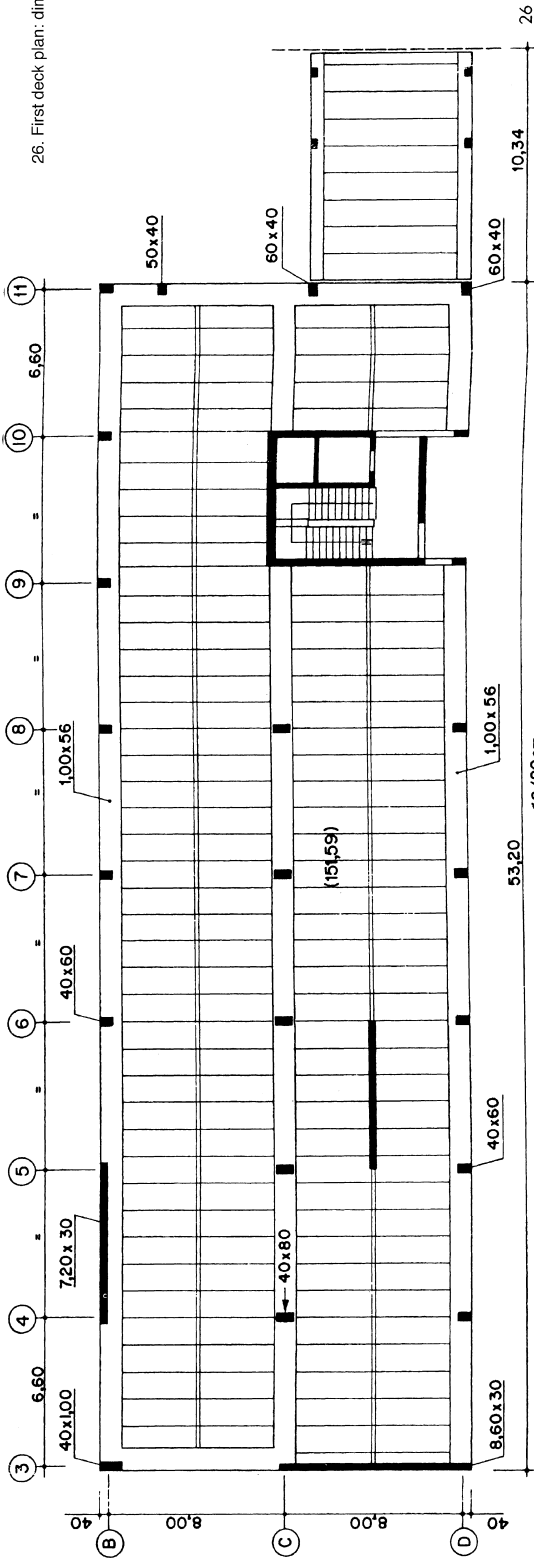


15-16. Vertical and horizontal sections of a pier: dimensions and reinforcement;
17. Hollow pier with twin columns

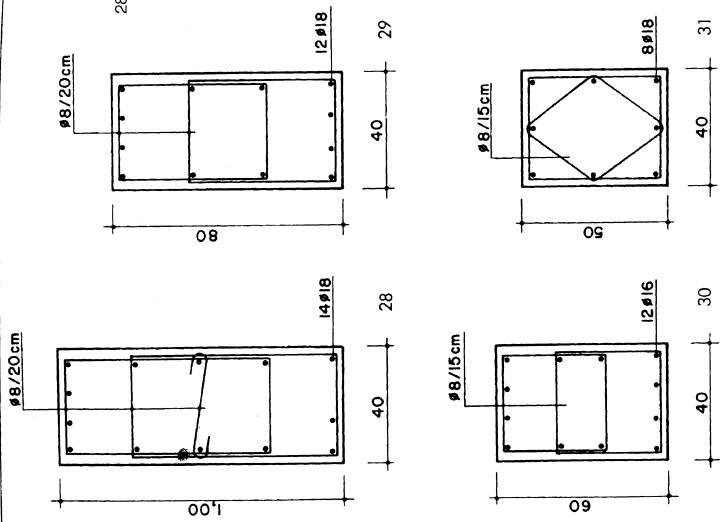
COLUMN DETAILS—II (BASED ON EC2)

SHEET NO. IV.17

26. First deck plan: dimensions:



28-31. Column sections: dimensions and reinforcement.



IV.2.3. Walls

There is no EC2 requirement for this, but a practical minimum of 175 mm.

IV.2.3.1. Minimum dimensions

IV.2.3.2. Vertical reinforcement

Minimum area, $A_{S_{v,min}}$

$$A_{S_{v,min}} \leq 0.004 A_c$$

Maximum area, $A_{S_{v,max}}$

$$A_{S_{v,max}} \geq 0.04 A_c$$

EC2 implies this to apply anywhere, including the laps.

Maximum spacing, S_{max}

$$S_{max} \geq 2h \text{ or } 300 \text{ mm}$$

IV.2.3.3. Horizontal reinforcement

Horizontal reinforcement to be placed between vertical reinforcement and face of wall.

Minimum area, $A_{S_{h,min}}$

$$A_{S_{h,min}} \leq \frac{A_{S_v}}{2}$$

Maximum spacing, S_{max}

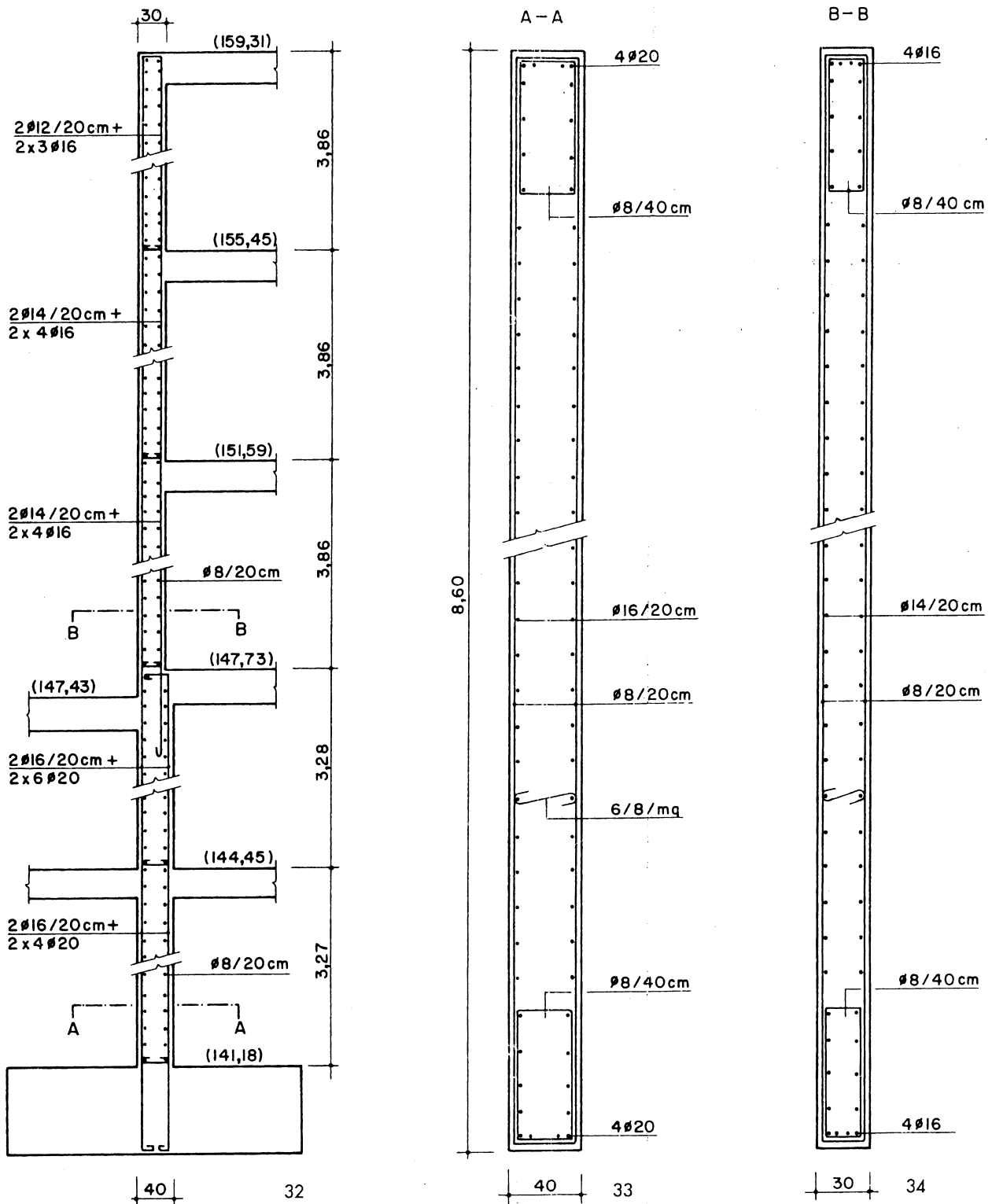
$$S_{max} = 300 \text{ mm}$$

IV.2.3.4. Transverse reinforcement

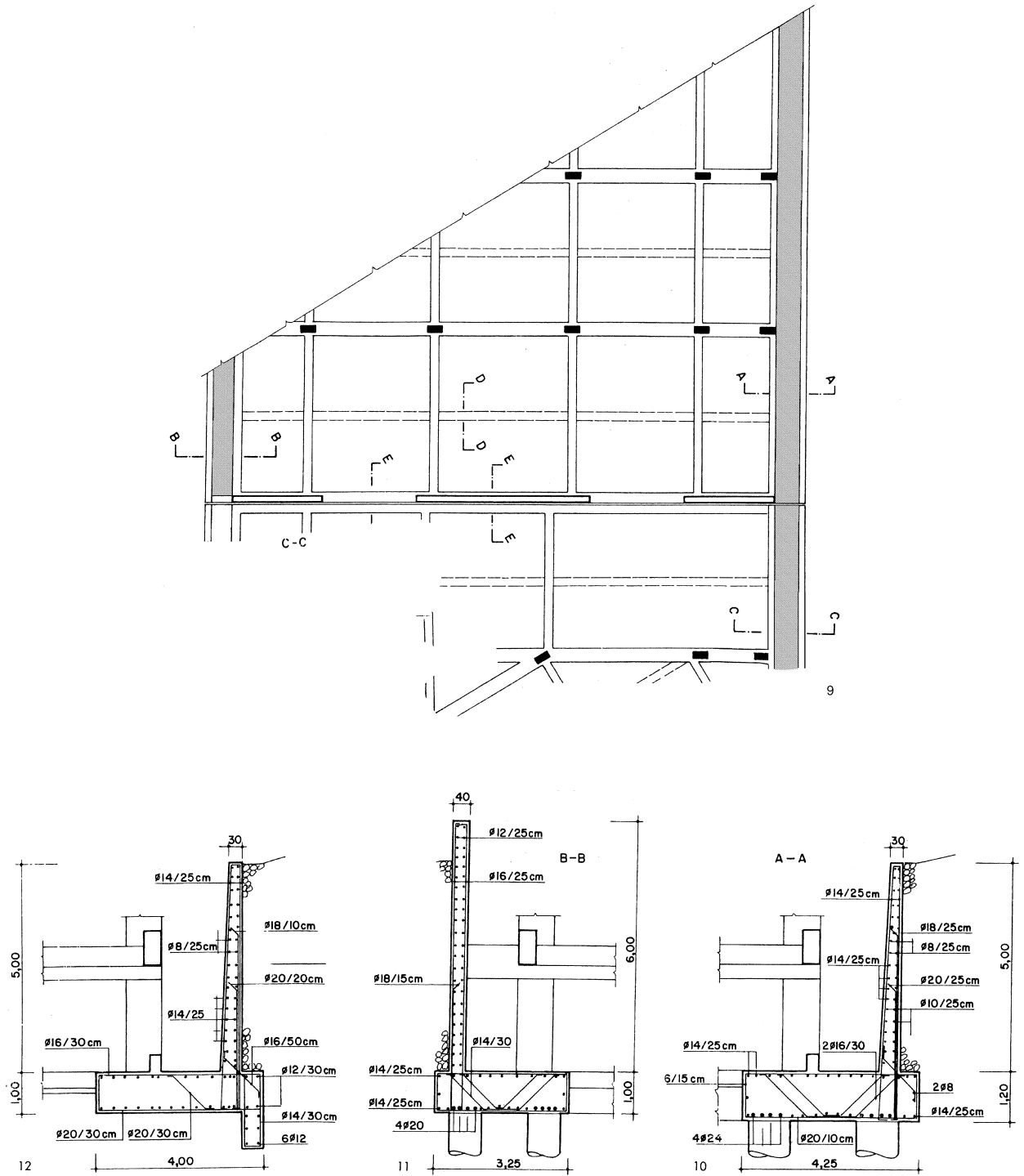
Where the area of vertical reinforcement exceeds $0.02A_c$, transverse reinforcement in the form of links should be provided in accordance with the requirements for columns. Sheet Nos IV.18 and IV.19 give isolated wall reinforcement details and plans as practised in European countries based on EC2.

WALL REINFORCEMENT DETAILS—I (BASED ON EC2)

SHEET NO. IV.18



32. Vertical section of an interior wall; reinforcement;
 33–34. Horizontal sections of interior walls; reinforcement.



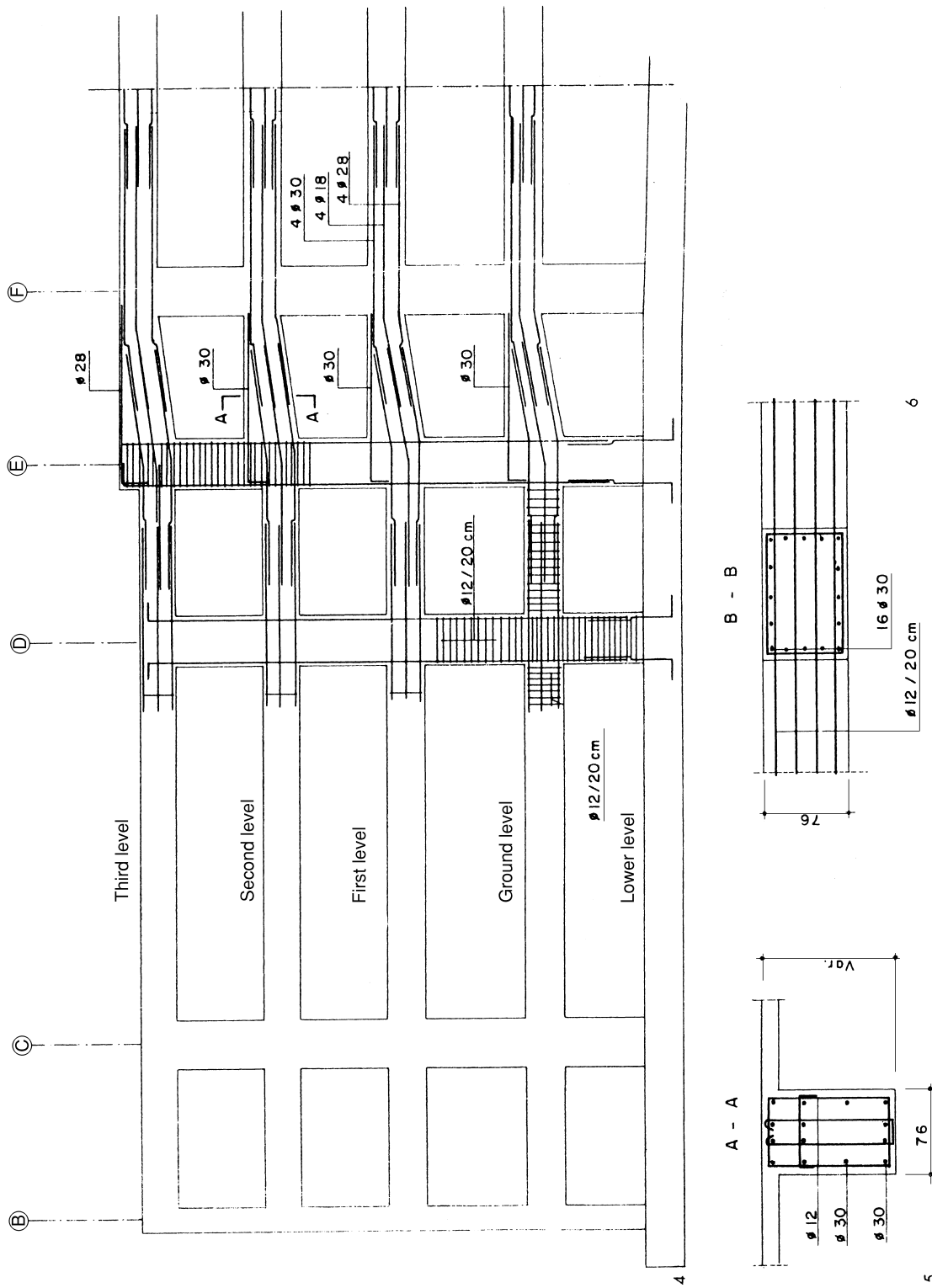
9. Plan of the first deck; 10–12. Vertical sections of the perimeter walls: dimensions and reinforcement.

IV.2.4. Frames

Sheet Nos IV.20 and IV.21 give detailing of frames based on EC2. The detailings of frames given on Sheet Nos IV.12 to IV.14 are acceptable by this code with minor modifications as shown on Sheet Nos IV.20 and IV.21.

RC FRAME DETAILING — 1 (BASED ON EC2)

SHEET NO. IV.20

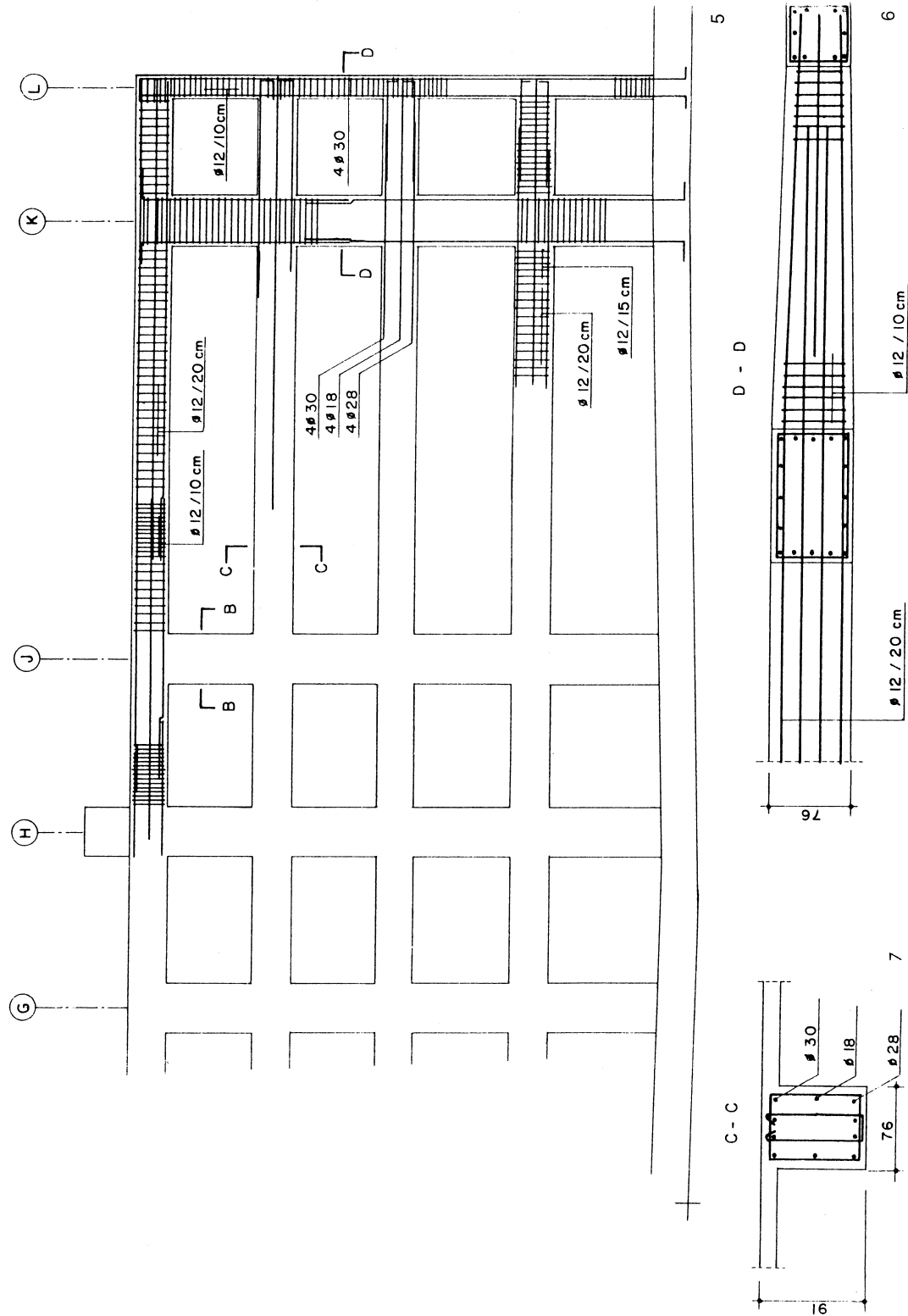


4. Longitudinal section of the structural frame; 5-7. Cross-sections.

RC FRAME DETAILING — II (BASED ON EC2)

SHEET NO. IV.21

5. Side elevation with reinforcement
6-7. Cross-sections



IV.3. Column, wall and frame detailing based on the American Concrete Institute codes

IV.3.1. Introduction

The ACI Building Code (2000) expresses the latest knowledge of reinforced concrete columns, walls and frames. This section will guide the users to various sections of the codes and at the same time encourage them to use computers for the design owing to the increasing complexity of the codes and new emergence of complex structures. Columns, walls and frames are designed and detailed using the latest computer software.

IV.3.2. Columns

Column designs must show the size of columns, number, locations, grade and size of reinforcement, and all necessary details where column section or reinforcement changes. Splicing must always be clearly defined, showing arrangement of splices whether butt or lapped, any staggers, and type of splicing required for butt splices. Orientation of reinforcement in two-way symmetrical columns must be shown when reinforcement is not two-way symmetrical. A reference is made to Sheet Nos IV.22 and IV.23.

IV.3.2.1. Column spirals

General

Code spirals must be provided with $1\frac{1}{2}$ extra turns at both top and bottom. The height (or length) of a spiral is defined as the distance out-to-out of coils, including the finishing turns top, and bottom, with a tolerance of plus or minus $1\frac{1}{2}$ in. (37 mm). Where a spiral cannot be furnished in one piece, it may be furnished in two or more sections to be field welded, or by additional length at each of the ends of each section to be tension lapped in the field 48 diameters minimum, but not less than 12 in. (180 mm). The sections must be properly identified by mark numbers to ensure proper assembly.

Spacers are used for maintaining the proper pitch and alignment of the spiral and shall conform to the ACI minimum requirements. Maximum length of spacers is that of the spiral plus one pitch.

The height of one piece assembled spirals for fabrication and shipping is limited to 25 ft (7.62 m) unless special handling arrangements are made. For greater heights, spirals must be field spliced by lapping or welding. Spacers are provided. Spirals are also used in piles, but these do not fall within the ACI 318-83 (revised 2000) definition of a spiral and are usually made of light wire and relatively large pitch. Spacers are not provided.

Buildings

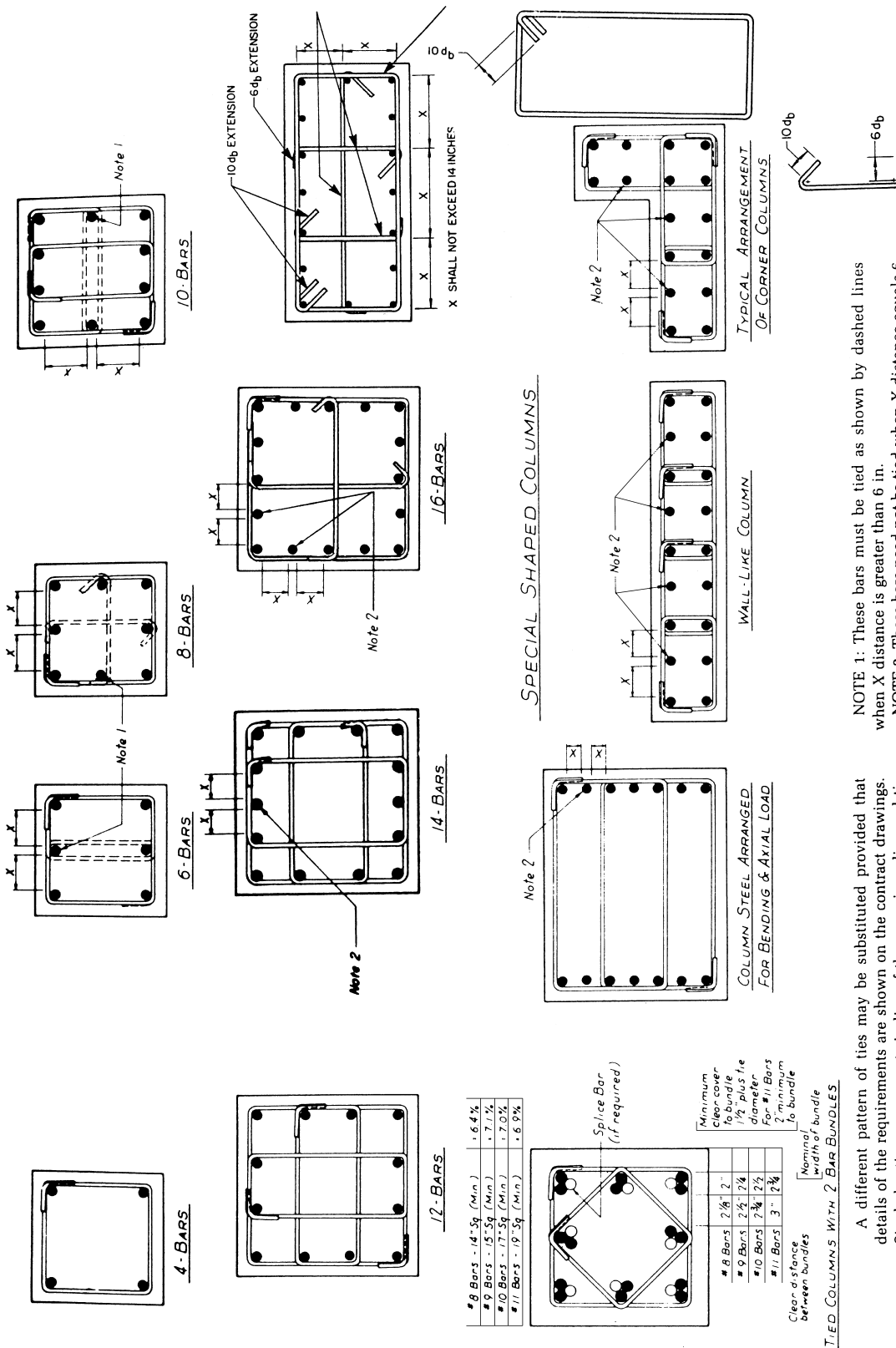
Unless otherwise specifically provided, spirals shall be detailed as extending from the floor level or top of footing or pedestal to the level of the lowest horizontal reinforcement in the slab, drop panel or beam above. In a column with a capital, the spiral shall extend to the plane at which the diameter or width of the capital is twice that of the column. If the engineering drawings require lateral reinforcement in the column between the top of the main spiral and the floor level above, it shall be provided by a stub spiral (short section of spiral) or by circular column ties. Where stub spirals are used, they must be attached to the main spiral for shipment or fully identified by mark numbers.

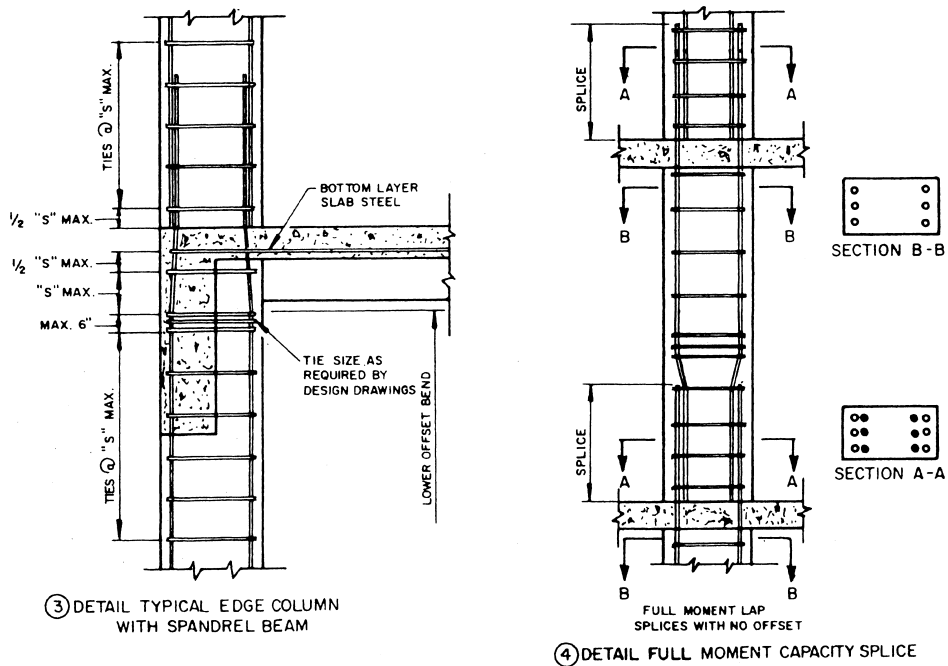
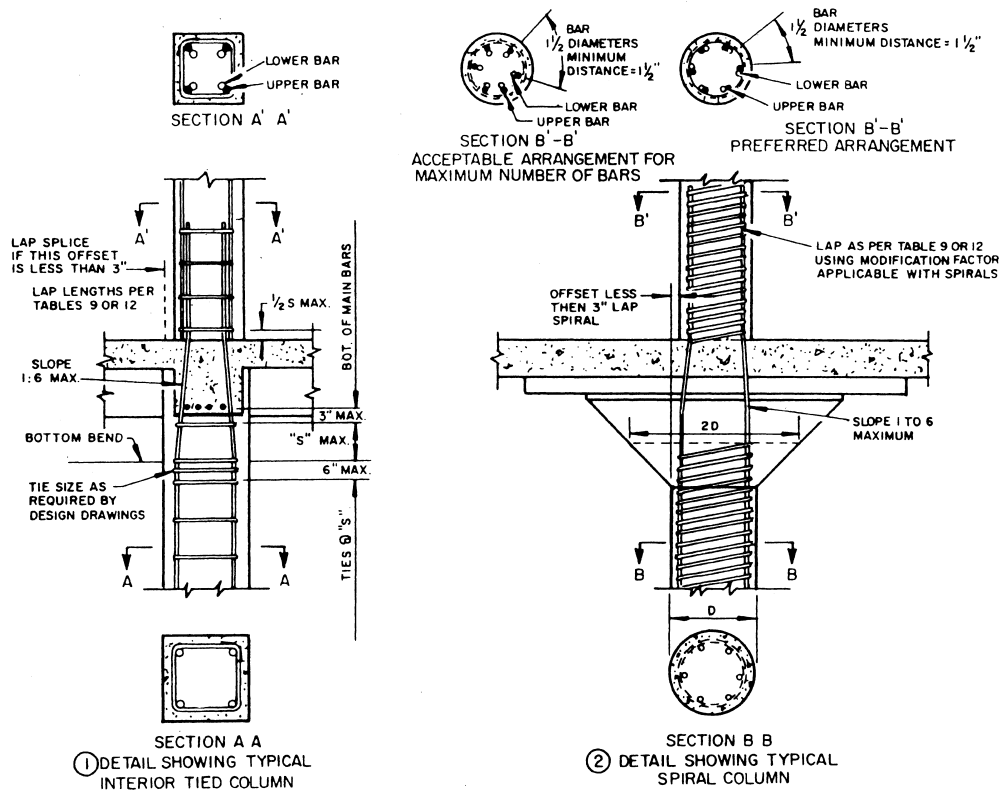
Offset between column faces

Where a column is smaller than the one below, vertical bars from below must be offset to come within the column above, or separate dowels must be used. The slope of the inclined portion must not exceed 1 to 6. In detailing offset column bars, a bar diameter plus clearance must be added to the desired

ONE PIECE COLUMN TIES AND LAP SPLICED
PRE-ASSEMBLED CAGES (BASED ON ACI CODE 318 (2000))

SHEET NO. IV.22





Where column size above is unchanged from the below, "upside down" offset bars are effective in maintaining the full moment capacity at the end of the column. In U.S. practice, this unusual detail is rare, and should be fully illustrated on design drawings to avoid misunderstandings, whenever its use is deemed necessary.

offset. In the corners of columns, bars are usually offset on the diagonal which requires that the offset be increased accordingly.

For any offset between column faces less than 3 in., the vertical bar shall be shop offset bent. When the offset is 3 in. (75 mm) or more, the vertical bars in the column below shall be terminated at the floor slab and separate straight dowels shall be provided.

Lapped splices

Typical arrangement of bars at a lapped splice is shown. Unless special details are provided on the engineering drawings, all column verticals to be lap spliced in square or rectangular columns must be shop offset.

There are no limits on the ratio of column cross-section dimensions. However, reductions prescribed for slender columns effectively limit the minimum size of practicable columns. Tables exist for short columns with specific loads, sizes, shapes, reinforcement concrete strengths and eccentricities. A number of computer programs and software are available for structural design and detailing of concrete structures.

IV.3.2.2. Spirals

Pitch or spacing of spirals should be given to the nearest quarter inch. According to ACI 318-83 (revised 1986), the clear spacing between spiral turns should not exceed 3 in. (75 mm) or be less than 1 in. or $\frac{1}{3}$ times the maximum size of coarse aggregate used. Spirals should be provided with $\frac{1}{2}$ (37 mm) extra turns at both top and bottom. If necessary to splice a spiral, it should be done by welding or by a tension lap splice of $48d_b$.

Minimum diameters to which standard spirals can be formed and minimum diameters which are considered collapsible are shown in Table IV.3 for various sizes of spiral bars (1 in. = 25.4 mm).

Table IV.3.

Spiral bar diameter: in.	Minimum outside diameter which can be formed: in.	Minimum outside diameter of collapsible spiral: in.
3/8	9	14
1/2	12	16
5/8	15	24
3/4 (special)	30	—

IV.3.3. Reinforced concrete walls

IV.3.3.1. Introduction

Reinforced concrete solid walls

A reference is made to ACI-318-2000 version for concrete walls. Concrete walls are defined (Section 2.1) as elements, usually vertical, used to enclose or separate spaces. Minimum reinforcement requirements given for walls in Section 14.3 of the code apply to walls which are used to separate spaces and which function in the structure as compression members. The provisions of Sections 14.4 and 14.5 of the code apply to walls which function as compression members. It should be noted that all minimum reinforcement requirements for walls in Chapter 14 of the code may be disregarded 'where structural analysis shows adequate strength and stability' (Section 14.2.7 of the code).

The required analysis for flexural strength (ACI 318.1, Section 7.1) can be employed. Temperature and shrinkage reinforcement can be likewise reduced (ACI 318.1, Section 6.3). The design of such walls must be based on lateral forces or any other loads to which they may be subjected (Section 14.2.1). A rational method of design for walls subject to flexure or both flexure and axial compression is provided (Section 14.4). An empirical method of design for

bearing walls with small moment (resultant compressive force within the middle-third of the wall thickness) is permitted (Section 14.5.1). The empirical design method is explained in detail (Section 14.5). Non-loadbearing walls whether precast or cast-in-place, ordinary reinforced or prestressed, must be designed by the rational method, taking into account all loading conditions (Sections 14.4, 16.3, and 18.1.3 of the code).

Special walls

Walls which principally resist horizontal shear forces (shear walls in low-rise buildings) in the plane of the wall and parallel to the length of the wall must conform to the requirements for shear (Section 11.10 of the code). Shear walls which resist forces from seismic accelerations must conform to the seismic requirements (Section A.5). Special requirements for walls designed as grade beams are provided (Section 14.7). In addition to the general provisions (Section 14.4) of the code, precast concrete wall panels must conform to requirements for precast concrete wall panels (Section 16.3). Cantilever retaining walls are designed for flexure under Chapter 10 (Section 14.1.2) of the code.

When bars larger than #5 are used, the minimum vertical and horizontal reinforcement must be increased 25% (Sections 14.3.2-b and 14.3.3-b of the code). The code does not require that minimum reinforcement be placed in the two faces of a wall (Sections 14.1.2, 14.2.7 and 14.4 of the code).

Table IV.4.

Thickness, <i>h</i> : in.	In single layer only*		In both faces†	
	Horizontal $\rho = 0.0020$	Vertical $\rho = 0.0012$	Horizontal $\rho = 0.0020$	Vertical $\rho = 0.0012$
6	#3 @ 9	#3 @ 18	—	—
8	#4 @ 12	# @ 11	—	—
10	#5 @ 15	#4 @ 16	#4 @ 18	#3 @ 18
12	#5 @ 12	#4 @ 13	#4 @ 16	#3 @ 15
14	#5 @ 11	#5 @ 18	#4 @ 14	#3 @ 13
16	#5 @ 10	#5 @ 16	#4 @ 12	#3 @ 11
18	#5 @ 8½	#5 @ 14	#5 @ 17	#4 @ 18

* Bars arranged in two-way mat, usually centred in wall.

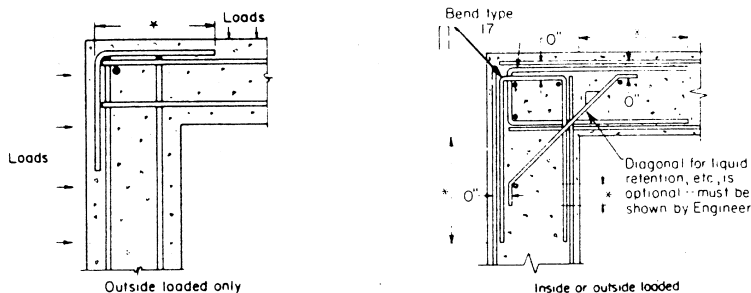
† Bars arranged in identical two-way mats, one near each face of wall.

Sheet No. IV.24 shows typical wall details. Note: when steel is placed in two layers, maximum spacing limitations apply separately, but minimum steel requirements (ρ) apply to the sum of steel areas in both faces. For wall panel detailing a reference is made to the separate section under precast concrete detailing in this text.

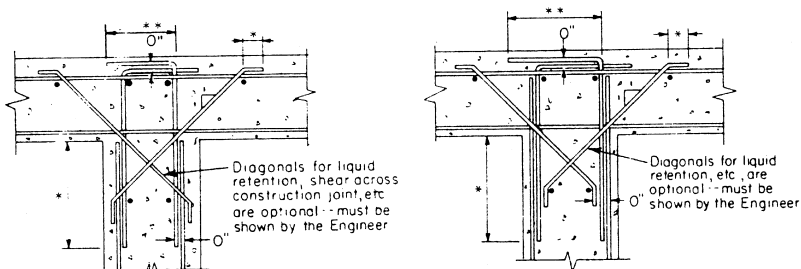
Different types of walls exist in order to perform different functions. The well knowns are lever retaining walls, area enclosure walls, exterior precast panel walls, tilt-up walls and cast-in-place walls for hydraulic structures. Such walls are subjected primarily to flexure or to flexure and relatively small axial compression loads with the resultant compressive forces outside the kern ($e > h/6$) of the section and must be designed by the rational method (Section 14.4 of the ACI code 318-83 (revised 2000)). Design for shear forces perpendicular to the face of such walls must conform to the requirements for slabs (Section 11.11 of the ACI code 318-83 (revised 2000)).

**TYPICAL WALL DETAILS
(BASED ON ACI CODES AND OTHER PRACTICES)**

SHEET NO. IV.24



Typical corner details

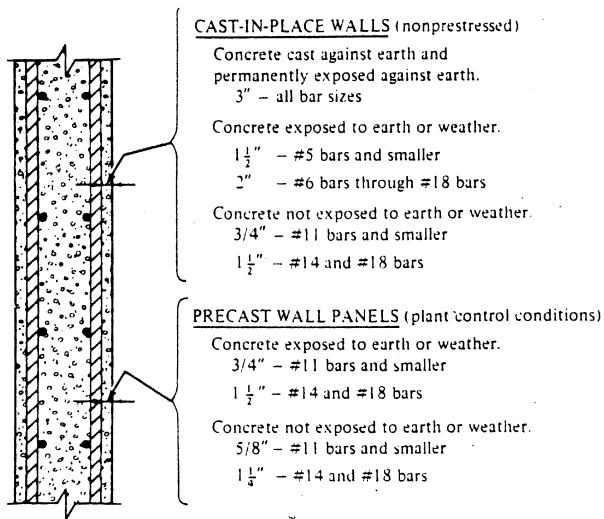


Typical intersection details for double curtain reinforcement

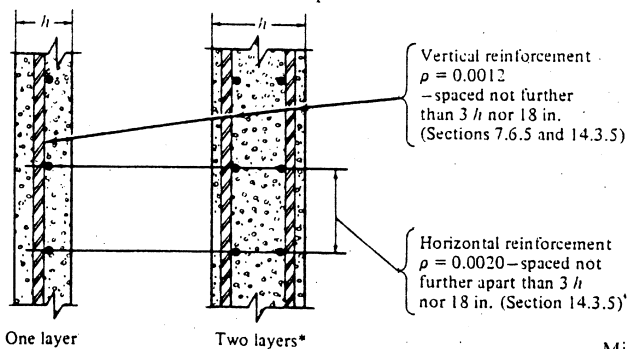
NOTES: All 90-deg bends as shown unless otherwise indicated on design drawings. Vertical bars shown at hooks only.

**If other than a standard 90-deg end hook, this dimension must be shown by the Engineer.

*This dimension must be shown or specified by Engineer.



Concrete protection for reinforcement.



*Required for walls except basement walls when h is greater than 10 inches.

Minimum wall reinforcing and maximum spacing

Slenderness effects

An approximate evaluation of slenderness effects was made (Section 10.11) for the wall examples designed by the rational method, but kl_u/h values greater than 100 were not considered to avoid making a special analysis (Section 10.10.1 of the ACI Code 318-83 (revised 2000)).

Two way reinforced walls

Walls for underground enclosures to resist lateral forces due to earth or liquid pressure frequently are supported on three or four sides (basements, sewage and water structures, etc.). A realistic two-way elastic analysis will usually reduce overall reinforcement requirements and cracking since two-way reinforcement will conform closely to the elastic analysis of bending. The specific requirements for two-way slab systems based on supports such as columns, or beams between columns, are difficult to extend to walls. Within the general code requirements two-way reinforcement proportioned by any of the methods specified in ACI 318-63, should be acceptable under ACI 318-83 as a 'rational' method.

IV.3.4. Reinforced concrete frames

IV.3.4.1. Introduction

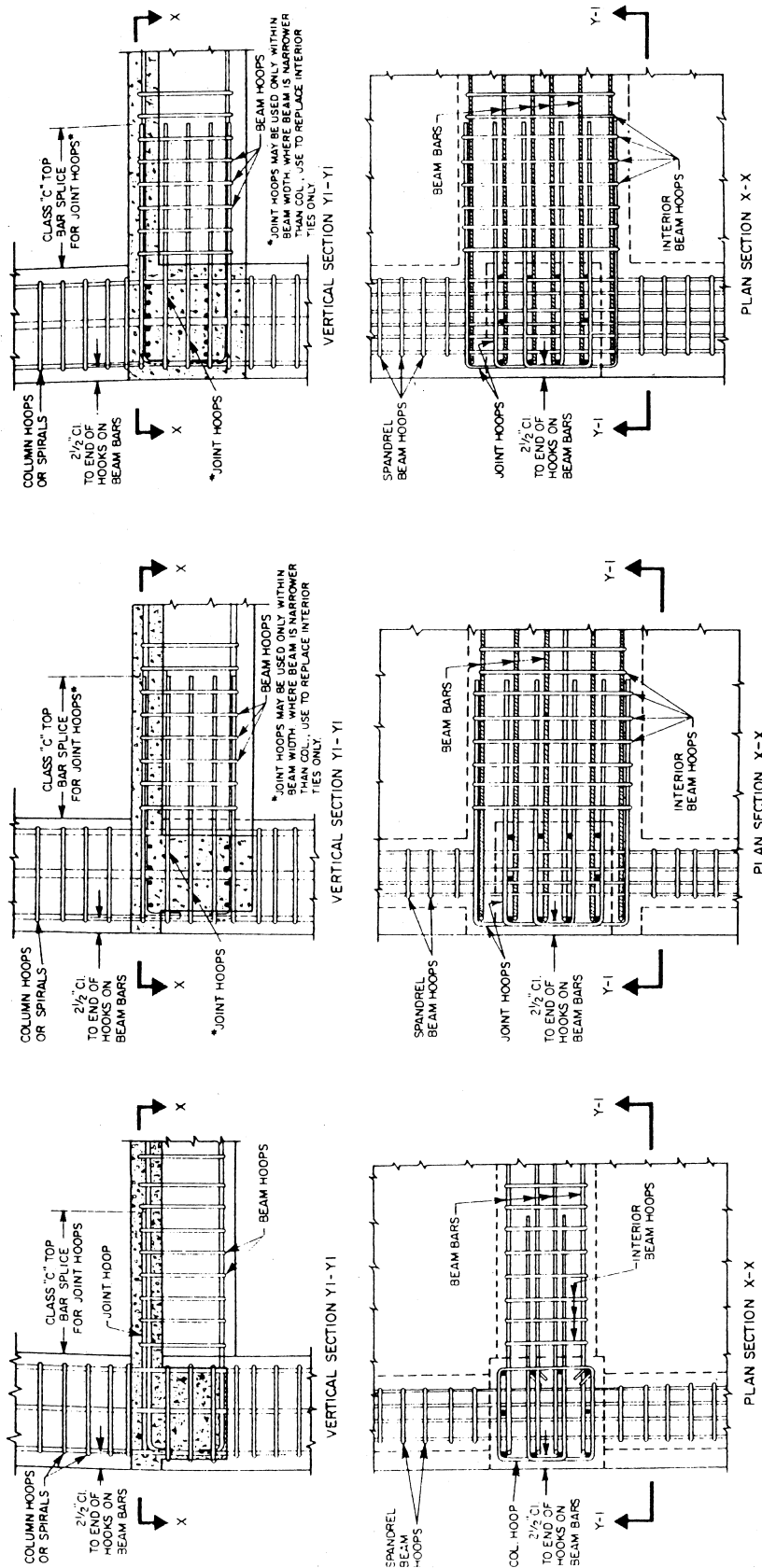
The analysis and design of various types of reinforced concrete frames are identical to those described for the British and the European codes and practices. Deviations do exist due to the ACI code requirements and such details are not within the scope of this text.

IV.3.4.2. Frame—spandrel joint details

For ductile frame, in the regions of high seismic risk, the interior spandrel beams shall be kept narrower than the column as are shown in case 1 on Sheet No. IV.25. For the region of moderate seismic risk in braced frames, the interior beam shall be kept wider than the column but the width of the spandrel beam shall be the same as the width of the column. This is shown in case 2 on Sheet No. IV.25. In case of braced frames with no change in case 2 for the interior beams, but spandrel beams are narrower than columns, the joint detailing shall conform onto case 3 on Sheet No. IV.25.

TYPICAL DUCTILE FRAME — SPANDREL JOINT DETAILS

SHEET NO. IV.25



Case 3: For regions of moderate seismic risk in braced frames. Interior beam wider than the column; spandrel beam narrower than column

Case 2: For regions of moderate seismic risk in braced frames. Interior beam wider than the column; spandrel beam same width as column

Case 1: For regions of high seismic risk. Interior and spandrel beams narrower than column

V. Prestressed concrete

V.1. General introduction

Prestressed concrete has attained worldwide recognition in the development of industrialized construction and design. Prestressing consists of introducing imposed deformations by tensioning prestressing wires, cables or strands and tendons to a high stress which decreases with time due to various losses such as shrinkage, creep, steel relaxation, friction and wobbling effects. The word *prestress* is associated with the following:

- (a) pretensioned concrete
- (b) post-tensioned concrete.

In the case of pretensioned concrete structures, the tensioning of the tendon is carried out before concreting. The prestressing force is held temporarily either by a specially-constructed prestressing bed or by a mould or form. When the concrete strength reaches the specified transfer strength, detensioning and stress transfer to such structures can be performed. In practice these structures are prefabricated.

In the case of post-tensioned concrete structures, the tensioning of the tendon is carried out after casting and hardening of the concrete. This method is more effective in the design and construction of high-rise and long-span concrete structures. The design and detailing of such structures are influenced by the serviceability classification, which includes the amount of flexural tensile stresses allowed while carrying out the design/detailing of such structures. They are then classified into individual classes which are given below:

Class 1: no flexural tensile stresses.

Class 2: flexural tensile stresses but no visible cracking.

Class 3: flexural tensile stresses but surface width of cracks not exceeding 0.1 mm for members in very severe environments and not exceeding 0.2 mm for all other members.

The structural detailing of prestressed concrete members must take into consideration durability, fire resistance and stability. The relevant codes include BS 8110 which should strictly be followed for the correct evaluation of design ultimate loads and the characteristic strength of concrete and steel.

Generally, high strength concrete is used for prestressed concrete work. The steel used in prestressed concrete is generally of a much higher strength than mild steel. This aspect is discussed later on in the choice and evaluation of prestressing systems.

Material data and prestressing systems are given on Sheet Nos V.1 to V.14. In such the prestressing tendons can be bonded and unbonded. The structural detailing is affected when the prestressed concrete structure is designed with bonded and unbonded tendons. Before the prestressing load is transmitted into various zones of concrete with bonded or unbonded tendons, it is necessary to protect the areas immediately under the anchorages against bursting effects caused by large loads generated by prestressing tendons.

STRAND CHARACTERISTICS**SHEET NO. V.1**

Nominal size	Nominal area	Characteristic strength	Characteristic load	Initial load	
mm	mm ²	f_{pu} N/mm ²	P_k kN	0.8 P_k kN	0.7 P_k kN
<u>Standard</u>	7 – wire (normal and low relaxation) to BS 3617				
12.5	94.2	1750	165	132	115
15.2	138.7	1640	227	182	159
<u>Standard</u>	19 – wire (normal and low relaxation) to BS 4757				
18.0	210	1760	370	296	259
<u>Super</u>	7 – wire (normal and low relaxation)				
12.9	100.6	1830	184	147	129
15.4	143.2	1750	250	200	175
<u>Dyform</u>	7 – wire (low relaxation)				
12.7	112	1870	209	167	146
15.2	165	1820	300	240	210
18.0	223	1700	380	304	266
Maximum relaxation for initial load				0.8 P_k	0.7 P_k
Normal relaxation strand				12%	7%
Low relaxation strand				3.5%	2.5%

DATA FOR MACALLOY SINGLE BAR STRESSING SYSTEM**SHEET NO. V.2**

Size of tendon Bar No./mm	Minimum centre of anchorage	Initial load		Size of bearing plate mm × mm	Size of duct (int. dia.) mm
		0.7 P _k kN	0.8 P _k kN		
1/20	100 single	228	260	100 × 100	40 for each bar
2/20		455	520	200 × 100	
3/20		683	780	300 × 100	
1/22		263	300	125 × 125	
2/22		526	600	250 × 125	
3/22		788	900	375 × 125	
1/25	120 single	350	400	125 × 125	40 for each bar
2/25		700	800	250 × 125	
3/25		1050	1200	375 × 125	
1/28	120 single	438	500	140 × 140	40 for each bar
2/28		876	1000	280 × 140	
3/28		1314	1500	420 × 140	
1/32	150 single	560	640	150 × 150	42.5 for each bar
2/32		1120	1280	300 × 150	
3/32		1680	1920	450 × 150	
4/32		2240	2560	300 × 300	
1/35	175	665	760	175 × 150	45 for each bar
2/35		1330	1520	350 × 175	
3/35		1995	2250	525 × 150	
1/40	350	875	1000	200 × 175	50 for each bar
2/40		1750	2000	400 × 175	
3/40		2625	3000	600 × 175	
4/40		3500	4000	400 × 350	

COLD WORKED HIGH TENSILE BARS**SHEET NO. V.3**

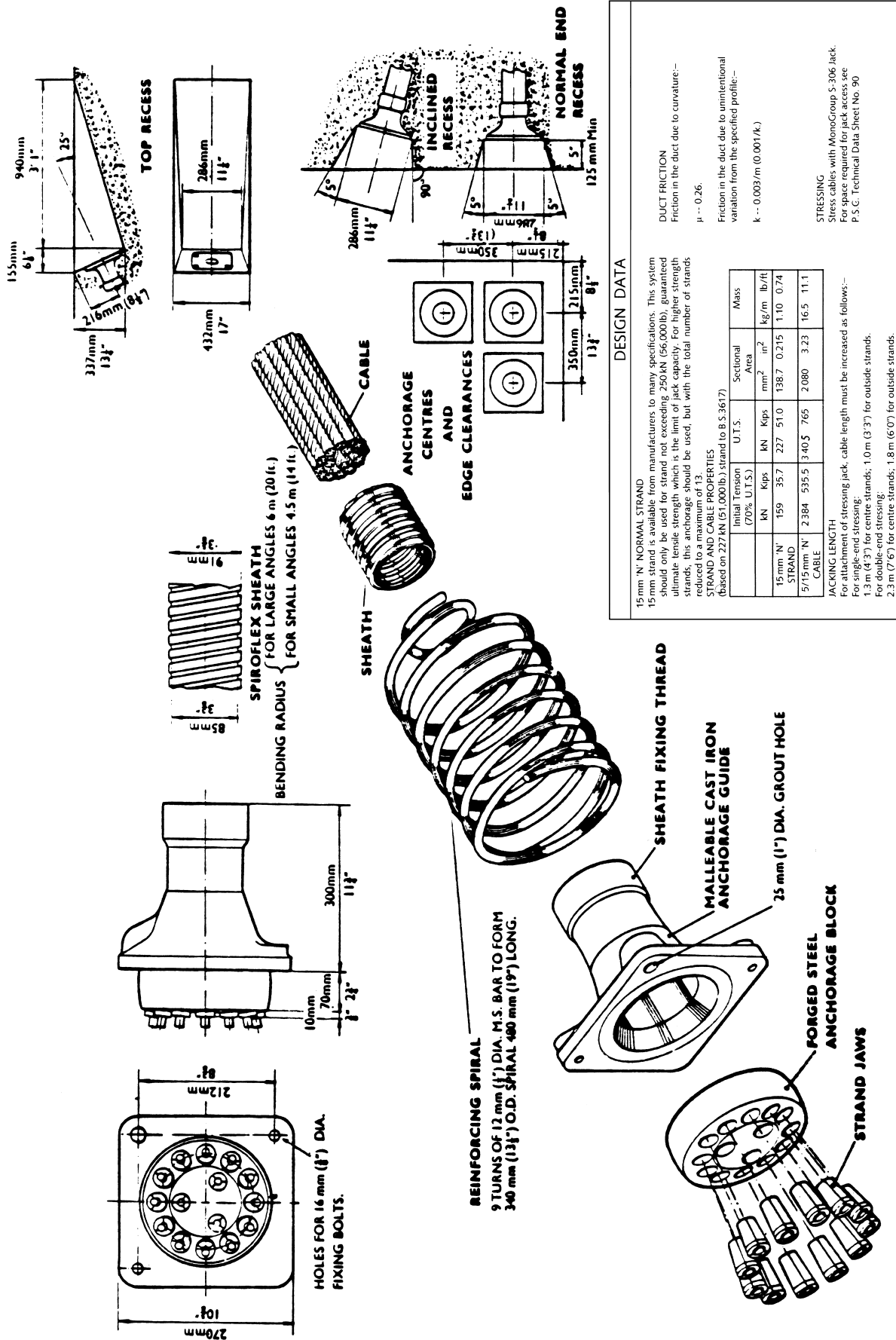
Nominal size	Nominal area	Characteristic strength	Characteristic load	Initial load	
mm	mm ²	f_{pu} N/mm ²	P_k kN	$0.8 P_k$ kN	$0.7 P_k$ kN
20	314	1035	325	260	228
25	491	1020	500	400	350
32	804	995	800	640	560
40	1257	995	1250	1000	875

**COLD DRAWN AND PRE-STRAIGHTENED WIRE
(NORMAL AND LOW RELAXATION)**

Nominal size	Nominal area	Characteristic strength	Characteristic load	Initial load	
mm	mm ²	f_{pu} N/mm ²	P_k kN	$0.8 P_k$ kN	$0.7 P_k$ kN
4	12.6	1720	21.7	17.3	15.2
5	19.6	1570	30.8	24.6	21.6
7	38.5	1570	60.4	48.3	42.3
Maximum relaxation for initial load				$0.8 P_k$	$0.7 P_k$
Normal relaxation wire				8.5%	5%
Low relaxation wire				3%	2%

'FREYSSI' MONOGROUP— 15/15 mm NORMAL STRAND SYSTEM

SHEET NO. V.4

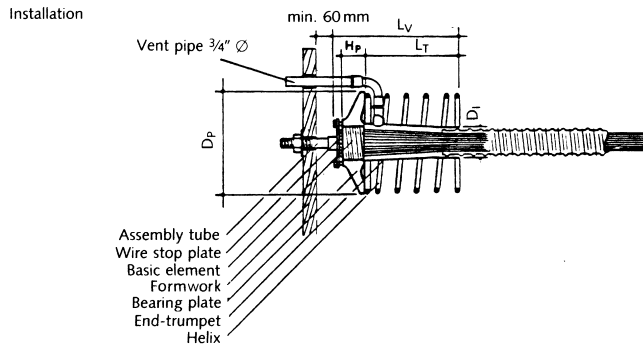


THE BBRV SYSTEM FOR PRESTRESSED CONCRETE

SHEET NO. V.5A

A fixed anchorage for use in conjunction with the Type 'C' stressing anchorage. The arrangement of the wires in the basic element and capacity are the same as in the Type 'C' anchorage.

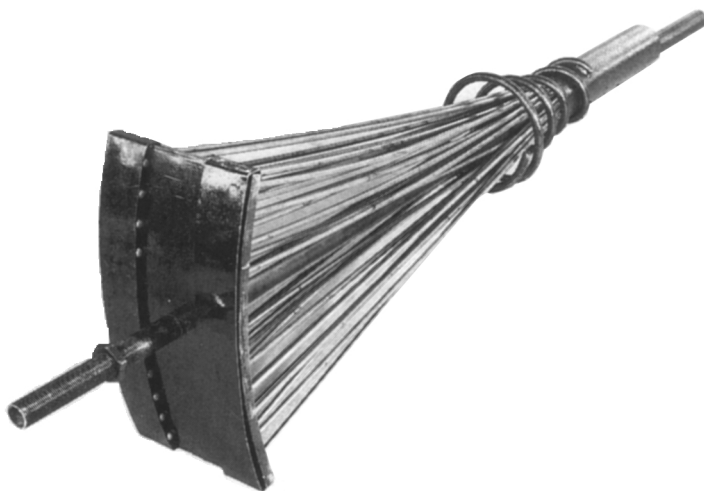
This type of fixed anchorage enables the full tendon force to be transmitted to the concrete immediately behind the bearing plate.



Type E anchorage	Type designation	E130	E170	E220				
Steel wire per anchorage, maximum number 7 mm (0.276") dia.		31	42	55				
		mm	in	mm	in	mm	in	
Bearing plate	outside diameter	D _P	235	9.25	270	10.63	300	11.81
	thickness	H _P	52	2.05	60	2.36	68	2.68
End-trumpet	length	L _T	215	8.47	245	9.65	265	10.43
	connection, outer dia.	D _I	60	2.36	70	2.76	80	3.15
Assembly tube	minimum length		130	5.12	130	5.12	130	5.12
Length of anchorage		L _V	288	11.34	326	12.84	354	13.94

**BBRV SINGLE FIXED ANCHORAGE SYSTEM
TYPE SS, SR AND SL**

SHEET NO. V.5B



BBRV TENDON SIZES AND THEIR PROPERTIES**SHEET NO. V.6**

Tendon reference	B16	B24	B34	C42	C55	L73	L85	L97	L109	L121	
No. of wires (7 mm dia.)	16	24	34	42	55	73	85	97	109	121	
Characteristic strength using 1570 N/mm ² wire	967	1450	2054	2538	3323	4411	5136	5861	6586	7311	
Jacking force (kN) at 80% of C.S.	773	1160	1643	2030	2659	3529	4109	4689	5269	5849	
Jacking force (kN) at 75% of C.S.	725	1088	1540	1903	2492	3308	3852	4396	4939	5483	
Jacking force (kN) at 70% of C.S.	677	1015	1438	1776	2326	3087	3595	4103	4610	5118	
<i>Bearing Plate</i> Side length (sq.)	A	178	220	250	280	300	335	360	385	405	425
Thickness	B	15	20	25	35	35	45	50	50	60	60
<i>Trumpet</i> Outside diameter	C	120	133	154	154	165	194	219	229	229	245
<i>Anchor Head</i> Thread diameter	D	100	115	130	90	98	125	130	145	150	155
Standard length	E	60	80	90	53	63	110	120	126	134	146
Overall diameter	F	—	—	—	—	—	160	165	185	190	200
<i>Pull Sleeve</i> Diameter	G	—	—	—	130	144	—	—	—	—	—
Standard length	H	—	—	—	118	138	—	—	—	—	—
<i>Lock Nut</i> Diameter	J	135	155	180	178	198	—	—	—	—	—
Thickness	K	30	40	50	45	53	—	—	—	—	—
<i>Chocks</i> Diameter	L	—	—	—	—	—	228	250	262	270	288
Min. thickness	M	—	—	—	—	—	48	52	55	59	62
Max. anchorage projection (Std. comps.)	N	67	87	97	118	138	205	219	228	240	255
<i>Sheathing</i> Internal diameter	O	40	50	60	65	75	85	95	100	105	110
External diameter		48	58	68	73	83	93	103	108	113	118

Length of trumpet 'P' \simeq Extension + 105 mm (B type anchors)
" + 120 mm (C type anchors)
" (L type anchors)

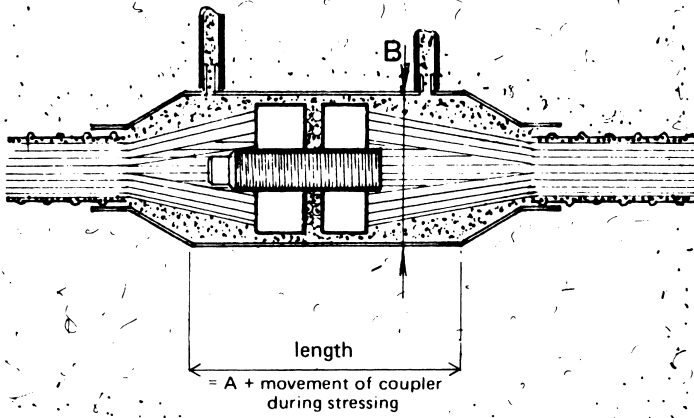
All dimensions are in millimetres.

BBRV COUPLING ANCHORS

SHEET NO. V.7

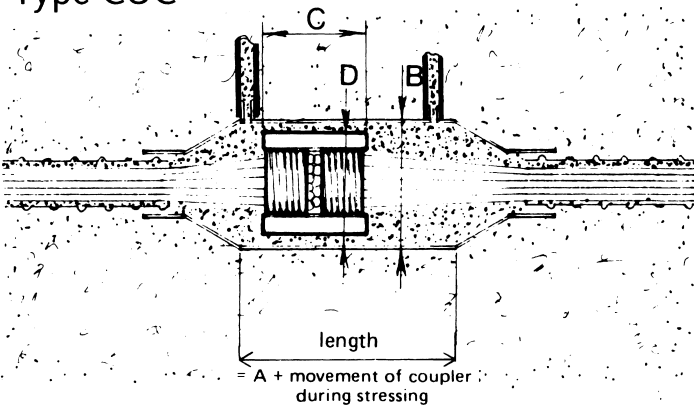
Movable (unstressed)

Type BUB



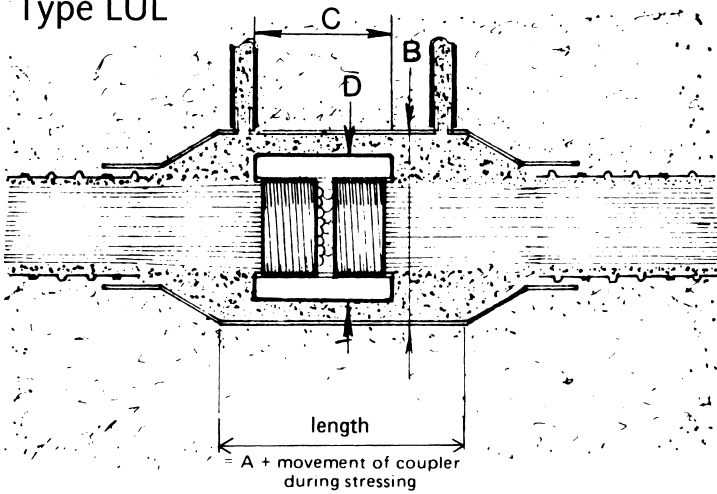
	A	B	C	D
B16	335	120	—	—
B24	405	133	—	—
B34	455	154	—	—

Type CUC



C42	160	154	140	130
C55	180	165	160	144

Type LUL



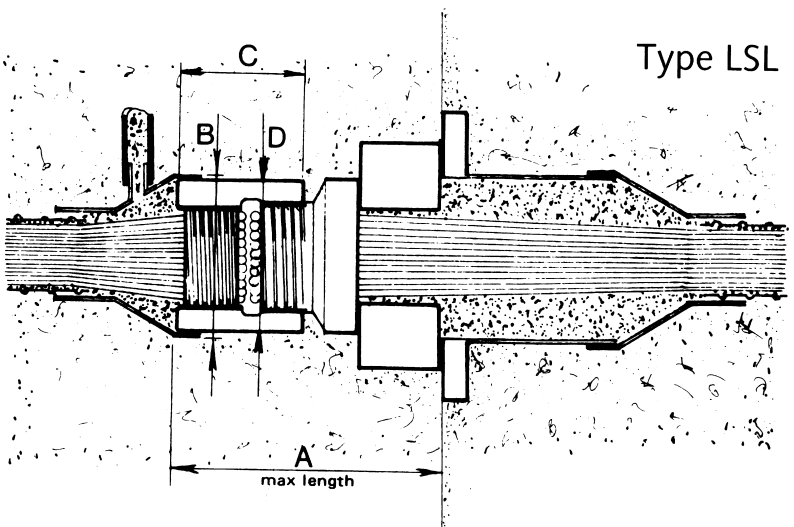
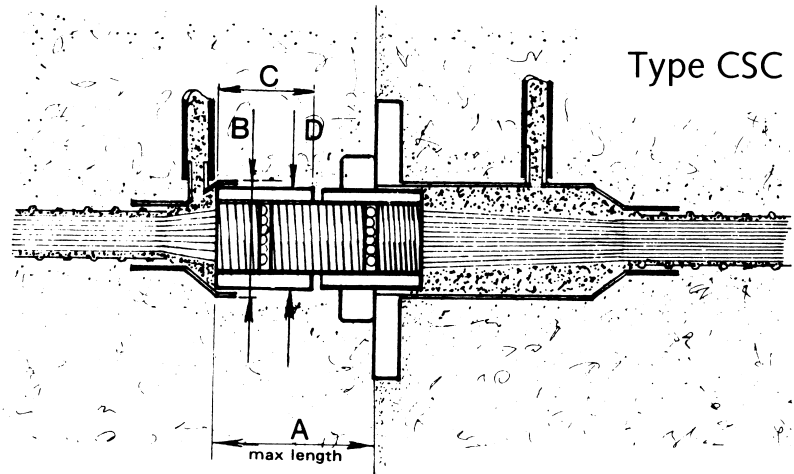
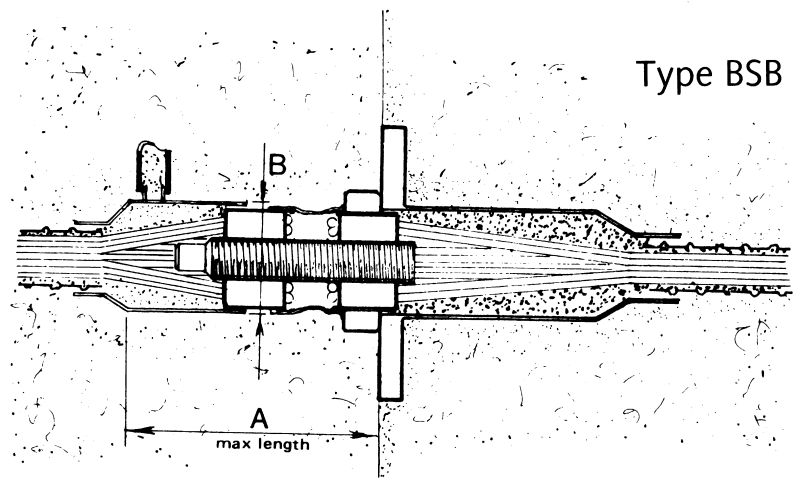
L73	210	180	195	161
L85	220	195	210	174
L97	235	205	215	186
L109	245	215	235	197
L121	255	230	250	208

BBRV COUPLING ANCHORS

Fixed (stressed)

A	B	C	D
260	120	—	—
315	133	—	—
350	154	—	—
290	135	160	130
330	150	180	144
315	170	190	165
340	180	210	175
351	195	215	190
368	205	230	200
388	215	245	210

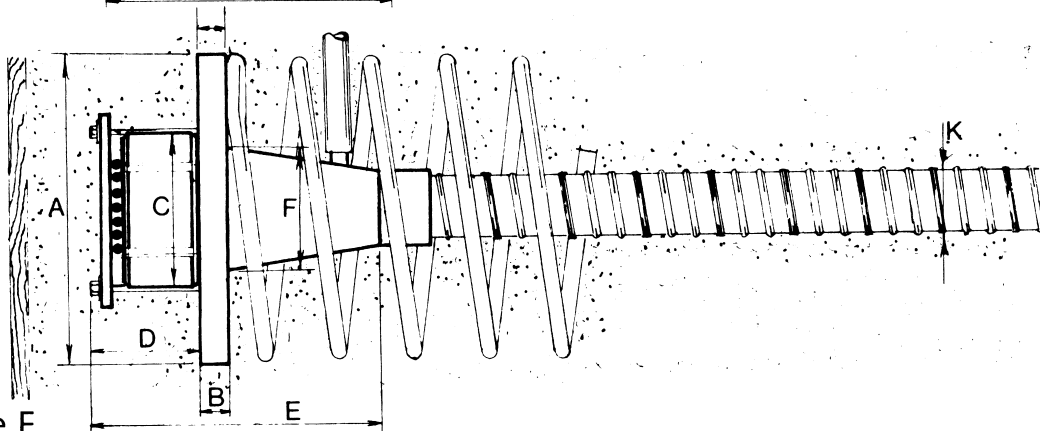
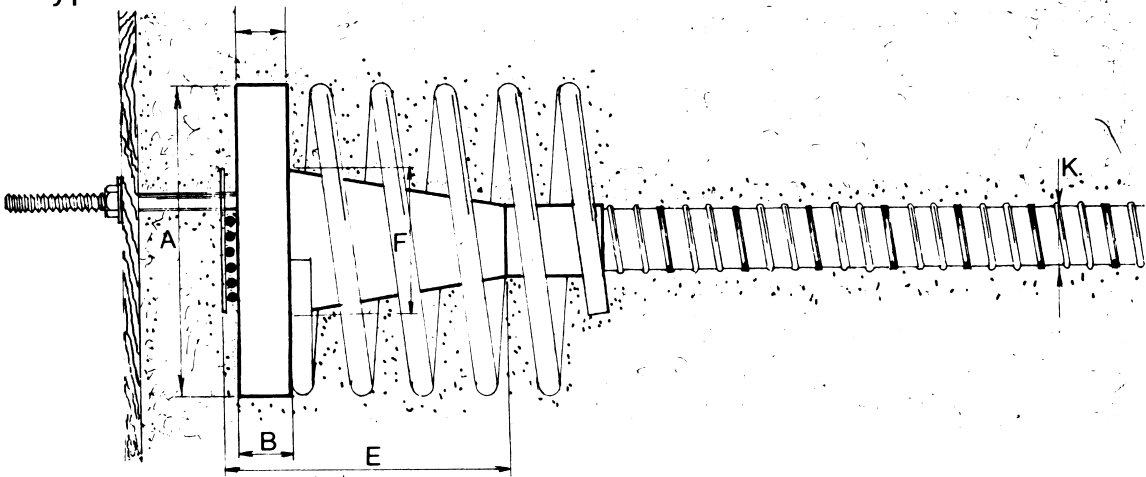
All dimensions are in mm.



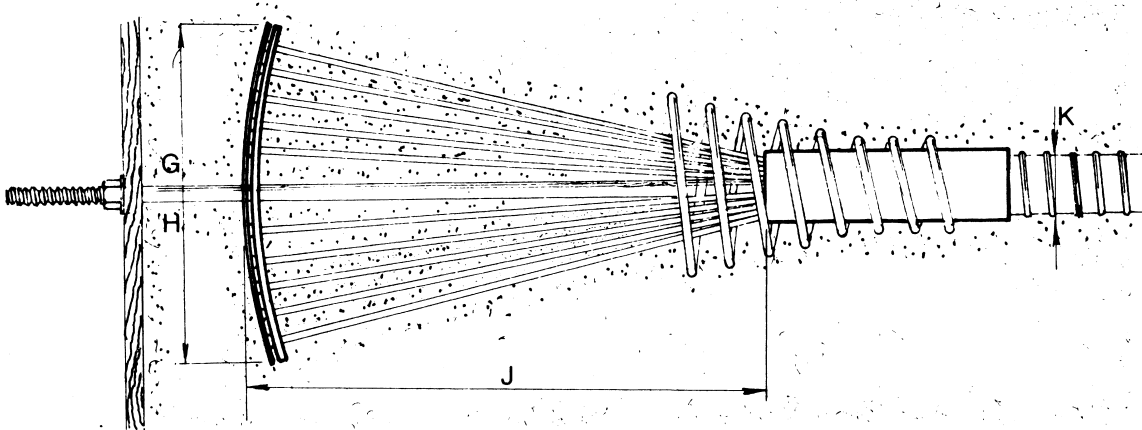
BBRV FIXED ANCHORS

SHEET NO. V.9

Type F



Type E

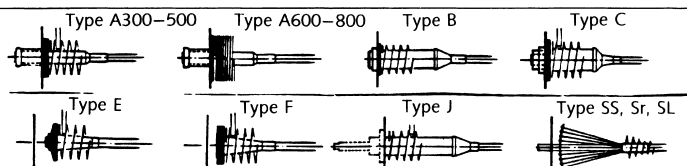


Type S

SMALL CAPACITY TENDONS UP TO 2658 kN

SHEET NO. V.10

kN	kN	No.	Type	Type	Fixed	Anchorage		Combined bearing plate		Stressing equipment clearance dimensions with dynamometer		
						Ref. No.	mm	mm	mm		mm	mm
						a × b	c × d	e	f × g dia.	h × g dia.		
483	387	8	B			32	150 × 150	138 × 138	76	30	1524 × 310	1344 × 310
			J				150 × 150					
			F				120 × 120					
			SR				120 × 150					
			SL				60 × 300					
966	773	16	B			64	175 × 175	171 × 171	89	40	1524 × 310	1344 × 310
			J				175 × 175					
			F				160 × 160					
			SR				150 × 220					
			SL				80 × 400					
1450	1160	24	B			100	200 × 200	197 × 197	108	50	1524 × 310	1344 × 310
			J				200 × 200					
			F				220 × 220					
			SS				220 × 220					
			SR				160 × 300					
1872	1498	31	C			130	250 × 250	235 × 235	127	55	1524 × 340	1324 × 340
			E				235 dia.					
			SR				260 × 260					
			SR				180 × 360					
			SL				120 × 560					
2054	1643	34	B			138	250 × 250	235 × 235	127	55	1524 × 340	1324 × 340
			J				250 × 250					
			F				260 × 260					
			SS				260 × 260					
			SR				180 × 360					
2537	2029	42	C			170	280 × 280	241 × 267	152	65	1880 × 440	1580 × 440
			E				270 dia.					
			SR				300 × 300					
			SR				200 × 450					
			SL				140 × 650					
3322	2658	55	C			220	300 × 300	267 × 305	152	75	1880 × 440	1580 × 440
			E				300 dia.					
			SR				340 × 340					
			SR				220 × 500					
			SL				160 × 700					



TYPE 'F' FIXED ANCHORAGE

SHEET NO. V.11

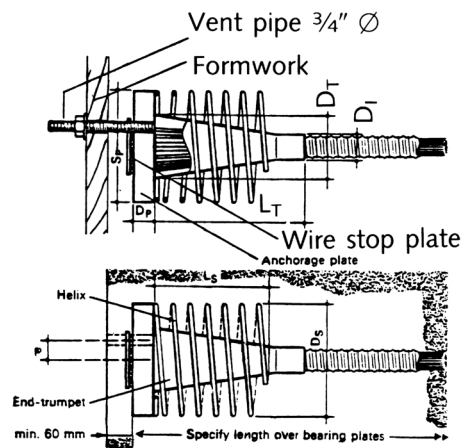
A fixed anchorage consisting of a rectangular steel plate drilled to receive the individual button-headed wires seated directly on this plate. A thin cover plate retains the button-heads during fixing.

With this anchorage, the prestressing force is transferred to the concrete through the plate.

Type 'F' anchorages are normally used in connection with movable anchorages of the Type 'B' series. If there are several tendons in one part of a structure, it is advisable to put half the Type 'F' anchorages at one end and half at the other.

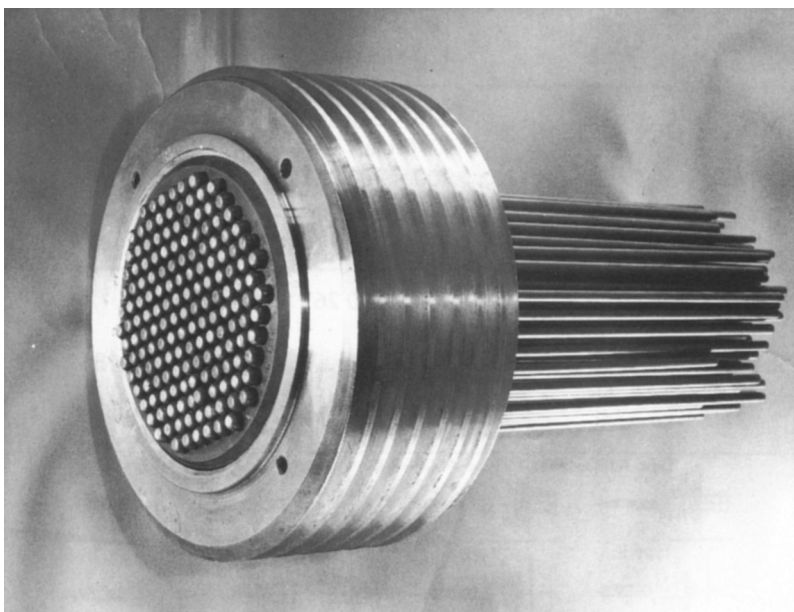
The anchorage is fastened to the formwork with a pipe which serves at the same time as a vent or as a grout connection.

This pipe passes through the plate and should be well-greased prior to fixing to facilitate removal after grouting.



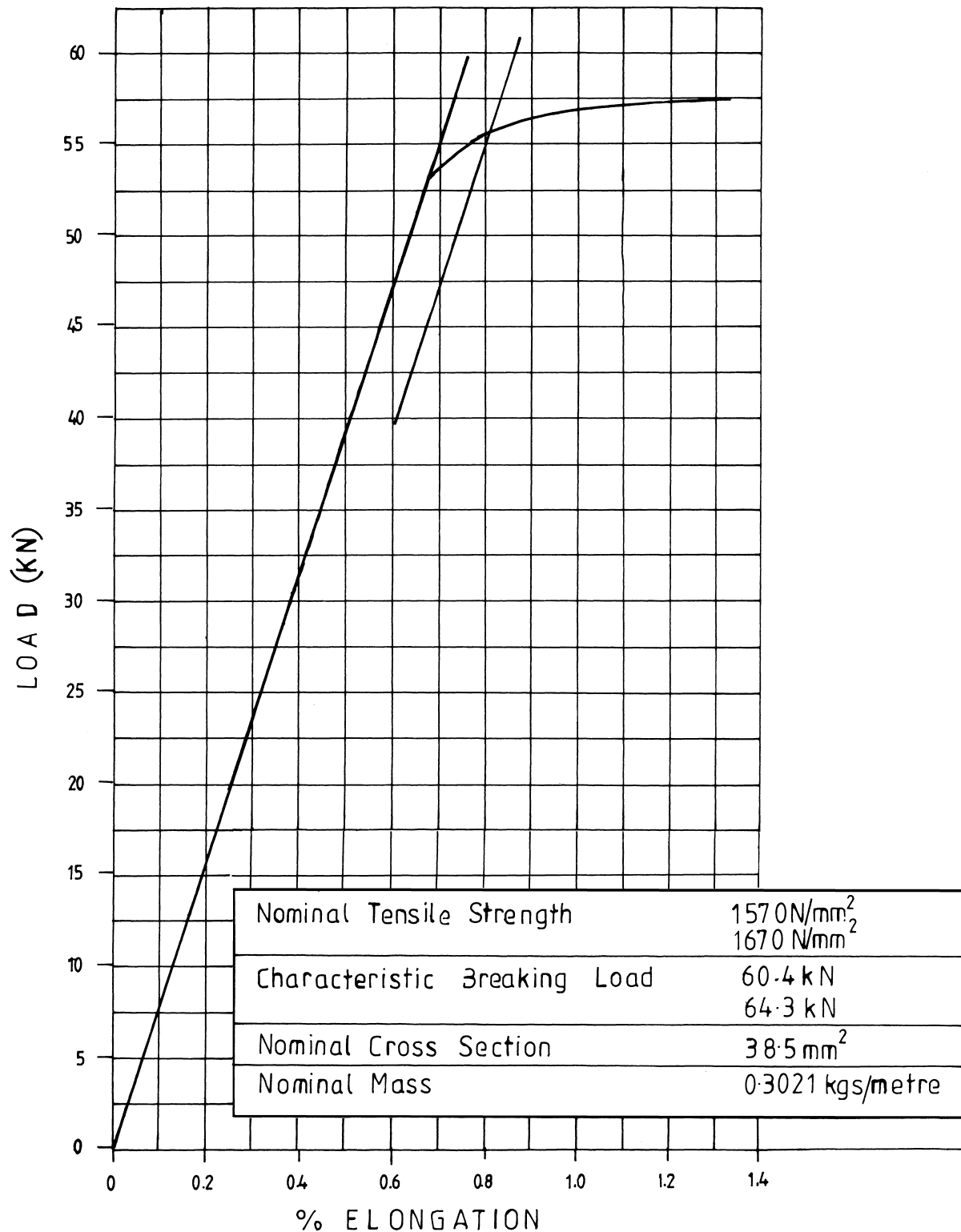
Type F anchorage	Type designation	F32	F64	F100	F138				
Steel wires per anchorage, maximum number 7 mm (0.276") dia.		8	16	24	34				
		mm	in	mm	in	mm	in	mm	in
Anchorage	side length	S _P	120 4.72	160 6.30	220 8.66	260 10.24			
	thickness	D _P	15 0.59	25 0.98	40 1.58	50 1.97			
End trumpet	outer diameter	D _T	87 3.43	112 4.41	128 5.04	148 5.83			
	connection, outer dia.	D ₁	35 1.38	45 1.77	55 2.17	60 2.36			
	length	L _T	200 7.87	250 9.84	300 11.81	350 13.78			
Helix	outer diameter	D _S	120 4.72	160 6.30	220 8.66	260 10.24			
	length (approx.)	L _S	250 9.84	250 9.84	250 9.84	250 9.84			
Distance of centre vent pipe to tendon axis		e	0 0	0 0	35 1.38	45 1.77			

BBRVF 163 No/7 mm Tendon



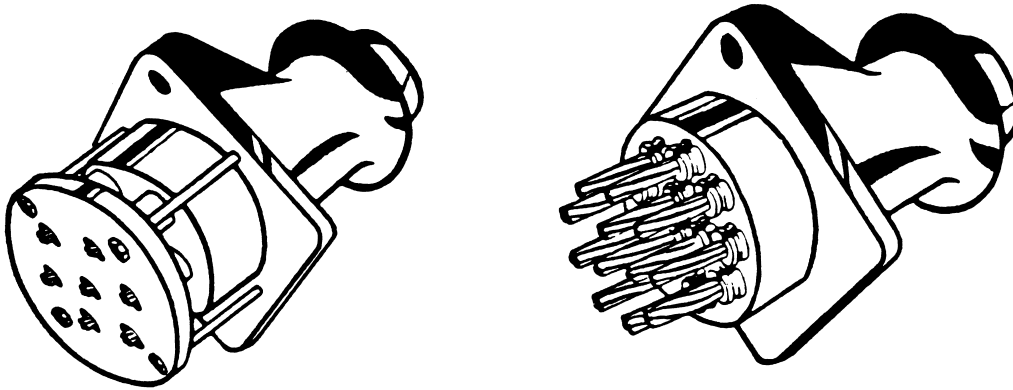
**TYPICAL TENSILE LOAD EXTENSION GRAPH
FOR 7 mm DIA. PLAN HIGH TENSILE WIRE**

SHEET NO. V.12



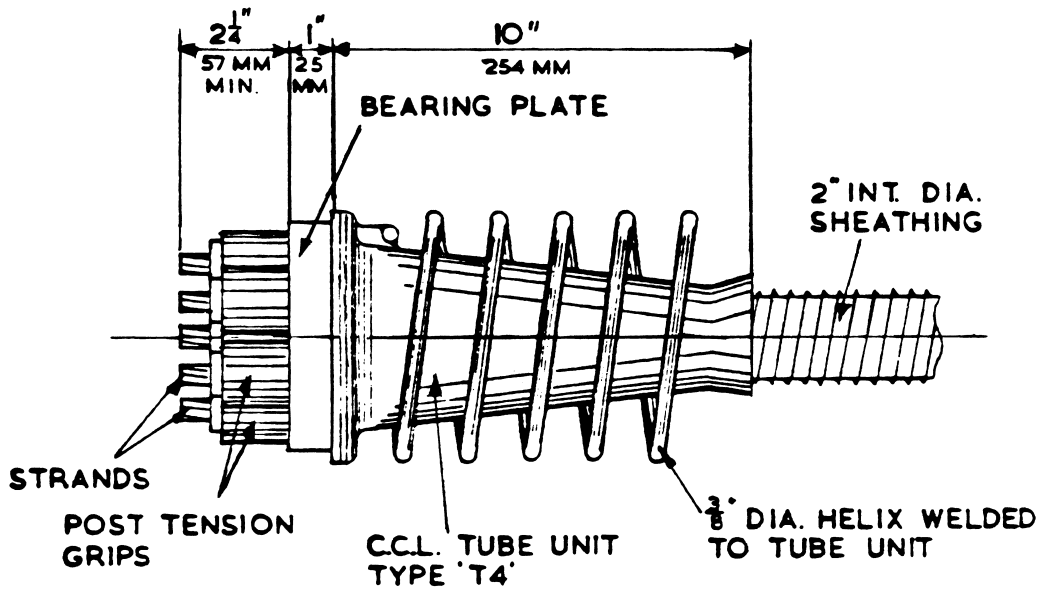
CCL SYSTEM

SHEET NO. V.13

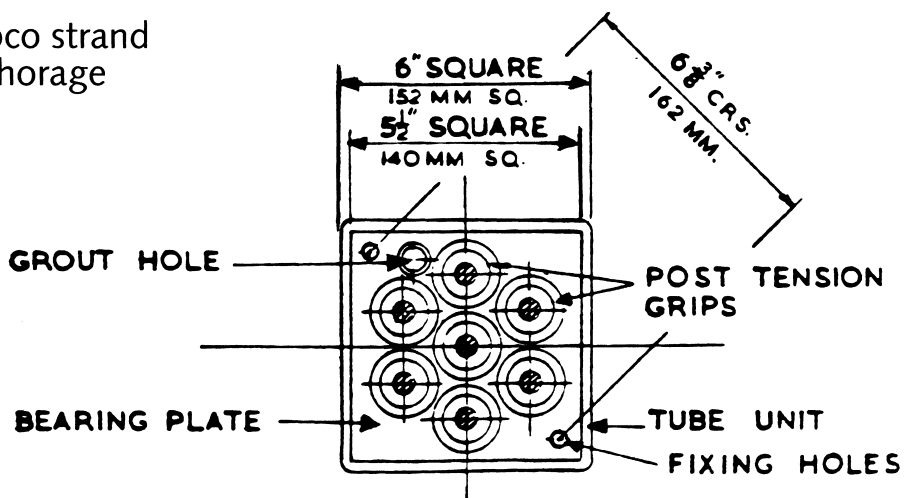


CABCO/MULTIFORCE
DEAD END (BURIED) ANCHORAGE

CABCO/MULTIFORCE
STRESSING ANCHORAGE



Cabco strand anchorage



CABCO PRESTRESSING TENDON MAIN DATA**SHEET NO. V.14**

Anchorage	System	Tendon*	Tendon Forces (kN)			Anchorage size mm	Sheath internal dia. mm
			P _k	0.8 P _k	0.7 P _k		
U1	Cabco	4/13 STD.	660.0	528.0	462.0	130	42
		4/13 SUP.	736.0	588.8	515.2	"	"
		4/13 DYF.	836.0	668.8	585.2	"	"
U2	Cabco	4/15 STD.	908.0	726.4	635.6	175	51
		4/15 SUP.	1000.0	800.0	700.0	"	"
		4/15 DYF.	1200.0	960.0	840.0	"	"
		7/13 STD.	1155.0	924.0	808.5	"	"
		7/13 SUP.	1288.0	1030.4	901.6	"	"
		7/13 DYF.	1453.0	1170.4	1024.1	"	"
		4/18 STD.	1480.0	1184.0	1036.0	175	51
		4/18 DYF.	1520.0	1216.0	1064.0	"	"
		7/15 STD.	1589.0	1271.2	1112.3	215	75
U3	Cabco	7/15 SUP.	1750.0	1400.0	1225.0	"	"
		7/15 DYF.	2100.0	1680.0	1470.0	"	63
		12/13 STD.	1980.0	1584.0	1386.0	215	81
	Multiforce	12/13 SUP.	2208.0	1766.4	1545.6	215	"
		12/13 STD.	1980.0	1584.0	1386.0	215	"
		12/13 SUP.	2208.0	1766.4	1545.6	215	"
U4	Cabco	7/18 STD.	2590.0	2072.0	1813.0	245	81
		7/18 DYF.	2660.0	2128.0	1862.0	"	75
		12/13 DYF.	2508.0	2006.4	1755.6	"	75
		12/15 STD.	2724.0	2179.2	1906.8	"	81
		12/15 SUP.	3000.0	2400.0	2100.0	"	81
		12/15 DYF.	3600.0	2880.0	2520.0	265	81
U5	Multiforce	19/13 STD.	3135.0	2508.0	2194.5	"	"
		19/13 SUP.	3496.0	2796.8	2447.2	"	"
		12/15 STD.	2724.0	2179.2	1906.8	245	"
		12/15 SUP.	3000.0	2400.0	2100.0	245	"
		12/15 DYF.	3600.0	2880.0	2520.0	265	"
		25/13 STD.	4125.0	3300.0	2887.5	300	90
U6	Multiforce	25/13 SUP.	4600.0	3680.0	3220.0	"	"
		13/15 DYF.	3900.0	3120.0	2730.0	"	84
		31/13 STD.	5115.0	4092.0	3580.5	335	100
U7	Multiforce	31/13 SUP.	5704.0	4563.2	3992.8	"	"
		19/15 STD.	4313.0	3450.0	3019.1	"	"
		19/15 SUP.	4750.0	3800.0	3325.0	"	"
		19/15 DYF.	5700.0	4560.0	3990.0	"	"
U8	Multiforce	19/18 DYF.	7220.0	5776.0	5054.0	"	"
		5/18 STD.	1776.0	1480.0	1295.0	362 × 89	114 × 25
		5/18 DYF.	1824.0	1520.0	1330.0	"	"
		10/18 STD.	3530.0	2960.0	2590.0	362 × 171	" twin
		10/18 DYF.	3648.0	3040.0	2660.0	"	" twin

* STD. Standard
 SUP. Super Strand
 DYF. Dyform Strand

A conventional steel in the form of reinforcing cages or helicals is provided below the anchorages in concrete to take much of the bursting effects. Codes give assistance in the design of such reinforcement, known as the *anchorage reinforcement*. The end areas where the anchorages rest are known as *anchorage* or *end blocks*. The purpose of such reinforcement is to transfer forces from anchorages smoothly into the concrete without causing internal cracks. The prestressing loads are affected by losses, elastic shortening of concrete, shrinkage and creep of concrete, relaxation of steel, anchorage slip and friction and wobbling effects. Again codes are consulted on these losses due to short- and long-term loads.

In order to sustain effectively the bending moments, deflections and shear, particularly in post-tensional systems, the tendon should be given a profile over its length where parasitic or secondary moments are to be avoided in a continuous structure. The tendon or cable shall then have a *concordant profile*.

V.2. Prestressing systems, tendon loads and material properties

V.2.1. Available systems

- (a) Wire/strand directly tensioned.
- (b) Macalloy System using high tensile bars.
- (c) Freyssinet System (France).
- (d) BBRV System (Switzerland).
- (e) CCL System (Britain).
- (f) KA System (Germany).
- (g) VSL System (Switzerland).

The details of the above systems are given below.

Sheet No. V.1 gives data for strand characteristics for standard, super and Dyform strands. Sheet No. V.2 is a table giving basic data for Macalloy bars listing tendon sizes, characteristic loads, bearing plates and duct sizes.

Sheet No. V.3 gives further data on cold worked high tensile bars and cold drawn and pre-straightened wire used generally in pretensioned concrete structures. Sheet No. V.4 is a monogroup for a well-known 15/15 mm normal strand system used in the Freyssinet System. A similar monogroup exists for other strand sizes. Sheet No. V.5 gives the BBRV System where a single tendon formed from individual wires is used. Sheet Nos V.6 to V.9 give comprehensive data on BBRV tendon systems. Data for the small capacity BBRV systems are given on Sheet No. V.10. Sheet No. V.11 shows the large tendon developed by BBRV to initially take up a 1000 t prestressing load. This type of tendon is used in the Dungeness B prestressed concrete pressure vessels. The stress-strain curve of this tendon and its basic data are given on Sheet No. V.12. Sheet Nos V.13 and V.14 give structural detailing and data for Cabco prestressing tendons manufactured by Cable Cover Ltd.

V.3. Structural detailing of prestressed concrete structures

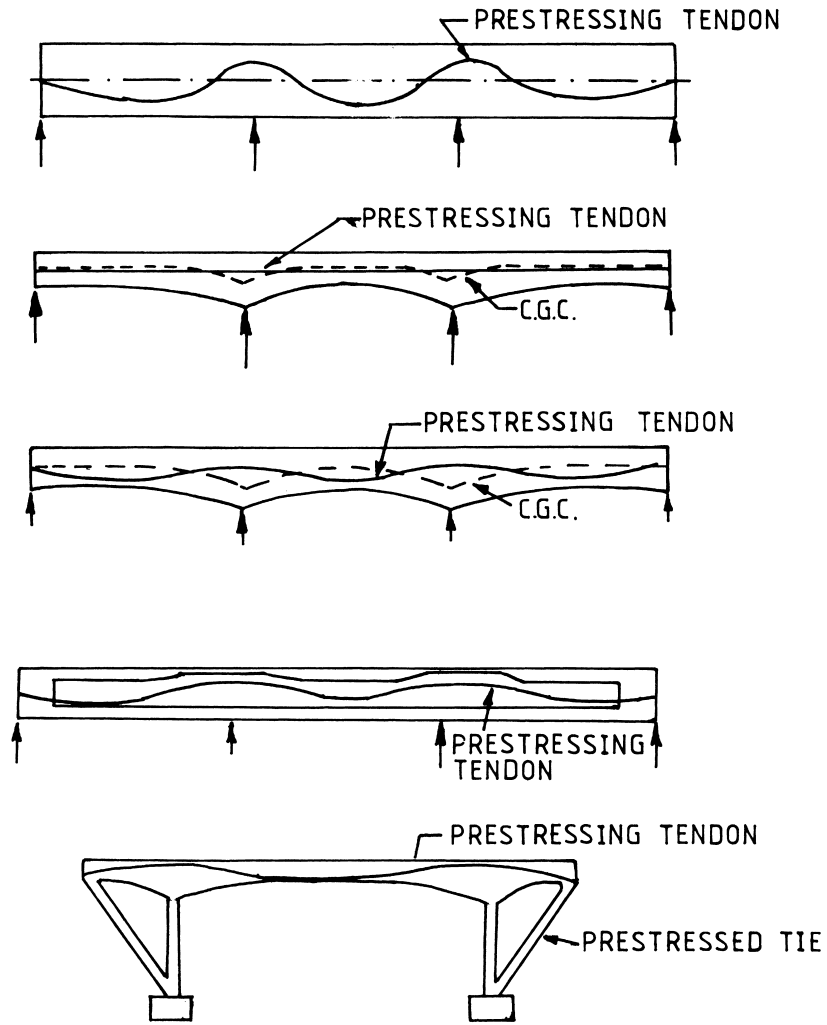
V.3.1. Detailing based on British codes

Prestressed concrete structural detailing of major structures is given later. In this section the reader is familiarized with basic problems. The first object is to lay out cables. Here many variations exist. Sheet Nos V.15 and V.16 show the tendon layouts for continuous beams, precast prestressed elements and cast-in-place (CIP) slabs on precast prestressed beams with continuous post-tensioned tendons. The prestressing systems given in this section are applicable to British, European and American practices.

Many variations exist in the design and detailing of prestressed beams. Sheet No. V.17 gives a detailed cross-section of a partially prestressed concrete beam. Sheet No. V.18 gives a typical example of anchorage

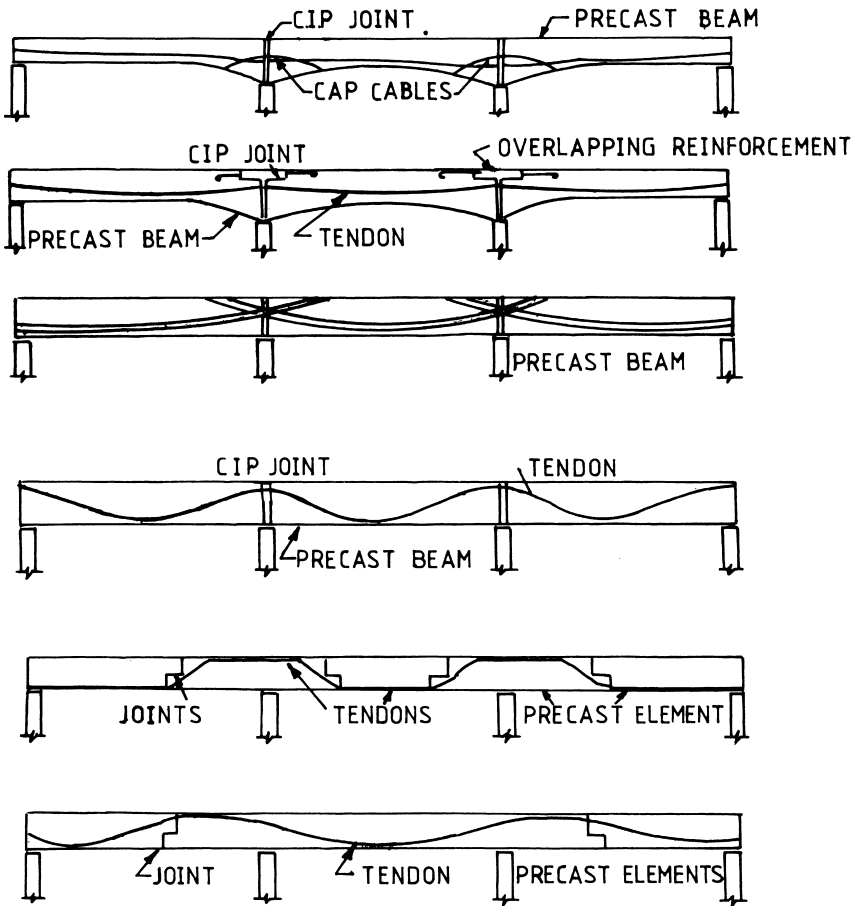
TYPICAL CONTINUOUS PRESTRESSED BEAMS

SHEET NO. V.15

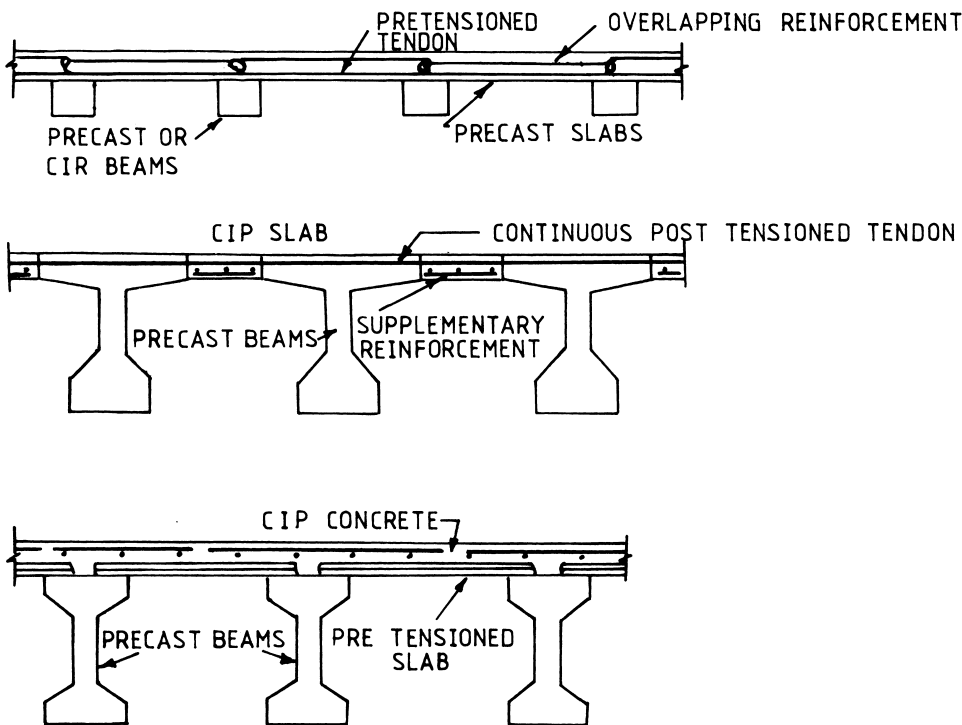


CONTINUOUS BEAMS OF PRECAST ELEMENTS

SHEET NO. V.16

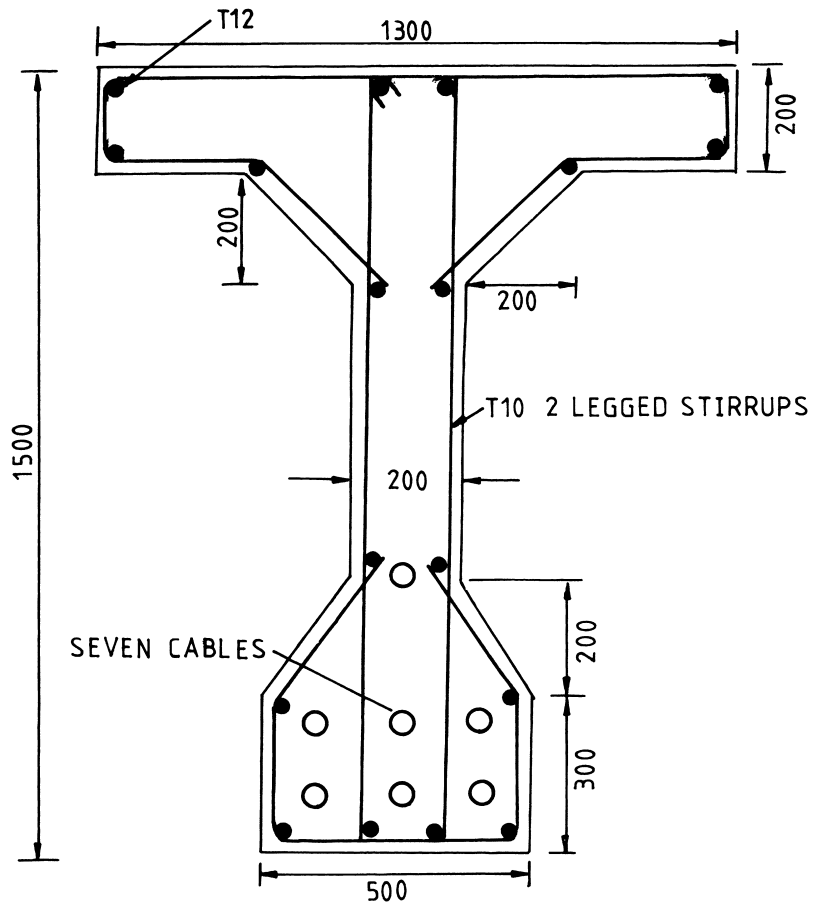


CONTINUOUS PRESTRESSED SLABS



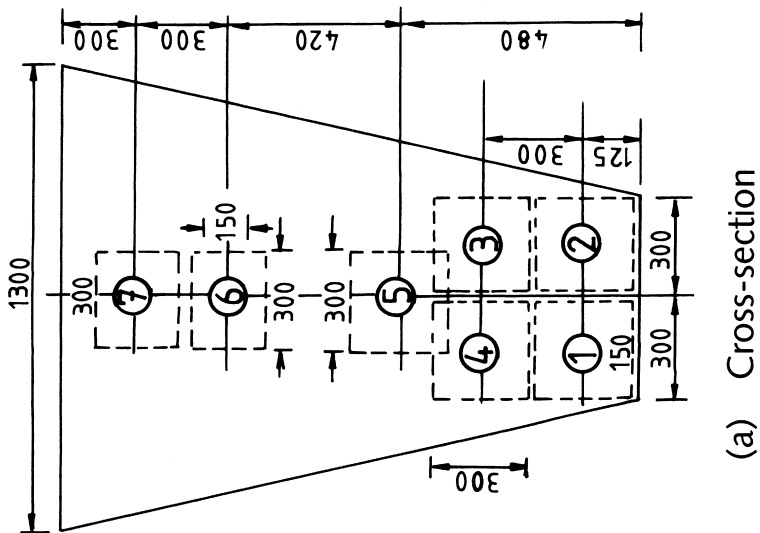
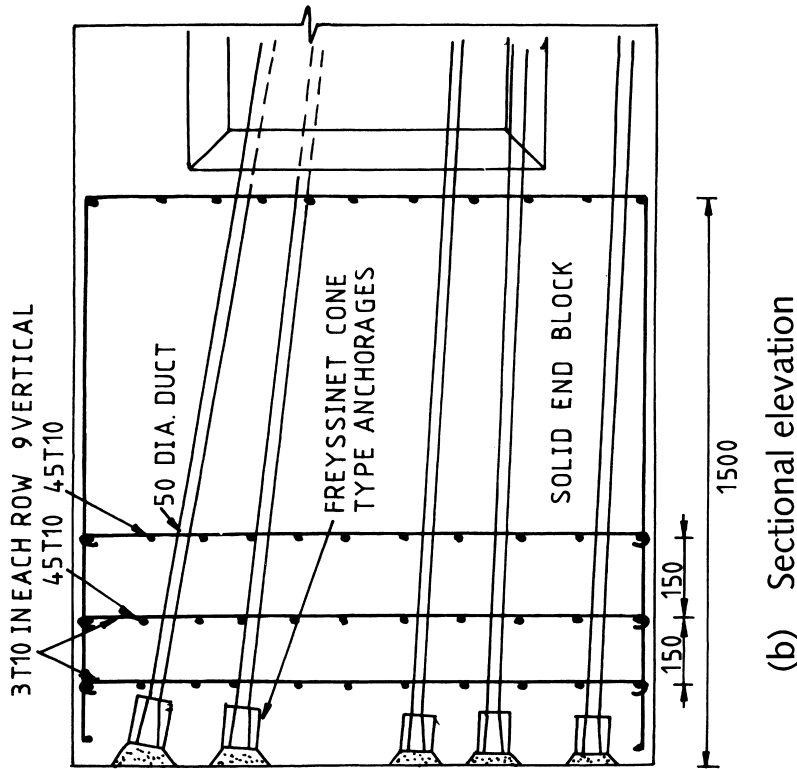
PARTIALLY PRESTRESSED CONCRETE BEAM

SHEET NO. V.17



ANCHORAGE ZONE REINFORCEMENT

SHEET NO. V.18



reinforcement and tendon details of a beam using Freyssinet cone type anchorages. They are based on BS 8110 and associated British codes and practices.

V.3.2. Detailing based on Eurocode 2

Refer to Sections V.1 and V.2 for prestressing systems and other relevant data. Some symbols that changed in the Eurocode 2 (EC2) are given below for comparison, together with some changes for material properties of prestressing steel.

V.3.2.1. Notation

F_{px}	Ultimate resisting force due to prestressing tendons and reinforcement in a cracked anchorage zone.
K	Unintentional angular displacement (per unit length) related to the profile of the tendons.
l_{bd}	Anchorage length over which the tendon force in pretensioned members is fully transmitted to the concrete.
l_{bp}	Transmission length, over which the prestressing force is fully transmitted to the concrete.
l_{bpd}	Design value for transmission length.
l_{bpo}	Length of a neutralised zone at the ends of pretensioned members, in the case of sudden release.
$l_{p,eff}$	Dispersion length, over which the concrete stresses gradually disperse to a linear distribution across the section (effective transfer).
n_1	Total number of wires or strands in a tendon.
n_2	Number of wires or strands transferring the radial load of all wires or strands in the tendon to the deviator.
z_{cp}	Distance between the centre of gravity of the concrete section and the tendons.
α	E_s/E_{cm} .
θ	Sum of angular displacements over a distance x (irrespective of direction or sign).
β_b	Coefficient relating transmission length of prestressing tendons to concrete strength.
$\epsilon_s(t, t_o)$	Estimated shrinkage strain.
$\sigma_{o,max}$	Maximum stress applied to a tendon.
$\sigma_{p,m,o}$	Stress in the tendon immediately after stressing or transfer.
σ_{pgo}	Initial stress in the tendons due to prestress and permanent actions.
σ_{cg}	Stress in the concrete adjacent to the tendons, due to self-weight and any permanent actions.
σ_{cpo}	Initial stress in the concrete adjacent to the tendons, due to prestress.
$\Delta\sigma_{p,c+s+r}$	Variation of stress in the tendons due to creep, shrinkage and relaxation at location x , at time t .
$\Delta\sigma_{pr}$	Variation of stress in the tendons at section x due to relaxation.

V.3.2.2. Material properties

Anchorage or coupler assemblies of tendons

1. Tendon anchorage assemblies and tendon coupler assemblies satisfying the performance requirements of Clause 3.4.1.2 of EC2 may be considered to withstand the full characteristic strength of the tendon.

Technological properties of prestressing steel

Relaxation

1. Certificates accompanying the consignments shall indicate the class and relevant relaxation data of the prestressing steel.
2. For design calculations, the values, which may be taken into account for losses at 1000 h are either those given in the certificate or those assumed in Figure 4.8 of the code for the three classes of steel shown. The long-term values of the relaxation losses may be assumed to be |three| times the relaxation losses after 1000 h.
3. An indication of how relaxation losses increase between 0–1000 hours is given in Table V.1.

Table V.1. Indication of relationship between relaxation losses and time up to 1000 hours

Time: h	1	5	20	100	200	500	1000
Relaxation losses as percentages of losses after 1000 h	15	25	35	55	65	85	100

4. Relaxation at temperatures of the structure over 20°C will be higher. A reference is made to the code.
5. Short-term relaxation losses at a temperature of the structure exceeding 60°C can be two to three times those at 20°C. However, in general, heat curing, over a short period, may be considered to have no effect on long-term relaxation results.

Loss of prestressing forces

Also refer to Sections V.1 and V.2.

Between columns the ribs are interrupted and solid strips are created where prestressing tendons are concentrated. This solution is also convenient for the design of the flat slab reinforcement for the effects of the earthquake action.

The main design criteria were to control the deflections due to permanent loads with the prestressing forces, and to limit the increase of long-term deflections to 25 mm for frequent load combination.

Initial prestressing force

1. The initial prestressing force shall be determined in accordance with Section 2.5.4 of EC2, which also lists relevant factors affecting loss of prestress.
2. The maximum force applied to a tendon P_o (i.e. the force at the active end, immediately after stressing, $x = 0$) shall not exceed $A_p \times \sigma_{o,max}$, where:

A_p is the cross-sectional area of the tendon and

$\sigma_{o,max}$ the maximum stress applied to the tendon

$$\sigma_{o,max} \leq \boxed{0.80} f_{pk} \text{ or } \leq \boxed{0.90} f_{p01k},$$

whichever is the lesser (V.1)

3. The prestressing force ($P_{m,o}$) applied to the concrete immediately after tensioning (post-tensioning) or after transfer (pre-tensioning) shall not exceed the lesser of the forces determined from:

$$A_p \times \sigma_{p,m,o} \leq \boxed{0.75} f_{pk} \times A_p, \text{ or } \leq \boxed{0.85} f_{p01k} \times A_p \quad (V.2)$$

where $\sigma_{p,m,o}$ is the stress in the tendon immediately after tensioning or transfer.

4. For pre-tensioned members, $P_{m,o}$, in P (3) above, is calculated from Equation (V.3) below:

$$P_{m,o} = P_o - \Delta P_c - \Delta P_{ir} [- \Delta P\mu(x)] \quad (V.3)$$

where ΔP_c and $\Delta P\mu(x)$ are defined and ΔP_{ir} is the short-term relaxation loss.

5. For post-tensioned members, $P_{m,o}$ is calculated by:

$$P_{m,o} = P_o - \Delta P_{sl} - \Delta P_c - \Delta P\mu(x) \quad (V.4)$$

6. Methods for evaluating ΔP_{sl} , ΔP_c , ΔP_{ir} and $\Delta P\mu(x)$ are given in Section 4.2.3.5.5 of the code.

Note: types of other losses for particular prestressing systems are given in Sections V.3.1 and V.3.2.

Transmission length of prestressing strands and wires

At the transfer, the transmission length of the prestressing strands and wires in relation to concrete strength is shown in Table V.2.

Table V.2.

Concrete strength at transfer: N/mm ²		25	30	35	40	45	50
β_b	Strands and indented wires	75	70	65	60	55	50
	Ribbed wires	55	50	45	40	35	30

Minimum strength class or prestressed normal weight concrete

1. The minimum class for post-tensioned members is [C25/3Q], and for pre-tensioned members is [C30/37].

V.3.2.3. Typical details

Sheet No. V.19 shows the longitudinal section of a deck segment continuous prestressing layout, cross-section of deck, with dimensions and reinforcement, and longitudinal cross-sections of one of the deck beams of the bridge with complete prestressing layout. Sheet Nos V.18 and V.20 show anchorage layouts for deck and pylon for a cable-stayed bridge. A typical example of the prestressing deck beam and ribbed slab are shown on Sheet No. V.21 and this detailing has been prepared by J. Appleton, J. Almeida, V. Lucio and A. Costa. The building in question was built in Lisbon in 1993 and holds the New Lisbon Stock Exchange. The building has 13 elevated floors and four underground floors with a total area of slab construction of 29 000 m².

The columns of a square section 0.7 m to 0.9 m wide are in general spaced 8.4 m in both directions and the ribbed slabs are 0.30 m thick. In the area of the Stock Exchange, a 16.8 m span was required and a 0.50 m prestressed ribbed slab was adopted. This slab was chosen to illustrate this example.

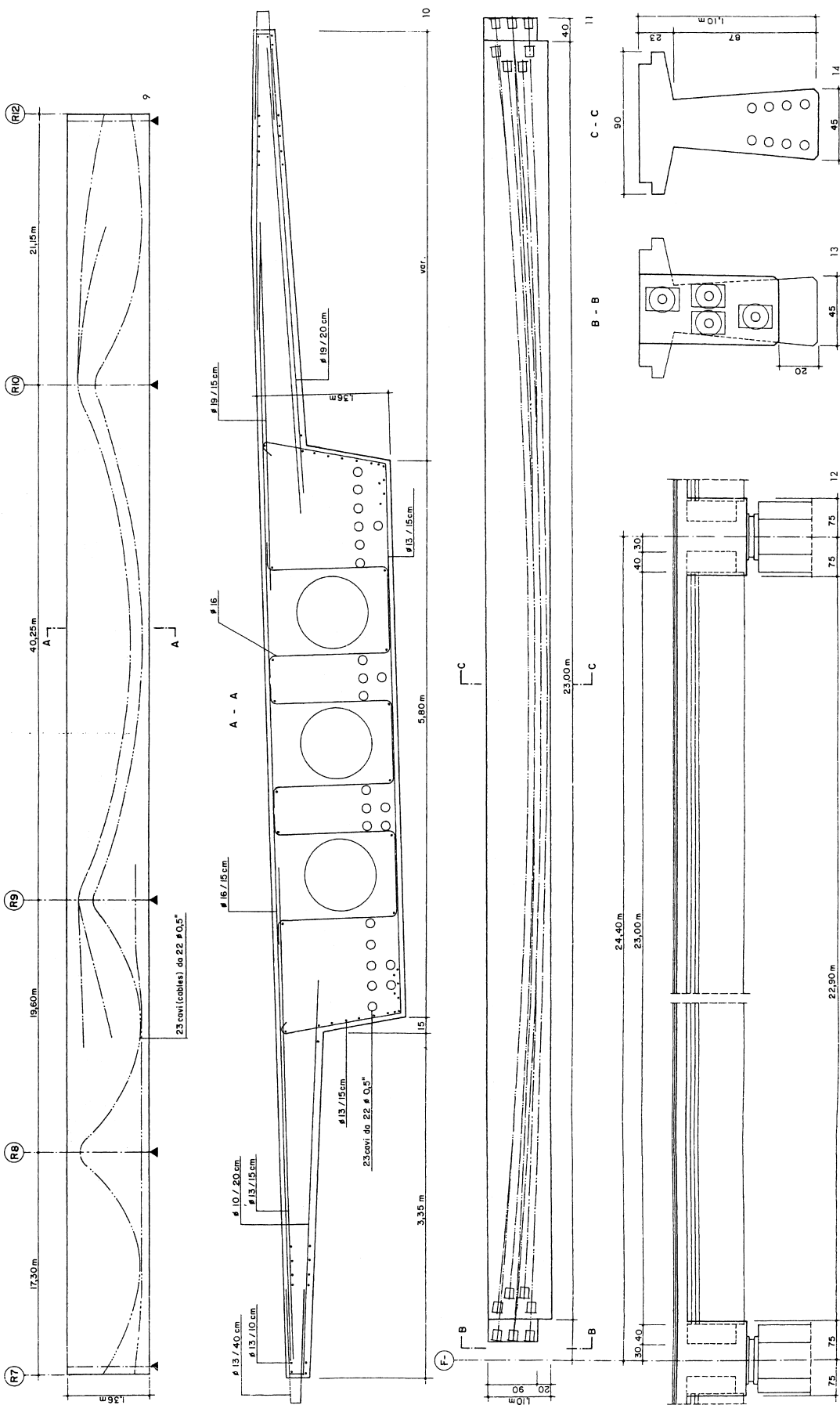
In Sheet No. V.21 a general layout of the slab and of the prestressing cables at level 01 are presented. For seismic resistance, shear walls were also introduced.

A slab with a 16.8 m span could only be conceived economically as a prestressed slab. The combination of a ribbed solution with prestressing reduces the weight of the structure and controls deflections in an effective way (Sheet No. V.21).

Although the slab mainly behaves in one direction, prestressing was also introduced in the transverse direction to control cracking caused by shrinkage

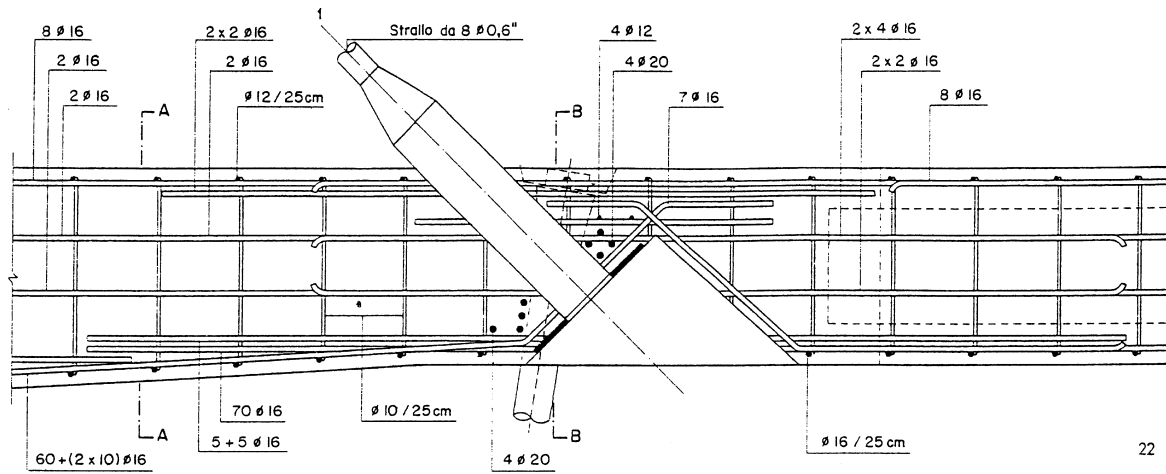
CONTINUOUS PRESTRESSED (BASED ON EC2)

SHEET NO. V.19

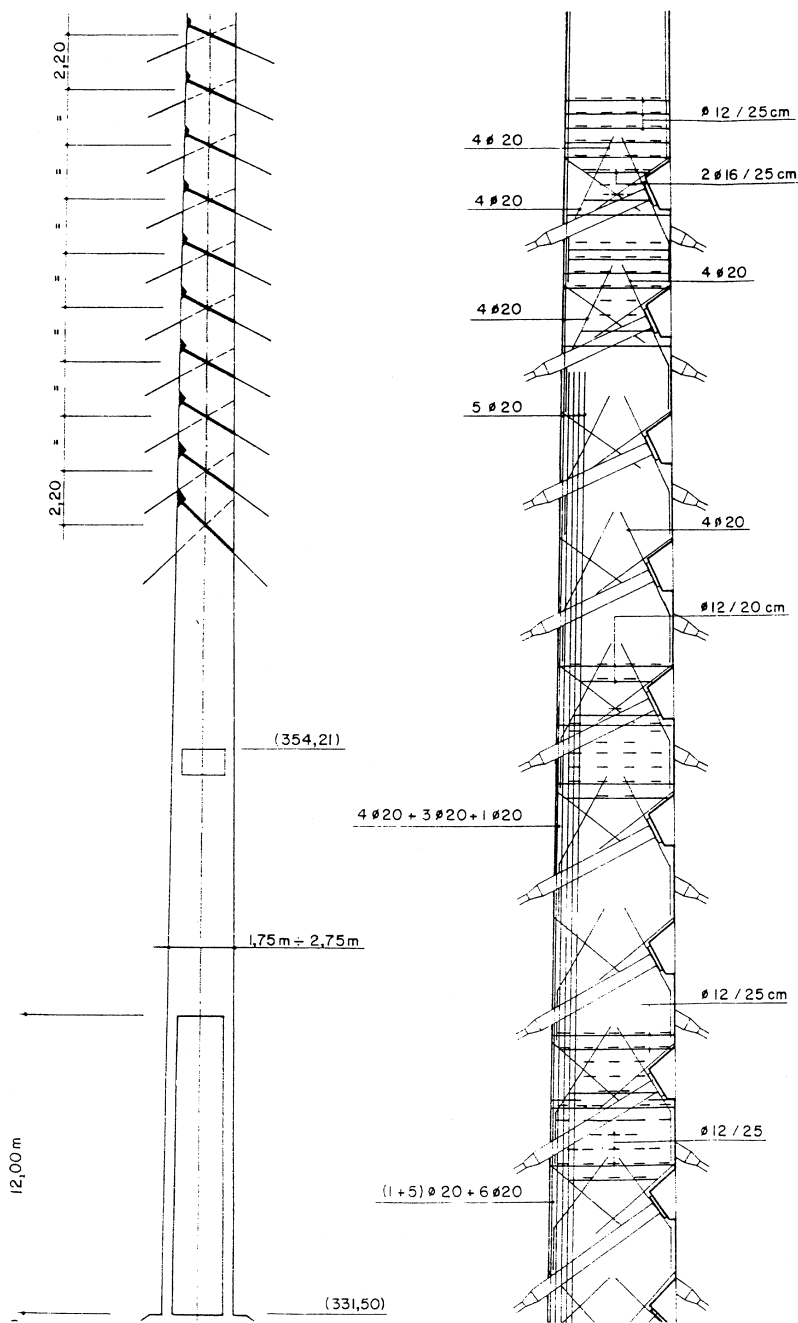


PRESTRESSED ANCHORAGE (BASED ON EC2)

SHEET NO. V.20

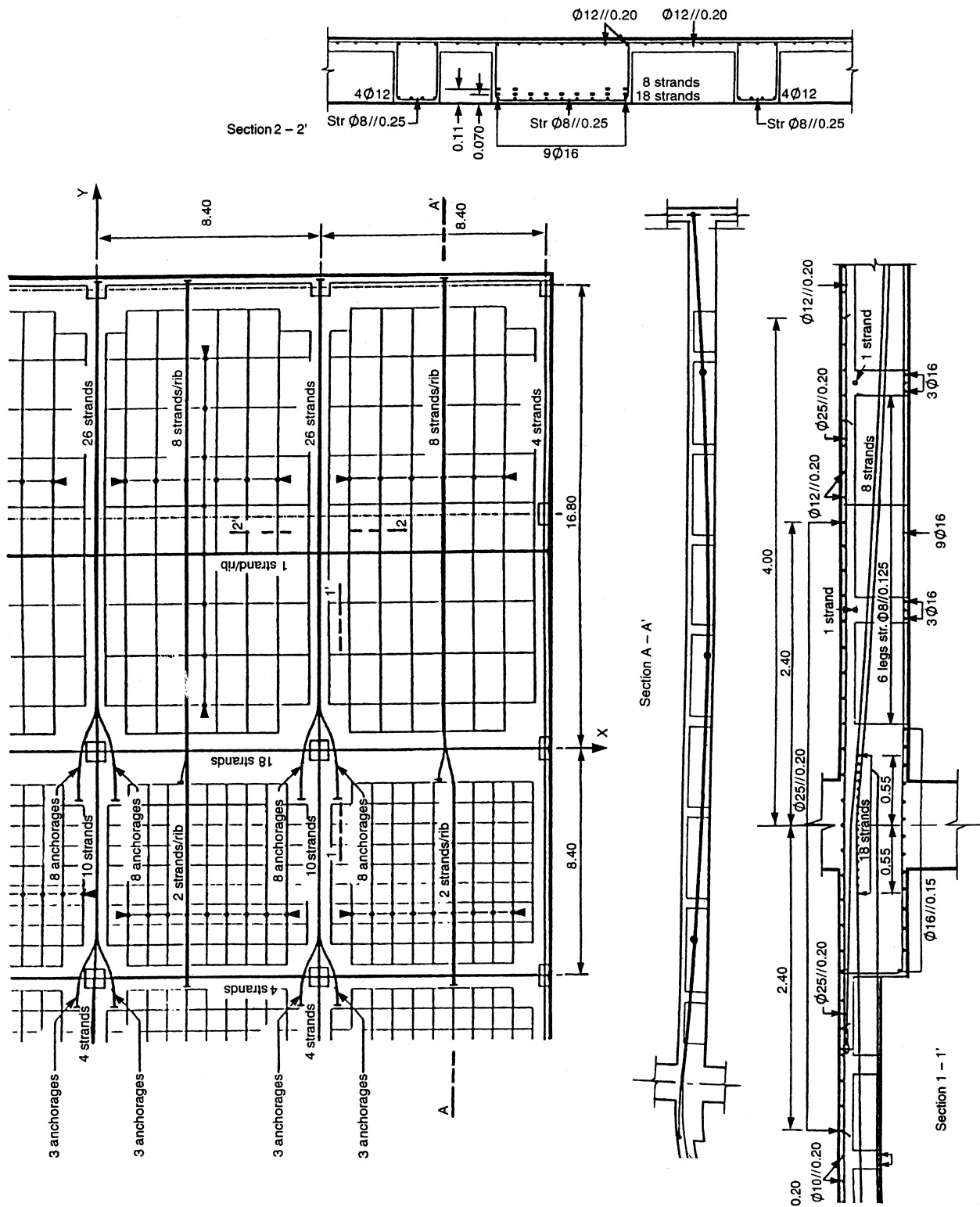


22



**PRESTRESSED CONCRETE RIBBED SLAB
(STOCK EXCHANGE LISBON)**

SHEET NO. V.21



and temperature. Unbonded prestressing monostrands were chosen due to ease of placement, to increase eccentricities and to avoid injection.

V.3.3. Detailing based on ACI and PCI codes and other American practices

V.3.3.1. Prestressing requirements

The provisions set forth in this section refer to the application and measurement of stresses to prestressed concrete members manufactured by the process of pre-tensioning, post-tensioning, or a combination of the two methods.

V.3.3.2. Tensioning of tendons

In all methods of tensioning, stress induced in the tendons shall be determined by monitoring applied force and independently by measurement of elongation. Applied force may be monitored by direct measurement using a pressure gauge piped into the hydraulic pump and jack system, dynamometer or load cell. The two control measurements shall agree with their computed theoretical values within a tolerance of $\pm 5\%$. If discrepancies are in excess of 5% between the two calculated forces, determined by elongation measurement and gauge reading, the tensioning operation shall be suspended and the source of error determined and evaluated by qualified personnel before proceeding. Additionally, the control measurements of force and elongation shall algebraically agree with each other within a 5% tolerance. If the measurements do not agree within 5%, a load cell may be added at the dead end and if force measurements agree within 5% between the gauge at the live end and the load cell at the dead end, the elongation agreement can be waived.

After an initial force has been applied to the tendon, reference points for measuring elongation due to additional tensioning forces shall be established. Location of reference points will vary with different methods of tensioning and with the physical characteristics of the equipment used.

Calculations for elongation and gauge readings must include appropriate allowances for friction in the jacking system, strand seating, movement of abutments, bed shortening if under load, thermal corrections, and any other compensation for the setup.

Hydraulic gauges, dynamometers, load cells or other devices for measuring the stressing load shall be graduated so they can be read within a tolerance of $\pm 2\%$. Gauges, jacks and pumps shall be calibrated as a system in the same manner they are used in tensioning operations. Calibrations shall be performed by an approved testing laboratory, calibration service or under the supervision of a registered professional engineer, and a certified calibration curve shall accompany each tensioning system. Pressure readings can be used directly if the calibration determines a reading is within a $\pm 2\%$ tolerance of actual load. Calibrations shall be performed at any time a tensioning system indicates erratic results, and in any case at intervals not greater than 12 months.

V.3.3.3. Tendons and anchorages

Requirements for all tendons (ASTM)

Certain fundamental requirements apply to all materials for prestressing tendons, including the following.

1. High ultimate tensile strength, approximately 150 000 to 160 000 psi for bars; 250 000 and 270 000 psi for strand; and 240 000 and 250 000 psi for wire.
2. Ductility as measured by an elongation of $3\frac{1}{2}\%$ minimum in a gauge length of 24 in. unless otherwise specified in ASTM or other applicable specifications.
3. Either mechanical or thermal stress relief of internal stresses induced by the processes of manufacturing high strength steels.

Tendon materials

Approved materials in common usage for prestressing tendons consist of the following.

1. Pretensioning:
 - (a) uncoated, stress-relieved strand, conforming to ASTM A 416, Grade 250 and Grade 270
 - (b) uncoated, low relaxation strand conforming to ASTM A 416 (Supplement) Grade 250 and Grade 270
 - (c) coated, stress-relieved strand ASTM 416, Grade 270 coated in accordance with ASTM Method G-12 tested in accordance with ASTM A 370.
2. Post-tensioning:
 - (a) strand as described above either singly or in multiple parallel strand units with wedge type or other adequate anchorages
 - (b) uncoated, stress-relieved wire conforming to ASTM A 421 in multiple parallel wire units with wedge-type, button head or other adequate anchorages
 - (c) high strength, stress-relieved bars conforming to ASTM A 722 with wedge type, threaded, or other adequate anchorages
 - (d) high strength, stress-relieved large cables with socketed or extruded ends fitted with anchorage nuts on the peripheries of the sockets.

Mill certificates from suppliers shall be on file at plant offices for tendon materials in current use.

Anchorage for post-tensioning

In brief, the basic requirements for tendon anchorages are as follows.

1. The anchorages for bonded tendons shall develop at least 95% of the minimum specified ultimate strength of the prestressing steel in an unbonded state without exceeding anticipated set.
2. The anchorages for unbonded tendons shall develop at least 95% of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set.
3. The minimum elongation of tendons tested in the unbonded state is not less than 2% when measured in a minimum gauge length of 10 ft (3.048 m).
4. Dynamic tests shall be performed on representative unbonded tendon specimens in conformance with PTI's *Guide Specifications for Post-Tensioning Materials*.

Post-tensioning anchorages not developing at least 95% of the guaranteed ultimate strength of the tendon shall be prohibited. The allowable jacking load during tensioning shall not exceed 80% of the strength of the prestressing steel nor 94% of the yield strength of the steel nor the maximum value recommended by the manufacturer of the steel or of the anchorages. Unless the tendon is effectively bonded to relieve the anchorage of stress fluctuation, the working stress shall also be a function of the strength of the anchorage.

Stress-strain or load-elongation curves

Wherever the terms stress-strain curves or relationship are used, it is to be understood that this implies either stress-strain or load-elongation. A reference is made to individual prestressing systems given in Section V.3.1 for stress-strain and load deformation curves of tendons.

Reinforcing steel and appurtenances

Steel bars

Steel reinforcing bars shall be of the designated sizes and grades and shall conform to the following applicable specifications as stipulated in the design:

ASTM A 184	Specification for Fabricated Concrete Reinforcement
ASTM A 615	Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
ASTM A 616	Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
ASTM A 617	Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
ASTM A 706	Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement

If bars other than the types listed above are to be used, their required properties shall be shown on the design drawings.

Steel wire

Steel wire reinforcement, other than tendons, shall conform to the following applicable specifications:

ASTM A 82	Specification for Cold-Drawn Steel Wire for Concrete Reinforcement
ASTM A 185	Specification for Welded Steel Wire Fabric for Concrete Reinforcement
ASTM A 496	Specification for Deformed Steel Wire for Concrete Reinforcement
ASTM A 497	Specification for Welded Deformed Steel Wire for Concrete Reinforcement

Manufacturing appurtenances

Form ties, inserts, bar chairs, spacers, bracing and similar appurtenances incidental to the manufacture of precast and prestressed concrete members shall be adequate for their intended purposes and of types resulting in a minimum marring of concrete surfaces.

Headed studs and deformed anchor studs

Studs for concrete anchors shall be manufactured in accordance with ASTM A 108 unless higher strengths are required by design.

VI. Composite construction, precast concrete elements, joints and connections

VI.1. Composite construction and precast elements

Composite construction consists of a combination of prefabricated unit and cast-in-situ concrete in a structure. The prefabricated unit may be in reinforced concrete, prestressed concrete or structural steel. To obtain composite action, shear connectors are placed in the form of studs, channels, spirals, etc., projecting from steel units such as steel beams or precast units. Their function is to transfer horizontal shear entirely from one element to another. The shear connectors are welded on to steel beams and concrete is cast in situ around them. In the case of precast concrete elements, such as beams and cast-in-situ slabs, a full horizontal shear is effected at the interface between these two elements when the deformation at the upper surface of the beam and the lower surface of the slab are the same.

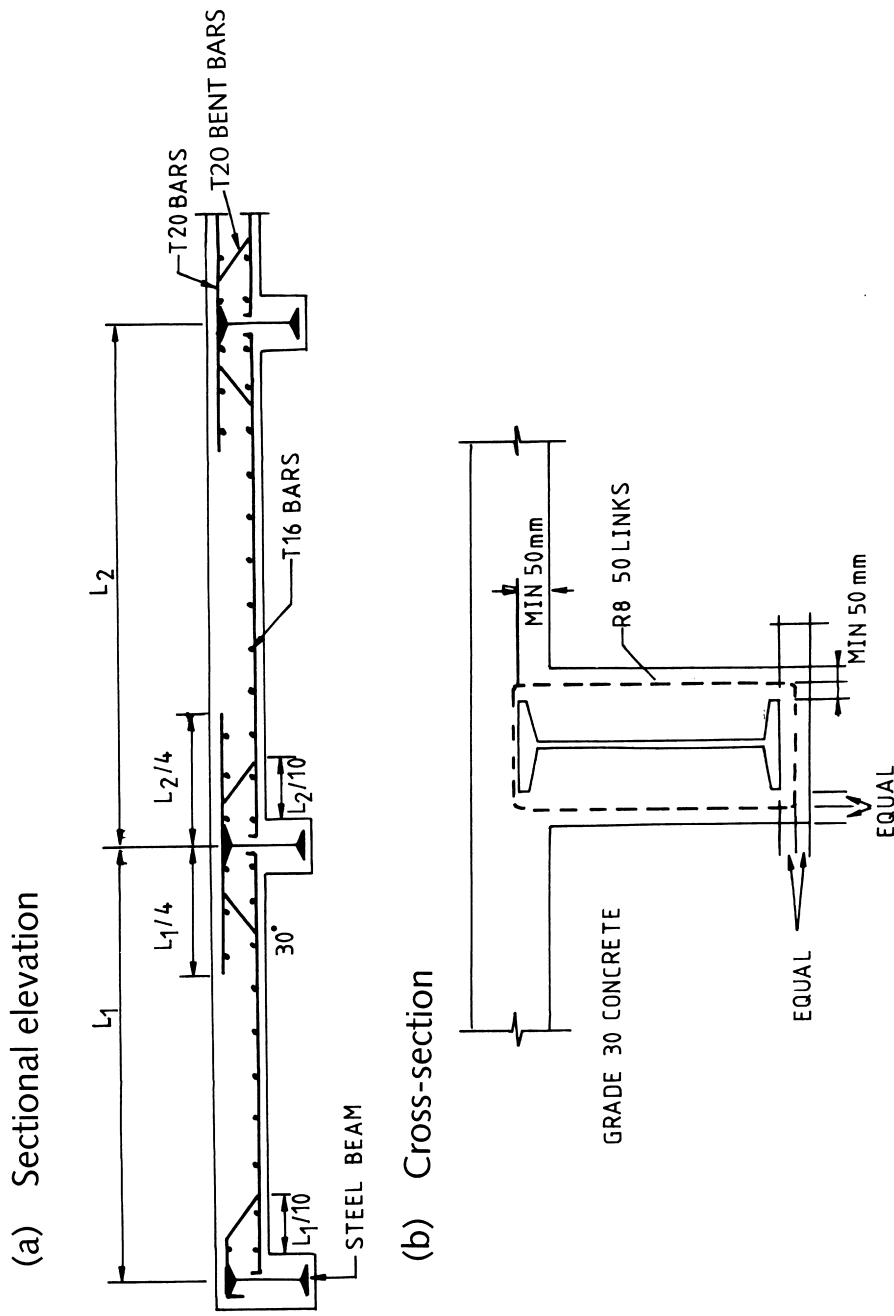
Precast construction consists of fabrication of various elements of a structure in a factory. Such a construction is commonly used in buildings and bridges. It results in an economy of formwork and scaffolding, economy in concrete, economy resulting from standardization and mass production of various elements and speedier construction. The disadvantages and shortcomings of such a construction are (a) transport costs, (b) the need for highly skilled labour, and (c) a reduction in the monolithic strength of the structure.

Sheet No. VI.1 shows a sectional elevation and a cross-section of a composite steel–concrete beam slab. The steel beam is encased in concrete. Codes give equations and specifications for designing and detailing such constructions.

Sheet No. VI.2 refers to composite connections between two identical precast beam elements. Both the isometric view and the sectional elevation show the two beams with opposite notches or nibs connected by mechanical fastenings or bolts. Shear stresses at the nib are reduced by the introduction of the bent bars. In addition, a bar cast in one end is projected outside to be inserted into the hole left in the other element. The two elements will achieve a monolithic structure of higher efficiency. A similar method is adopted by connecting two precast concrete beam elements by means of high tensile bars as tie rods leaving grooves at the top of the concrete and filling them with a specified filler. At the bottom a steel bar, as shown on Sheet No. VI.3, is welded to a steel plate which is then welded to a bearing plate on top of the precast column bonded by a steel bar. Sheet No. VI.4 shows a composite beam connected to the rib floor units using bars either with sleeve joints and links or bent bars welded to each other or a bar loop between the elements. In the latter case, in Sheet No. VI.4(a) the common plates are welded. An isometric detailing in 3D is given for main beams connected to secondary beams using inserted bars in Sheet No. 1.3(b). Typical cross-sections for VI.4(b) are given in VI.4(c).

COMPOSITE STEEL—CONCRETE BEAM SLAB

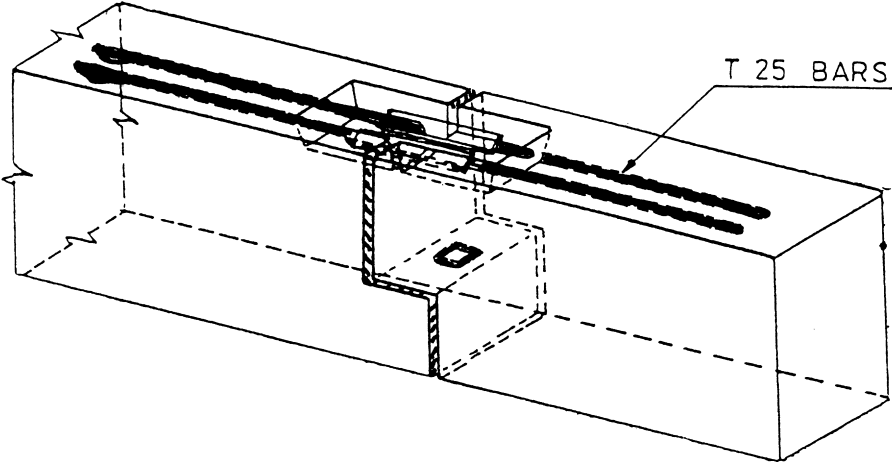
SHEET NO. VI.1



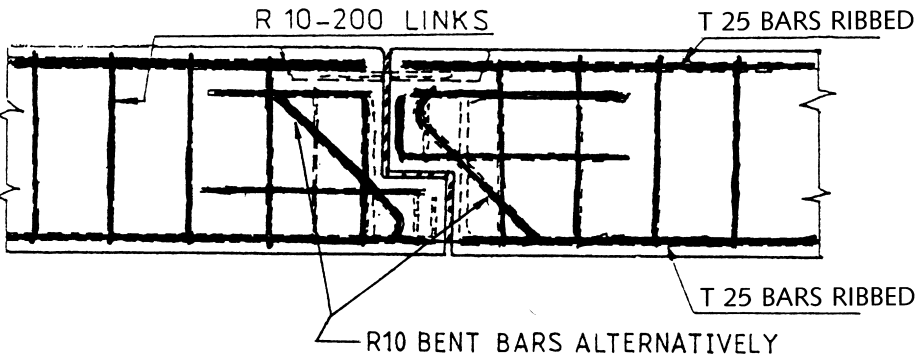
BEAM TO BEAM COMPOSITE CONNECTION (BARS WELDED)

SHEET NO. VI.2

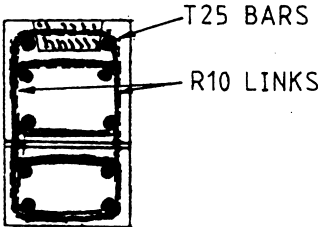
(a) Isometric view



(b) Sectional elevation

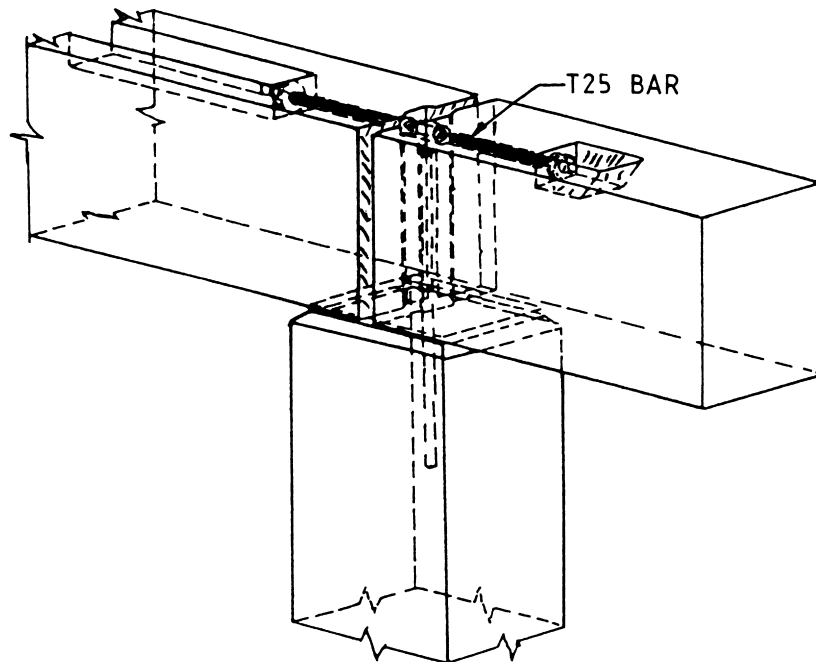


(c) Cross-section

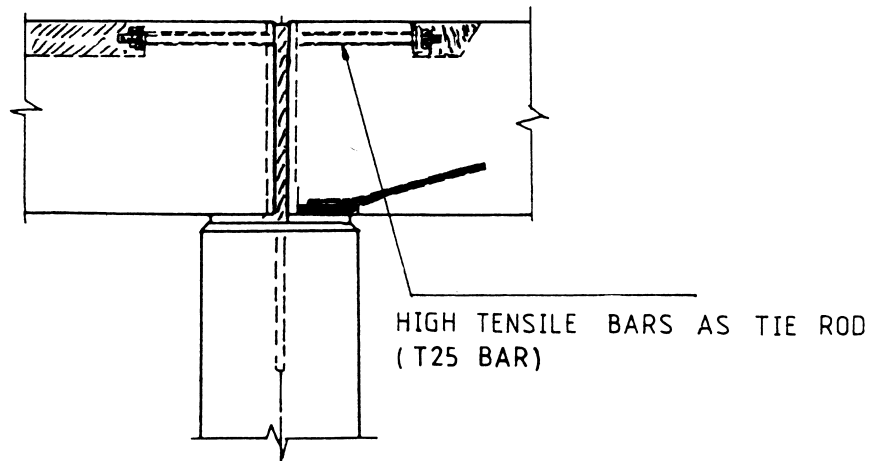


BEAM TO COLUMN CONNECTION USING HIGH TENSILE BARS AS TIE RODS SHEET NO. VI.3 AS TIE RODS

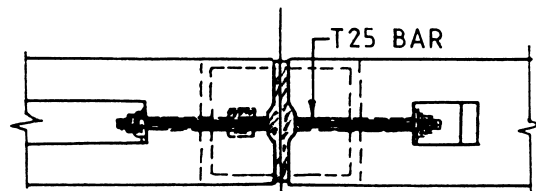
(a) Isometric view



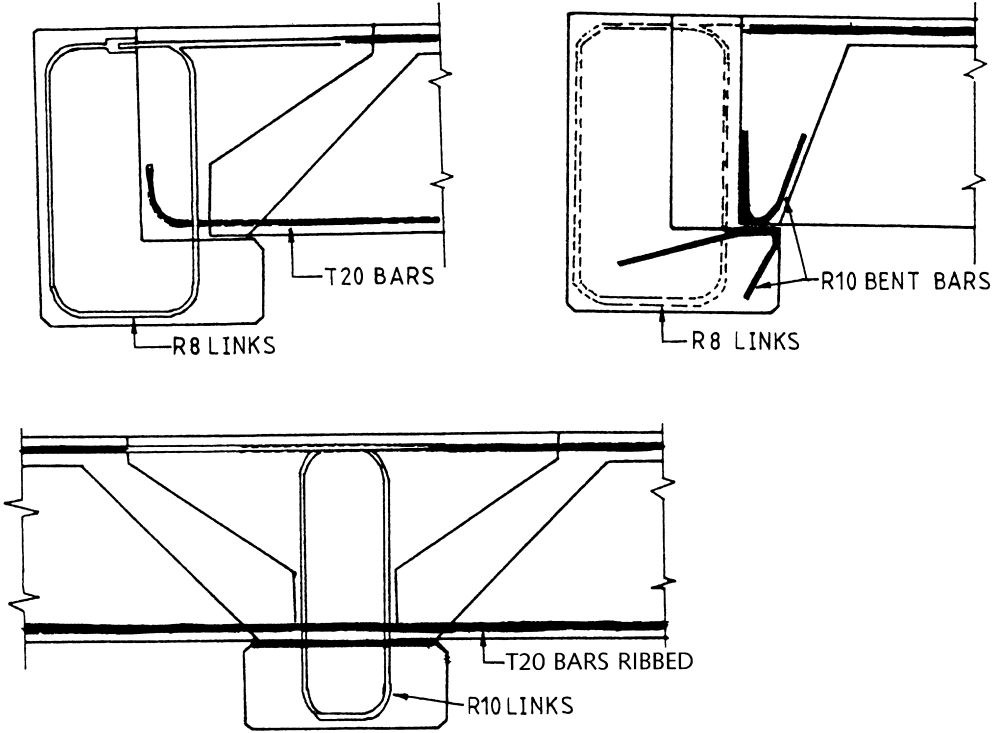
(b) Sectional elevation



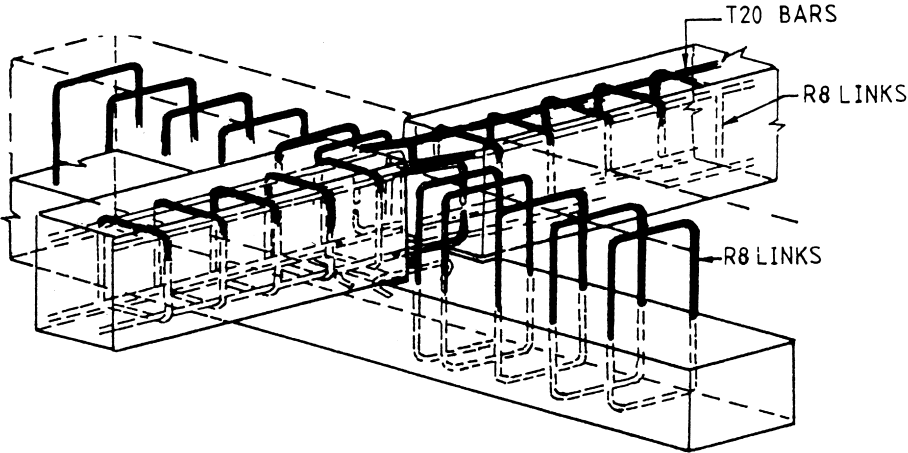
(c) Top plan



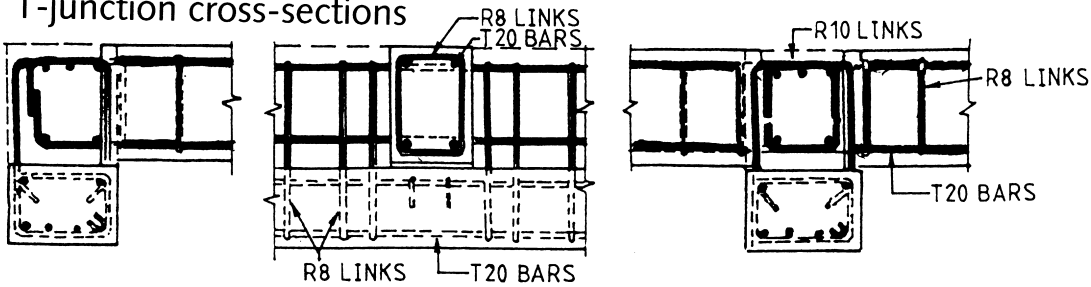
(a) Composite beam connected to ribbed floor units with skirted ends



(b) Composite main beam to secondary beams



(c) T-junction cross-sections



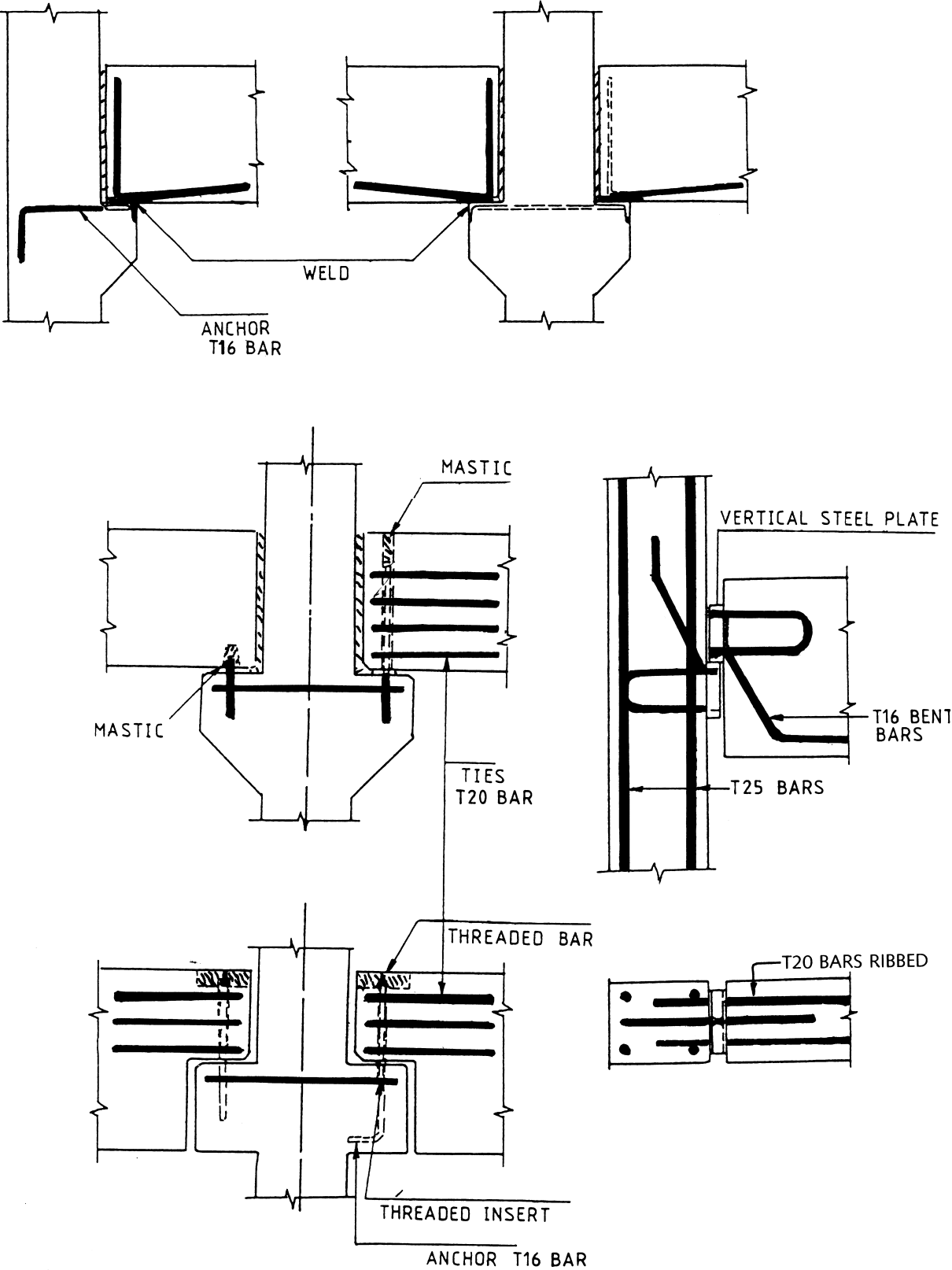
Precast concrete beams resting on column brackets or corbels are detailed using plates and dowel bars as threaded inserts as shown on Sheet No. VI.5. Where holes are left because of threaded bars, they are filled with an approved mastic. Details shown on Sheet No. VI.5 represent a few methods for connecting precast elements.

Sometimes factory-made column elements are to be connected to make larger columns of specified lengths. Dowel bars, as shown on Sheet No. VI.6, are used to erect such columns. Injection holes are left to fill in hollow areas with grout.

Where prestressing is used to connect precast elements, such as those shown on Sheet No. VI.5, instead of tie rods a post-tensioned cable or tendon is introduced. This is shown on Sheet No. VI.7. In such circumstances plastic or steel ducts are left with special holes in the top part of columns, and cables are finally pushed through and stressed. The steel plates at corbels are welded afterwards. A nominal weld of 6 mm would in most cases be enough.

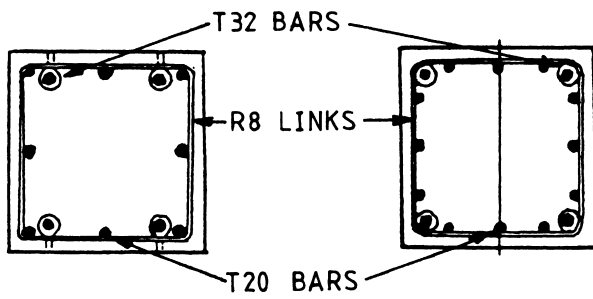
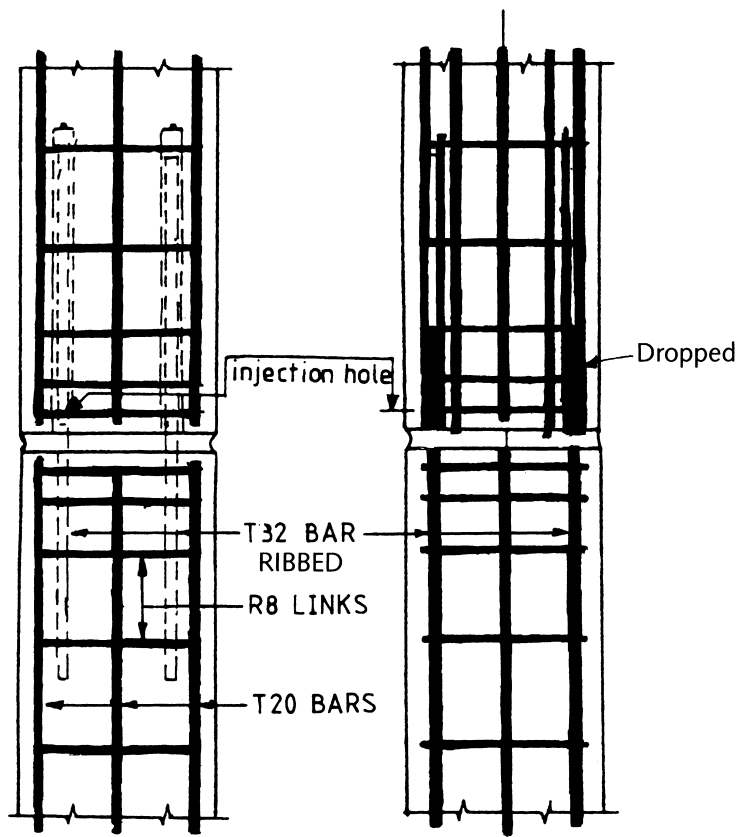
BEAM TO COLUMN: USING STEEL BEARING PLATES, VERTICAL PLATES AND DOWELS

SHEET NO. VI.5



COLUMN TO COLUMN CONNECTION USING DOWEL BARS

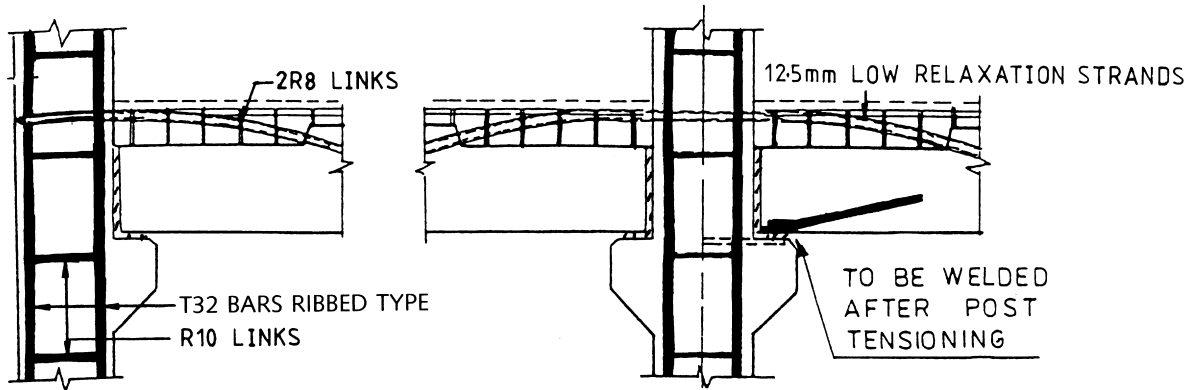
SHEET NO. VI.6



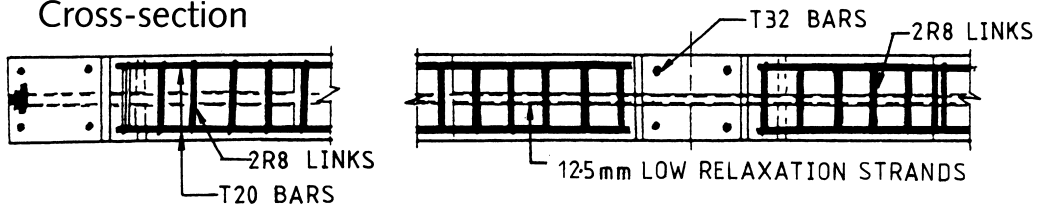
BEAM TO COLUMN: COMPOSITE BEAMS BY POST-TENSIONING SUPPORTED ON CONCRETE CORBELS

SHEET NO. VI.7

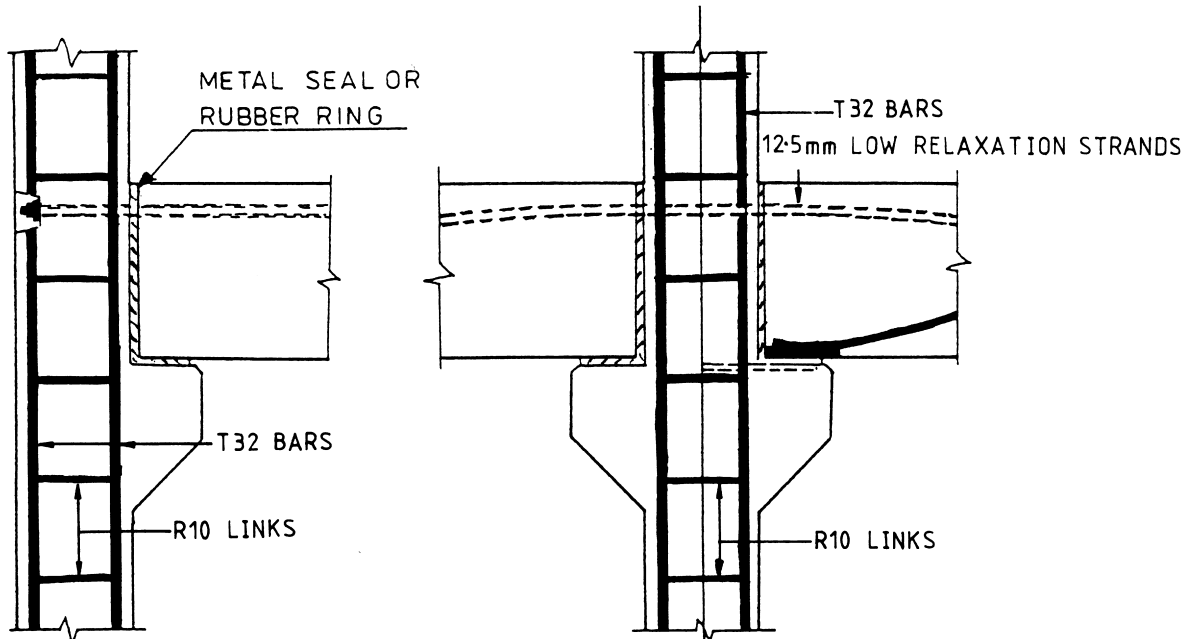
(a) Side view



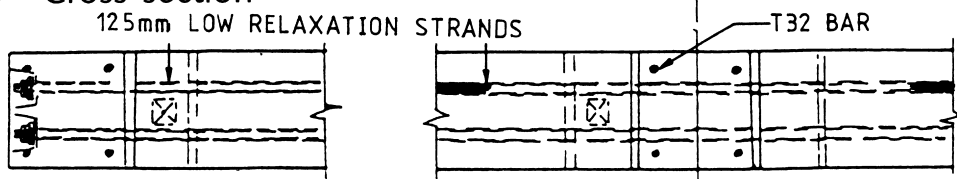
(b) Cross-section



(c) Side view



(d) Cross-section



VI.2. Joints and connections

A sequence of particular construction causes joints in a structure. Joints can be between old and fresh concrete and can be between two parts of a structure. Construction joints must be so positioned that the strength of a completed member is not affected. The most suitable place for a construction joint in a simple structure is where a bending moment is zero or a shear force is maximum. A construction joint may be at the junction of a rib and slab of a T-beam or a smaller beam at a short distance from the junction of intersecting beams. Joints can also be possible where columns at different floors are to be integrated with slab/beam construction. As shown in Sheet No. VI.8, a number of possibilities exist for lapping and splicing of bars at or around floor levels. They are detailed on column elevations on Sheet No. VI.8.

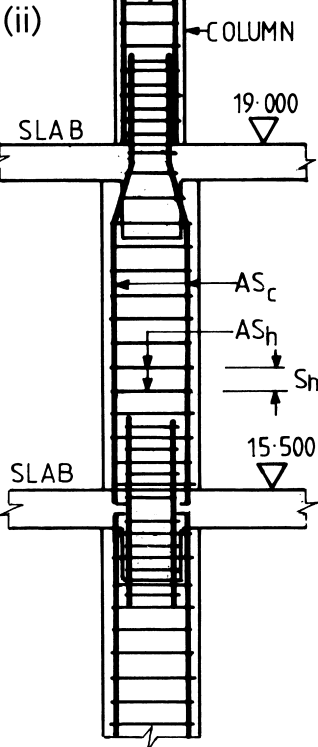
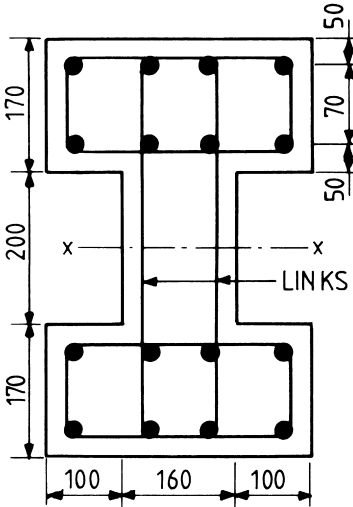
As explained in Section VI.1, beams are connected to columns using different methods. It depends where and at what level a beam (or beams) can be connected where corbels are involved. Their design and detailing, as shown on Sheet No. VI.9, must depend upon induced loads and member sizes. Sheet VI.9(a) gives a layout of a bracket or corbel showing the main reinforcement (thick bar) and links for shear. Columns can also be connected directly to beams or vice versa. Some of them are given in VI.9(b). Where reinforced concrete ties are required in frames and trusses, some possible layouts and structural detailing of them are given on Sheet No. VI.10. Another important family of joints are the expansion joints in a bridge. Sheet No. VI.11 shows three different types of expansion joint used in bridges. They are classified on the basis of their movement but their function is common—to stop the creation of deformation in bridge decks or other structures subject to traffic loads and environmental loads. Other types of joint are those needed in walls, columns and floors. They need to be watertight. A family of such joints in concrete are detailed on Sheet No. VI.12. In such circumstances, it is important to give data on water bars made in rubber or plastics. Sheet No. VI.13 gives basic data on such water bars.

Precast detailing under Eurocode 2 and ACI/PCI codes are given in Sheet Nos VI.14 to VI.17.

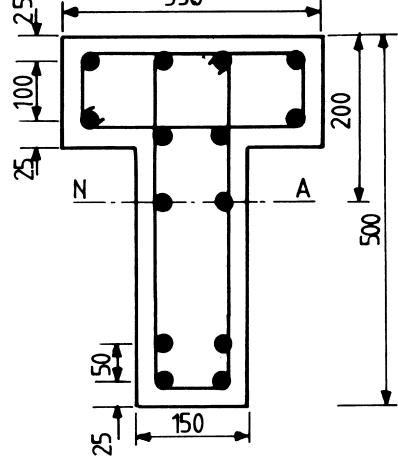
COLUMN ELEVATION WITH REINFORCEMENT AND SPLICE BARS

SHEET NO. VI.8

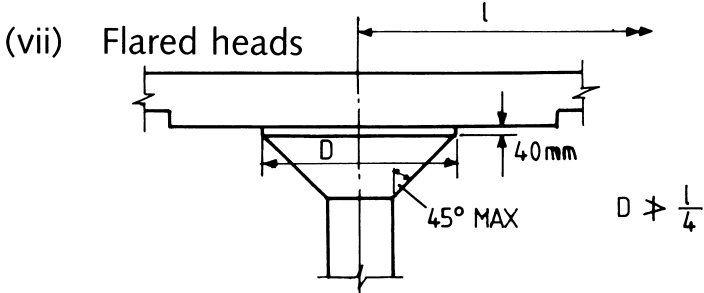
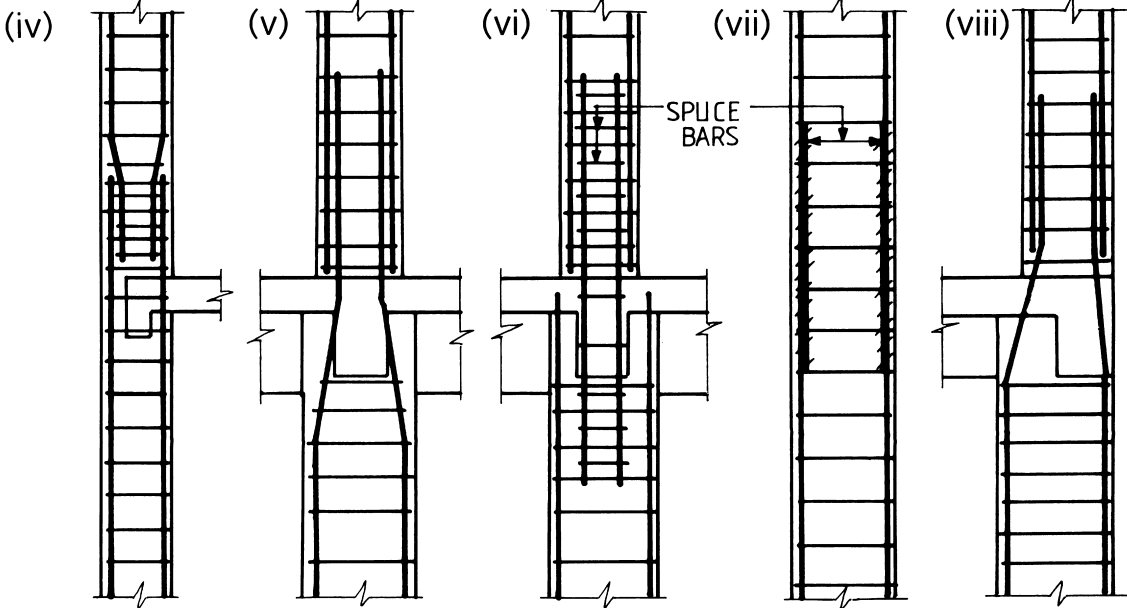
(i) I-Section



(iii) T-Section



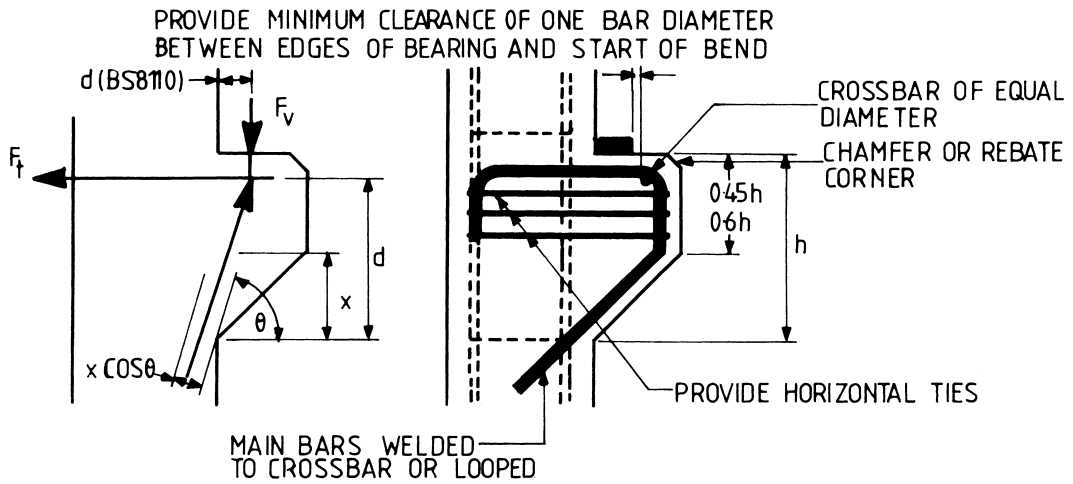
Column splice details



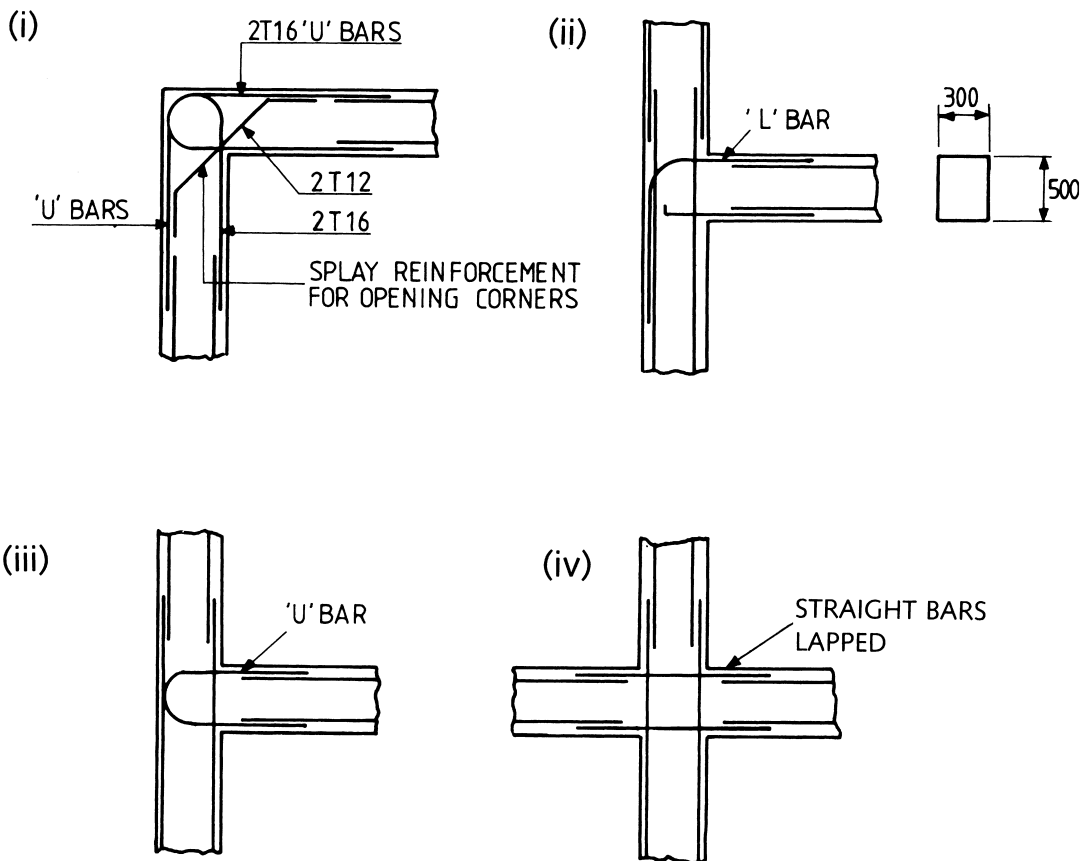
COLUMN BRACKETS AND CONNECTIONS

SHEET NO. VI.9

(a) Brackets

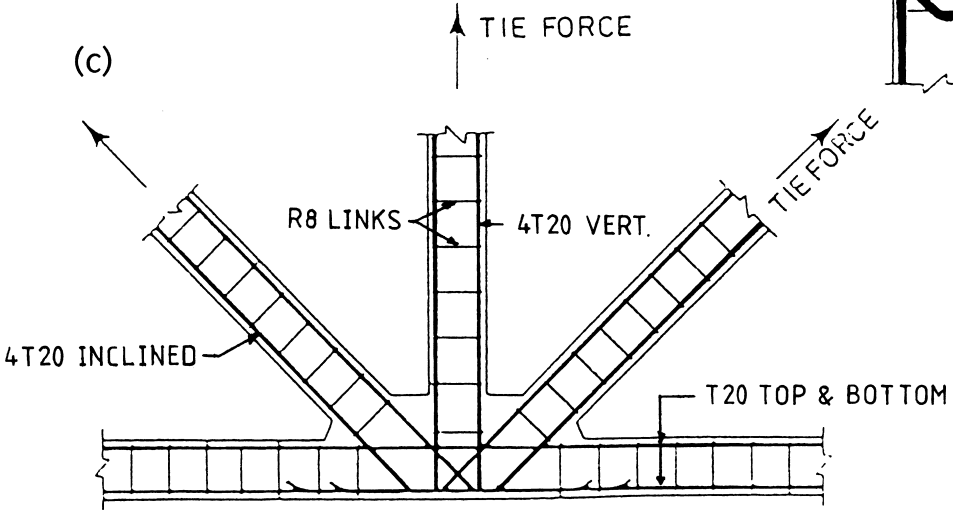
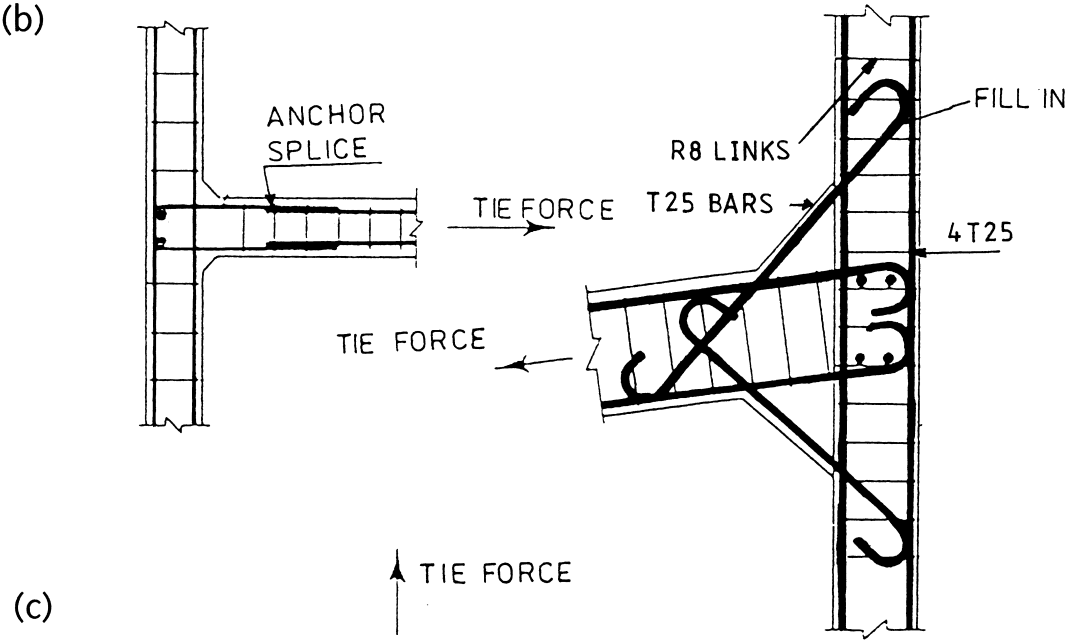
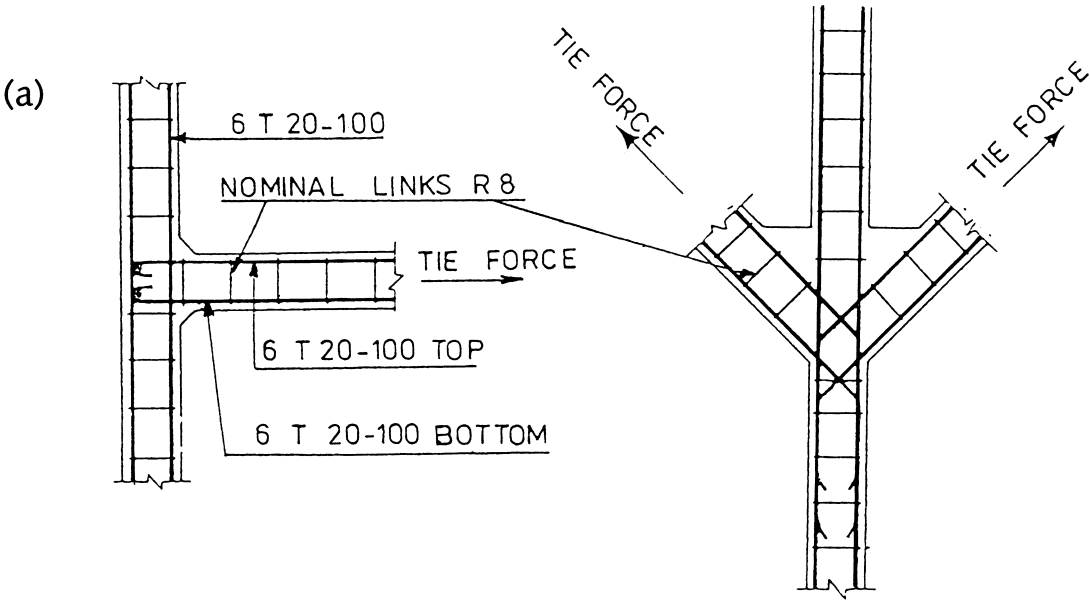


(b) Column connections

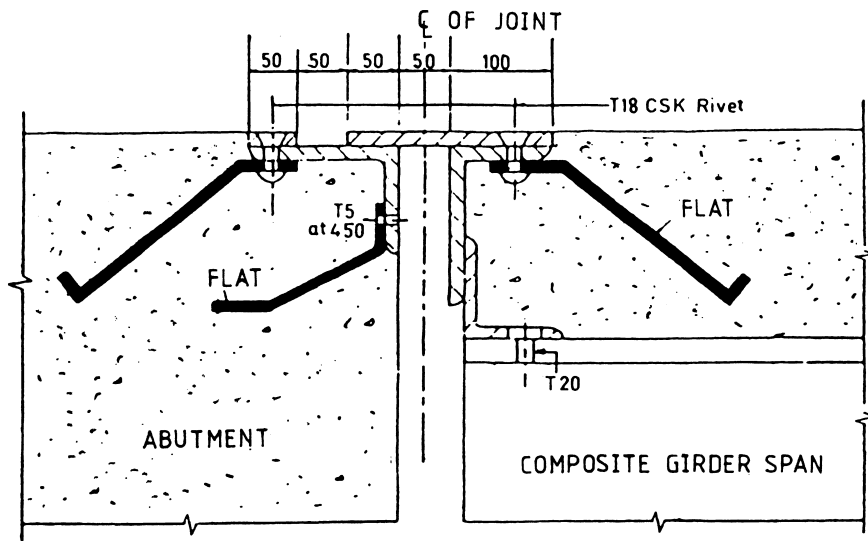


TIES

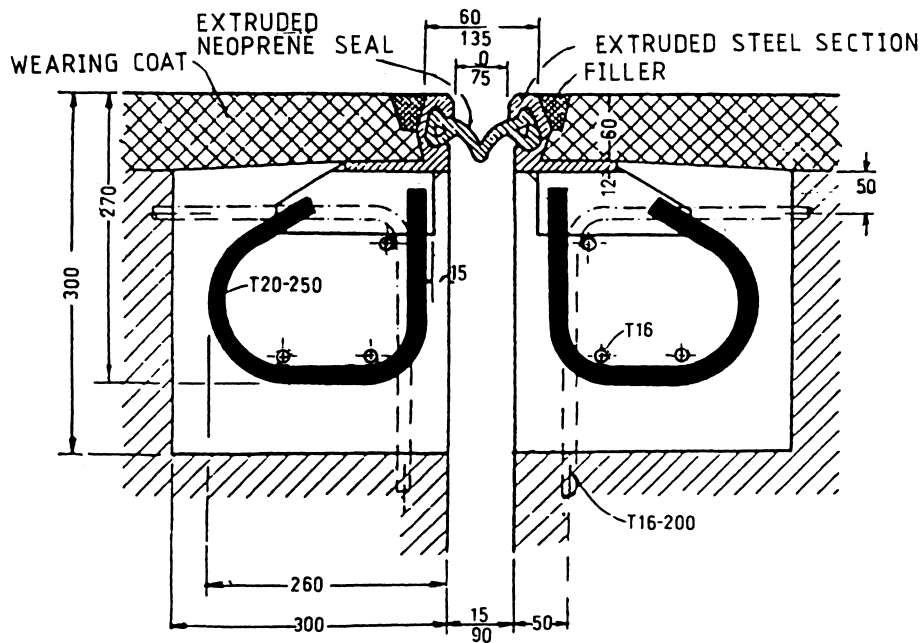
SHEET NO. VI.10



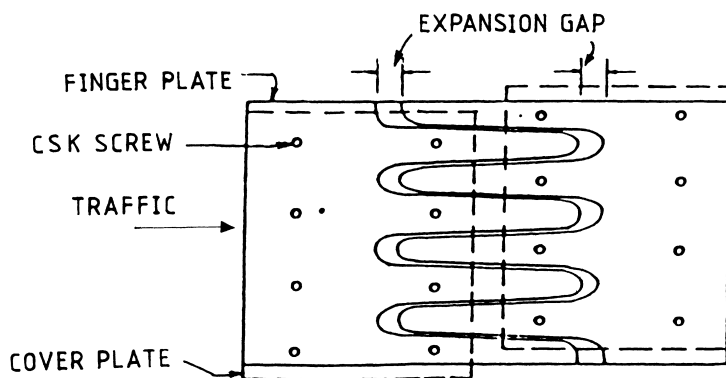
EXPANSION JOINTS FOR BRIDGES



(a) Expansion joint for movement up to 60 mm



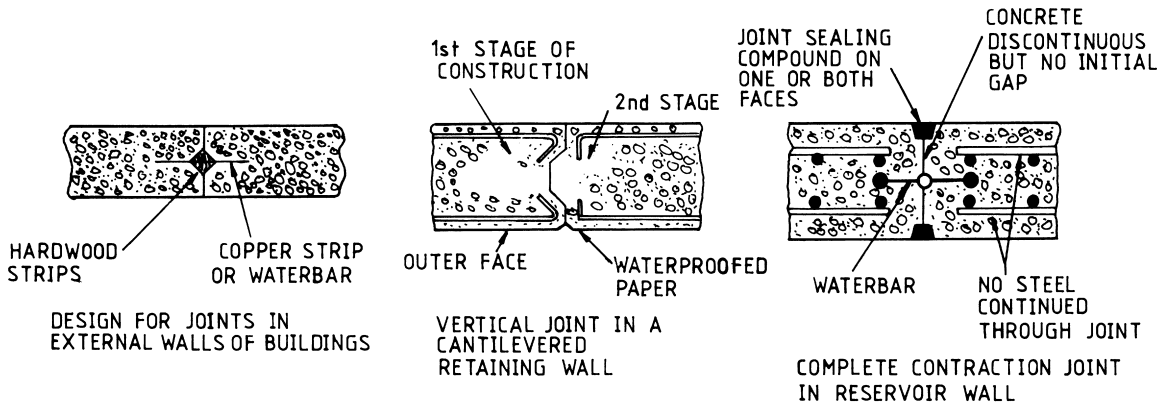
(b) Expansion joint for movement up to 75 mm



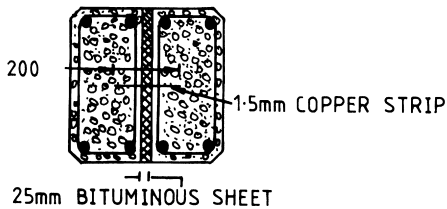
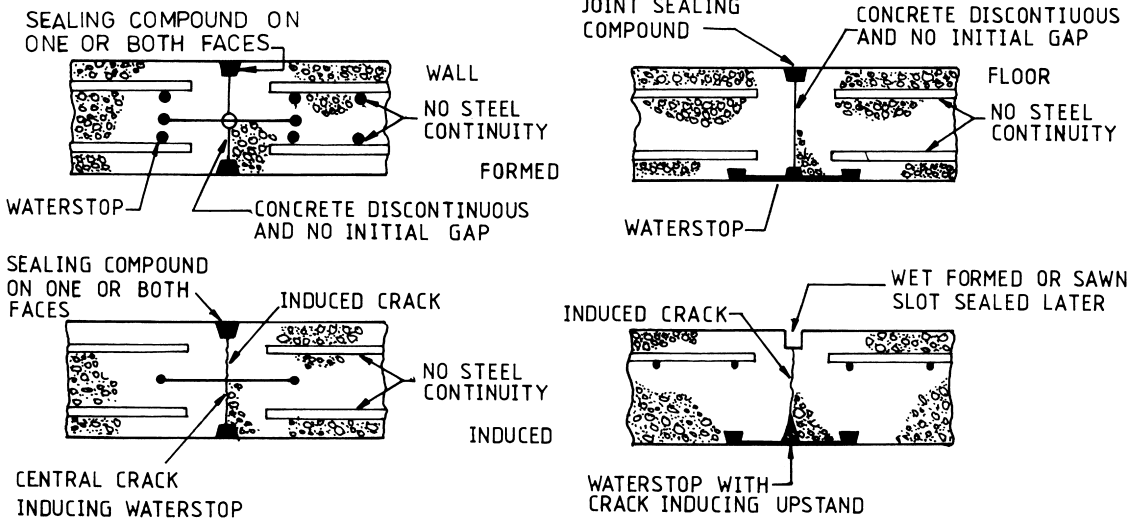
(c) Plan view of onset of finger plates for expansion joint for movement up to 150 mm

WATER JOINTS IN CONCRETE

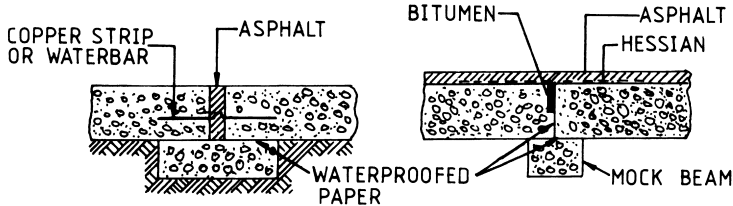
SHEET NO. VI.12



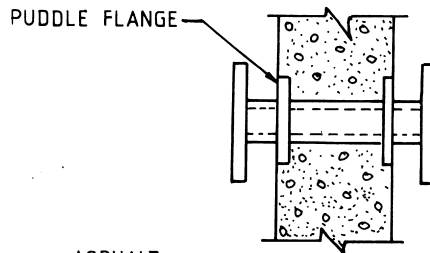
EXPANSION JOINTS



EXPANSION JOINT AT COLUMN



DESIGN FOR JOINTS IN FLOOR LAID ON THE GROUND



PLASTIC WATER BARS FOR EXPANSION JOINTS

SHEET NO. VI.13

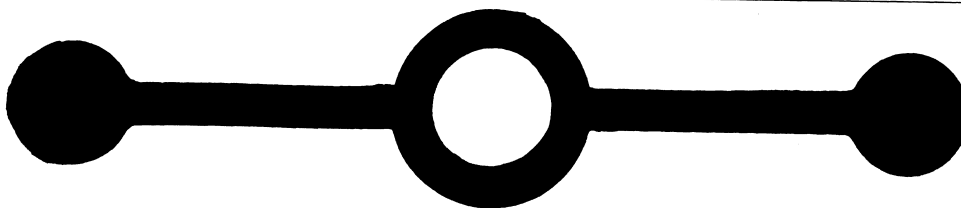
(a) Plain plastic water bar

Section		Sizes			
(i)	Width	140 mm	190 mm	240 mm	305 mm
(ii)	Web thickness	4.8 mm	4.8 mm	4.8 mm	4.8 mm
(iii)	Size of edge bulbs	12.5 mm	19 mm	19 mm	19 mm
(iv)	Width(int.) of centre bulb	12 mm	21 mm	21 mm	28 mm
Placing details					
(v)	Minimum radius to which it can be bent on flat	8.25 m	14 m	16 m	29 m
(vi)	Minimum radius to which it can be bent on edge	75 mm	150 mm	150 mm	230 mm



(b) Rubber central bulb water bar

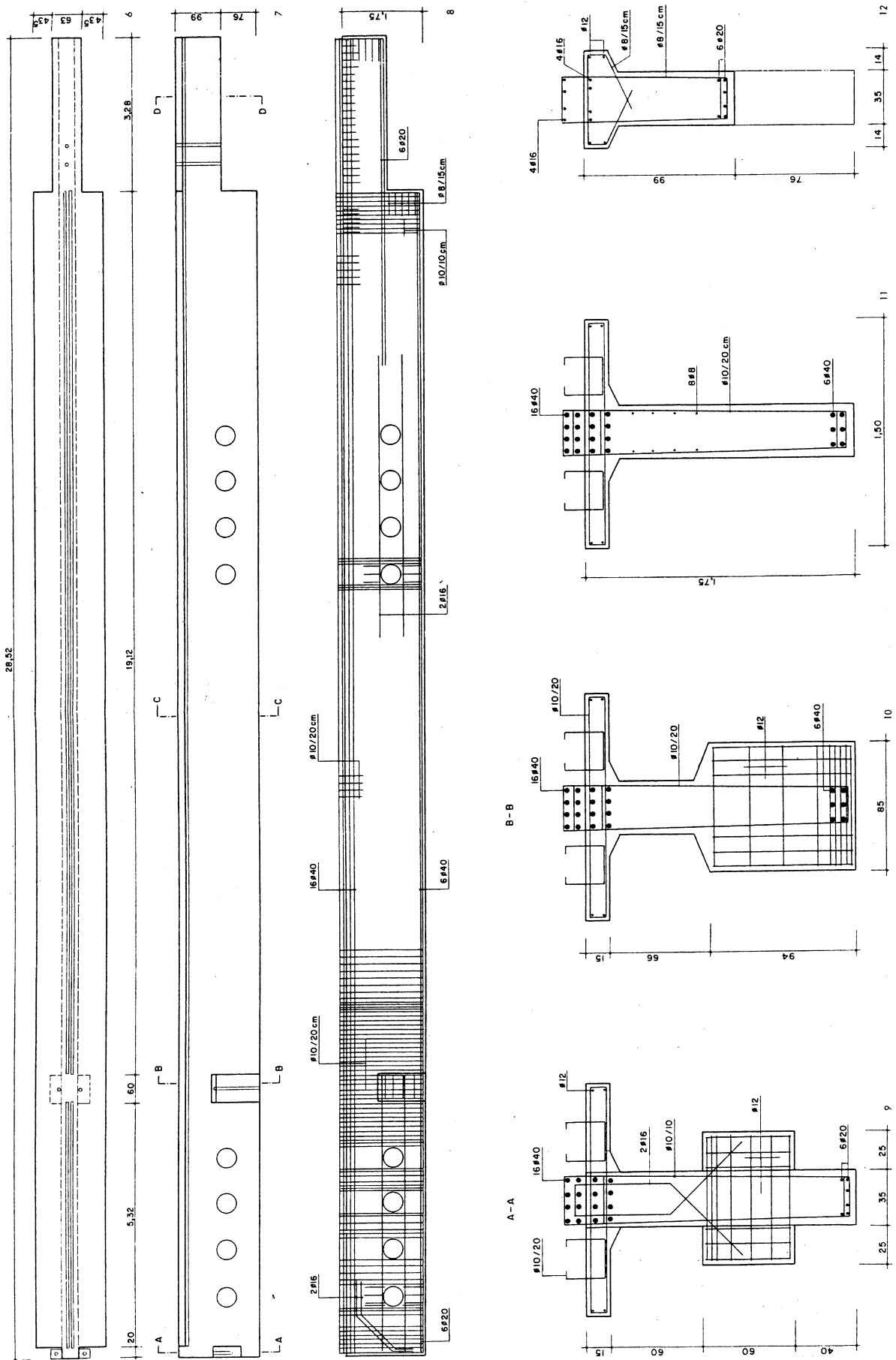
Section		Sizes			
(i)	Width	115 mm	150 mm	230 mm	305 mm
(ii)	Web thickness	6.4 mm	9.5 mm	9.5 mm	9.5 mm
(iii)	Edge bulb diameter	16 mm	19 mm	25 mm	25 mm
(iv)	Centre bulb diameter	29 mm	29 mm	38 mm	50 mm
(v)	Centre core diameter	19 mm	16 mm	19 mm	32 mm
Placing details					
(vi)	Minimum radius to which it can be bent on flat	3.25 m	7 m	8 m	8.75 m
(vii)	Minimum radius to which it can be bent on edge	75 mm	150 mm	150 mm	230 mm



**RC PRECAST BEAM DETAILING
(BASED ON EC2)**

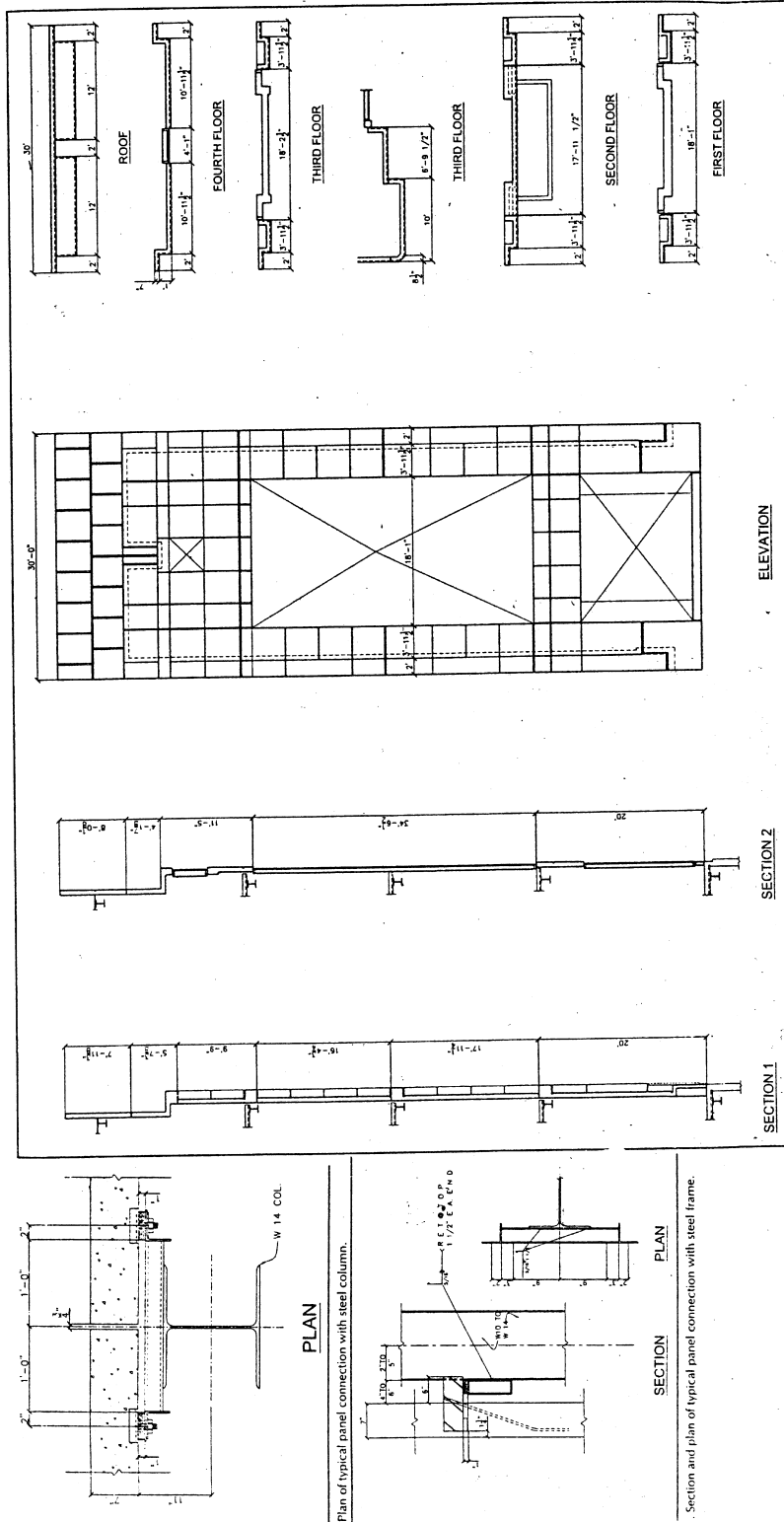
SHEET NO. VI.14

6, 7. Plan and elevation of a casting precast beam, dimensions, 8 longitudinal section of the beam, reinforcement, 9, 10, 11, 12. Cross sections of the beam, reinforcement.



**PRECAST CONCRETE PANELS
(BASED ON PCI CODES)**

SHEET NO. VI.15



CREDITS

Owner: Federated Department Stores,
Cincinnati, Ohio
Architect: Cooper Carry Inc., Atlanta,
Georgia
Engineer: Structural Engineering
Corp., Pittsburgh, Pennsylvania
General Contractor: Turner Construc-
tion Co., Pittsburgh, Pennsylvania
Precast/Prestressed Concrete Manufac-
turer: Modern Mosaic Ltd., Niagara
Falls, Ontario, Canada
Precast Concrete Erector: Alpha Steel
Erectors Inc., Worthington, Pennsylv-
ania

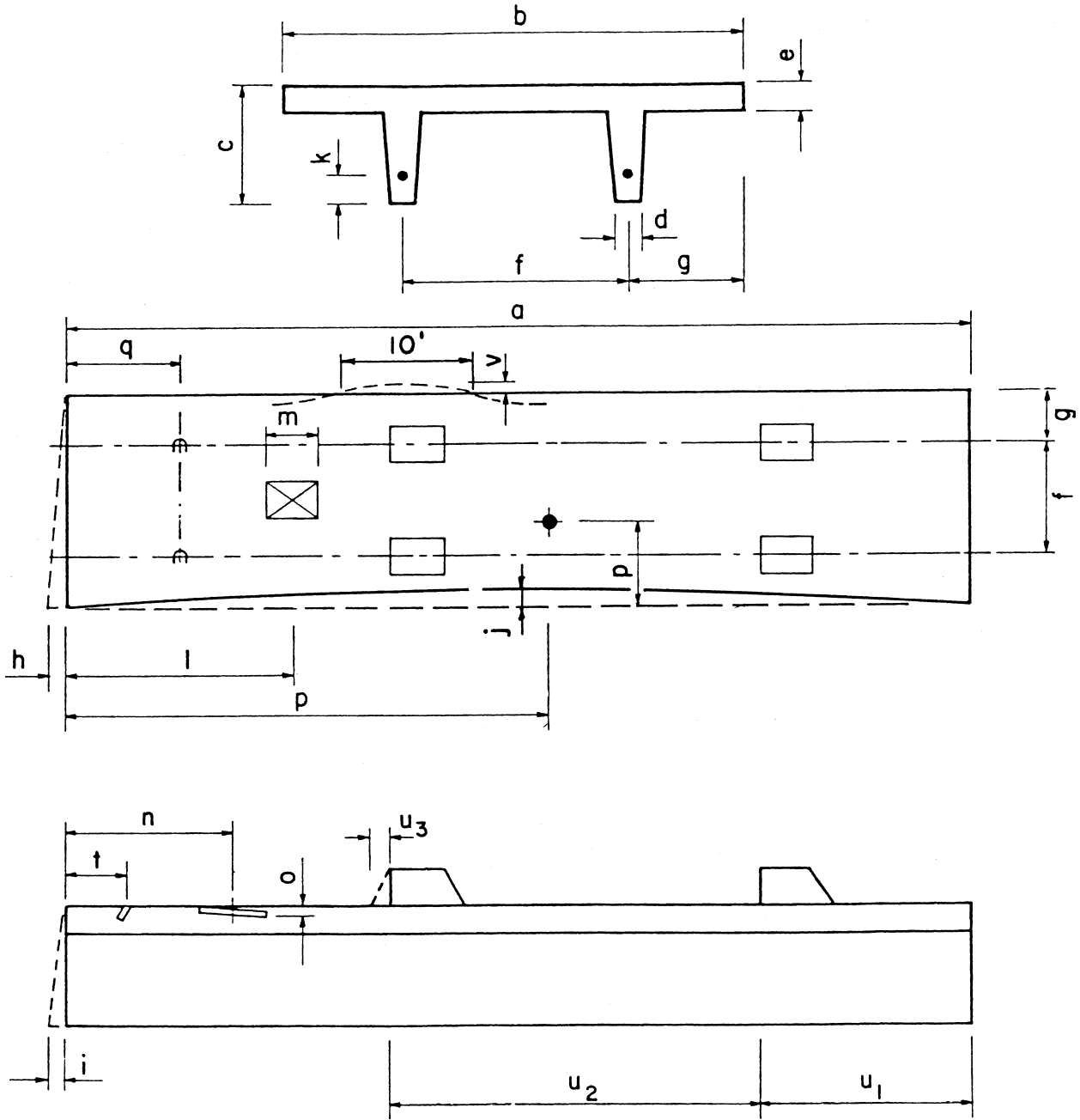
ELEVATION
**LAZARUS DEPARTMENT STORE
PITTSBURGH, DA. USA.**

September-October 2000

PCI JOURNAL

RIBBED WALL PANELS

SHEET NO. VI.16



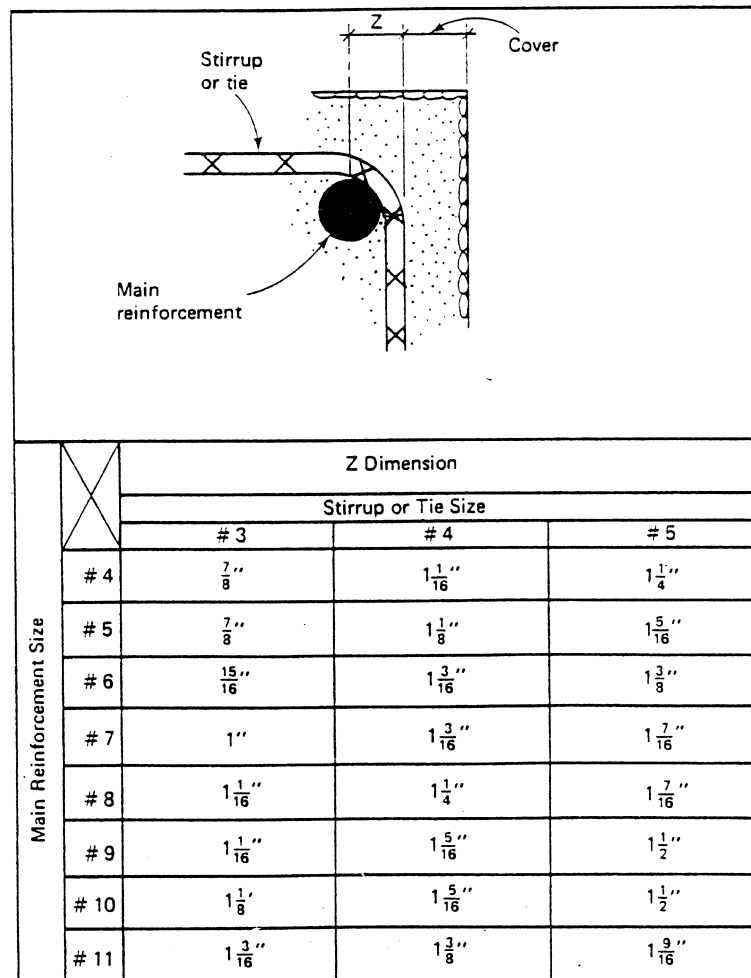
- a = Length ± 1/2 in.
- b = Width ± 1/4 in.
- c = Depth ± 1/4 in.
- d = Stem width ± 1/8 in.
- e = Flange thickness + 1/4 in., - 1/8 in.
- f = Distance between stems ± 1/8 in.
- g = Stem to edge of top flange ± 1/8 in.
- h = Variation from specified flange
squareness or skew ± 1/8 in. per
12 in. width, ± 1/4 in. max.
- i = Variation from specified end squareness
or skew ± 1/8 in. per 12 in.
- j = Sweep (variation from straight line parallel to
centerline of member)
Up to 40 ft. member length ± 1/4 in.
40 ft. and greater
member length ± 3/8 in.
- k = Position of tendons ± 1/4 in.
- l = Position of blockouts ± 1 in.
- m = Size of blockouts
Finished opening ± 1/2 in.
Rough opening ± 1 in.

- n = Position of plates ± 1 in.
- o = Tipping and flusbness of plates ± 1/4 in.
- p = Position of inserts for
structural connections ± 1/2 in.
- q = Position of handling devices
Parallel to length ± 6 in.
Transverse to length ± 1 in.
- r = Bowing L/360 max.
- s = Differential bowing between adjacent
panels of the same location 1/2 in.
- t = Position of flashing reglets ± 1/4 in.
- u = Haunches (noncumulative)
- u₁ = Bearing elevations from
bottom of panel ± 1/4 in.
- u₂ = Relative position of bearing elevation
in vertical plane ± 1/8 in.
- u₃ = Haunch bearing surface squareness perpen-
dicular to applied major load ± 1/8 in. per
18 in., ± 1/4 in. max.
- v = Local smoothness 1/4 in. in 10 ft.
any surface (does not apply to visually con-
cealed surfaces)
- w = Warping 1/16 in. per foot of distance
from nearest adjacent corner

**MINIMUM COVER REQUIREMENTS FOR PRECAST CONCRETE SHEET NO. VI.17
(BASED ON ACI CODES)**

Condition	Minimum Cover	
	Nonprestressed Reinforcement	Prestressed Reinforcement
Exposed to earth or weather		
Wall panels	#11 and smaller	3/4 in.
Other members	#6 through #11	1 1/2 in.
	# 5, W31 or D31 wire, and smaller	1 1/4 in.
Not exposed to earth or weather		
Wall panels, slabs and joists	#11 and smaller	5/8 in.
Beams and columns	Diameter of bar, but not less than 5/8 in., and need not exceed 1 1/2 in.	1 1/2 in.
Main steel		
Ties, stirrups or spirals	All sizes	3/8 in.

Location of main reinforcement



VII. Concrete foundations and earth-retaining structures

VII.1. General introduction

An essential requirement in foundations is the evaluation of the load which a structure can safely bear. The type of foundation selected for a particular structure is influenced by the following factors:

- (a) the strength and compressibility of the various soil strata
- (b) the magnitude of the external loads
- (c) the position of the water table
- (d) the need for a basement
- (e) the depth of foundations of adjacent structures.

The types of foundations generally adopted for buildings and structures are spread (pad), strip, balanced and cantilever or combined footings, raft and pile foundations. The foundations for bridges may consist of pad, piles, wells and caissons.

VII.2. Types of foundations

VII.2.1. Isolated spread foundation, pad footing and combined pad foundations

These are generally supporting columns and may be square or rectangular in plan and in section, they may be of the slab, stepped or sloping type. The stepped footing results in a better distribution of load than a slab footing. A sloped footing is more economical although constructional problems are associated with the sloping surface. The isolated spread footing in plain concrete has the advantage that the column load is transferred to the soil through dispersion in the footing. In reinforced concrete footings, i.e. pads, the slab is treated as an inverted cantilever bearing the soil pressure and supported by the column. Where a two-way footing is provided it must be reinforced in two directions of bending with bars of steel placed in the bottom of the pad parallel to its sides. If clearances permit, two-way square footings are used to reduce the bending moments. Where more than one column is placed on pads (combined footing), their shapes may be rectangular or trapezoidal; the latter produces a more economical design where large differences of magnitude of the column loads exist or where rectangular footings cannot be accommodated. (Sheet No. II.23 gives section and plan together with a tabular method for reinforcement designation and scheduling.) Sheet No. VII.1 gives pad-type combined footings and their behaviour under external loads and bearing pressures; typical reinforcement detailing for two different combined footings in plan and sectional elevations. The specifications and quantities may change depending on the spread area and the column loads. In order to keep a record of the types of footings in particular areas and the bar schedule, a *tabular method* given on Sheet No. VII.1 is recommended using given headings.

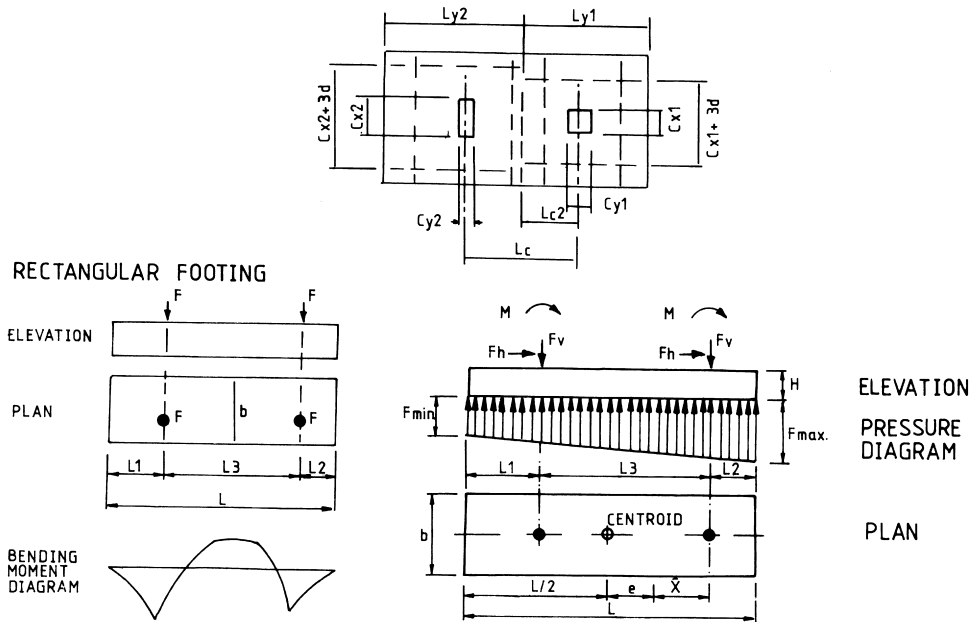
FOOTINGS

SHEET NO. VII.1

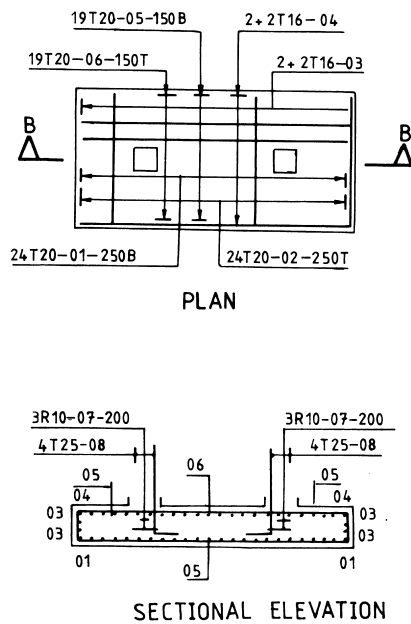
TABULAR METHOD FOR FOOTINGS

COLUMN REF	NO OFF	LEVEL	BASE REINFORCEMENT	SECTION	LINKS	STARTERS

PAD TYPE COMBINED FOOTING



TYPICAL REINFORCEMENT DETAILS



VII.2.2. Cantilever, balanced and strip foundations

Foundations under walls or under closely spaced rows of columns sometimes require (because of restrictions in one direction) a specific type of foundation, such as cantilever and balanced footings and strip footings. The principles and reinforcement details are given on Sheet No. VII.2.

VII.2.2.1. National Application Document

Introduction

This National Application Document has been prepared by Technical Committee B/526 to enable ENV 1997-1 (Eurocode 7: Part 1) to be used for the design of geotechnical structures to be constructed in the United Kingdom. It has been developed from:

- (a) a textual examination of ENV 1997-1, and
- (b) trial calculations, including parametric calibration against relevant UK codes and standards, to assess its ease of use and to provide numerical factors that produce designs in general conformity with UK practice.

Definitions

For the purposes of this National Application Document, the following definitions apply.

Action

Force (load) applied to the structure (direct action); or an imposed or constrained deformation (indirect action). Note: for example, caused by temperature changes, moisture variation or uneven settlement.

Execution

Activity of creating a building or civil engineering works. Note: the term covers work on-site; it may also signify the fabrication of components off-site and their subsequent erection on-site.

VII.2.2.2. Normative references

BSI publications (British Standards Institution, London):

- BS 5573: 1978 Code of practice for safety precaution in the construction of large diameter boreholes for piling and other purposes
- BS 6031: 1981 Code of practice for earthworks
- BS 8002: 1994 Code of practice for earth retaining structures
- BS 8004: 1986 Code of practice for foundations
- BS 8006 Code of practice for strengthened/reinforced soils and other fills

VII.2.2.3. CEN publication

European Committee for Standardization (CEN), Brussels. ENV 1991-1: 1994 Eurocode 1—Basis of design and actions on structure—Part 1: Basis of design.

VII.2.2.4. Values of partial factors with actions and material properties

In this clause, values of partial factors currently do not differ from those used in ENV 1997-1. Sheet No. VII.3 gives factors for the design of various foundations. Note: in the state of development of this National Application Document at July 1995 no deviations from boxed values are proposed.

VII.2.2.5. Foundations of concrete structures (ACI manual, 2000)

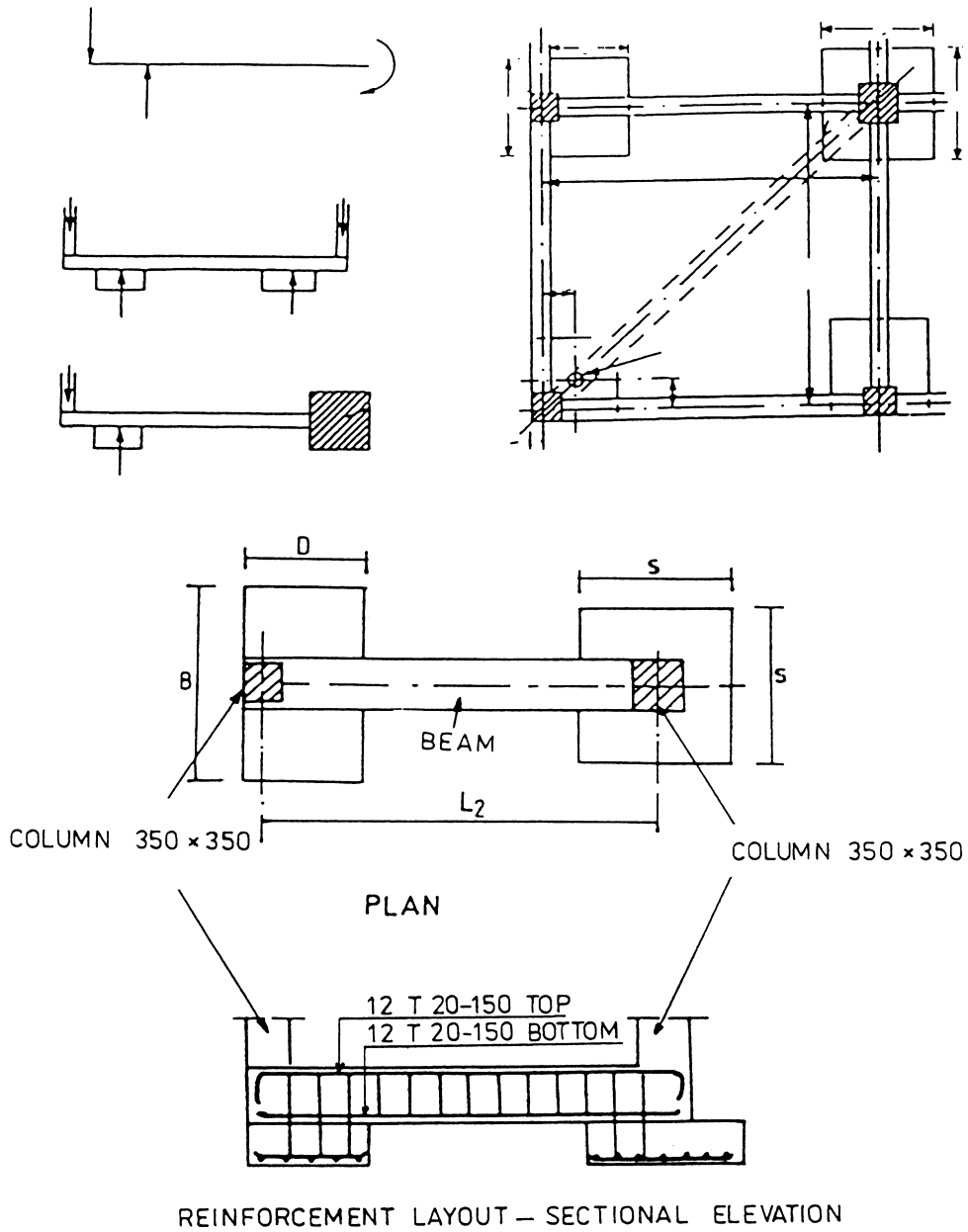
Introduction

The American concrete Institute (ACI) has produced in Section 14 a comprehensive treatment of concrete foundations. Sections include, with detailed explanation and soils strength, site exploration, shallow footings, mat footings, pile foundations, caissons and drilled piers. The 'ACI Detailing Manual' provides details and detailing of concrete reinforcement (ACI 315-1986) with supporting data based on ACI 315R-1988.

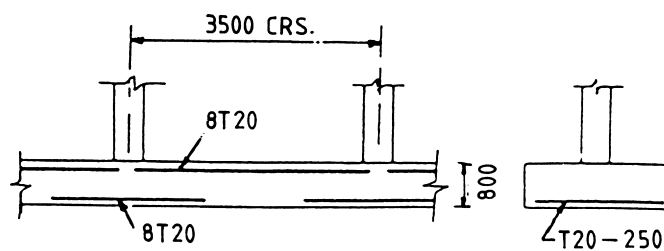
CANTILEVER, BALANCED AND STRIP FOUNDATIONS

SHEET NO. VII.2

(a) Cantilever and balanced footings



(b) Strip footings



**PARTIAL SAFETY FACTORS
(BASED ON NAD AND DD ENV 1997)**

SHEET NO. VII.3

Table 1. Partial factors — ultimate limit states in persistent and transient situations

Case	Actions			Ground properties			
	Permanent		Variable	tan ϕ	c'	c_u	$q_u^{1)}$
	Unfavourable	Favourable	Unfavourable				
A	1.0	0.95	1.5	1.1	1.3	1.2	1.2
B	1.35	1.0	1.5	1.0	1.0	1.0	1.0
C	1.0	1.0	1.3	1.25	1.6	1.4	1.4

¹⁾ Compressive strength of soil or rock.

Table 2. Factors to derive the ultimate characteristic bearing resistance

	Number of load tests		
	1	2	>2
Factor ξ on mean R_{cm}	1.5	1.35	1.3
Factor ξ on lowest R_{cm}	1.5	1.25	1.1

Table 5. Factors to derive ultimate characteristic resistance from anchorage tests

	Number of assessment tests		
	1	2	>2
Factor ξ on mean R_{am}	1.5	1.35	1.3
Factor ξ on lowest R_{am}	1.5	1.25	1.1

Table 3. Factors to derive the ultimate design bearing resistance

Component factors	γ_b	γ_s	γ_t
Driven piles	1.3	1.3	1.3
Bored piles	1.6	1.3	1.5
CFA piles	1.45	1.3	1.4

Table 4. Factors to derive characteristic ultimate tensile pile resistance from tests

	Number of load tests		
	1	2	>2
Factor ξ on mean R_m	1.5	1.35	1.3
Factor ξ on lowest R_m	1.5	1.25	1.1

Placement drawings

This document provides standards of practice for both the engineer and reinforcing bar detailer with tables and figures. Apart from variations in actual production of placement and design drawings, some notation and dimensional representations of course, do vary from the British and European practices. The basic principles of the design are identical to the British and European practices. These can be compared among themselves by the intensive aptitude of the reader in the section of the text.

VII.2.3. Circular and hexagonal footings

For specific structures, it is sometimes economical to arrange the footings or foundations to suit their shapes. This applies to many cylindrical structures such as concrete pressure and containment vessels, hoppers/bunkers/silos and cells for offshore gravity platforms, etc. Sheet No. VII.4 gives the structural detailing of such foundations.

VII.2.3.1. Reinforced concrete machine foundation

The design of a machine foundation is more complex than that of a foundation which supports only static loads. In machine foundations, the designer must consider, in addition to the static loads, the dynamic forces caused by the working of the machine. These dynamic forces are, in turn, transmitted to the foundation supporting the machine. The designer should, therefore, be well conversant with the method of load transmission from the machine as well as with the problems concerning the dynamic behaviour of the foundation and the soil underneath the foundation. Based on the design criteria of their foundations, machines may be classified as follows:

- (a) those producing impact forces, e.g. forge hammers, presses
- (b) those producing periodical forces, e.g. reciprocating engines such as, compressors
- (c) high speed machinery such as turbines and rotary compressors
- (d) other miscellaneous machines, the shapes, loads and other criteria must be followed for the proper design of their foundations.

Considering their structural form, machine foundations are generally classified as follows:

- (a) block-type foundations consisting of a pedestal of concrete on which the machine rests
- (b) box or caisson-type foundations consisting of a hollow concrete block supporting the machinery on its top
- (c) wall-type foundations consisting of a pair of walls which support the machinery on their top
- (d) framed-type foundations consisting of vertical columns supporting on their top a horizontal framework which forms the seat of essential machinery.

Machines producing impulsive and periodical forces at low speeds are generally mounted on block-type foundations, while those working at high speeds and the rotating type of machinery are generally mounted on framed foundations. However, owing to certain conditions, these may not always be possible, in which case alternative types may be adopted.

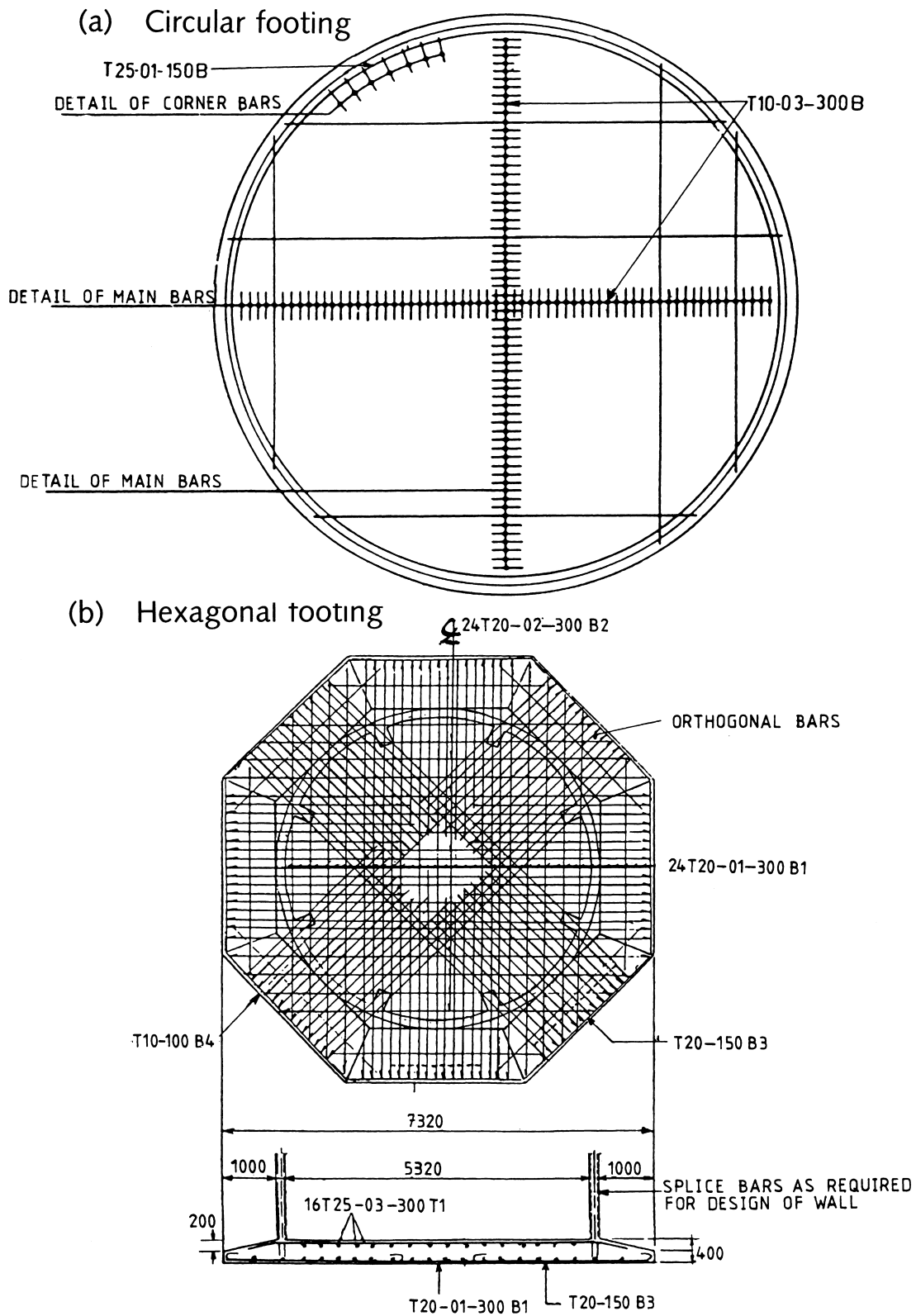
Based on their operating frequency, machines may be divided into three categories:

- | | |
|--------------------------------|-----------------------|
| 1. Low to medium frequencies: | 0–500 rpm |
| 2. Medium to high frequencies: | 300–1000 rpm |
| 3. Very high frequencies: | Greater than 1000 rpm |

-

CIRCULAR AND HEXAGONAL FOOTINGS

SHEET NO. VII.4



Group 1 contains large reciprocating engines, compressors and large blowers. Reciprocating engines generally operate at frequencies ranging within 50–250 rpm. For this group, foundations of block type with large contact area with the soil are generally adopted.

Group 2 consists of foundations of medium-sized reciprocating engines, such as diesel and gas engines. Block foundations resting on springs or suitable elastic pads are generally suggested for this group in order to maintain the natural frequencies of the foundation considerably below the operating frequency.

Group 3 includes high-speed, internal combustion engines, electric motors and turbogenerator sets. Where massive block foundations are used, small contact surfaces and suitable isolation pads are desirable to lower the natural frequencies. Turbo-machinery requires framed-type foundations which accommodate the necessary auxiliary equipment between the columns.

VII.2.3.2. General requirements for the machinery foundations

The following general requirements shall be satisfied and results checked prior to detailing the foundations.

1. The foundation should be able to carry the superimposed loads without causing shear or crushing failure.
2. The settlements should be within the permissible limits.
3. The combined centre of gravity of machine and foundation should, as far as possible, be in the same vertical line as the centre of gravity of the base plane.
4. No resonance should occur, hence the natural frequency of the foundation–soil system should be either too large or too small compared to the operating frequency of the machine. For low-speed machines, the natural frequency should be high.
5. The amplitudes under service conditions should be within permissible limits which are prescribed by the machine manufacturers.
6. All rotating and reciprocating parts of a machine should be so well balanced as to minimize the unbalanced forces or moments.
7. Where possible, the foundation should be planned in such a manner as to permit a subsequent alteration of natural frequency by changing base area or mass of the foundation as may subsequently be required.

From the practical point of view, the following requirements should be fulfilled.

1. The groundwater table should be as low as possible and groundwater level deeper by at least one-fourth of the width of foundation below the base plane. This limits the vibration propagation, groundwater being a good conductor of vibration waves.
2. Machine foundations should be separated from adjacent building components by means of expansion joints.
3. Any steam or hot air pipes, embedded in the foundation must be properly isolated.
4. The foundation must be protected from machine oil by means of acid-resisting coating or suitable chemical treatment.
5. Machine foundations should be taken to a level lower than the level of the foundations of adjoining buildings.

Design data

The specific data required for design vary depending upon the type of machine. The general requirements of data for the design of machine foundations are, however, as follows.

1. Loading diagram showing the magnitude and positions of static and dynamic loads exerted by the machine on its foundation.
2. Power engine and operating speed.
3. Drawings showing the embedded parts, openings, grooves for foundation bolts, etc.
4. Nature of soil and its static and dynamic properties as required in design calculations.

Some typical examples for certain machinery are noted in Table VII.1.

Table VII.1. Block foundation for horizontal compressors

Machine data	
Operating speed of engine	150 rpm
Horizontal unbalanced force in the direction of the piston	12 t
Weight of machine	36 t
The horizontal unbalanced force acts at a height of 0.6 m above the top of the foundation	
Soil data	
Nature of soil	Sandy
Bearing capacity of soil	200 KN/m ²
Coefficient of elastic uniform shear	250 KN/m ²
Grade of concrete	C30–C50

A typical design drawing for a block foundation is shown on Sheet No. VII.5.

VII.2.3.3. Foundation for a tube mill on soil

The schematic arrangement of the foundation for a tube mill is shown on Sheet No. VII.5 (item 2). The site investigation should give the value of permissible stress of soil. The data of machinery are as follows (where t=tonne).

1. Weight of the cylindrical tube without charge: 80 t
2. Weight of steel balls: 40 t
3. Maximum weight of material to be pulverized: 8 t
4. Dynamic loading = 0.2 (i)
5. Weight of the machinery at the discharge end, including motor and gear-box assembly: 30 t

The inlet and discharge ends of the mill are provided on separate foundations as shown on Sheet No. VII.5.

VII.2.3.4. Hammer foundation resting on soil

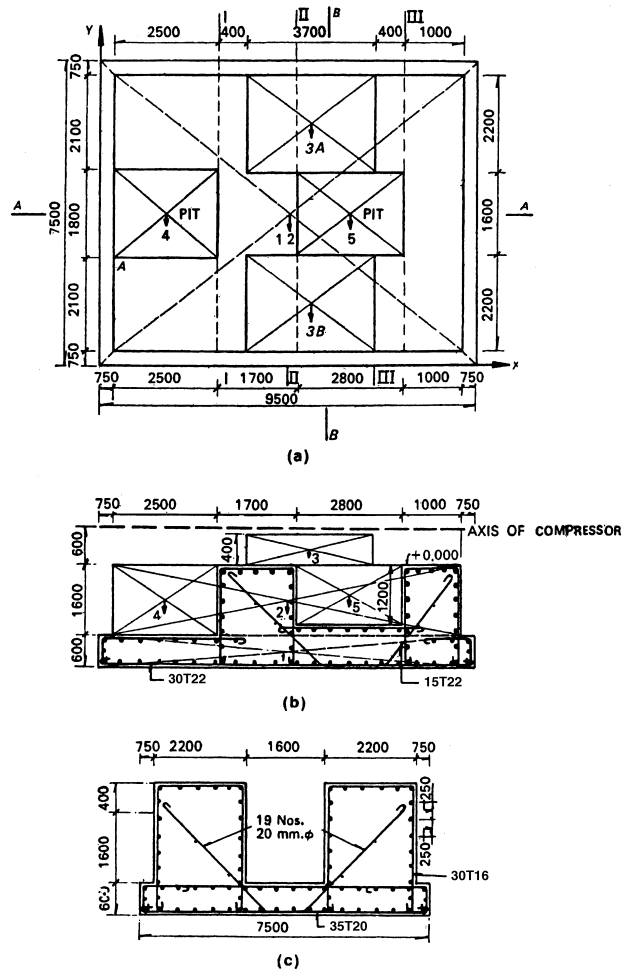
1. Weight of tup: 3400 Kg
2. Weight of anvil: 75 000 Kg
3. Weight of frame resting on foundation block: 38 350 Kg
4. Area of anvil base: 8.32×10^4 cm²
5. Thickness of timber layer under anvil: 60 cm
6. Elasticity of timber layer: 13×10^3 Kg/cm²
7. Velocity of fall of tup: 600 cm/s
8. Coefficient of restitution: 0.25
9. Bearing capacity of soil: 3.5 Kg/cm²
10. Coefficient of elastic compression of soil: 3.8 Kg/cm³

Sheet No. VII.6 shows a typical layout of the hammer foundation.

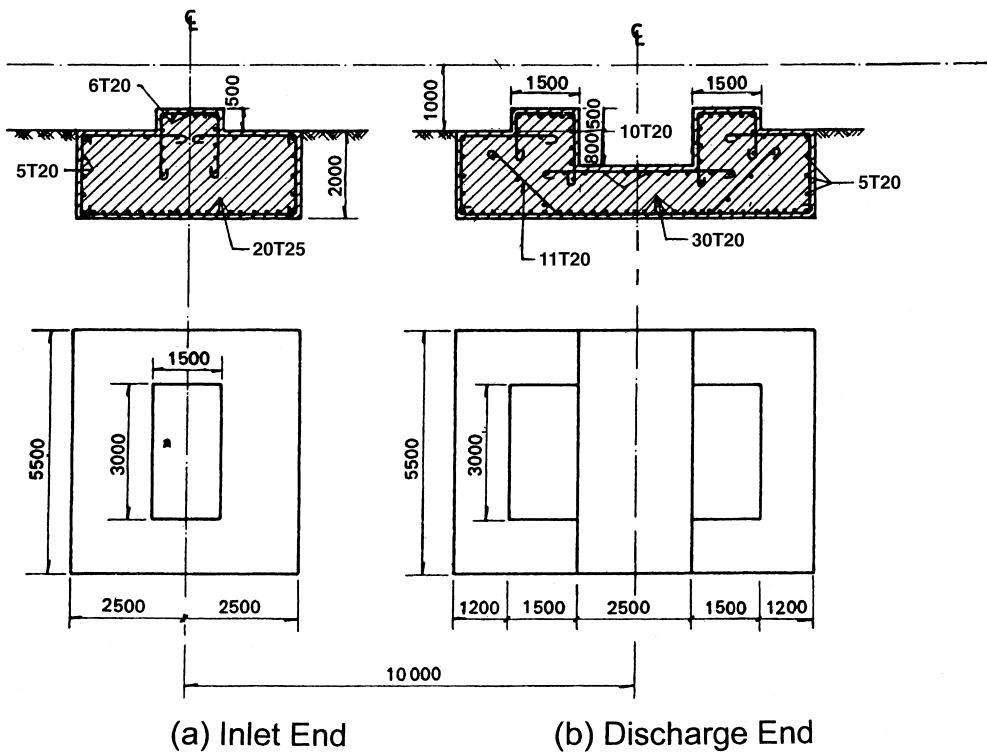
MACHINE FOUNDATIONS

SHEET NO. VII.5

1. Horizontal compressor

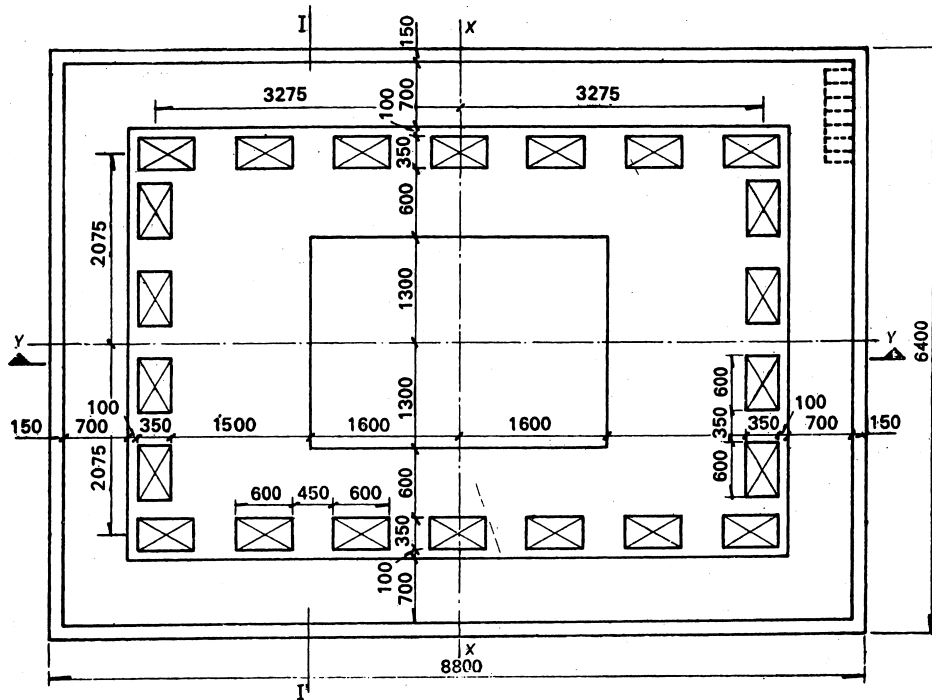


2. Foundation for a tube mill

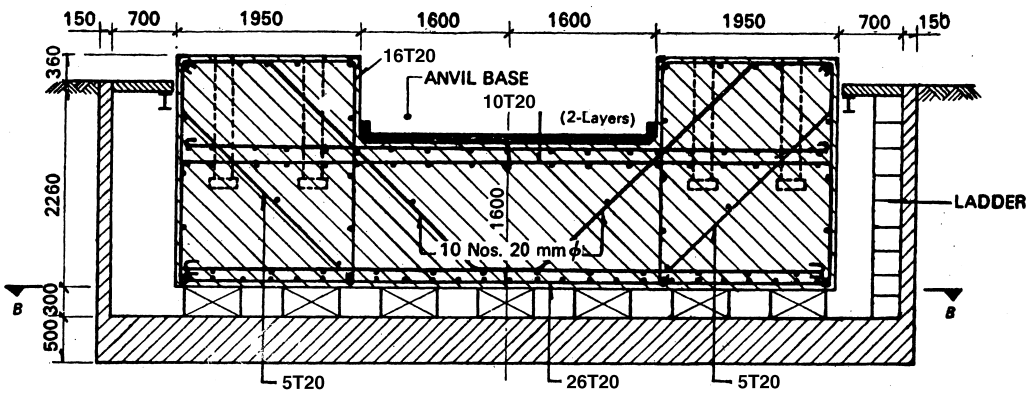


HAMMER FOUNDATION ON SOILS

SHEET NO. VII.6



PLAN AT B-B



SECTION Y-Y

VII.2.3.5. Preliminary dimensioning

1. Minimum thickness of foundation under anvil: 1.60 m
2. Thickness of elastic pad under anvil: 0.6–1.2 m (Generally 0.6)
3. Calculated dynamic force: 896 t

A typical structural details and reinforcement layout is shown on Sheet No. VII.7.

Testing machine with pulsator

1. Weight of machine complex: 8.3 t. Permissible bearing capacity of soil (broadly classified as sandy).
2. Data for unbalanced forces in machine:
 - (a) moving weight of pulsator: 45 Kg
 - (b) stroke length (S_p): ± 3.5 cm
 - (c) moving weight of testing machine: 700 Kg
 - (d) stroke length: ± 0.5 cm
 - (e) operating frequency: from 300–750 cps
3. Permissible amplitude: 0.5 mm.
4. Maximum operating frequency: 750 cpm.

Sheet No. VII.7 shows structural details of this kind of foundation. The foundation from the surrounding walls should be provided with 35 mm thick insulation boards as hatched.

VII.2.3.6. Pile details**Tips**

The tips of piles must be strong and rigid enough to resist distortion. Adequate wall thickness, reinforced as necessary, should be used for cast-in-place shells. Steel plate tips must have sufficient plate thickness to withstand local distortion. The connection (weldment) between tip plate and shell must be adequate to withstand repeated impact. The tip may be filled with concrete (precast) prior to driving.

Flat tips drive straighter and truer than pointed tips. Pointed or wedge shaped tips may aid penetration through overlying trash, etc., and may also be used to help penetration into decomposed rock. However, such tips may guide the pile off axial alignment. Blunt (rounded) tips will often accomplish the penetration through rock, etc., with a minimum of misalignment and point breakage.

Shoes

Pile shoes of cast or fabricated steel are used to protect and reinforce the tip of precast piles. They may be purposely sharpened to aid in cutting through buried timbers, etc.

Modern high strength concrete usually requires no shoe except in such special cases as:

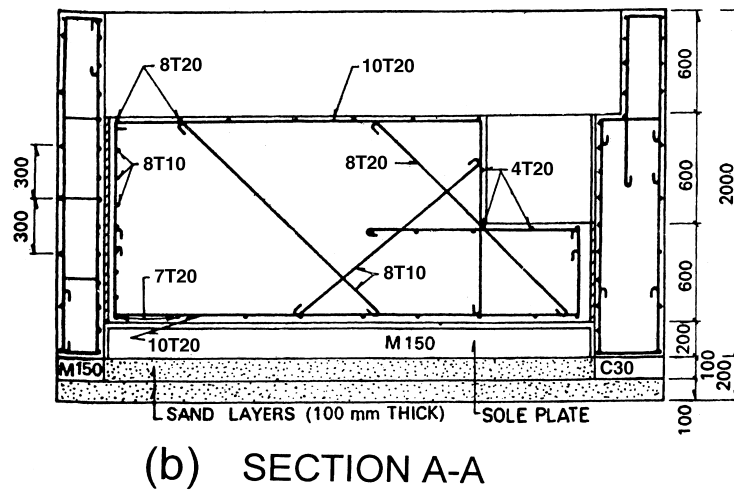
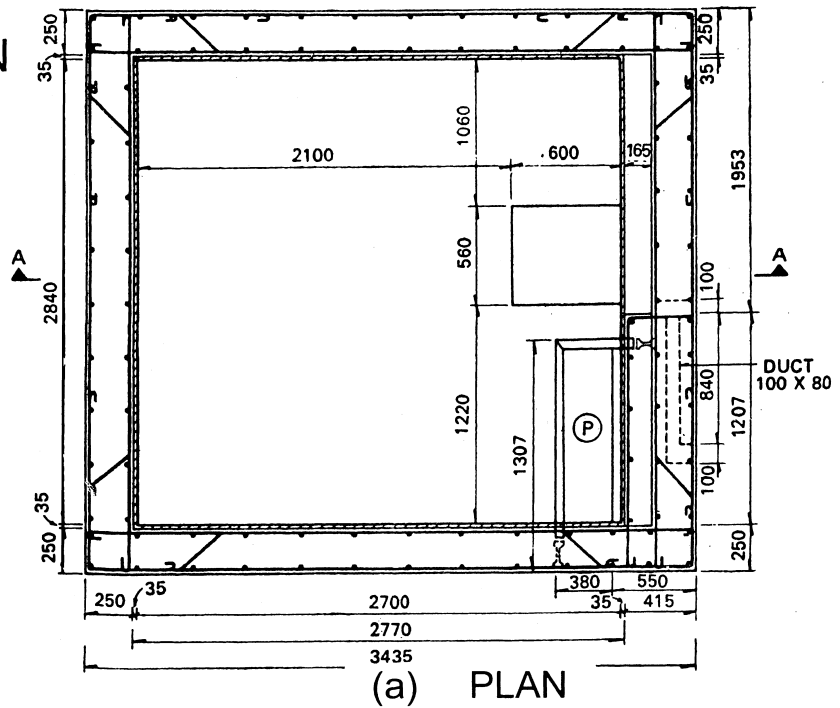
- (a) in driving through riprap, corner protection is desirable to prevent spalling
- (b) to aid in penetration into bedrock or decomposed rock.

Shoes should be securely anchored to the body or the pile. Particular care should be taken to ensure dense and full compaction of concrete in the shoe.

FOUNDATION FOR PULSATER

SHEET NO. VII.7

(A) PULSATER
FOUNDATION



The shoe should have a hole for escape of trapped air and water during casting of the pile.

Head details on precast piles

Attention is called to the following.

1. Adequate chamfers.
2. Extra lateral ties.
3. Square ends—no protuberances.
4. Adequate pile cushioning material.
5. Proper loose fit driving head; no ‘cocking’ of driving head.
6. Dowels in head.

Typical details of precast concrete piles are given on Sheet No. VII.8.

Splices

During driving and in service, splices should develop the requisite strength in compression, bending, tension, shear, and torsion at the point of splice. Splices are usually so located as to minimize these requirements; often direct bearing (compression) will be the only one requiring full pile strength.

Welded splices in shells or precast pile joints must consider the effect of repeated impact. Welding rod and techniques shall be selected for impact conditions. When welded splices are used with precast piles, the effect of heat and consequent splitting and spalling near the splice must be overcome.

Doweled splices, using cement or epoxy grout have been successfully used with precast piles under widely varying conditions. Adequate curing before driving must be attained.

When splicing load-bearing steel shells, backup plates or other suitable techniques should be employed to ensure full weld penetration especially for shells $\frac{3}{8}$ in. or thicker.

When splicing precast or prestressed piles, special care must be taken to avoid a discontinuity at the point of splice, which will result in tensile destruction of the pile. The use of epoxy grout and a doweled splice accomplishes continuity if properly installed. Use of a centre bearing plate is recommended with all-welded splices of solid piles and a central bearing ring must be used with all-welded splices of cylinder piles. Care must be taken in splicing to ensure concentric alignment.

Wedge splices or drive sleeves are successfully used with both precast piles and steel pipe shells. With pipe shells, inside drive sleeves may form an obstruction and care in the placement of concrete is required. With precast piles, care must be taken in soft driving to be sure the wedge is fully seated, so that the bottom section is not driven off.

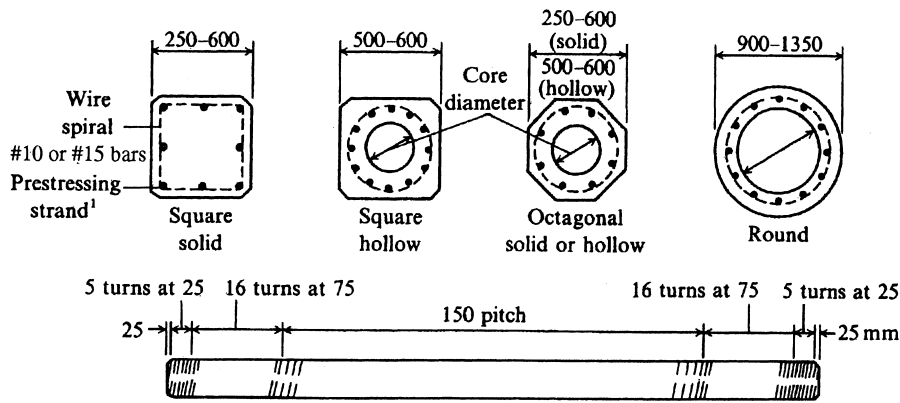
Stubs

Steel stubs are often installed protruding from the tip of concrete piles. They may be used to break-up hard strata, such as coral, ahead of the pile or to secure penetration of soft or disintegrated rock. The stubs may be H-piles, X-sections, or steel rail sections. They must have adequate wall thickness to prevent distortion. They must be selected so as to provide an adequate ratio of stiffness and strength to length of protrusion.

Heavy lateral ties are needed around the embedded portion of the stub. Concrete placement must be thorough, with good consolidation, and holes must be provided for the escape of air and water. Stubs may be combined with shoes to both facilitate penetration and protect the tip.

**PRECAST PILES
(BASED ON US PRACTICES)**

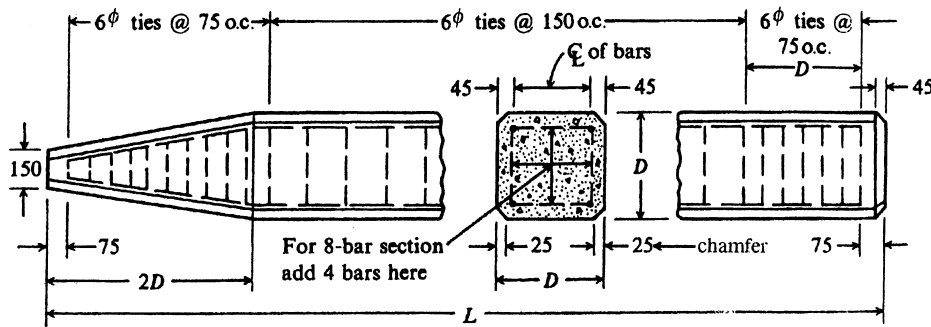
SHEET NO. VII.8



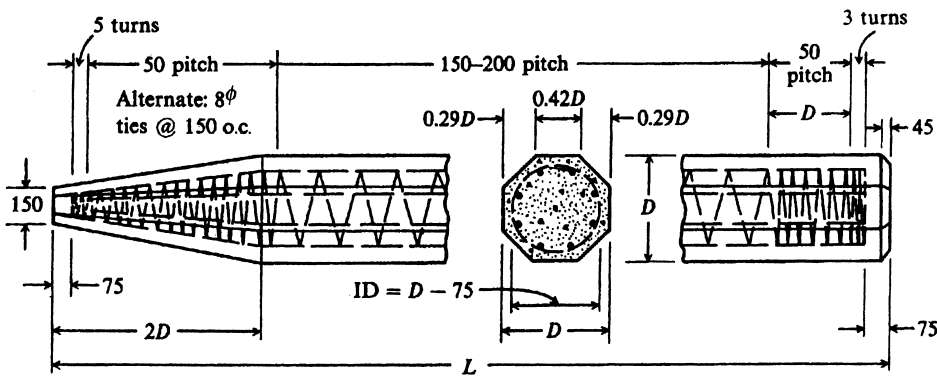
Strand: 9.5–12.7 mm ($\frac{3}{8}$ to $\frac{1}{2}$ in.) nominal diam., $f_u = 1860$ MPa

Typical prestressed concrete piles

dimensions in millimeters.



Square piles



Octagonal piles

Spiral wire			
D, mm	400	500	600
US. bar	#5	#4	#3
SI bar	15	10	10

$6\phi = 6$ mm diameter (not standard diameter SI bar)

Typical details of precast piles. Note all dimensions in millimeters. [After PCA (1951).]

Cut-off precast piles

Precast piles should be cutoff at the required elevation by suitable techniques that will prevent spalling or weakening of the concrete. A circumferential cut around the pile head will permit the use of pavement breakers without spalling. Clamps of timber or steel help prevent spalling. A slightly roughened surface of the cut head and sides to be embedded will aid bond between pile and cap. Explosives may cause damage to the adjoining concrete, so should generally not be permitted.

VII.3. Pile foundations

Where the bearing capacity of the soil is poor or the imposed loads are very heavy, piles, which may be square, circular or other shapes are used for foundations. If no soil layer is available, the pile is driven to a depth such that the load is supported through the surface friction of the pile. Sheet No. VII.9 gives the general layout of piles and pile caps and a typical generalized layout of pile and pile-cap reinforcement is shown in VII.9(a) and (b). Two-pile, three-pile, four-pile and six-pile foundation reinforcement details are given on Sheet Nos VII.9–VII.14. These details form part of the drawings to be submitted for the superstructures. For other types, see Sheet Nos VII.10 to VII.14.

The piles can be precast or cast in situ. One way of cast-in-situ construction is to drive into the soil a hollow tube with the lower end closed with a cast iron shoe or with a concrete plug. After the tube has reached the required depth, a steel reinforcement case is introduced in the pile and it is gradually filled with concrete. In the case of bored piles, a tube is driven into the ground and the soil inside the tube is removed with augers, etc. The rest of the procedure for the steel case and concrete is the same as discussed earlier.

Where precast piles are used, they are designed and detailed to:

- (a) bear the imposed load
- (b) withstand impact load during driving
- (c) withstand bending moments caused by self-load during handling.

Numerous empirical formulae exist for the evaluation of the bearing capacity of a pile. Texts and codes relevant to this area can be consulted for a better design of piles and pile caps. Sheet Nos VII.9 and VII.10 gives the reinforcement detailing of two types of piles. The nominal pile sections in millimetres are: 300×300 , 350×350 , 400×400 and 450×450 . For a 350×350 mm pile section, the bar diameter varies from 20 to 32 mm for pile lengths of 9 to 15 m. Sections 400×400 mm and above are provided with bars of diameter 20 to 40 mm for pile lengths of 9 to 15 m. The link spacings are generally 150 mm unless otherwise specified. The loads on piles range from 400 kN to 1000 kN.

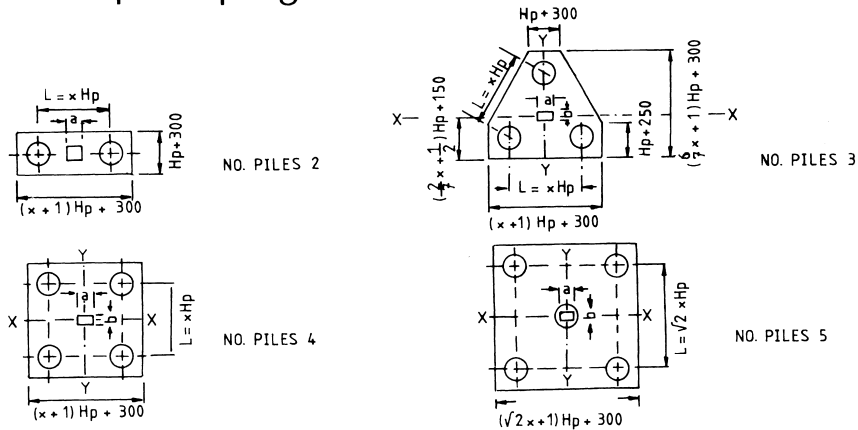
At both the head and the foot of the precast concrete pile, the volume of lateral reinforcement shall not be less than 1% for lengths of at least three times the size of the pile. The pile must have at least a safety factor of two.

For minimum bending during handling, the points of suspension may be taken for one-point suspension, $0.29l$, from one end and for two-point suspension $0.21l$ from two ends where l is the length of the pile.

PILE FOUNDATION

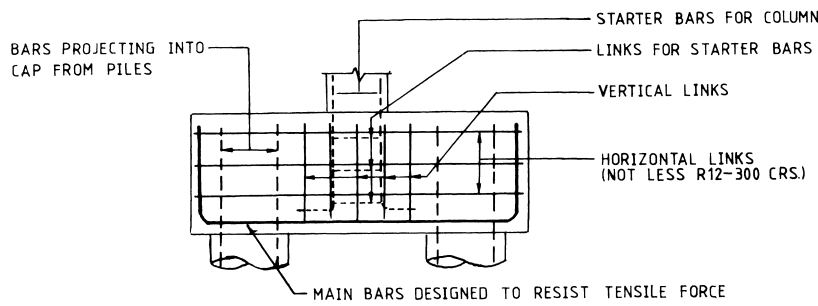
SHEET NO. VII.9

(a) Piles and pile caps: general recommendations

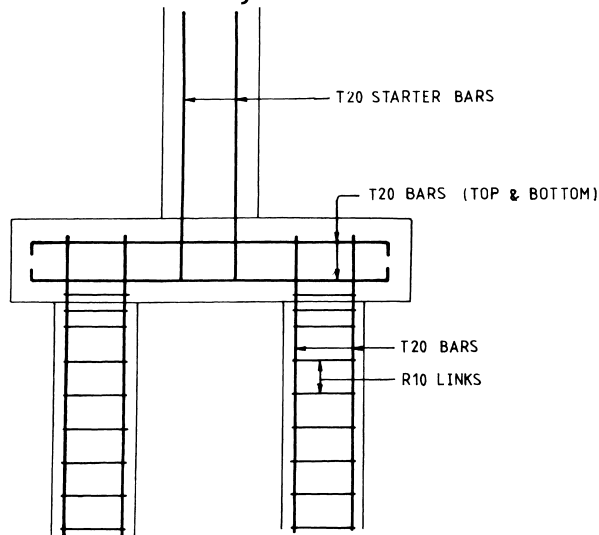


H_p DIA OF PILE: a, b DIMENSION OF COLUMN: x SPACING FACTOR OF PILES

(b) General reinforcement layout



(c) Two-pile cap reinforcement layout

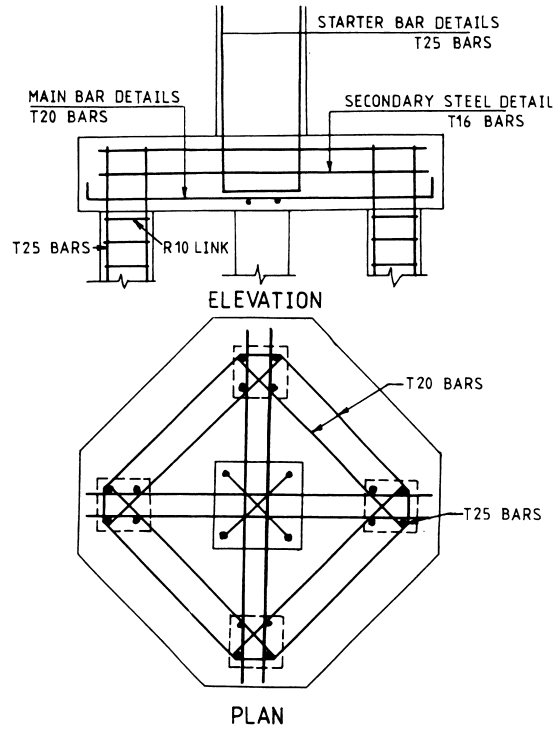
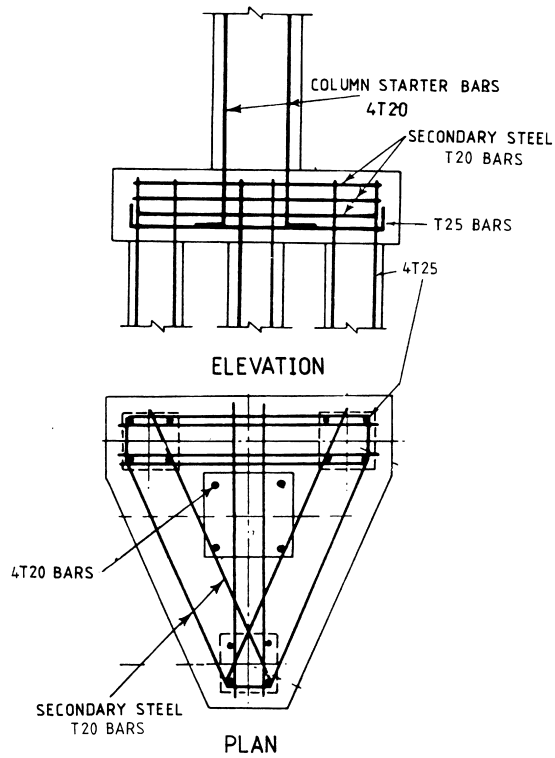


PILE FOUNDATION

SHEET NO. VII.10

Three-pile cap

Four-pile cap



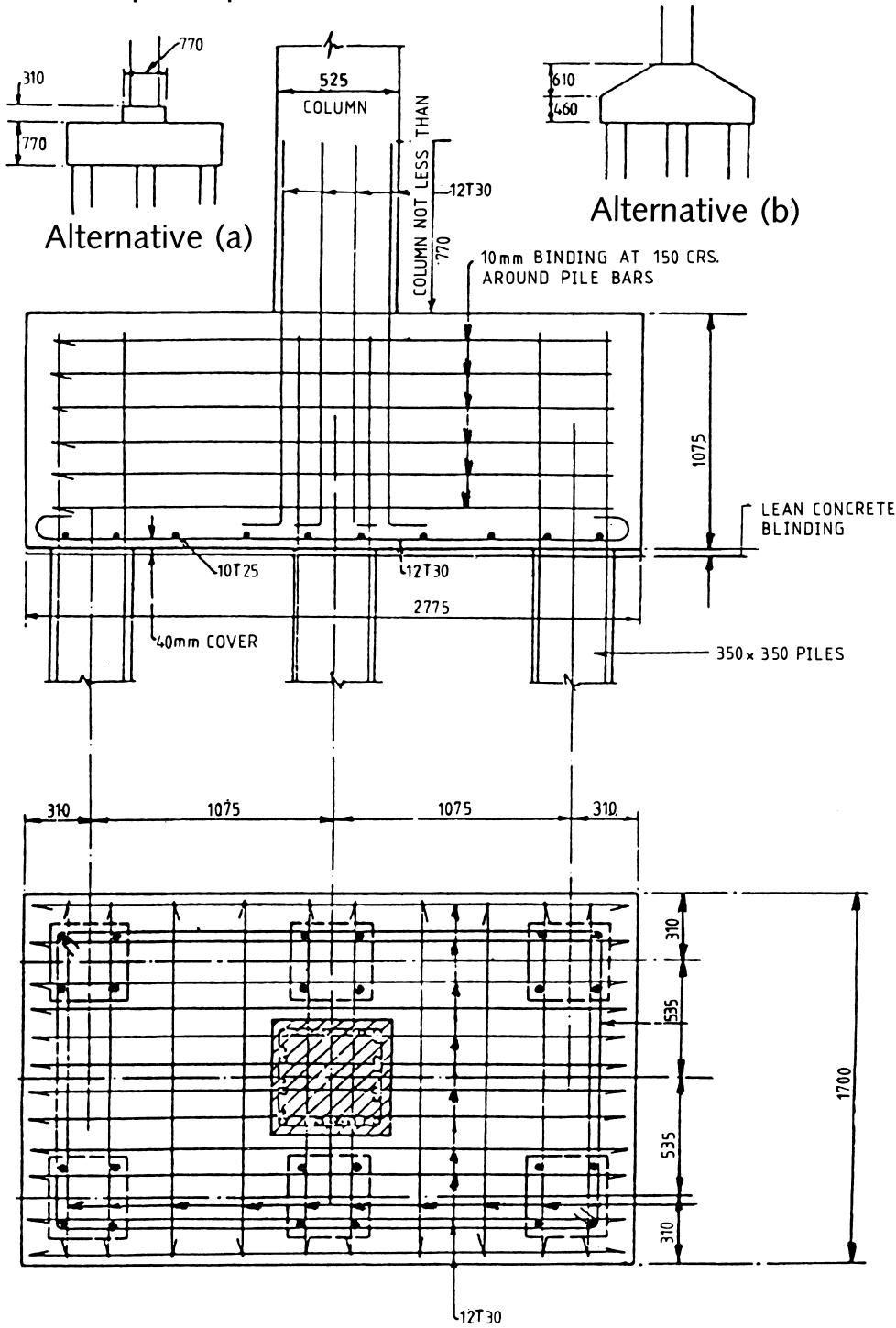
Tabular method

COLUMN REFERENCE	BASE TYPE	NO-OFF	BASE LEVEL	CUT OFF LEVEL	BASE REINFORCEMENT			SECTION	LINKS D	STARTER BARS
					A	B	C			

PILE FOUNDATION

SHEET NO. VII.11

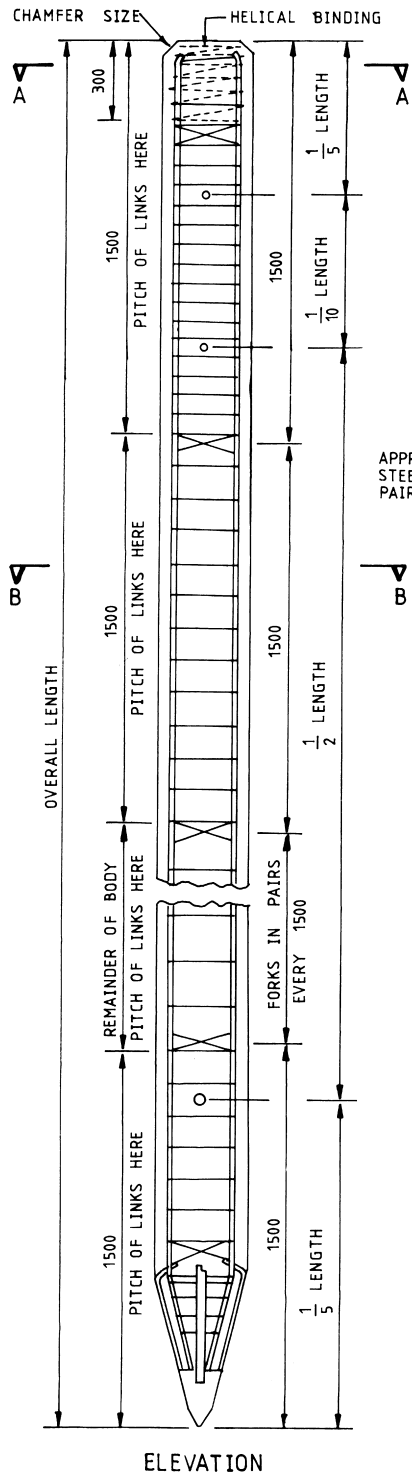
Six-pile cap



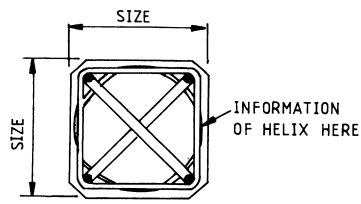
Details of pile cap

PRECAST CONCRETE PILES

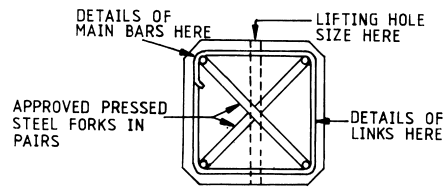
SHEET NO. VII.12



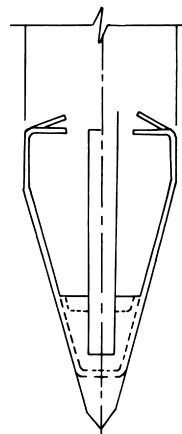
(a) Square pile



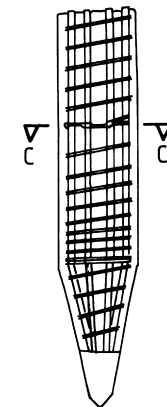
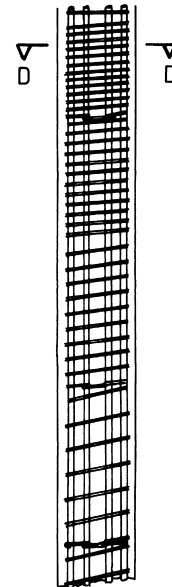
SECTION A-A



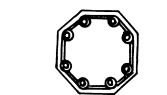
SECTION B-B



SHOE



ELEVATION

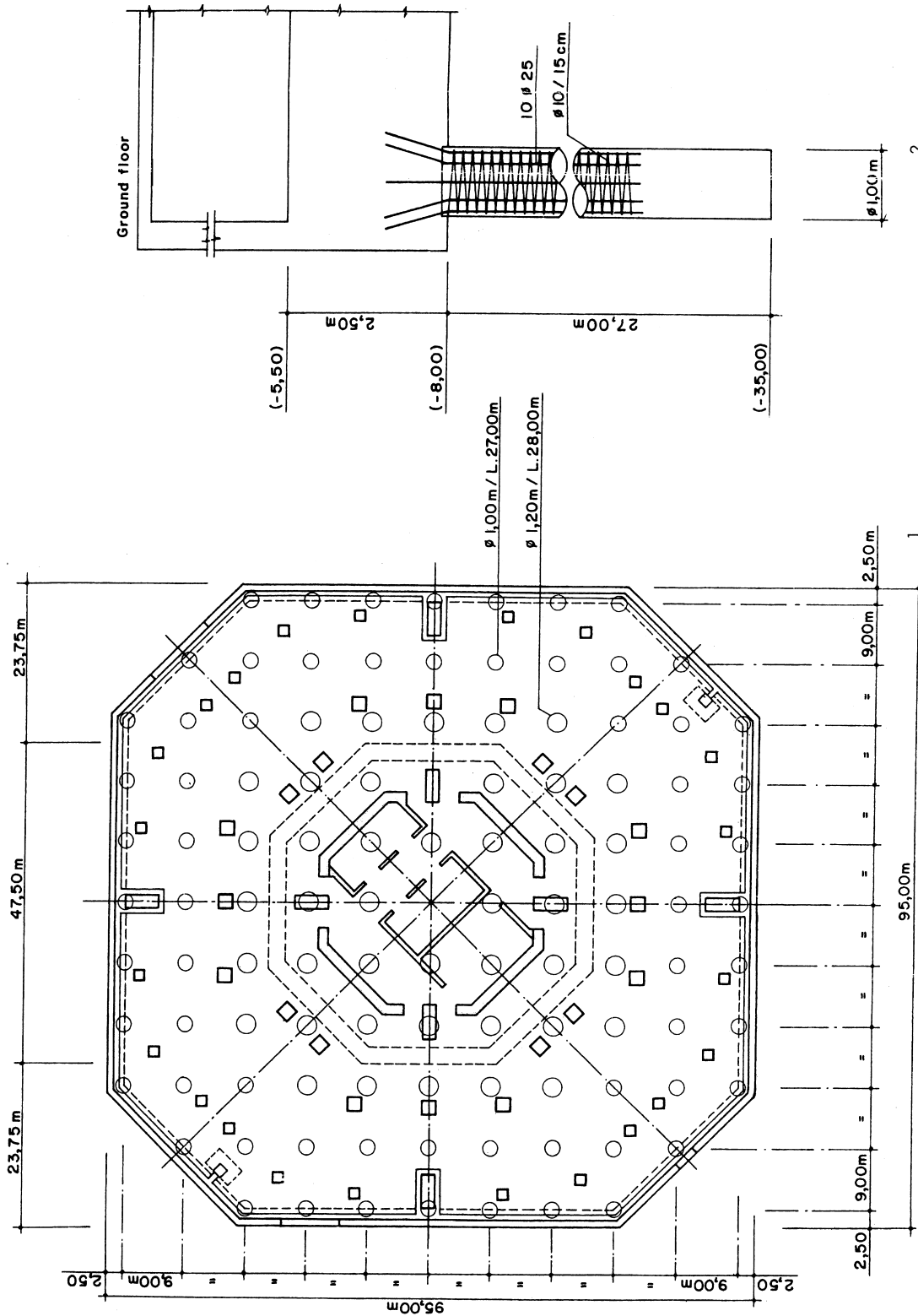


(b) Octagonal pile

FOUNDATION PLAN WITH TYPICAL BORED-PILE STEEL (BASED ON EC2)

SHEET NO. VII.13

1. Foundation plan; 2. Section of a bored-pile: reinforcement

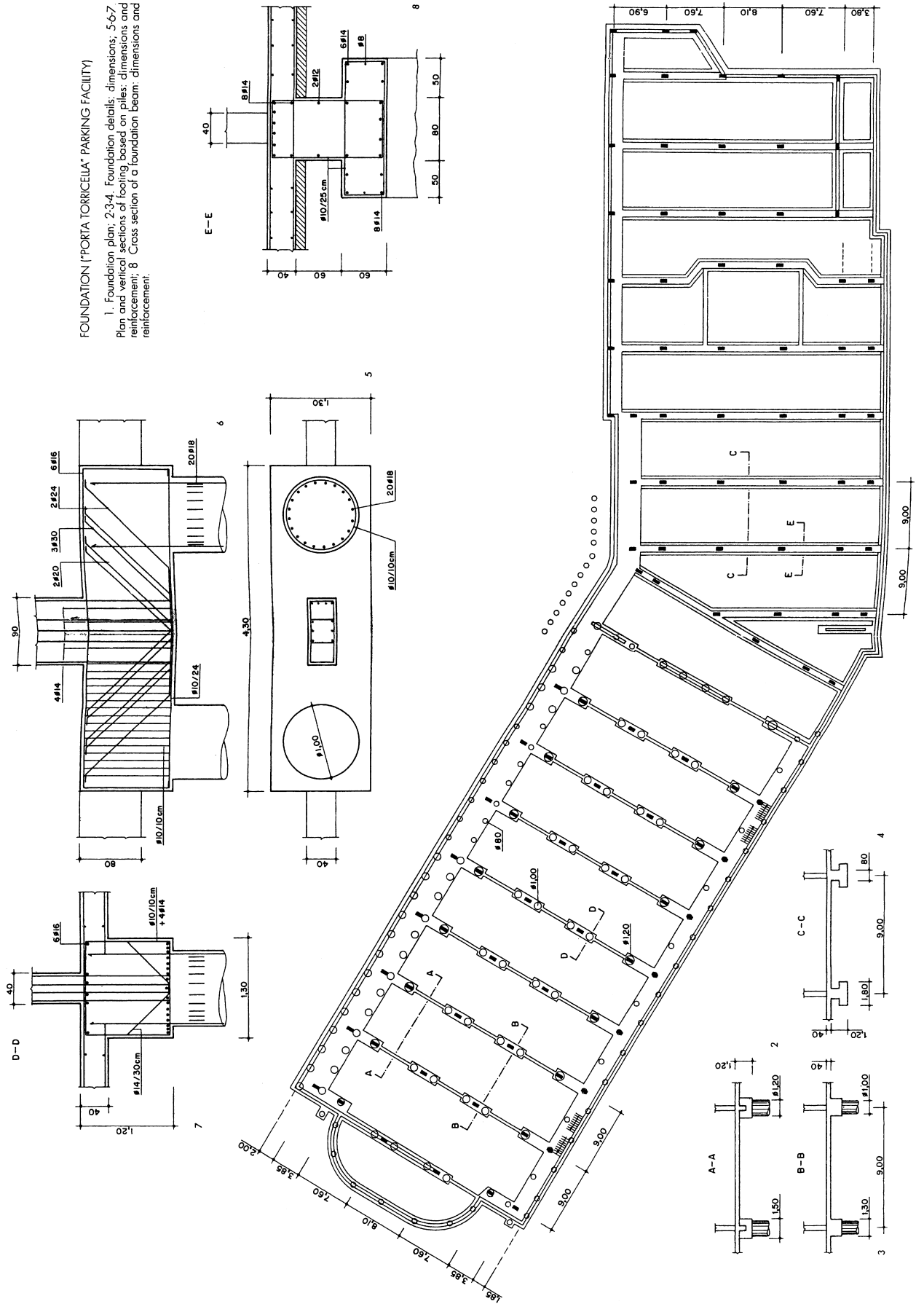


FOUNDATION FOR PARKING STRUCTURES

SHEET NO. VII.14

FOUNDATION ("PORTA TORRICELLA" PARKING FACILITY)

1. Foundation plan; 2,3,4. Foundation details: dimensions, 5-6-7. Plan and vertical sections of footing based on piles; dimensions and reinforcement; 8. Cross section of a foundation beam; dimensions and reinforcement.



Based on ENV 1997-1: 1994, the pile foundations must be checked against the following limit states.

1. Loss of overall stability.
2. Bearing resistance failure of the pile foundation.
3. Uplift or insufficient tensile resistance of the pile foundation.
4. Failure in the ground due to transverse loading of the pile foundation.
5. Structural failure of the pile in compression, tension, bending, buckling or shear.
6. Combined failure in ground and in the pile foundation.
7. Combined failure in ground and in the structure.
8. Excessive settlements.
9. Excessive heave.
10. Unacceptable vibrations.

Ground in which piles are located may be subject to displacement caused by consolidation, swelling, adjacent loads, creeping soil, landslides or earthquakes. These phenomena affect the piles by causing downdrag (negative skin friction), heave, stretching, transverse loading and displacement. For these situations, the design values of the strength and stiffness of the moving ground shall usually be upper values.

The ground displacement is treated as an action while an upper bound to the force which the ground could transmit to the pile or pile group shall be introduced as the design action.

If design calculations are carried out treating the downdrag force (negative skin friction) as an action, its value shall be the maximum which could be generated by large settlement of the ground relative to the pile.

Heave may take place during construction, before piles are loaded by the structure, and may cause unacceptable uplift or structural failure of the piles. In considering the effect of heave, or upward forces which may be generated along the pile shaft, the movement of the ground shall generally be treated as an action.

Transverse ground movements exert transverse loading on pile foundations. This transverse loading shall be considered if one or a combination of the following situations occur.

1. Different amounts of surcharge on either side of a pile foundation.
2. Different levels of excavation on either side of a pile foundation.
3. A pile foundation located at the edge of an embankment.
4. A pile foundation constructed in a creeping slope.
5. Inclined piles in settling ground.
6. Piles in a seismic region.

Transverse loading on pile foundations should normally be evaluated by considering the piles as beams in a deforming soil mass.

To demonstrate that the foundation will support the design load with adequate safety against bearing resistance failure, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations:

$$F_{CD} \leq R_{CD}$$

where F_{CD} is the ultimate limit state axial design compression load and R_{CD} is the sum of all the ultimate limit state design bearing resistance components of the pile foundation against axial loads, taking into account the effect of any inclined or eccentric loads.

In principle F_{CD} should include the weight of the pile itself and R_{CD} should include the overburden pressure of the soil at the foundation base. However,

these two items may be disregarded if they cancel approximately. They may not cancel when:

- (a) downdrag is significant
- (b) the soil is very light, or
- (c) the pile extends above the surface of the ground.

For piles in groups, two failure mechanisms shall be considered:

- (a) bearing resistance failure of the piles individually
- (b) bearing resistance failure of the piles and the soil contained between them acting as a block.

The design bearing resistance shall be taken as the lower of these two values.

In order to derive the ultimate design bearing resistance, the characteristic value, R_{ck} should be divided into components of base resistance, R_{bk} , and shaft resistance, R_{sk} such that:

$$R_{ck} = R_{bk} + R_{sk}$$

The ratio of these components may be derived from the load test results, e.g. when measurements of these components have been performed, or estimated using the methods of Section 7.6.3.3. of the code.

The design bearing resistance R_{cd} shall be derived from:

$$R_{CD} = \frac{R_{BK}}{\gamma_b} + \frac{R_{SK}}{\gamma_s}$$

where γ_b and γ_s are taken from Table VII.2.

Table VII.2. Values of γ_b , γ_s and γ_c

Component factors	γ_b	γ_s	γ_c
Driven piles	[1.3]	[1.3]	[1.3]
Bored piles	[1.6]	[1.3]	[1.5]
CFA (continuous flight auger) piles	[1.5]	[1.3]	[1.4]

To demonstrate that the foundation will support the design load with adequate safety against failure in tension, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations:

$$F_{td} \leq R_{td}$$

where F_{td} is the ultimate limit state axial design tensile load and R_{td} is the ultimate limit state design tensile resistance of the pile foundation.

For tension piles, two failure mechanisms shall be considered:

- (a) pull out of the piles from the ground mass
- (b) uplift of the block of the ground containing the piles.

For isolated tension piles or a group of tension piles, failure may occur by pulling out of a cone of ground especially for a pile with an oversized base or rock socket.

To demonstrate that there is adequate safety against failure of piles in tension by uplift of the block of soil containing the piles, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations:

$$F_{td} \leq W_d - (U_{2d} - U_{1d}) + F_d$$

where F_{td} is the design tensile force acting on the group of piles, W_d is the design weight of the soil block (including the water) and the piles, F_d is the design shear resistance at the boundary of the block of soil, U_{1d} is the design downward force due to the water pressure on the top of the pile foundation, and U_{2d} is the design upward force due to the water pressure on the base of the soil block.

Based on the above specifications, Sheet No. VII.9 and VII.13 give details for the foundation plans with typical steel in bore piles. Sheet Nos VII.10 and VII.11 show the foundation plans with details of vertical sections with reinforcement based piles. A cross-section of the foundation begun with reinforcement is also shown. Also, the sheet shows a plan of the footing and the reinforcement for piles.

The following are based on American Practices (ASCE, ACI and US Corps of Engineers).

VII.3.1. Types of concrete piles

The two most common types of concrete piles are (1) precast and (2) cast in situ. Of these, precast piles may be constructed to specifications at a casting yard or at the site itself if a large number of piles are needed for the particular construction. In any case, handling and transportation can cause intolerable tensile stresses in precast, concrete piles. Hence, one should be cautious in handling them so as to minimize bending moments in the pile.

Cast-in-situ piles are of two classes:

- (a) cased type, these are piles that are cast inside a steel casing that is driven into the ground
- (b) uncased type, these are piles that are formed by pouring concrete into a drilled hole.

VII.3.2. Precast piles

Piles can be manufactured in a casting yard adjacent to the construction site for a large project, or may be made in a central plant and transported to the site by rail, barge, or truck. See Sheet No. VII.12 for reinforcement layouts.

VII.3.3. Square and octagonal piles

Except for a taper at the point, piles are of uniform section throughout their length. Choice of square or octagonal pile is a matter of personal preference. Hexagonal and octagonal piles are similar. Octagonal piles are more prevalent as their flexural strength is the same in all direction, rotation during driving is not as noticeable, lateral ties are easily made in the form of a continuous spiral, and can be made in a wood or steel form without a need for chamfers.

Advantages of square piles are that the longitudinal steel is better located to resist flexure, easy to form in banks or tiers, have more square area for volume of concrete and easier to place concrete.

The size of piles can vary to suit almost any condition. Piles, 24 in. (610 mm) square over 100 ft (30 m) long are common and piles as small as 6 in. (150 mm) are also made. Reinforcement consists of longitudinal bars in combination with spiral winding or hoops as shown on the plans and are designed to resist handling and driving loads as well as service loads. Coverage over the steel is at least 2 in. (50 mm) except for small piles of exceptionally dense concrete where less cover is permitted. Piles exposed to sea water or freezing and thawing cycles while wet are required to have 3 in. (76 mm) of cover. Splices of longitudinal steel are required to be staggered.

The ends of the pile are called the head and the point. The head should be carefully made, smooth, and at right angles to the axis of the pile. It is deeply chamfered on all sides, and extra reinforcements are provided for a distance equal to the diameter of the pile. The shape of the point depends on the expected soil.

VII.3.4. Hollow cylindrical piles

A cylindrical pile is made of a series of precast sections placed end to end and held together by post-tensioned cables. The pile sections are manufactured by centrifugal spinning and contain a small amount of longitudinal and spiral reinforcement to facilitate handling. Longitudinal holes for the prestressing wires are formed into the wall of each section by means of a mandrel enclosed in a rubber hose.

After curing, the sections are assembled end to end on the stressing racks with the ducts lined up and high strength polyester resin adhesive is applied to the joints surfaces. Post-tensioning wires are threaded through the ducts, tensioned, the ducts are pressure grouted, and the completed pile remains in the rack until the grout cures. Following curing of the grout, the prestressing wires are burned off and the pile is ready for shipping or driving.

VII.3.5. Cast-in-place piles

These piles are constructed by drilling a shaft in the earth, placing the reinforcement cage and filling with concrete or by driving a metal shell by means of a mandrel. After driving, the sheet metal shell is filled with concrete.

Cylinder piles are manufactured in standard diameters of 36 and 54 in. (915 to 1370 mm) with wall thickness and number of stressing cables depending on the project requirements. The 36 in. (915 mm) piles have wall thickness from 4 to 5 in. (100 to 125 mm) and contain 8, 12 or 16 prestressing cables. The 54 in. (1370 mm) piles have a thickness of $4\frac{1}{2}$, 5 or 6 in. (115, 125 or 150 mm) and contain 12, 16 or 24 cables.

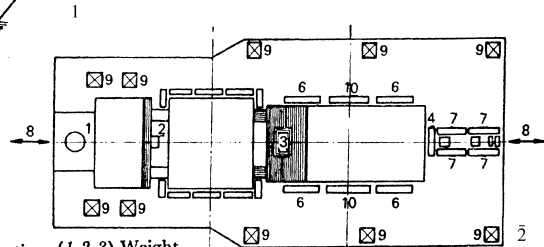
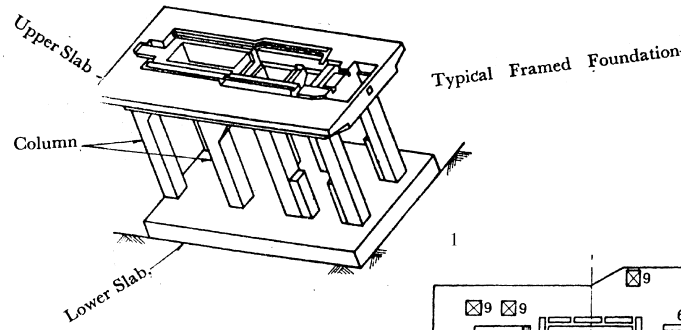
VII.3.6. Framed foundations for high-speed machinery *VII.3.6.1. Introduction*

High-speed machines such as turbo-generators are generally mounted on framed-type foundations (Sheet No. VII.15). The turbo-generator foundation is a vital and expensive part in a power plant complex. It is, therefore, essential that the foundation is designed adequately for all possible combinations of static and dynamic loads. While the mechanical engineers usually furnish the layout diagram—showing the broad outlines of the foundation and clearances required for piping, linkages, etc., and also the loading diagram—it is the task of the structural designer to check the adequacy of the foundation under static and dynamic conditions. At times, it may become necessary to suitably alter the dimensions of the foundation as suggested by the machine manufacturers so as to satisfy the design requirements. Any alterations thus found necessary must, however, have the prior concurrence of the mechanical engineers to ensure that these changes do not affect the erection or operation of the machine. It is desirable, therefore, to have a close coordination between the foundation designers and the erection staff (of the mechanical and electrical installations) from the early planning stage until the foundation is completed and the machine installed. Various framed foundation types are given on Sheet Nos VII.15 to VII.18.

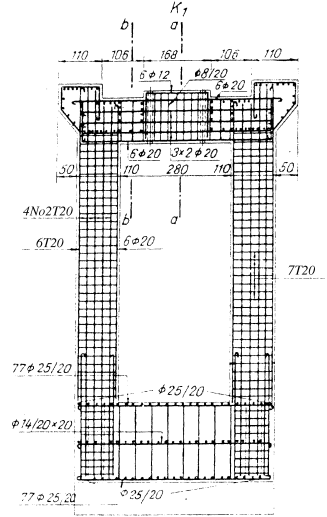
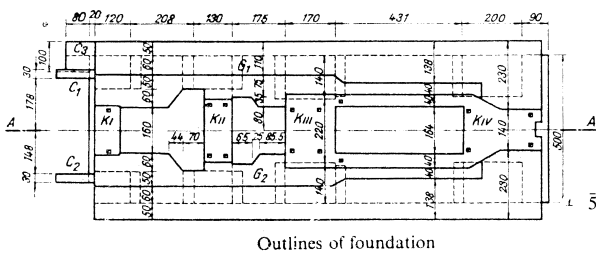
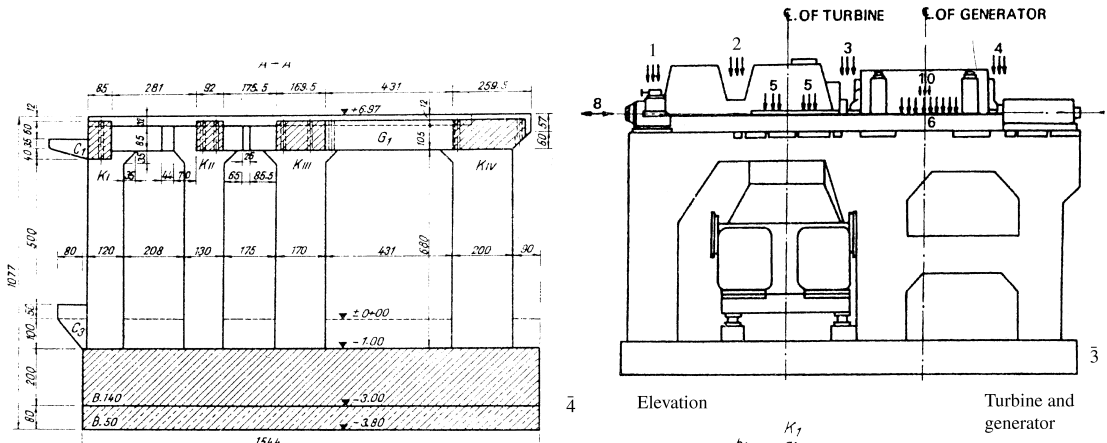
The conventional framed foundation consists of a heavy foundation slab (called 'sole plate') which is supported from underneath by soil (or piles) and which supports on it the top series of columns. The columns are connected at their top ends by longitudinal and transverse beams forming a rigid table (called 'upper plate' or 'table plate') on which rest the turbine and generator. The condensers rest generally on independent supports below the turbine portion on the foundation slab.

FRAMED FOUNDATIONS (BASED ON EC2)

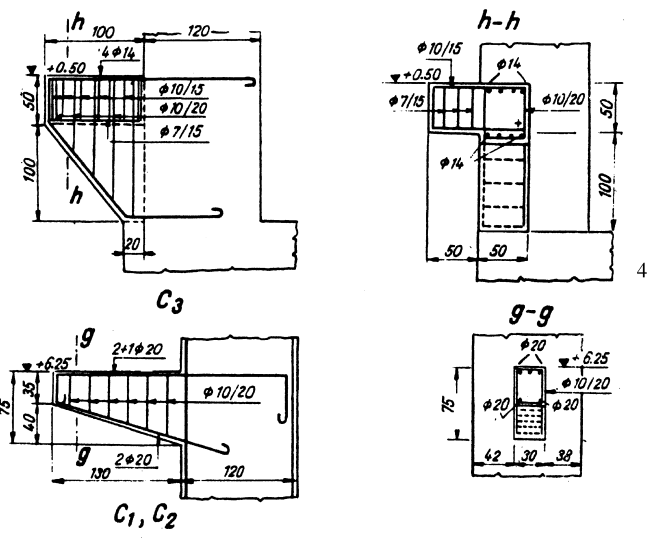
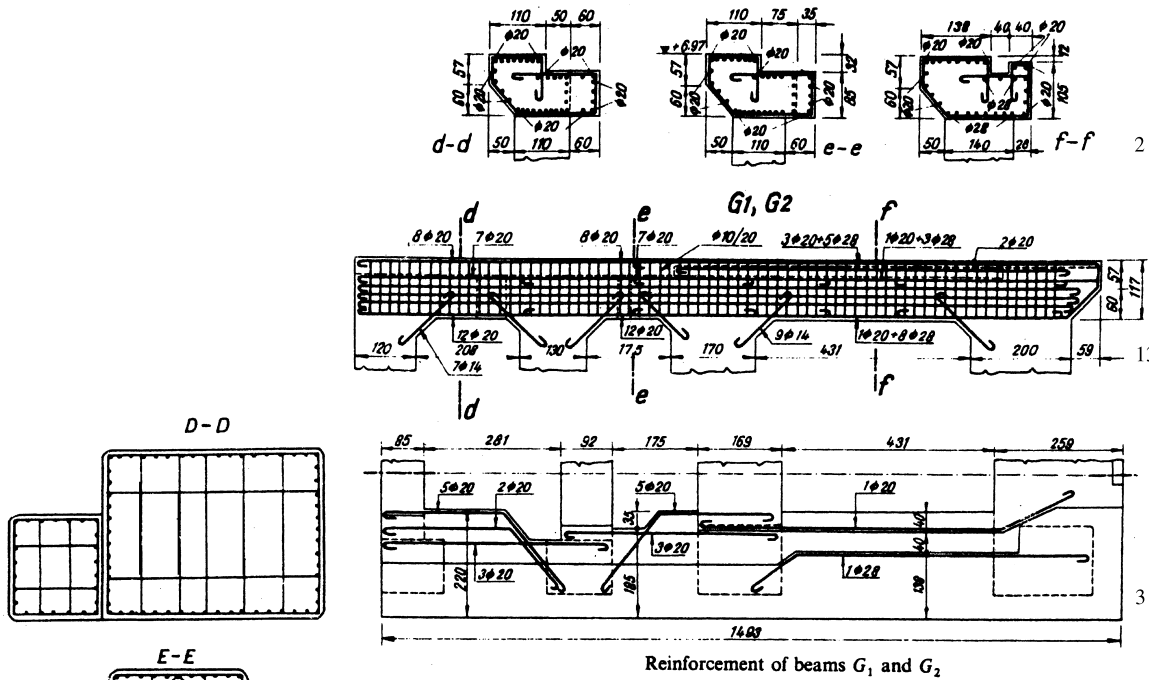
SHEET NO. VII.15



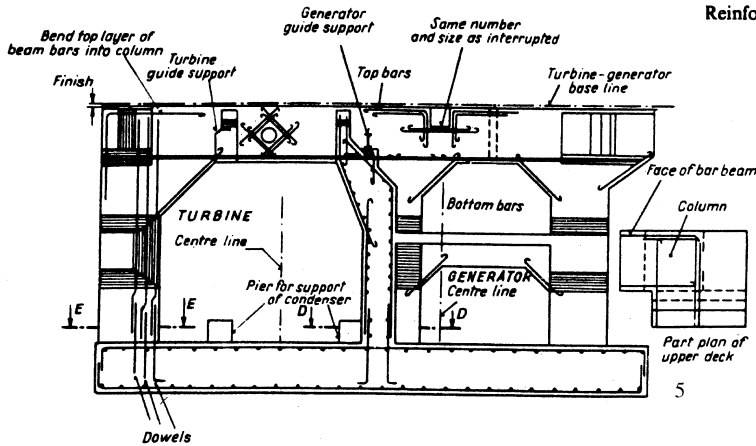
Loading Diagram for a Turbo-generator Foundation—(2) Plan, (3) Elevation. (1,2,3) Weight Due to Cylinder, Turbine Rotor and Foundation Plates, (3,4) Weight Due to Cylinder, Bearing Plate and Rotors of Generator.



Note: all dimensions cm

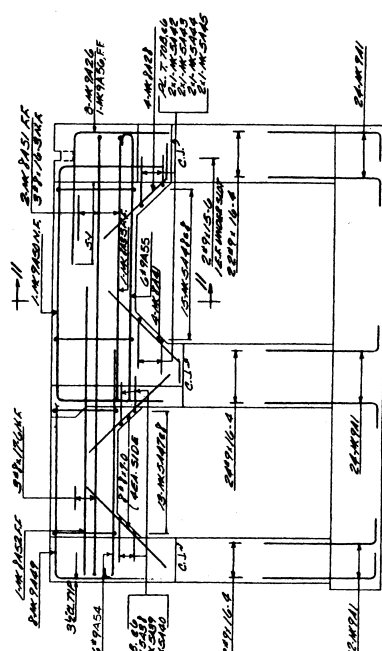
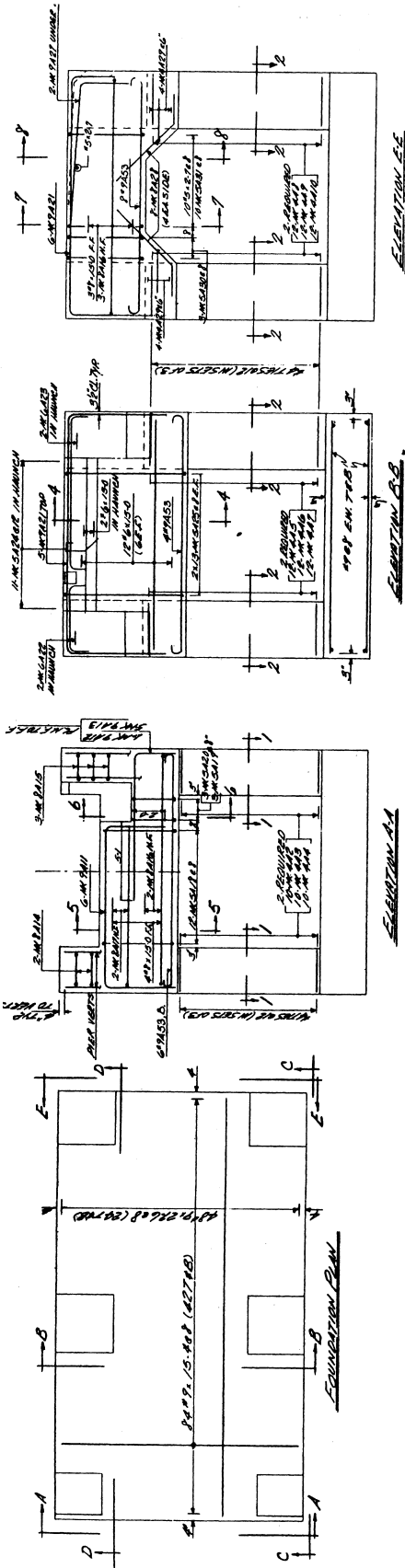


Note: all dimensions cm

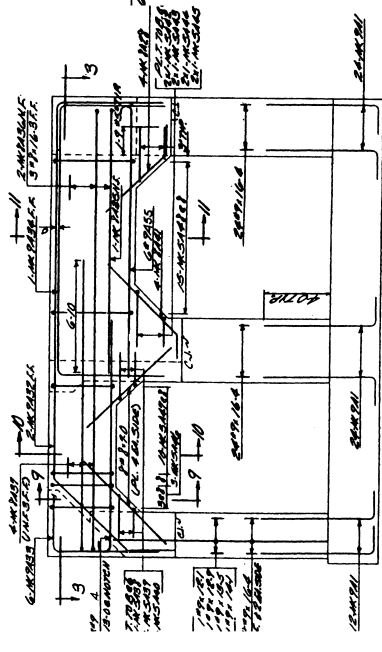


TURBINE PEDESTAL FOUNDATIONS

SHEET NO. VII.17

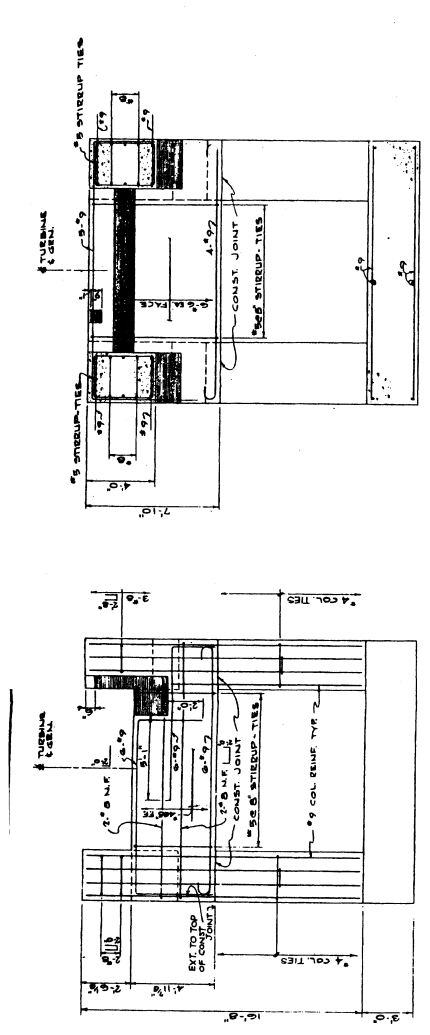


ELEVATION D.D



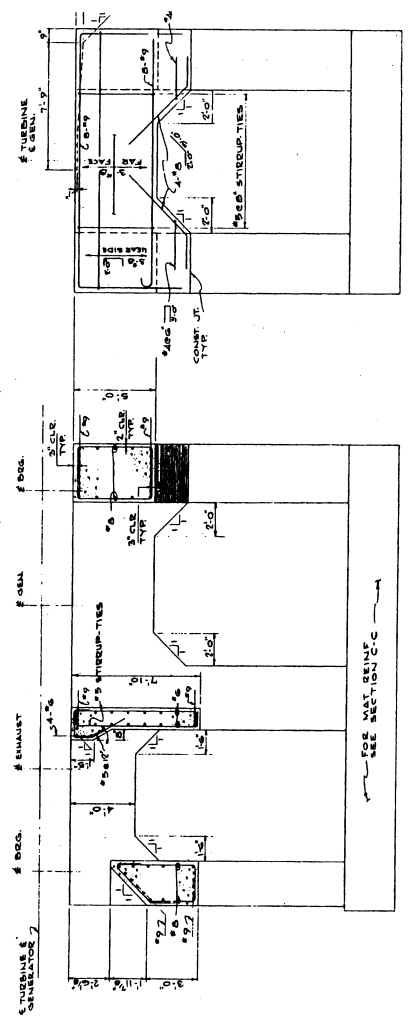
ELEVATION C.C

TURBINE PEDESTAL FOUNDATIONS (CONTD)



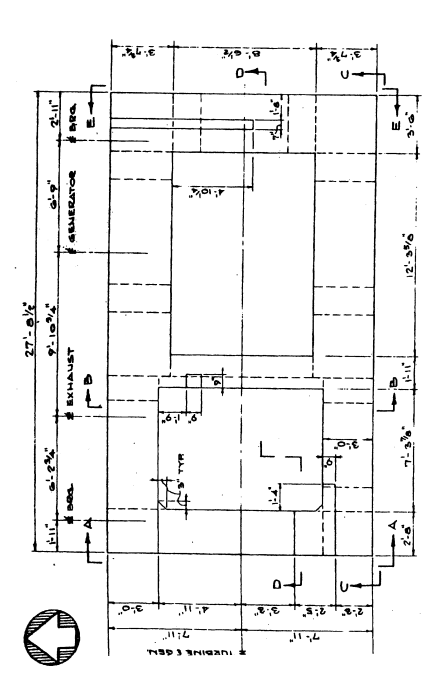
SECTION B-B
SCALE: 3/8" = 1'-0"

SECTION A-A
SCALE: 3/8" = 1'-0"

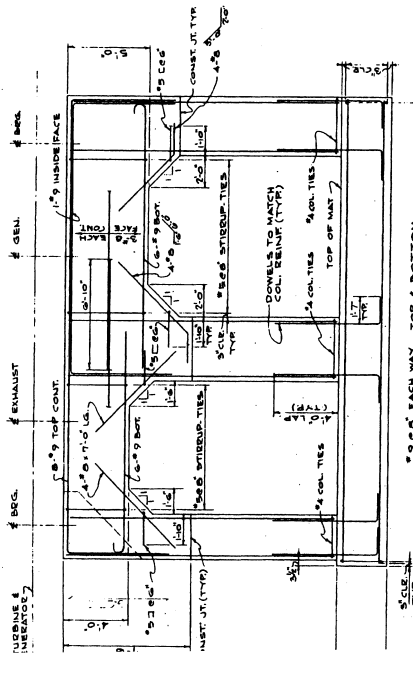


SECTION E-E
SCALE: 3/8" = 1'-0"

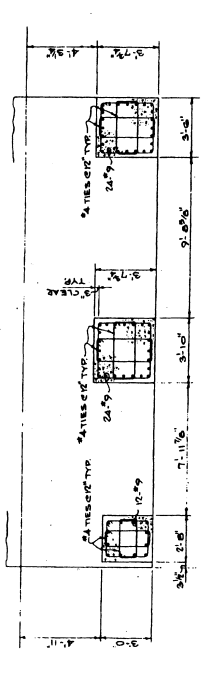
SECTION D-D
SCALE: 3/8" = 1'-0"



PLAN - TOP OF PEDESTAL
SCALE: 3/8" = 1'-0"



SECTION C-C
SCALE: 3/8" = 1'-0"



VII.3.6.2. Design data

Machine data

1. The data required include a detailed loading diagram showing the magnitude and position of all loads (static and rotating loads separately) acting on the foundation. The loading diagram should contain not only the loads but also the area over which the loads will be distributed on the foundation.
2. The rated capacity of the machine.
3. Operating speed of machine.
4. The layout of auxiliary equipment and platforms at the floor level of the machine hall.
5. The distribution of pipe lines and the temperature of their outer surfaces.
6. A detailed drawing showing the sizes and location of all anchor bolts, pipe lines, chases, pockets, inserts, etc.

Soil data

1. Soil profile and characteristics of soil up to at least thrice the width of the turbine foundation or until hard strata are reached.
2. The relative positions of the groundwater table in different seasons of the year.

VII.3.7. Special considerations in planning

1. The foundation should be completely separated from the main building or other neighbouring foundations by providing a clear gap all around. This avoids transfer of vibrations to the surroundings.
2. All the beams and columns of the foundation should be provided with adequate haunches to avoid stress concentration and to ensure rigidity.
3. The base slab should be rigid to prevent non-uniform settlement of soil.
4. The transverse frames shall be located directly under bearings thus avoiding the eccentric loading on transverse girders.
5. The minimum thickness of the slab shall not be less than 2 m.
6. The foundations should be dimensional so that from all parts the resultant of the load passes through the centre of the base area in contact with the soil.

VII.3.8. Turbine pedestal using American practice

All design practices are identical to the British and European practices. Sheet Nos VII.17 and VII.18 show drawings as examples of heavy construction. Due to the complexity of bar arrangement, the detailer found it necessary to draw complete elevations and cut sections through every member.

The beams change in size and some of the beams are recessed or cut away. It is important to show the bar arrangement in each case. For instance, notice the top of the elevation. A sloping trough interrupts half of the length of the beam. This required two of the top #9 bars to be bent below the trough and the beam stirrups to be arranged around the sloping recess.

Another unusual detail is shown in elevation A–A. A portion of the beam has been cut away which in turn has caused a considerable rearrangement of beam bars and stirrups.

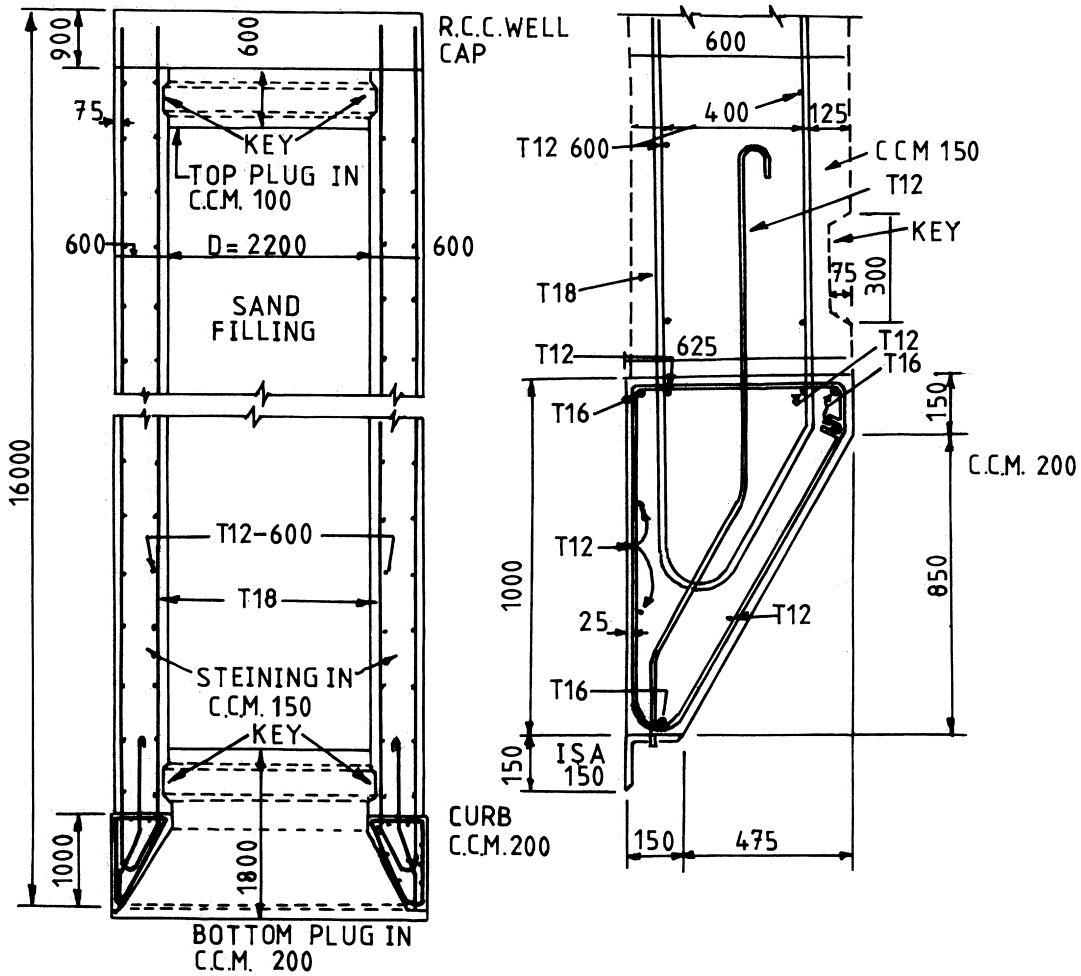
VII.4. Well foundations and caissons

These foundations are used for supporting bridge piers and abutments. The well foundation generally consists of steining, bottom plug, sand filling, top plug and well cap. The well cap is described in the next section on bridges.

The structural detailing of such a foundation is shown on Sheet No. VII.19. The well kerb carrying the cutting edge is made up of reinforced concrete. The

WELL FOUNDATION

SHEET NO. VII.19

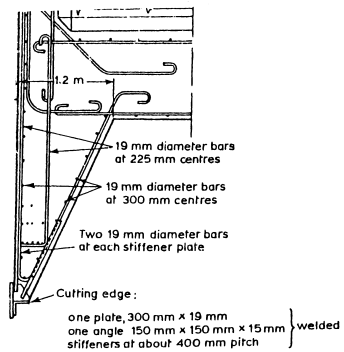
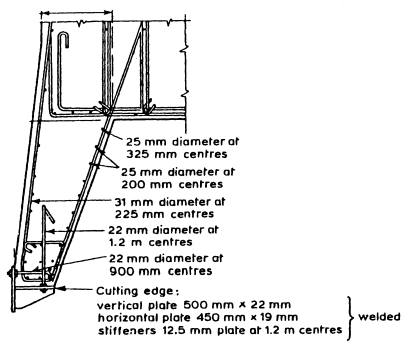


Well

Cutting edge for a caisson forming part of a dock wall, designed for use in boulder clay.

Detail of curb

Cutting edge for pump-house caisson sunk through sand to rock:



cutting edge consists of a mild steel angle of 150 mm side. Where boulders are expected, the vertical leg is embedded in steining with the horizontal leg of the angle remaining flush with the bottom of the kerb. The steining consists of either brick masonry or reinforced concrete. The thickness of the steining should not be less than 450 mm nor less than given by the following equation:

$$t = k(0.01H + 0.1D)$$

where:

- t = minimum thickness of steining
- H = full depth to which the well is designed
- D = external diameter of the well
- K = subsoil constant
 - = 1.0 sandy strata
 - = 1.1 soft clay
 - = 1.25 hard clay
 - = 1.30 for hard soil with boulders.

The concrete steining shall be reinforced with longitudinal and hoop bars on both faces of the well. The bottom plug is provided up to a height of 0.3 m for small-diameter and 0.6 m for large-diameter wells. The concrete shall not be less than grade 30. A top plug is provided with a thickness of about 0.3 m beneath the well cap and on top of the compacted sand filling. The space inside the well between the bottom of the plug and top of the bottom plug is usually filled with clean sand. This is needed to provide stability and to prevent the well overturning.

VII.4.1. Caissons

The word *caisson* is derived from the French *caisse*, which means a chest or box. When applied to foundation engineering, it describes a prefabricated hollow box or cylinder that is sunk into the ground to some desired depth and then filled with concrete, thus forming a foundation. Caissons have most often been used in the construction of bridge piers and other structures that require foundations beneath rivers and other bodies of water because the caissons can be floated to the job site and sunk into place.

Caissons must be designed to resist the various loads imparted during construction, as well as the structural and hydrodynamic loads from the completed structure. In addition, it must have sufficient weight to overcome the side friction forces as it descends into the ground.

VII.4.1.1. Open caissons

An *open caisson* is one that is open to the atmosphere. They may be made of reinforced concrete, and normally have pointed edges at the bottom to facilitate penetration into the ground as shown for well foundations.

VII.4.1.2. Pneumatic caissons

When the excavation inside open caissons extends well below the surrounding water level, water flowing into the bottom can produce a quick condition in the soils. This is most likely to occur in clean sands and is caused by the upward seepage forces of the flowing water.

This is often necessary when constructing jetties or cooling water intake shafts over water. Before construction can commence it is necessary to build a temporary jetty strong enough to carry the cylinder until it becomes supported in the ground, together with a kentledge frame and all excavating

machinery. The shape of a caisson will, in most cases, be dictated by the requirements of the superstructure.

VII.4.1.3. Pier construction with box caissons

Box caissons are hollow structures with a closed bottom designed to be buoyant for towing to the bridge site and then sunk on to a prepared bed by admitting water through flooding valves. In sheltered conditions the top can be left open until sinking and ballasting is completed, or a closed top can be provided for towing in rough water.

Box caissons are unsuitable for founding on weak soils, or for sites where erosion can undermine the base, They are eminently suitable for founding on compact granular soils not susceptible to erosion by scour, or on a rock surface which is dredged to remove loose material, trimmed to a level surface and covered with a blanket of crushed rock. Skins are provided to allow the caisson to bed into the blanket and a cement-sand grout is injected through.

Sheet No. VII.20 shows a typical box section caisson for the Queen Elizabeth II Bridge at Dartford, designed and constructed by Trafalgar House Technology. The single box caisson at each pier position is 59 m × 28.6 m × 24.1 m deep resisting the impact load of a 65 000 t ship travelling at 10 knots. The size of the base is governed by the resistance to horizontal shear on an assumed plane of weakness.

The walls of caissons should be set back for a distance of 25–75 mm from the shoe. The thickness of the wall is dictated by the need for great rigidity to resist severe stresses which may occur during sinking, and the need to provide sufficient weight for overcoming skin friction. Lightness of the wall construction for the stages of floating-out and lowering is achieved by hollow steel plate construction, using 6–7 mm thick skin plates stiffened by vertical and horizontal trusses as provided in the caisson shoes. The plating should be arranged in strakes about 1.2–2.4 m high. These are convenient heights of lift for sinking and concreting in a 24-hour cycle.

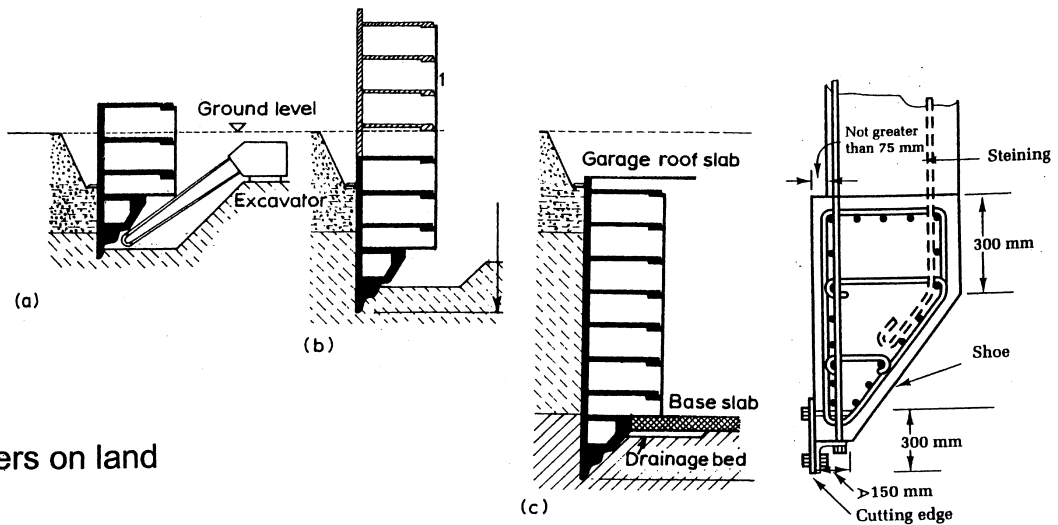
Pneumatic caissons are used in preference to open-well caissons in situations where dredging from open wells would cause loss of ground around the caisson, resulting in settlement of adjacent structures. They are also used in sinking through variable ground or through ground containing obstructions where an open caisson would tend to tilt or refuse further sinking. Pneumatic caissons have the advantage that excavation can be carried out by hand in the 'dry' working chamber, and obstructions such as tree trunks or boulders can be broken out from beneath the cutting edge. Also the soil at foundation level can be inspected, and if necessary bearing tests made directly upon it.

The foundation concrete is placed under ideal conditions in the dry, whereas with open-well caissons the final excavation and sealing concrete is almost always carried out underwater.

Pneumatic caissons have the disadvantage, compared with open-well caissons, of requiring more plant and labour for their sinking, and the rate of sinking is usually slower. There is also the important limitation that men cannot work in air pressures much higher than 3.5 bar, which limits the depth of sinking to 36 m below the water table, unless some form of groundwater lowering is used externally to the caisson. If such methods are used to reduce air pressures in the working chamber they must be entirely reliable, and the dewatering wells must be placed at sufficient distance from the caisson to be unaffected by ground movement caused by caisson sinking.

CAISSONS

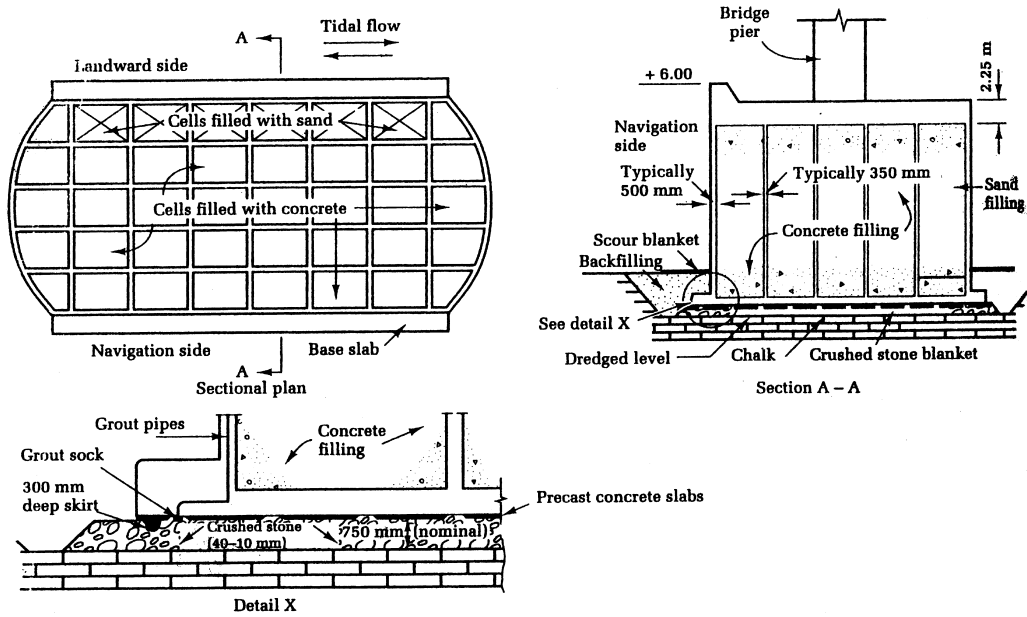
SHEET NO. VII.20



(A) Cylinders on land

- (a) Concrete placed for cutting edge and three storeys of basement;
- (b) concreting, excavation and sinking proceed simultaneously;
- (c) concrete base slab placed.

(B) Box Caissons



Box caisson for Queen Elizabeth II Bridge, Dartford (courtesy of Trafalgar House Technology).

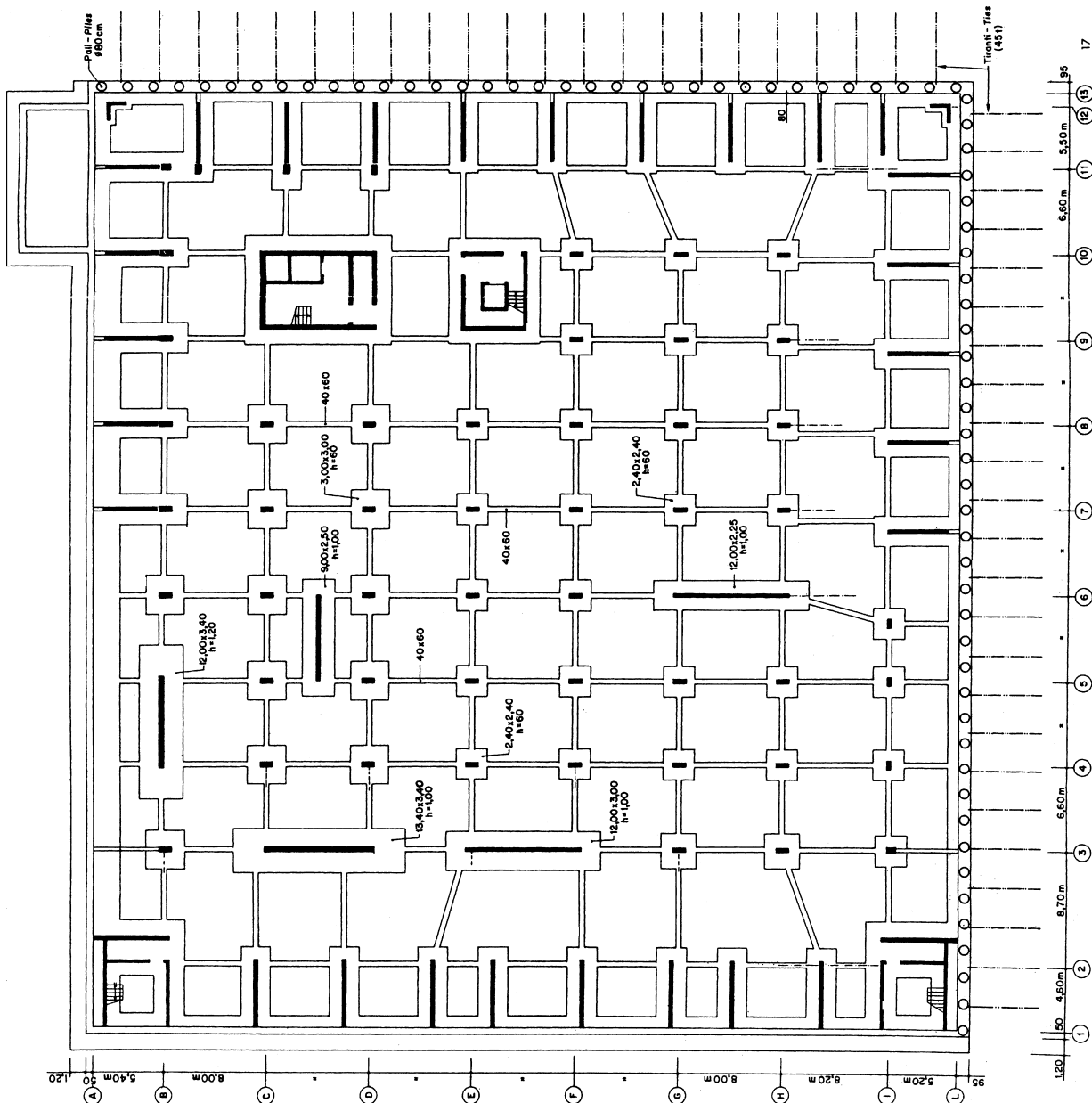
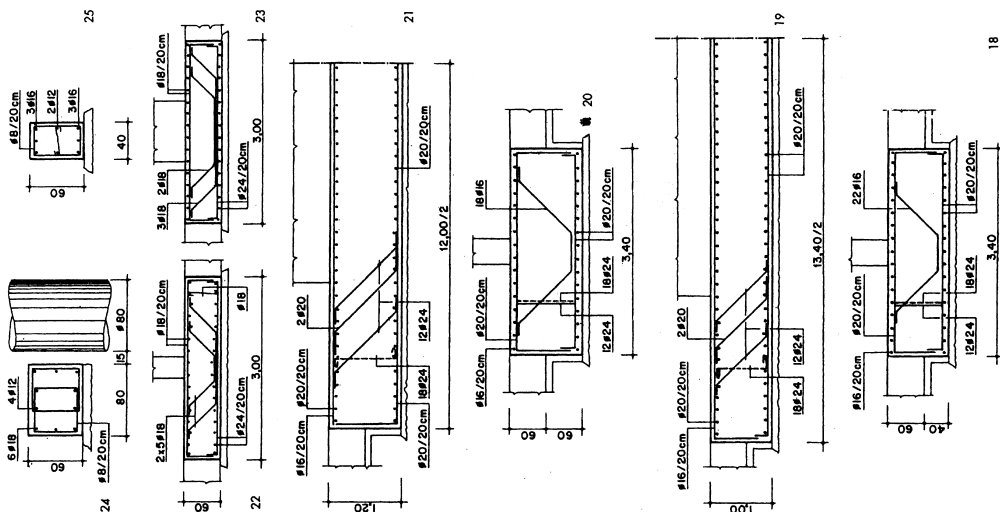
VII.5. Raft foundations

When the spread footings occupy more than half the area covered by the structure and where differential settlement on poor soil is likely to occur, a raft foundation is found to be more economical. This type of foundation is viewed as the inverse of a one-storey beam, slab and column system. The slab rests on soil carrying the load from the beam/column system which itself transmits the loads from the superstructure. Sheet No. VII.21 gives structural detailing based on EC2.

RAFT FOUNDATION (BASED ON EC2)

SHEET NO. VII.21

FOUNDATION (EX G.I.L. - PARKING FACILITY)
17. Foundation plan: 18-19-20-21-22-23. Foundation details: dimensions and reinforcement; 24-25. Cross sections of the foundation beams. Dimensions and reinforcement.



VII.6. Ground and basement floor foundations

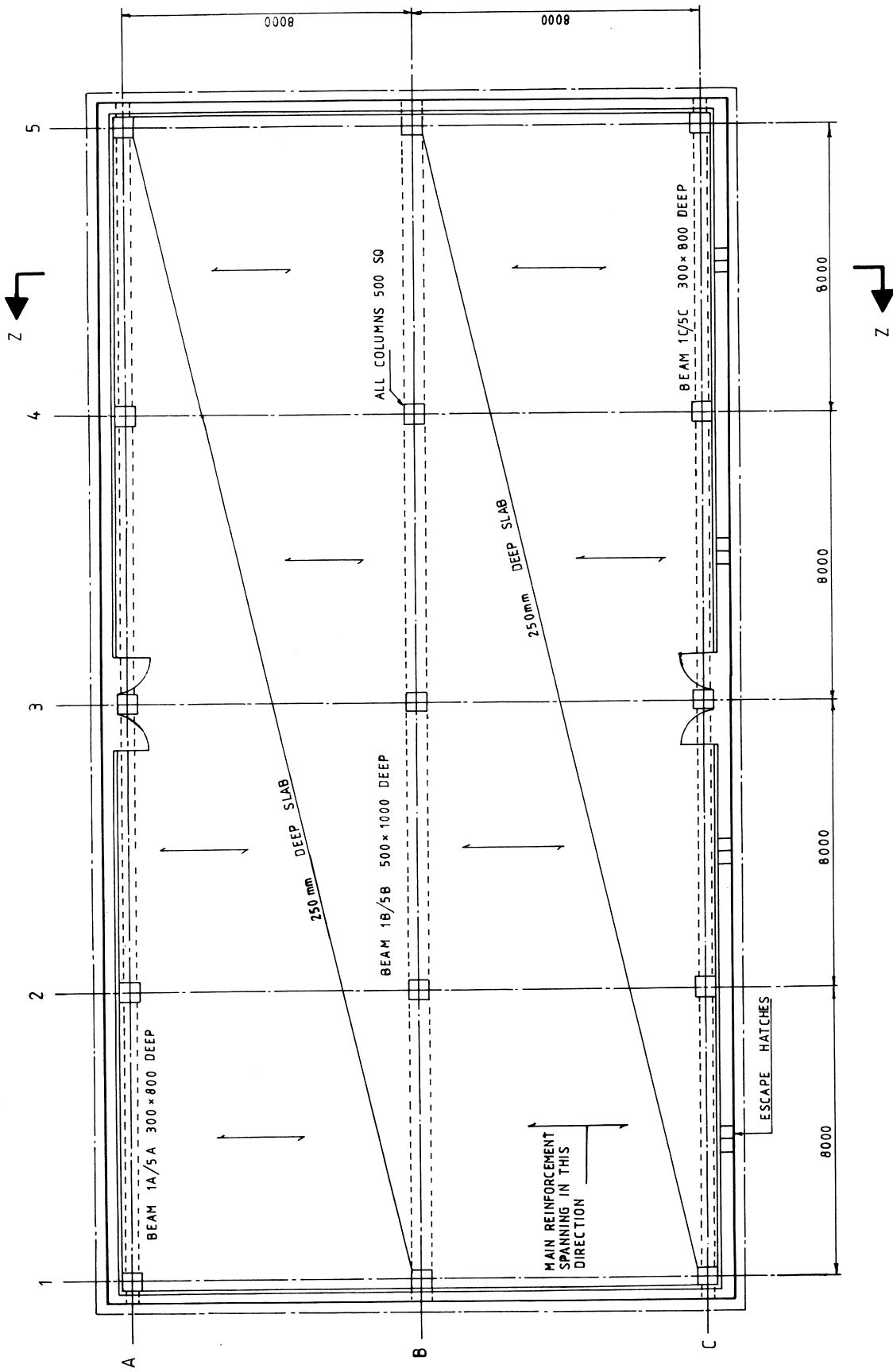
Most building constructions have basements. A scheme is presented through drawings on Sheet Nos VII.22 to VII.24 giving the layout of the plans of the ground and basement floors, a typical sectional elevation of a scheme and reinforcement details of the basement floor slab and the adjacent retaining wall.

In such a scheme, building loads (imposed and dead), soil-bearing pressures, water table, buoyancy and equipment loads are included in the design.

In this scheme, wherever possible, special keys are introduced with pedestals to take the direct column load and to avoid punching failure occurring in adjacent parts of the slab. This may well also be treated as a raft foundation.

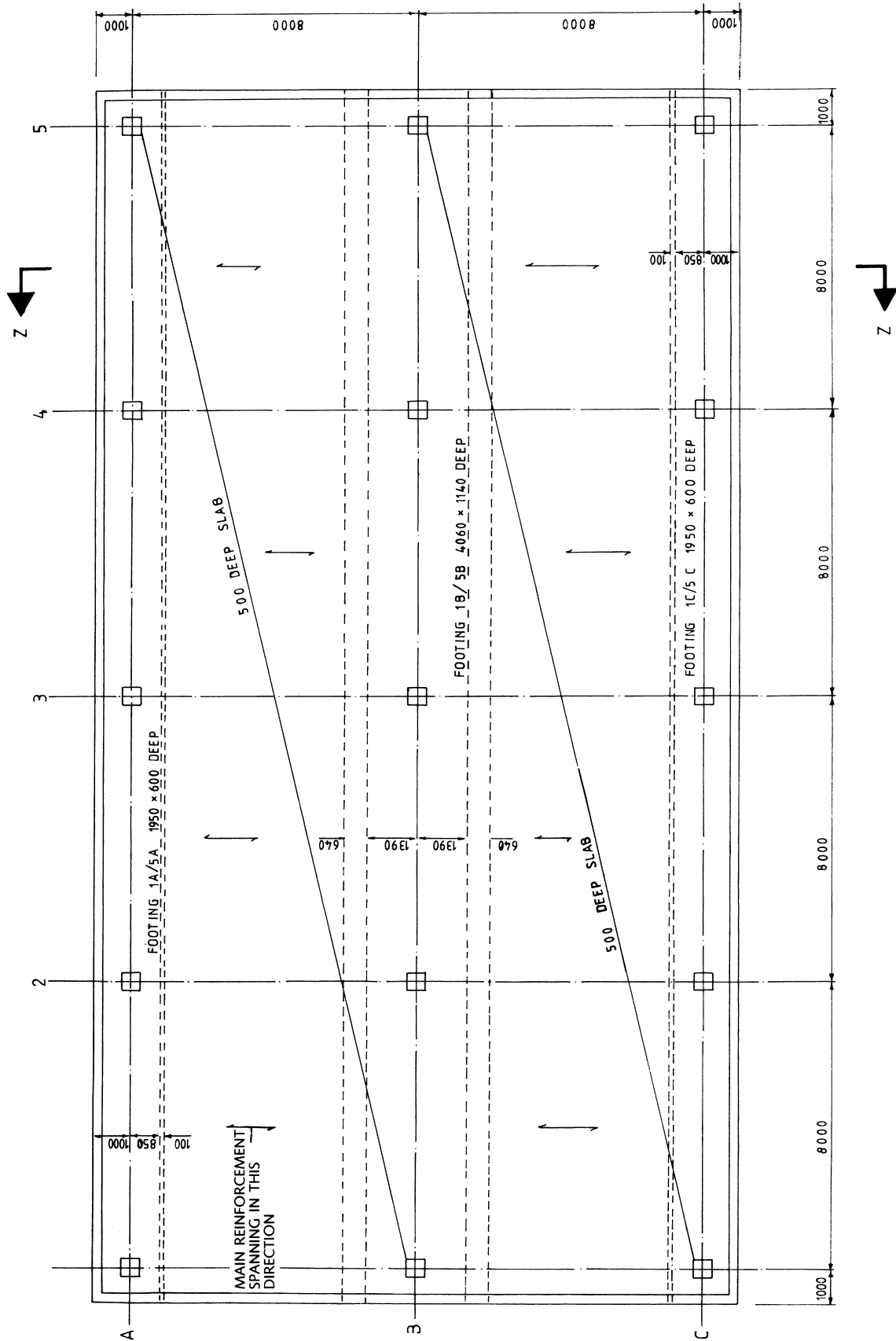
PLAN OF GROUND FLOOR

SHEET NO. VII.22



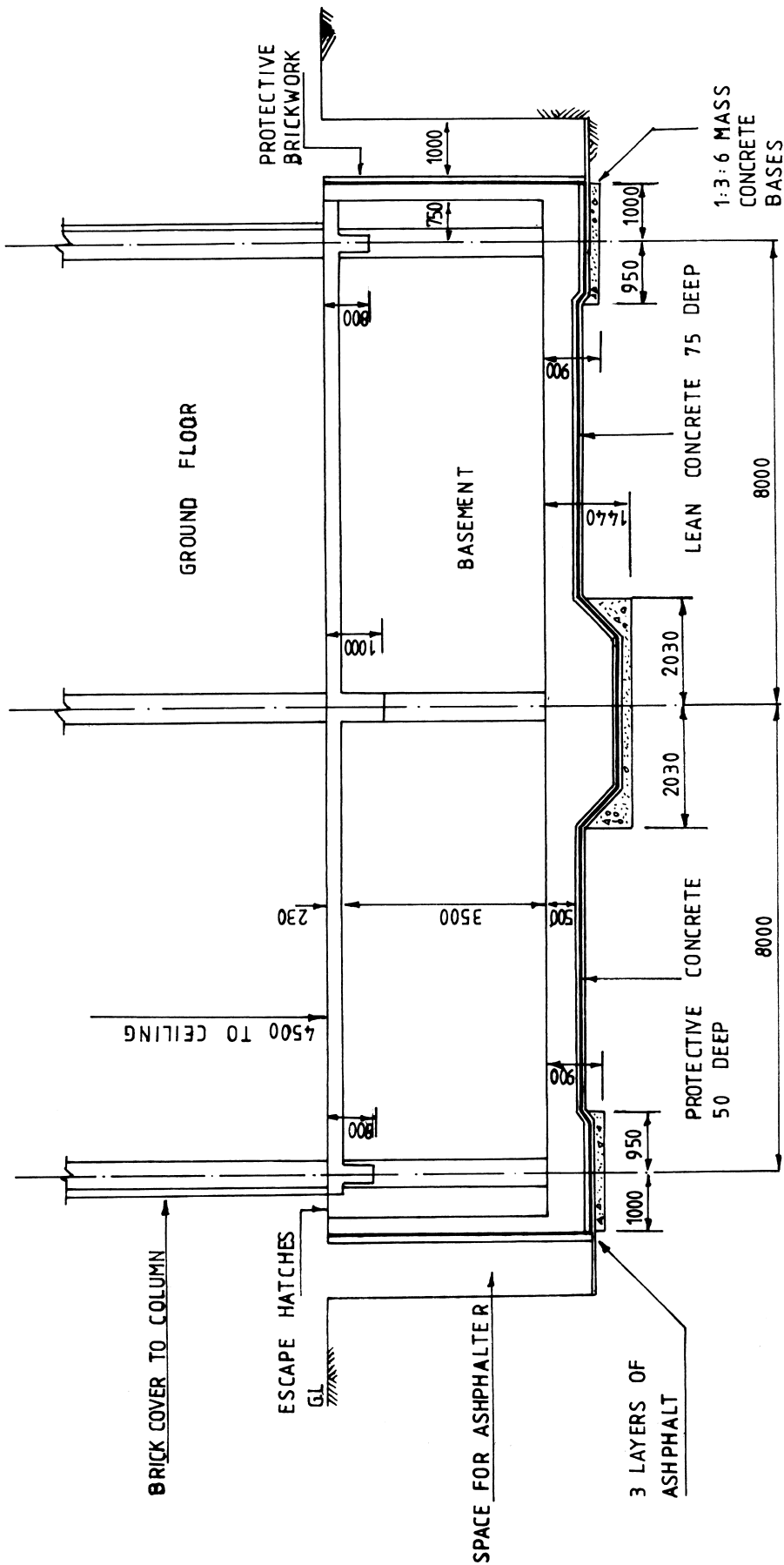
PLAN OF BASEMENT FLOOR

SHEET NO. VII.23



SECTION Z-Z

SHEET NO. VII.24



VII.7. Earth-retaining structures

Walls for retaining earth sustain horizontal pressures exerted by the earth material. Retaining walls without supports may be broadly classified into two types, cantilever and counterfort. The *cantilever retaining wall*, as shown on Sheet No. VII.25, may have its base in front of the wall or at the back. The back-base cantilever retaining wall is generally used to retain stored material. The soil under the front base has to sustain the vertical pressure induced in the base by the horizontal earth pressure on the vertical wall. In addition, the soil has to resist the horizontal sliding force due to the earth pressure on the vertical wall. Several codes exist to give recommendations for the design and detailing of such walls. The reinforcement layout and concrete thickness depend on the applied loads. The details given on Sheet No. VII.26 are modified while keeping the same optimum layout.

When the height exceeds 5 to 6 m, *counterfort retaining walls* are more economical. The counterforts extend beyond the vertical wall and the base at intervals of about 5 to 6 m. The vertical wall is designed as a continuous slab spanning between successive counterforts. Most of the time the counterforts act as tension members between the wall and the base. Sheet No. VII.27 gives the structural detailing of a counterfort retaining wall.

The retaining wall with shelves and sheet pile walls are outside the scope of this book. The former is seldom adopted, the latter is made in corrugated steel or wood.

VII.7.1. Retaining structures based on ENV 1997-1 (1994)

In considering the design of retaining structures it may be appropriate to distinguish between the following three main types of retaining structures.

1. Gravity walls are walls of stone or plain or reinforced concrete, having a base footing with or without heel, ledge or buttress. The gravity of the wall itself, sometimes including stabilizing soil or rock masses, plays a significant role in the support of the retained material. Examples of such walls include concrete gravity walls having constant or variable thickness, spread footing reinforced concrete walls, buttress walls, etc.
2. Embedded walls are relatively thin walls of steel, reinforced concrete or timber, supported by anchors, struts and/or passive earth pressure. The bending capacity of such walls plays a significant role in the support of the retained material while the role of the weight of the wall is insignificant. Examples of such walls include: cantilever steel sheet pile walls, anchored or strutted steel or concrete sheet pile walls, diaphragm walls, etc.
3. Composite retaining structures include walls composed of elements from the above two types of walls. A large variety of such walls exists. Examples include double sheet pile wall cofferdams, earth structures reinforced by tendons, geotextiles or grouting structures with multiple rows of ground anchors or soil nails, etc.

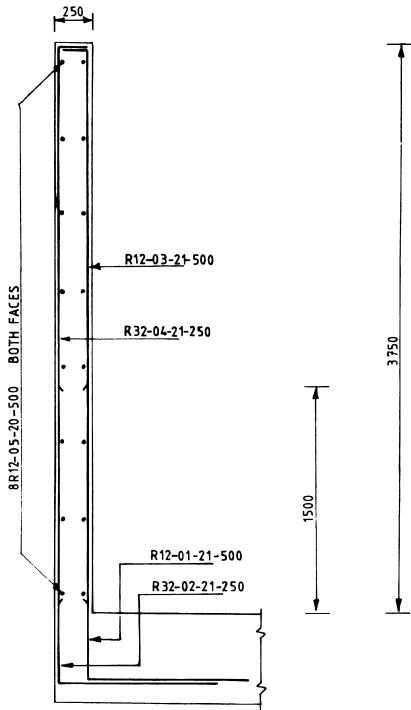
VII.7.2. Limit states

A list of limit states to be considered shall be compiled. As a minimum, the following limit states shall be considered for all types of retaining structures.

1. Loss of overall stability.
2. Failure of a structural element, such as a wall or strut, or failure of the connection between such elements.
3. Combined failure in ground and in structural element.
4. Movements of the retaining structure which may cause collapse or affect the appearance or efficient use of the structure, nearby structures or services which rely on it.

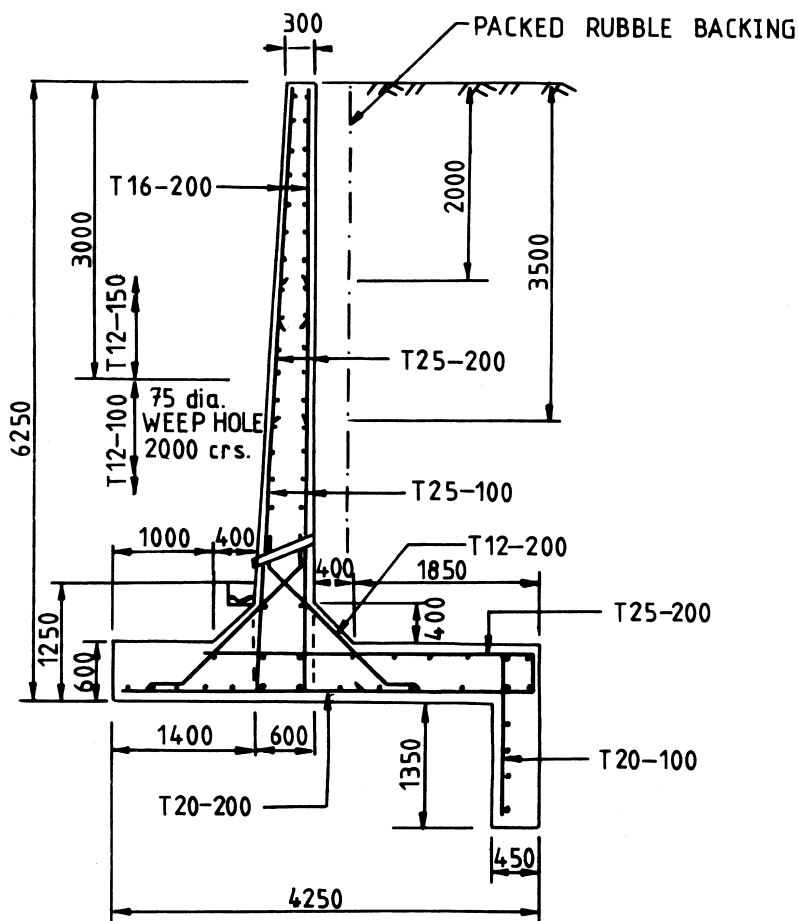
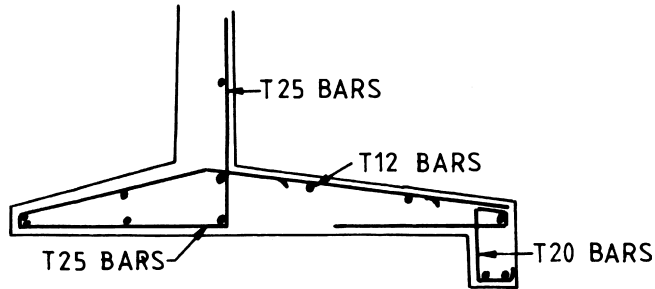
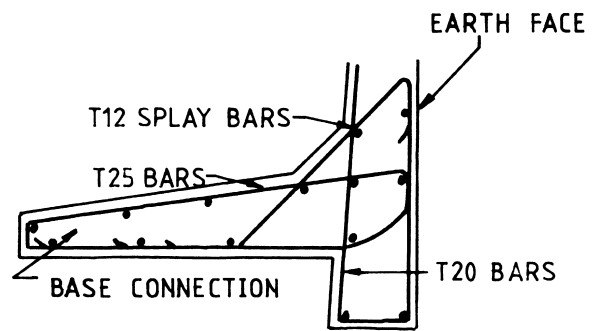
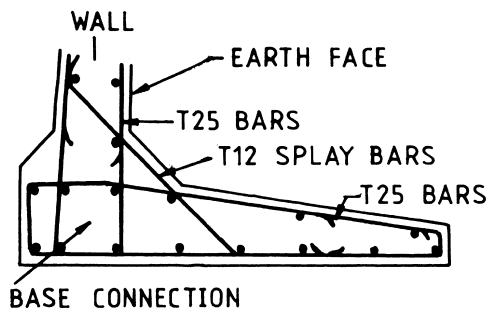
**TYPICAL SECTION THROUGH RETAINING WALLS
(BASED ON BS8110)**

SHEET NO. VII.25



CANTILEVER RETAINING WALLS

SHEET NO. VII.26



5. Unacceptable leakage through or beneath the wall.
6. Unacceptable transport of soil grains through or beneath the wall.
7. Unacceptable change to the flow of groundwater.

In addition the following limit states shall be considered for gravity retaining structures and for composite retaining structures.

1. Bearing resistance failure of the soil below the base.
2. Failure by sliding at the base of the wall.
3. Failure by toppling of the wall.

For embedded retaining structures the following limit states shall be considered.

1. Failure by rotation or translation of the wall or parts thereof.
2. Failure by lack of vertical equilibrium of the wall.

For all types of retaining structures, combinations of the above mentioned limit states shall be considered.

Design of gravity retaining structures often encounters the same type of problems encountered in the design of spread foundations and embankments and slopes. When considering the limit states for gravity retaining structures, the principles of the code should therefore be applied as appropriate. Special care should be taken to account for bearing capacity failure of the ground below the base of the wall under loads with large eccentricities and inclinations.

VII.7.3. Actions, geometrical data and design situations

VII.7.3.1. Actions

In selecting the actions for calculation of limit states, the actions listed shall be considered.

Weight of backfill material

Design values for the unit weight of backfill material shall be estimated on the basis of knowledge of the material available for backfilling. The *Geotechnical Design Report* shall specify the checks which shall be made during the construction process to verify that the actual field values are no worse than those assumed in the design.

VII.7.3.2. Surcharges

Determination of design values for surcharges shall take account of the presence on or near the ground surface of nearby buildings, parked or moving vehicles or cranes, stored granular material, goods, containers, etc.

Care is needed in the case of repeated surcharge loading such as crane rails supported by a quay wall. The pressures induced by such surcharges may significantly exceed those due to the first loading or those resulting from static application of a load of equal magnitude.

VII.7.3.3. Weight of water

Design values for unit weight of water shall reflect whether the water is fresh, saline or charged with chemicals or contaminants to an extent that the normal value needs amendment.

Local conditions such as salinity and the content of mud may significantly influence the unit weight of water.

VII.7.3.4. Wave forces

Design values for wave and wave impact forces shall be selected on the basis of locally available data for the climatic and hydraulic conditions at the site of the structure.

VII.7.3.5. Supporting forces

The components of forces caused by prestressing operations shall be regarded as actions. Design values shall be selected taking into account the effect of overstressing the anchor and the effect of a relaxation of the anchor.

VII.7.3.6. Collision forces

Determination of design values for impact loads shall take account of the energy absorbed by the retaining system on impact.

For lateral impacts on retaining walls, it is normally necessary to consider the increased stiffness exhibited by the retained ground when resisting an impact on the face of the wall. Furthermore, the risk of the occurrence of liquefaction due to lateral impact on embedded walls should be investigated.

The impact load of an ice floe colliding with a retaining structure shall be calculated on the basis of the compressive strength of the ice and the thickness of the ice floe. The salinity and homogeneity of the ice shall be considered in calculating the compressive strength.

VII.7.3.7. Temperature effects

Design of retaining structures shall take into account the effect of abnormal temperature differences over time and space. The effects of changes in temperature should especially be considered when determining the loads in struts and props.

Fire effects are dealt with in the Structural Fire Design parts of the material-related Eurocodes. The design values for ice forces acting on retaining structures from a sheet of ice covering water shall be calculated taking into account:

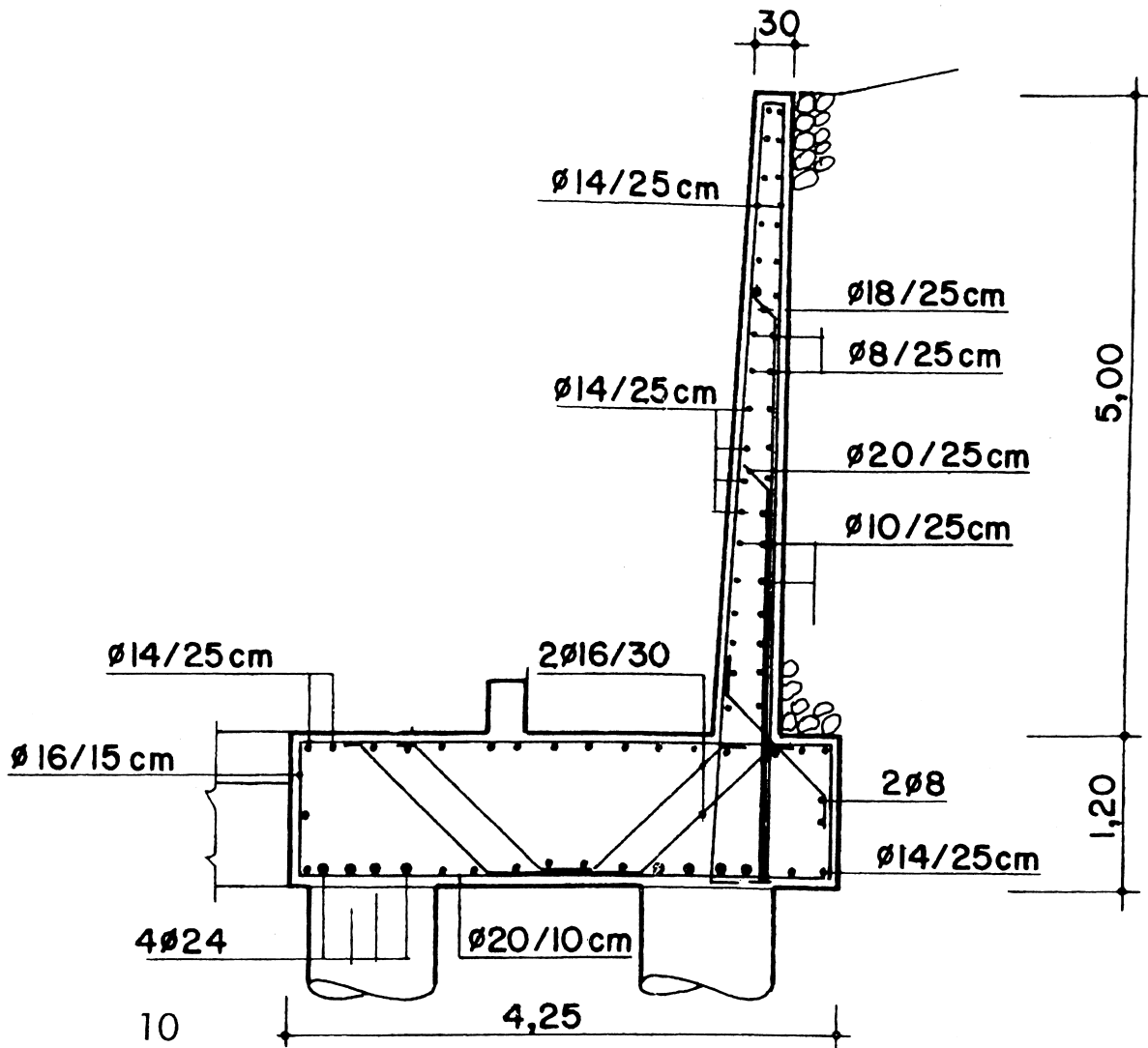
- (a) the initial temperature of the ice before warming begins
- (b) the rate at which the temperature increases
- (c) the thickness of the ice.

Special precautions, such as selection of suitable backfill material, drainage or insulation, shall be taken to prevent ice lenses forming in the ground behind retaining structures.

Sheet Nos VII.28 and VII.29 show typical retaining structures with structural details based on BS 8110. Sheet No. VII.30 shows a retaining wall with the key. Sheet No. VII.28 indicates the reinforcement layout for a cantilever wall with a base pad of 4.25 m resting on piles. Sheet No. VII.29 gives a reinforcement layout acting as a buttress to the basement wall.

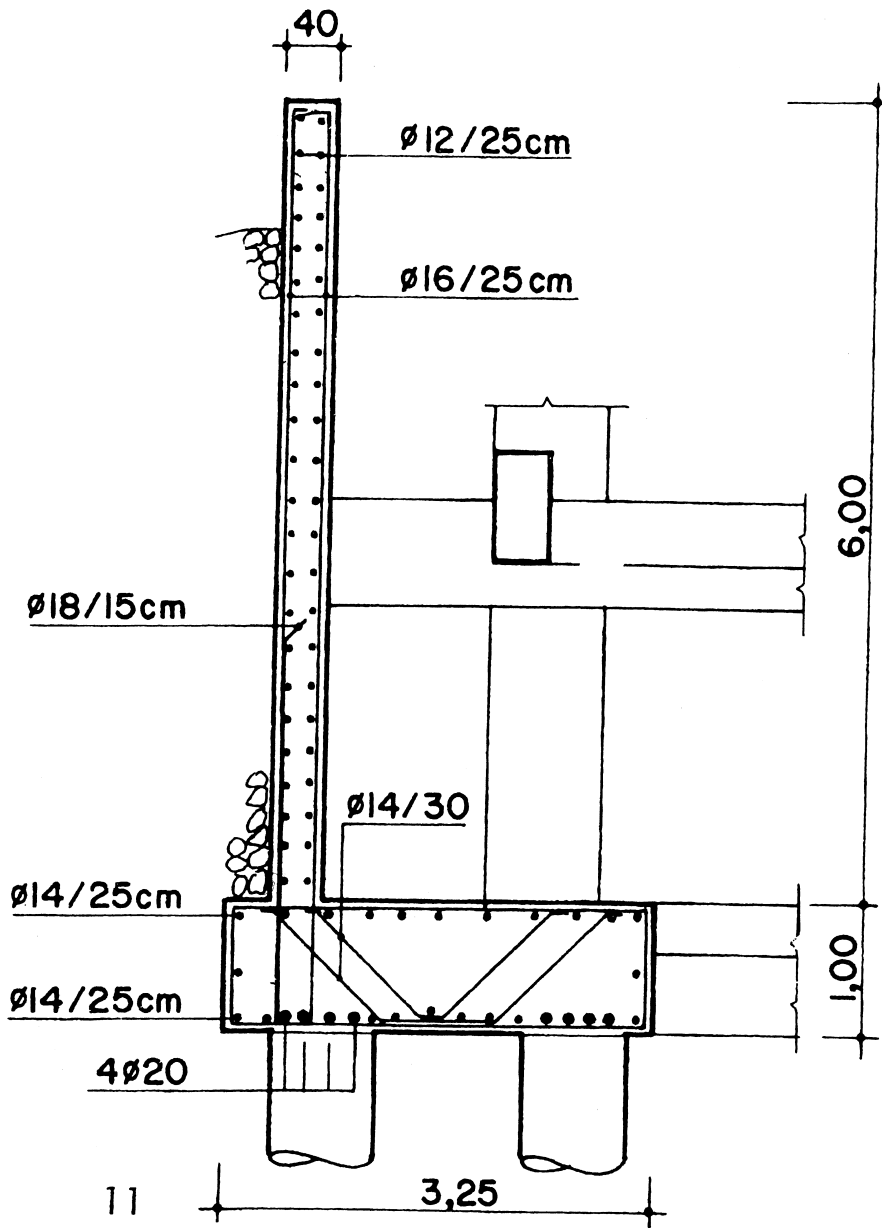
**CANTILEVER WALL ON PILES — REINFORCEMENT LAYOUT
(BASED ON EC2)**

SHEET NO. VII.28

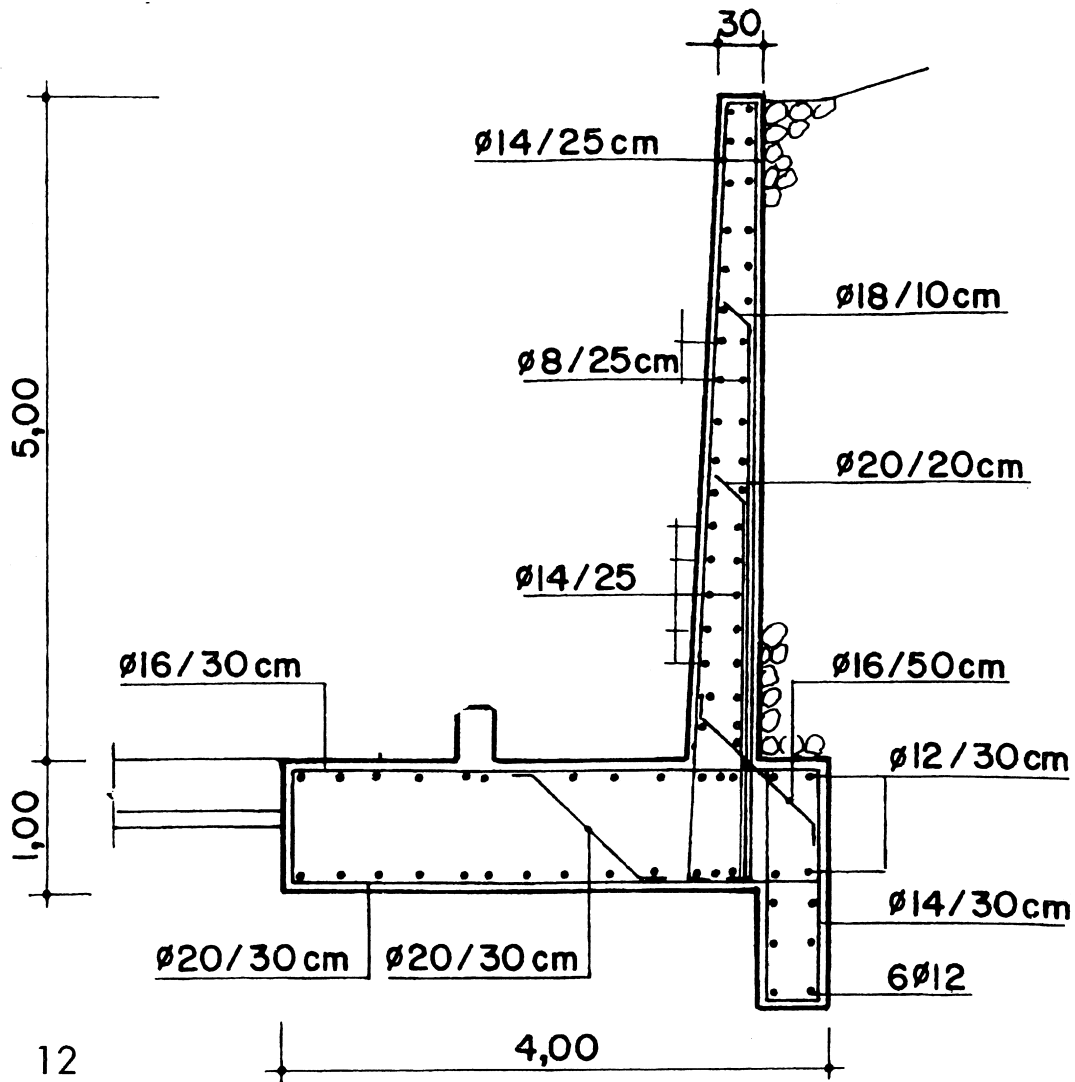


**BUTTRESSED WALL — REINFORCEMENT LAYOUT
(BASED ON EC2)**

SHEET NO. VII.29



**A CANTILEVER WALL WITH A KEY REINFORCEMENT LAYOUT SHEET NO. VII.30
(BASED ON EC2)**



VIII. Special structures: case studies

VIII.1. Bridges

VIII.1.1. General introduction to types of bridges

A bridge is subdivided into (a) the superstructure, (b) the substructure and (c) the foundation. The bridge deck system is the part of the superstructure directly carrying the vehicular loads. It is furnished with balustrades or parapets, crash barriers, highway surfacing, footpaths, traffic islands, railway tracks on ties, expansion joints and drainage systems. The substructure comprises piers, columns or abutments, capping beams and bearings. The foundations consist of reinforced concrete footings, spread foundations, rafts bearing directly on soil or rock and capping slabs supported on piles, wells and caissons. The superstructure of the bridge deck system can be any one or a combination of the following: slabs, coffered slabs, grids, beams, girders, cantilevers, frames, trusses and arches and cable-stayed.

Deck surface members may be classified into the three groups which may be of precast, cast-in-situ and composite construction. They may be of conventional reinforcement, partially or fully prestressed or composite construction. The following classified system lists fully the types of bridges constructed in reinforced, prestressed and composite materials.

(a) *Slabs:*

- (i) solid slabs } Supported directly on piers, with or without
- (ii) void slabs } haunches or drop heads
- (iii) coffered slabs — they act like a grid
- (iv) above with beams of reinforced concrete and prestressed concrete (precast or in-situ beams).

(b) *Beams:**

- (i) longitudinal stringed beams with webs spaced apart and integral with the deck slab
- (ii) longitudinal and transverse beams forming a grid system integral with the deck slab
- (iii) inverted longitudinal beams, trusses and girders, fully or partially integral with the deck
- (iv) a single central longitudinal spine beam, T-beam; truss and girder composite or monolithic with deck.

* Note

T-beams (precast beam slab deck):

- (a) T-beam with in-situ concrete topping
- (b) 'tophat' beams with in-situ concrete topping
- (c) continuous beams.

Span range for:

- (a) precast post-tensioned I-beam 20–35 m
- (b) precast post-tensioned T-beam ranges up to 45 m.

- (c) *Boxes*:
 (i) a single longitudinal box beam or several box beams with and without cantilevered top flanges comprised of a double webbed single unicellular box, and twin or multiple unicellular boxes with or without cross-beams or diaphragms.
- (d) *Frames* (with or without struts)
 They may have members in one or more plane. They may be portal frames (single or multiple), vierendeel girders trestle piers, spill through abutments and towers for cable-stayed or suspension bridges. } Short-span bridges over highways or rivers or flyovers over freeways.
- (e) *Arches*
 They are classified as:
 (i) solid arches
 (ii) open spandrel arches
 (iii) solid spandrel arches
 (iv) tied arches
 (v) funicular arches
 (vi) strut-frame with inclined legs.
- (f) *Suspension and cable-stayed bridges*
 Suspension bridges with draped cables and vertical or triangulated suspender hangers are adopted for spans exceeding 300 m. *Cable-stayed bridges* are economical over the span range of the order of 100 to 700 m with concrete deck, pylons and frames. For cable-stayed, the elevational and transverse arrangements are given below.
 (i) elevational arrangement: single, radiating, harp, fan, star and combination.
 (ii) transverse arrangement:
 single plane
 (vertical—central or eccentric)
 double plane
 (vertical or sloping) } No cables single, double, triple, multiple or combined.

VIII.1.2. Types of loads acting on bridges

They are classified as follows.

1. *Permanent and long-term loads*: dead; superimposed; earth pressure and water pressure of excluded or retained water.
2. *Transient and variable loads (primary type)*: vehicular loading; railway loading; footway loading and cycle loading.
3. *Short-term load*: erection loads; dynamic and impact loads.
4. *Transient forces*: braking and traction forces; forces due to accidental skidding and vehicle collision with parapet or with bridge supports.
5. *Lurching and nosing by trains*.
6. *Transient forces due to natural causes*: wind action; flood action and seismic forces.
7. *Environmental effects*: Loads generated due to creep, shrinkage of concrete; prestress parasitic moments or reactions and prestrain and temperature range or gradient.

Relevant codes are consulted for the application of these loads on bridge structures.

VIII.1.3. Substructures supporting deck structures

The deck structures are supported directly on:

- (a) mass concrete or masonry gravity abutments
- (b) closed-end abutments with solid or void walls such as cantilevers, struts or diaphragms
- (c) counterforted or buttressed walls or combinations
- (d) open-end or spill-through abutments with trestle beams supported on columns.

The intermediate piers can be of the following type:

- (a) solid or void walls with or without capping beams
- (b) single solid or void columns with or without caps
- (c) trestles and bents
- (d) specially shaped columns, e.g. V-shaped or fork shaped, etc.

In most cases bridge bearings are needed to transmit deck loads to substructures and to allow the deck to respond to environmental and vehicle loads.

VIII.1.4. Bridges—case studies

This section contains the relevant specifications of the British, European and American codes on the design/detailing of steel bridges and their important components. In some cases the practices are self-explanatory and need no additional text to clarify them. For thorough explanations of theory, design analysis and structural detailing refer to *Prototype bridge structures: analysis and design* by the author, published by Thomas Telford, London, 1999.

VIII.1.4.1. Bridge loadings and specifications

VIII.1.4.1.1. Highway bridge live loads based on British practice

General

Standard highway loading consists of HA and HB loading. HA loading is a formula loading representing normal traffic in Great Britain. HB loading is an abnormal vehicle unit loading. Both loadings include impact.

Loads to be considered

The structure and its elements shall be designed to resist the more severe effects of either:

- (a) design HA loading (Fig. VIII.1)
- (b) design HA loading combined with design HB loading.

Notional lanes, hard shoulders. etc.

The width and number of notional lanes, and the presence of hard shoulders, hard strips, verges and central reserves are integral to the disposition of HA and HB loading. Requirements for deriving the width and number of notional lanes for design purposes are specified in the highway codes. Requirements for reducing HA loading for certain lane widths and loaded length are specified.

Distribution analysis of structure

The effects of the design standard loadings shall, where appropriate, be distributed in accordance with a rigorous distribution analysis or from data derived from suitable tests. In the latter case, the use of such data shall be subject to the approval of the appropriate authority.

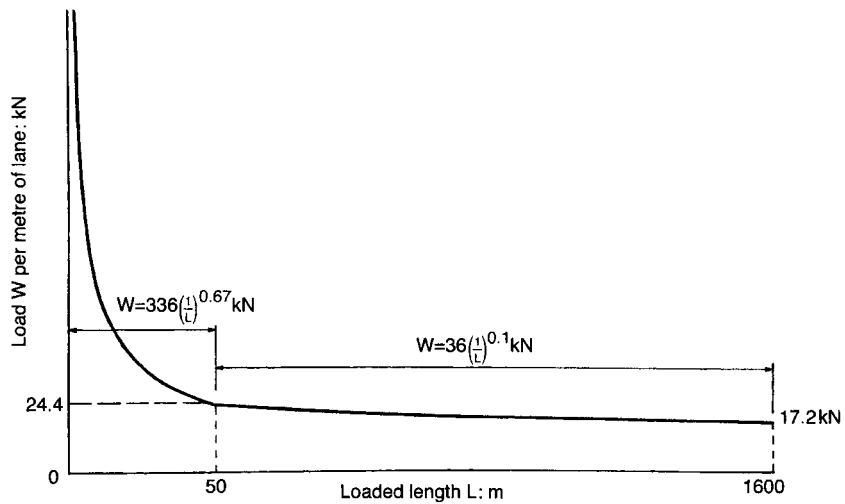


Fig. VIII.1. HA loading

Type HA loading

Type HA loading (Tables VIII.1 and VIII.2) consists of a uniformly distributed load (see Clause 8.1.2 of the code) and a knife edge load combined, or of a single wheel load.

Table VIII.1. Factors for limit state for combination of loads (HA type)

Design HA loading — for design HA load considered alone, VFL shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
For combination 1	1.50	1.20
For combinations 2 and 3	1.25	1.00

Table VIII.2. Type HA uniformly distributed load

Loaded length: m	Load: kN/m	Loaded length: m	Load: kN/m	Loaded length: m	Load: kN/m
2	211.2	55	24.1	370	19.9
4	132.7	60	23.9	410	19.7
6	101.2	65	23.7	450	19.5
8	83.4	70	23.5	490	19.4
10	71.8	75	23.4	530	19.2
12	63.6	80	23.2	570	19.1
14	57.3	85	23.1	620	18.9
16	52.4	90	23.0	670	18.8
18	48.5	100	22.7	730	18.6
20	45.1	110	22.5	790	18.5
23	41.1	120	22.3	850	18.3
26	37.9	130	22.1	910	18.2
29	35.2	150	21.8	980	18.1
32	33.0	170	21.5	1050	18.0
35	31.0	190	21.3	1130	17.8
38	29.4	220	21.0	1210	17.7
41	27.9	250	20.7	1300	17.6
44	26.6	280	20.5	1400	17.4
47	25.5	310	20.3	1500	17.3
50	24.4	340	20.1	1600	17.2

Nominal uniformly distributed load (UDL)

For loaded lengths up to and including 50 m, the UDL, expressed in kN per linear metre of notional lane, shall be derived from the equation:

$$W = 336 \left(\frac{1}{L} \right)^{0.67} \quad (\text{VIII.1})$$

and for loaded lengths in excess of 50 m but less than 1600 m the UDL shall be derived from the equation

$$W = 36 \left(\frac{1}{L} \right)^{0.1} \quad (\text{VIII.2})$$

where L is the loaded length (in m) and W is the load per metre of notional lane (in kN). For loaded lengths above 1600 m, the UDL shall be agreed with the appropriate authority.

Nominal knife edge load (KEL)

The KEL per notional lane shall be taken as 120 kN.

Distribution

The UDL and KEL shall be taken to occupy one notional lane, uniformly distributed over the full width of the lane and applied as specified in Clause 6.4.1 of the code.

Dispersal

No allowance for the dispersal of the UDL and KEL shall be made.

VIII.1.4.1.2. Nominal HB loading**Type HB loading**

For all public highway bridges in Great Britain, the minimum number of units of type HB loading that shall normally be considered is 30, but this number may be increased up to 45 if so directed by the appropriate authority.

The overall length of the HB vehicle shall be taken as 10, 15, 20, 25 or 30 m for inner axle spacings of 6, 11, 16, 21 or 26 m respectively (Fig. VIII.2), and the effects of the most severe of these cases shall be adopted. The overall width shall be taken as 3.5 m. The longitudinal axis of the HB vehicle shall be taken as parallel with the lane markings.

Contact area

Nominal HB wheel loads shall be assumed to be uniformly distributed over a circular contact area, assuming an effective pressure of 1.1 N/mm².

Design HB loading

For design HB load, y_{fl} shall be taken as shown in Table VIII.3.

Table VIII.3. Factors for limit state for combination of loads (HB type)

	For the ultimate limit state	For the serviceability limit state
For combination 1	1.30	1.10
For combinations 2 and 3	1.10	1.00

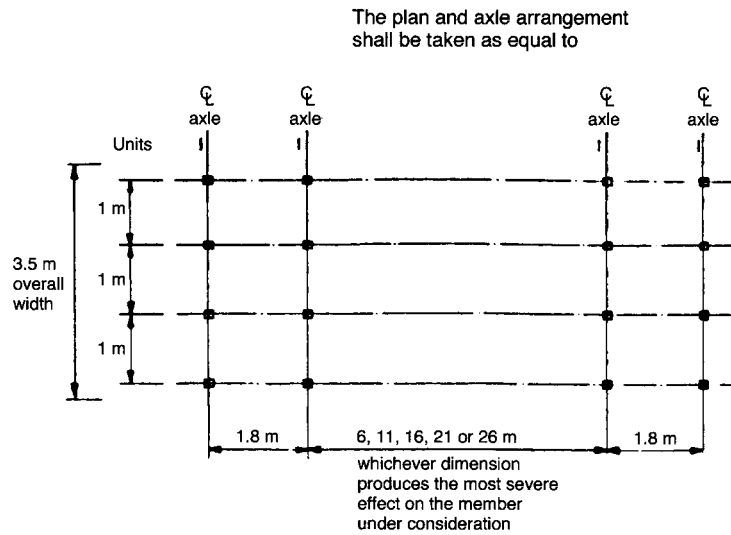


Fig. VIII.2. Dimensions of HB vehicle for 1 unit of nominal loading (1 unit = 10 kN per axle — i.e. 2.5 kN per wheel)

VIII.1.4.1.3. Railway bridge live load

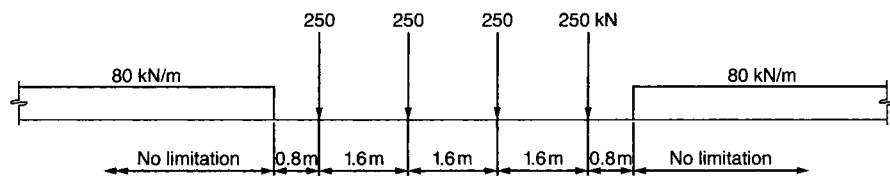
General

Standard railway loading consists of two types, RU and RL. RU loading allows for all combinations of vehicles currently running or projected to run on railways in Europe, including the UK, and is to be adopted for the design of bridges carrying main line railways of 1.4 m gauge and above.

RL loading is a reduced loading for use only on passenger rapid transit railway systems on lines where main line locomotives and rolling stock do not operate.

Type RU loading

Nominal type RU loading consists of four 250 kN concentrated loads preceded, and followed, by a uniformly distributed load of 80 kN/m. The arrangement of this loading is as shown in Fig. VIII.3.



Note: see Dynamic effects, below, for effect of additions to this loading.

Fig. VIII.3. Type RU loading

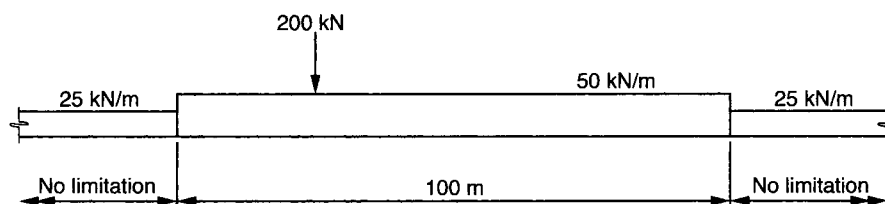
Type RL loading

Nominal type RL loading consists of a single 200 kN concentrated load coupled with a uniformly distributed load of 50 kN/m for loaded lengths up to 100 m. For loaded lengths in excess of 100 m the distributed nominal load shall be 50 kN/m for the first 100 m and shall be reduced to 25 kN/m for lengths in excess of 100 m, as shown in Fig. VIII.4.

Alternatively, two concentrated nominal loads, one of 300 kN and the other of 150 kN, spaced at 2.4 m intervals along the track, shall be used on deck elements where this gives a more severe condition. These two concentrated loads shall be deemed to include dynamic effects.

VIII.1.4.1.4. Dynamic effects

The standard railway loadings specified above (except the 300 kN and 150 kN concentrated alternative RL loading) are equivalent static loadings and shall be



Note: see Dynamic effects, below, for effect of additions to this loading.

Fig. VIII.4. Type RL loading

multiplied by appropriate dynamic factors to allow for impact, oscillation and other dynamic effects including those caused by track and wheel irregularities.

Type RU loading

The dynamic factor for RU loading applied to all types of track and shall be as given in Table VIII.4.

Table VIII.4. Dynamic factor for type RU loading

Dimension L : m	Dynamic factor for evaluating bending moment	Dynamic factor for evaluating shear
Up to 3.6	2.00	1.67
From 3.6 to 6.7	$0.73 + \frac{2.16}{\sqrt{(L - 0.2)}}$	$0.82 + \frac{1.44}{\sqrt{(L - 0.2)}}$
Over 67	1.00	1.00

In deriving the dynamic factor, L is taken as the length (in m) of the influence line for deflection of the element under consideration. For unsymmetrical influence lines, L is twice the distance between the point at which the greatest ordinate occurs and the nearest end point of the influence line. In the case of floor members, 3 m should be added to the length of the influence line as an allowance for load distribution through track.

Type RL loading

The dynamic factor for RL loading, when evaluating moments and shears, shall be taken as 1.20, except for unballasted tracks where, for rail bearers and single-track cross girders, the dynamic factor shall be increased to 1.40.

VIII.1.4.1.5. Road traffic actions and other actions specifically for road bridges—ENV 1991-3: 1995

Models of road traffic loads

Loads due to the road traffic, consisting of cars, lorries and special vehicles (e.g. for industrial transport), give rise to vertical and horizontal, static and dynamic forces. The load models defined in this section do not describe actual loads. They have been selected so that their effects (with dynamic amplification included unless otherwise specified) represent the effects of the actual traffic. Where traffic outside the scope of the load models specified in this section needs to be considered, then complementary load models, with associated combination rules, should be defined or agreed by the client.

Separate models are defined below for vertical, horizontal, accidental and fatigue loads.

Loading classes

The actual loads on road bridges result from various categories of vehicles and from pedestrians. Vehicle traffic may differ between bridges depending on

traffic composition (e.g. percentages of lorries), density (e.g. average number of vehicles per year), conditions (e.g. jam frequency), the extreme likely weights of vehicles and their axle loads, and, if relevant, the influence of road signs restricting carrying capacity.

These differences justify the use of load models suited to the location of a bridge. Some classifications are defined in this section (e.g. classes of special vehicles). Others are only suggested for further consideration (e.g. choice of adjustment factors α and β defined in Clause 4.3.2(7) of the code for the main model and in Clause 4.3.3 for the single axle model) and may be presented as loading classes (or traffic classes).

Divisions of the carriageway into notional lanes

The widths w_1 of notional lanes on a carriageway and the greatest possible whole (integer) number n_1 of such lanes on this carriageway are shown in Table VIII.5.

Table VIII.5. Number and width of lanes

Carriageway width, w	Number of notional lanes	Width of a notional lane	Width of the remaining area
$w < 5.4$ m	$n_1 = 1$	3 m	$w - 3$ m
$5.4 \text{ m} \leq w < 6$ m	$n_1 = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_1 = \text{Int} \left(\frac{w}{3} \right)$	3 m	$w - 3 \times n_1$

Note: for example, for a carriageway width of 11 m, $n_1 = \text{Int} \left(\frac{w}{3} \right) = 3$, and the width of the remaining area is $11 - 3 \times 3 = 2$ m.

VIII.1.4.1.6. Highway loads based on EC3 loadings

Location and numbering of the lanes for design (EC3) (ENV 1995)

The location and numbering of the lanes should be determined in accordance with the following rules:

- (a) the locations of notional lanes are not necessarily related to their numbering
- (b) for each individual verification (e.g. for a verification of the ultimate limit states of resistance of a cross-section to bending), the number of lanes to be taken into account as loaded, their location on the carriageway and their numbering should be so chosen that the effects from the load models are the most adverse.

Vertical loads—characteristic values

General and associated design situations

Characteristic loads are intended for the determination of road traffic effects associated with ultimate limit-state verifications and with particular serviceability verifications (see ENV 1991-1, 9.4.2 and 9.5.2, and ENV 1992 to 1995). The load models for vertical loads represent the following traffic effects.

1. Load model 1: concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model is intended for general and local verifications.

2. Load model 2: a single axle load applied on specific tyre contact areas which covers the dynamic effects of normal traffic on very short structural elements. This model should be separately considered and is only intended for local verifications.
3. Load model 3: a set of assemblies of axle loads representing special vehicles (e.g. for industrial transport) which may travel on routes permitted for abnormal loads. This model is intended to be used only when, and as far as required by the client, for general and local verifications.
4. Load model 4: a crowd loading. This model should be considered only when required by the client. It is intended only for general verifications. However, crowd loading may be usefully specified by the relevant authority for bridges located in or near towns if its effects are not obviously covered by load model 1.

Load models 1 and 2 are defined numerically for persistent situations and are to be considered for any type of design situation (e.g. for transient situations during repair works). Load models 3 and 4 are defined only for some transient design situations. Design situations are specified as far as necessary in design Eurocodes and/or in particular projects, in accordance with definitions and principles given in ENV 1991-1. Combinations for persistent and transient situations may be numerically different.

Main loading system (load model 1)

The main loading system consists of two partial systems as detailed below.

Double-axle concentrated loads (tandem system: TS), each axle having a weight:

$$\alpha_Q Q_k$$

where α_Q are adjustment factors.

No more than one tandem system should be considered per lane; only complete tandem systems shall be considered. Each tandem system should be located in the most adverse position in its lane (see, however, below and Fig. VIII.5). Each axle of the tandem model has two identical wheels, the load per wheel being therefore equal to $0.5\alpha_Q Q_k$. The contact surface of each wheel is to be taken as square and of size 0.40 m.

Uniformly distributed loads (UDL system), having a weight density per square metre:

$$\alpha_q q_k \tag{VIII.3}$$

where α_q are adjustment factors.

These loads should be applied only in the unfavourable parts of the influence surface, longitudinally and transversally.

Load model 1 should be applied on each notional lane and on the remaining areas. On notional lane number 1, the load magnitudes are referred to as $\alpha_{Q1} Q_{ik}$ and $\alpha_{q1} q_{ik}$ (Table VIII.6). On the remaining areas, the load magnitude is referred to as $\alpha_{qr} q_{rk}$.

Unless otherwise specified, the dynamic amplification is included in the values for Q_{ik} and q_{ik} , the values of which are given in Table VIII.6.

For the assessment of general effects, the tandem systems may be assumed to travel along the axes of the notional lanes.

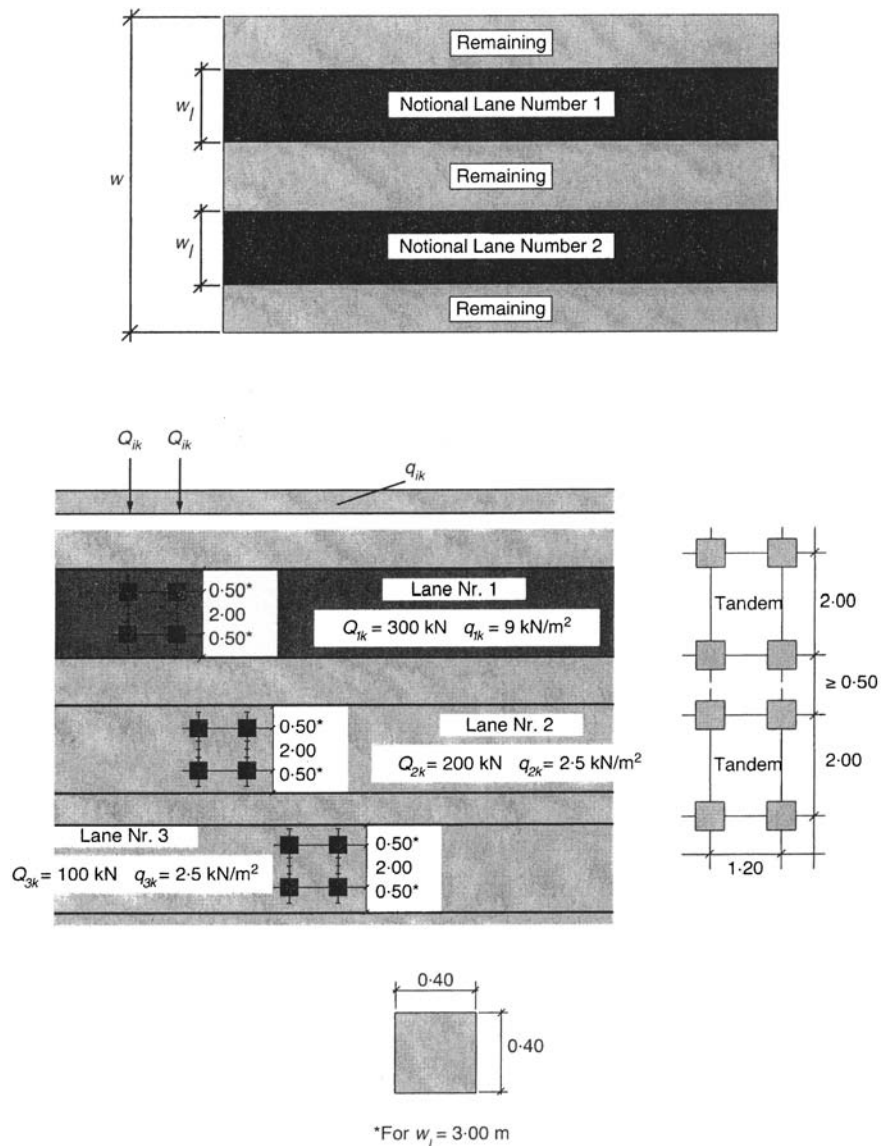


Fig. VIII.5. Example of lane numbering in the most general case and load model 1

Table VIII.6. Basic values

Location	Tandem system axle loads, Q_{ik} : kN	UDL system q_{ik} (or q_{rk}): kN/m ²
Lane number 1	300	9
Lane number 2	200	2.5
Lane number 3	100	2.5
Other lanes	0	2.5
Remaining area (q_{rk})	0	2.5

Where general and local effects can be calculated separately, and unless otherwise specified by the client, the general effects may be calculated:

- (a) by replacing the second and third tandem systems by a second tandem system with axle weight equal to:

$(200\alpha_{Q2} + 100\alpha_{Q3})$ kN (although relevant authorities may restrict the application of this simplification), or

- (b) for span lengths greater than 10 m, by replacing each tandem system in each lane by a one-axle concentrated load of weight equal to the total weight of the two axles. However, the relevant authorities may restrict the application of this simplification. The single axle weight is:

- 600 α_{Q1} kN on lane number 1
- 400 α_{Q2} kN on lane number 2
- 200 α_{Q3} kN on lane number 3.

The values of the factors α_{Qi} , α_{qi} and α_{qr} (adjustment factors) may be different for different classes of route or of expected traffic. In the absence of specification, these factors are taken as equal to 1. In all classes, for bridges without road signs restricting vehicle weights:

$$\alpha_{Q1} \geq 0.8 \text{ and}$$

for: $i \geq 2$, $\alpha_{qi} \geq 1$; this restriction is not applicable to α_{qr} . Note that α_{Qi} , α_{qi} and α_{qr} factors other than 1 should be used only if they are chosen or agreed by the relevant authority.

Single axle model (load model 2)

This model consists of a single axle load $\beta_Q Q_{ak}$ with Q_{ak} equal to 400 kN, dynamic amplification included, which should be applied at any location on the carriageway. However, when relevant, only one wheel of 200 β_Q (kN) may be considered. Unless otherwise specified, β_Q is equal to α_{Q1} .

Unless it is specified that the same contact surface as for load model 1 should be adopted, the contact surface of each wheel is a rectangle of sides 0.35 m and 0.60 m as shown in Fig. VIII.6.

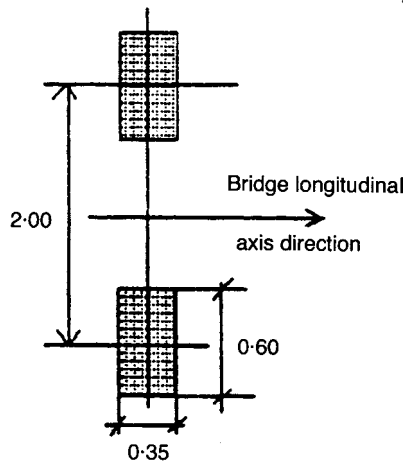


Fig. VIII.6. Load model 2

Set of models of special vehicles (load model 3)

When one or more of the standardized models of this set is required by the client to be taken into account, the load values and dimensions should be as described in annex A of the code concerned.

The characteristic loads associated with the special vehicles should be taken as nominal values and should be considered as associated solely with transient design situations.

Unless otherwise specified the following should be assumed.

1. Each standardized model is applicable on one notional traffic lane (considered as lane number 1) for the models composed of 150 or 200 kN axles, or on two adjacent notional lanes (considered as lane numbers 1 and 2—see Fig. VIII.8) for models composed of heavier axles. The lanes are located as unfavourably as possible in the

carriageway. For this case, the carriageway width may be defined as excluding hard shoulders, hard strips and marker strips.

2. Special vehicles simulated by the models are assumed to move at low speed (not more than 5 km/h); only vertical loads without dynamic amplification have therefore to be considered.
3. Each notional lane and the remaining area of the bridge deck are loaded by the main loading system. On the lane(s) occupied by the standardized vehicle, this system should not be applied at less than 25 m from the outer axles of the vehicle under consideration.

Crowd loading (load model 4)

Crowd loading, if relevant, is represented by a nominal load (which includes dynamic amplification). Unless otherwise specified, it should be applied on the relevant parts of the length and width of the road bridge deck, the central reservation being included where relevant. This loading system, intended for general verifications, is associated solely with a transient situation.

Dispersion of concentrated loads

The various concentrated loads to be considered for local verifications, associated with load models 1, 2 and 3, are assumed to be uniformly distributed across their whole contact area. The dispersion through the pavement and concrete slabs is taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the level of the centroid of the structural flange below (see Figs VIII.7 and VIII.8).

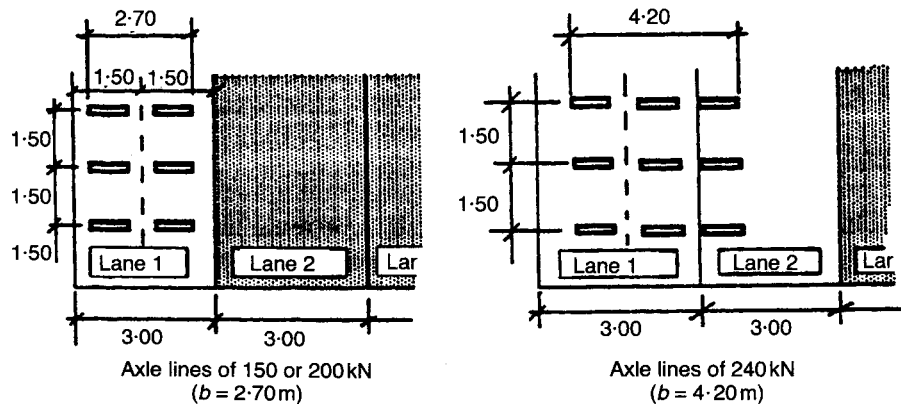


Fig. VIII.7. Location of special vehicles

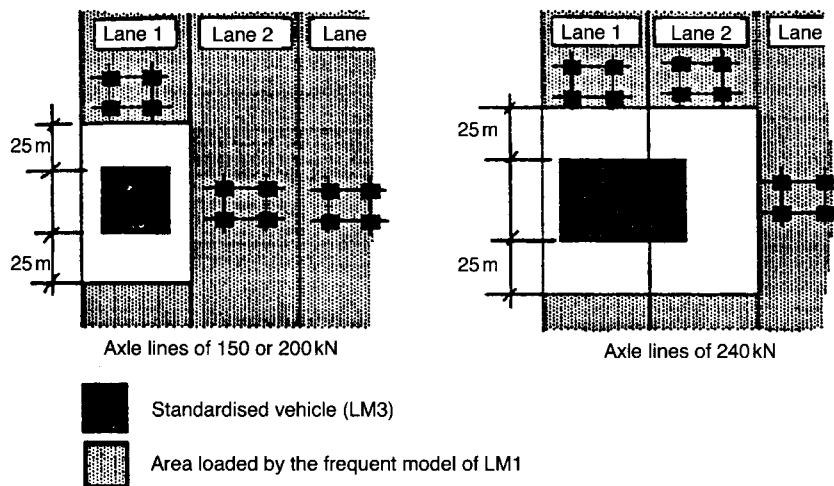


Fig. VIII.8. Simultaneity of load models 1 and 3

VIII.1.4.2. Structural details

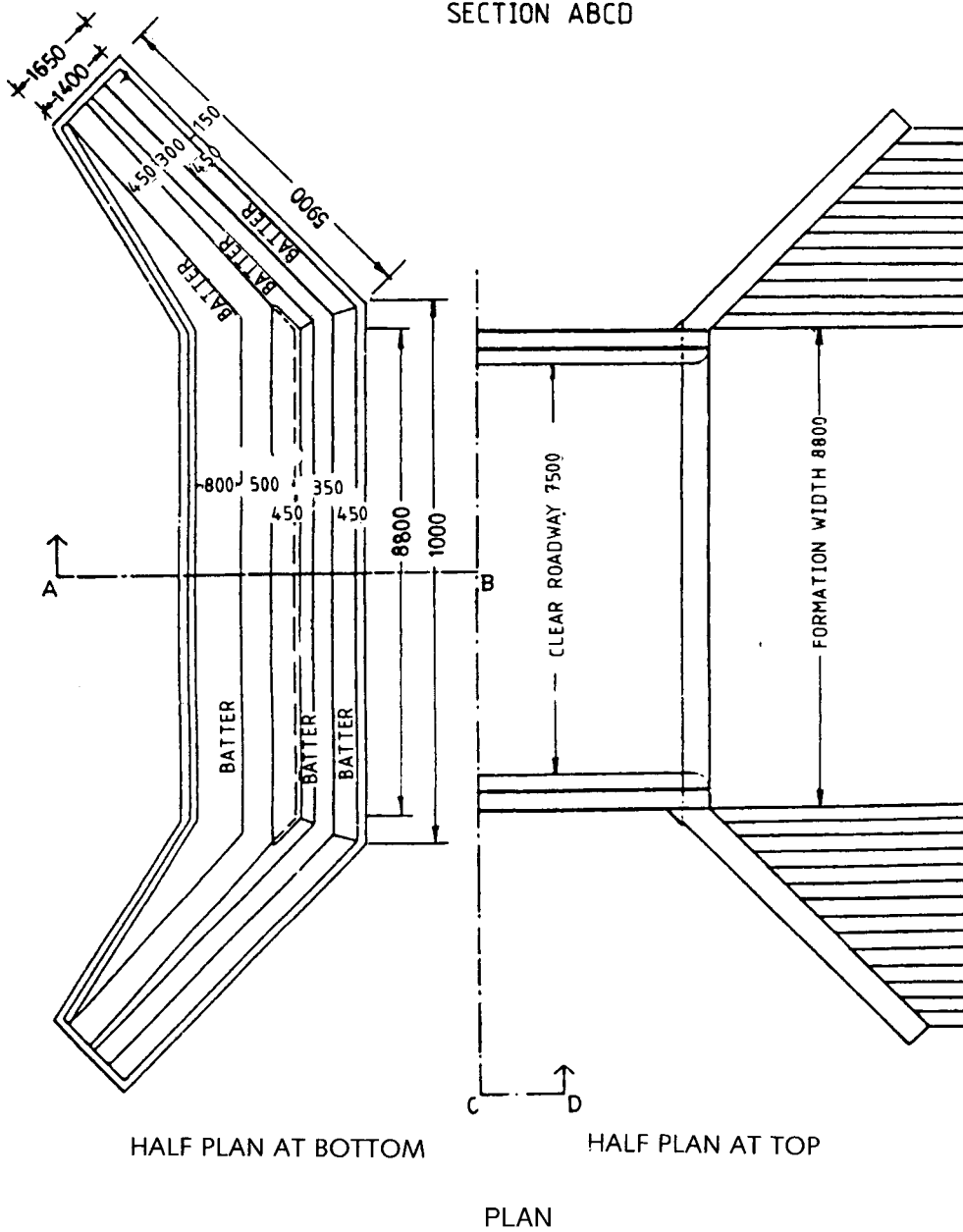
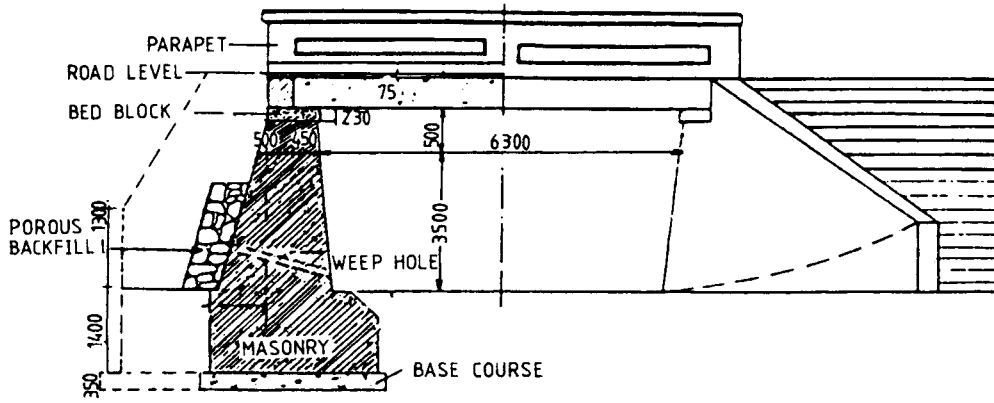
Bridge engineering is a vast field and based on site and other requirements no two bridges can be totally the same. Here the reader is given examples of some reinforced, composite and prestressed concrete bridges with simplified structural details.

Sheet No.

- VIII.1.1 shows a reinforced slab culvert layout with sectional elevation and plan
- VIII.1.1.a shows a reinforced concrete beam/slab bridge on skew
- VIII.1.2 shows a reinforced concrete T-beam bridge superstructure in cross-section, the longitudinal section of a central beam and a plan for a reinforced concrete deck slab integral with main beams
- VIII.1.3 show deck and girder details of a continuous reinforced concrete and girder bridge
- VIII.1.4
- VIII.1.5 shows a reinforced concrete twin-box bridge deck with parapets
- VIII.1.6 shows a composite steel beam concrete deck bridge with a typical longitudinal elevation and cross-section with shear connectors
- VIII.1.7 shows structural details of a reinforced concrete rigid frame with footings. The road surface rests on this frame
- VIII.1.8 shows a reinforced concrete bow-string bridge showing arches and suspenders with their reinforcement details. Cross-beams and cross-sections of the bow-string at various zones are fully detailed.
- VIII.1.9 shows typical bridge decks with post-tensioned girders and pretensioned beams. They are shown in relation to their respective reinforced concrete decks
- VIII.1.10 shows additional bridge decks with post-tensioned girders and also gives an articulated prestressed concrete balanced cantilever bridge. For the arrangement of prestressing and the tendon profile see Section V
- VIII.1.11 show elevations and cross-sections of an open spandrel arch to bridge scheme and relevant structural details of parts where prestressing and conventional steel are recommended
- VIII.1.15
- VIII.1.16 shows a choice of bridge substructure comprising piers and bed blocks
- VIII.1.17 show reinforcement details of a typical pier bent and well cap and for a pier consisting of several wells
- VIII.1.18

**REINFORCED CONCRETE SLAB CULVERT LAYOUT
(BRITISH PRACTICE)**

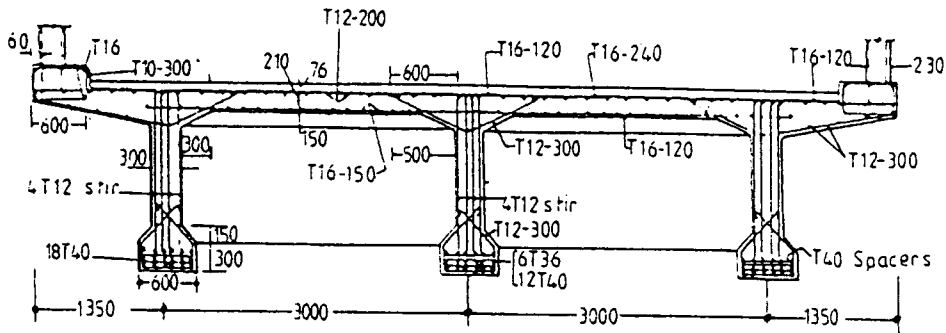
SHEET NO. VIII.1.1



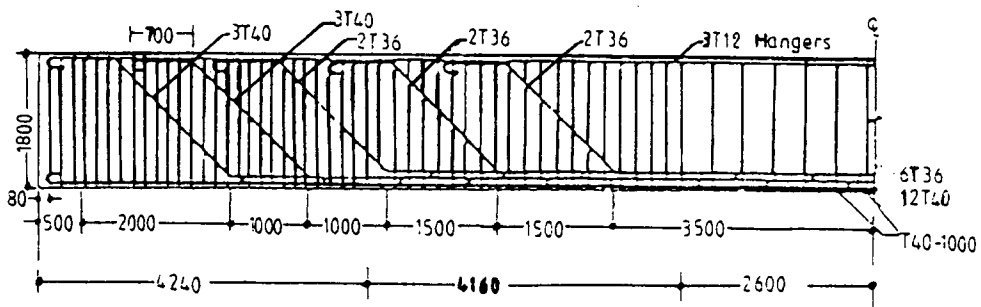
**T-BEAM BRIDGE SUPERSTRUCTURE
(BRITISH PRACTICE)**

SHEET NO. VIII.1.2

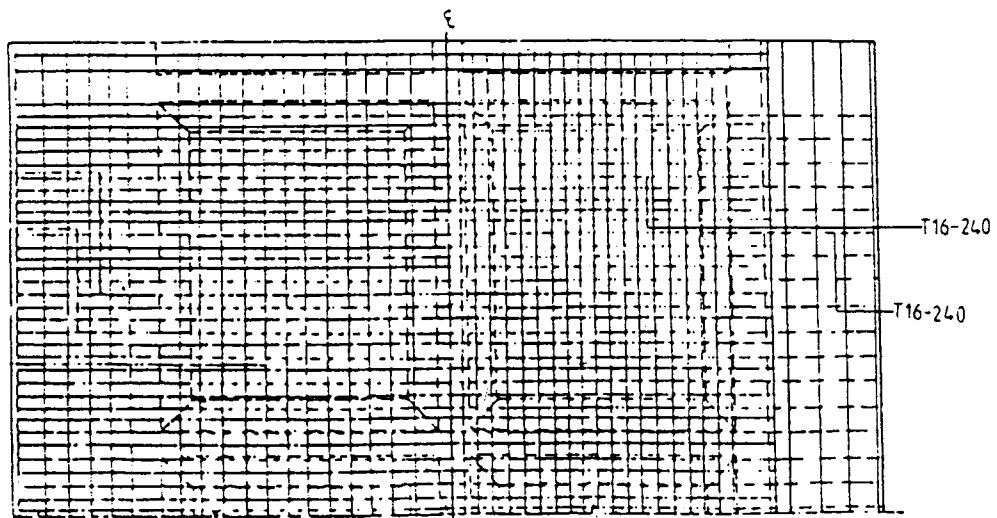
(a) Cross-section



(b) Longitudinal section of central beam



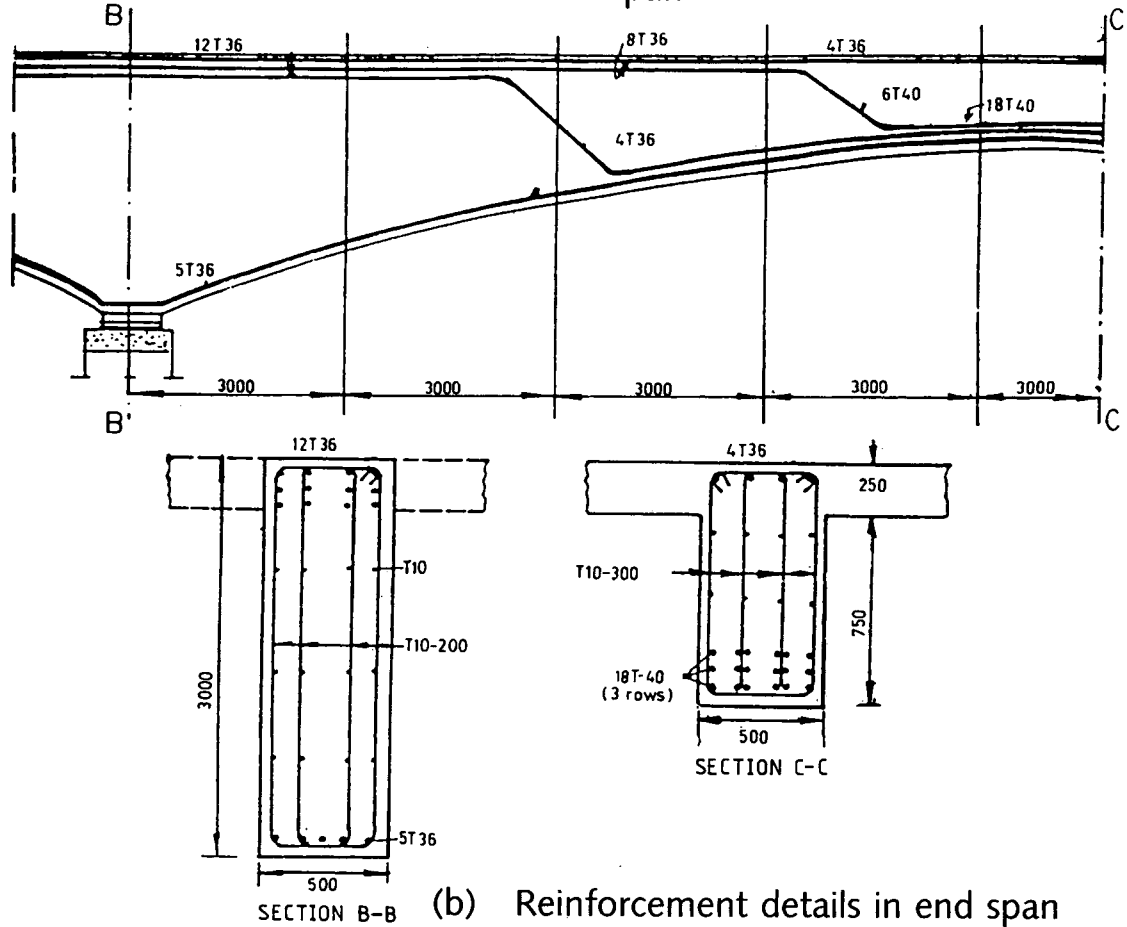
(c) Detail of deck slab



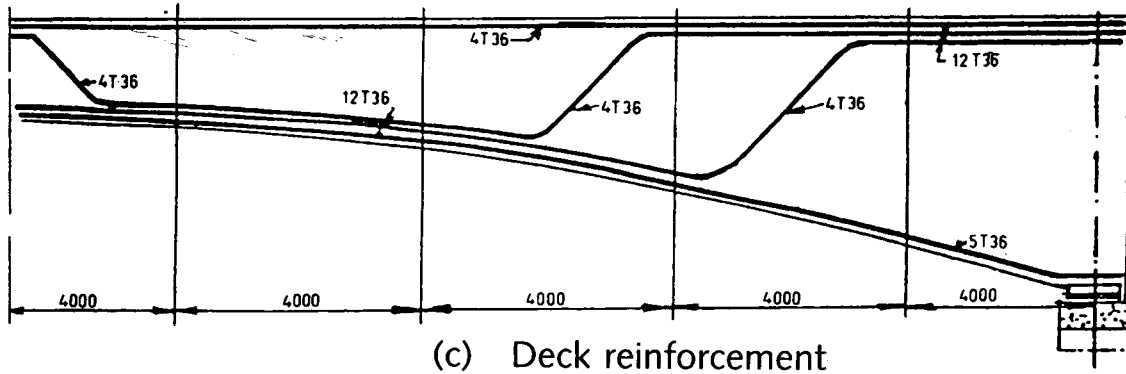
**CONTINUOUS RC GIRDER BRIDGE
(BRITISH PRACTICE)**

SHEET NO. VIII.1.3

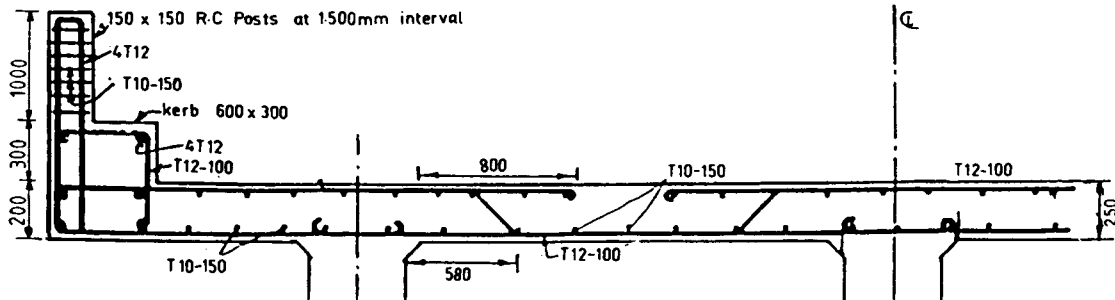
(a) Reinforcement details in central span



(b) Reinforcement details in end span

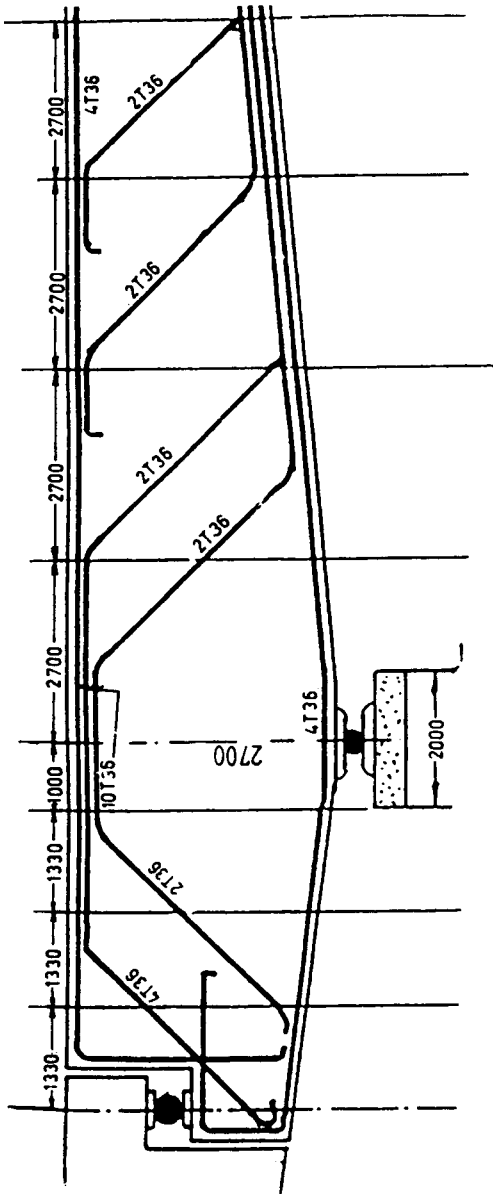


(c) Deck reinforcement

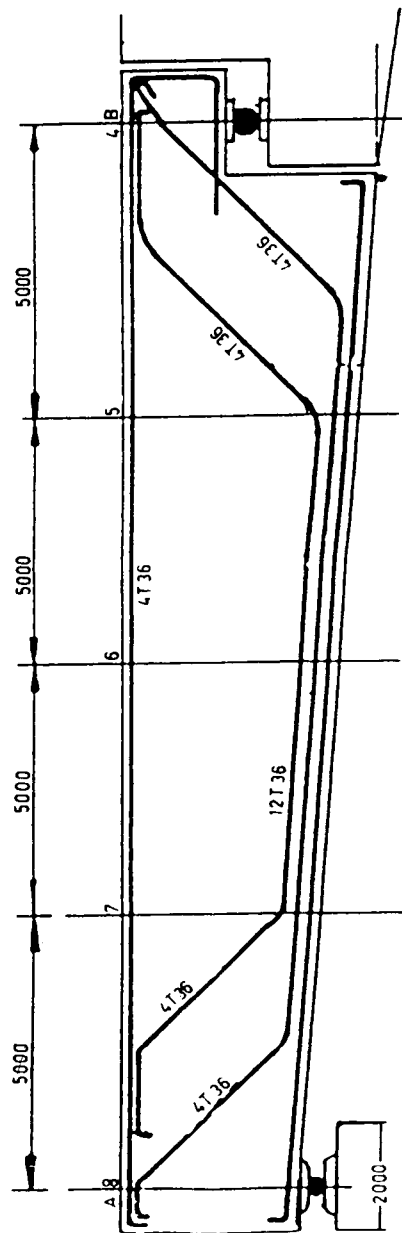


**CONTINUOUS RC GIRDER BRIDGE (CONTD)
(BRITISH PRACTICE)**

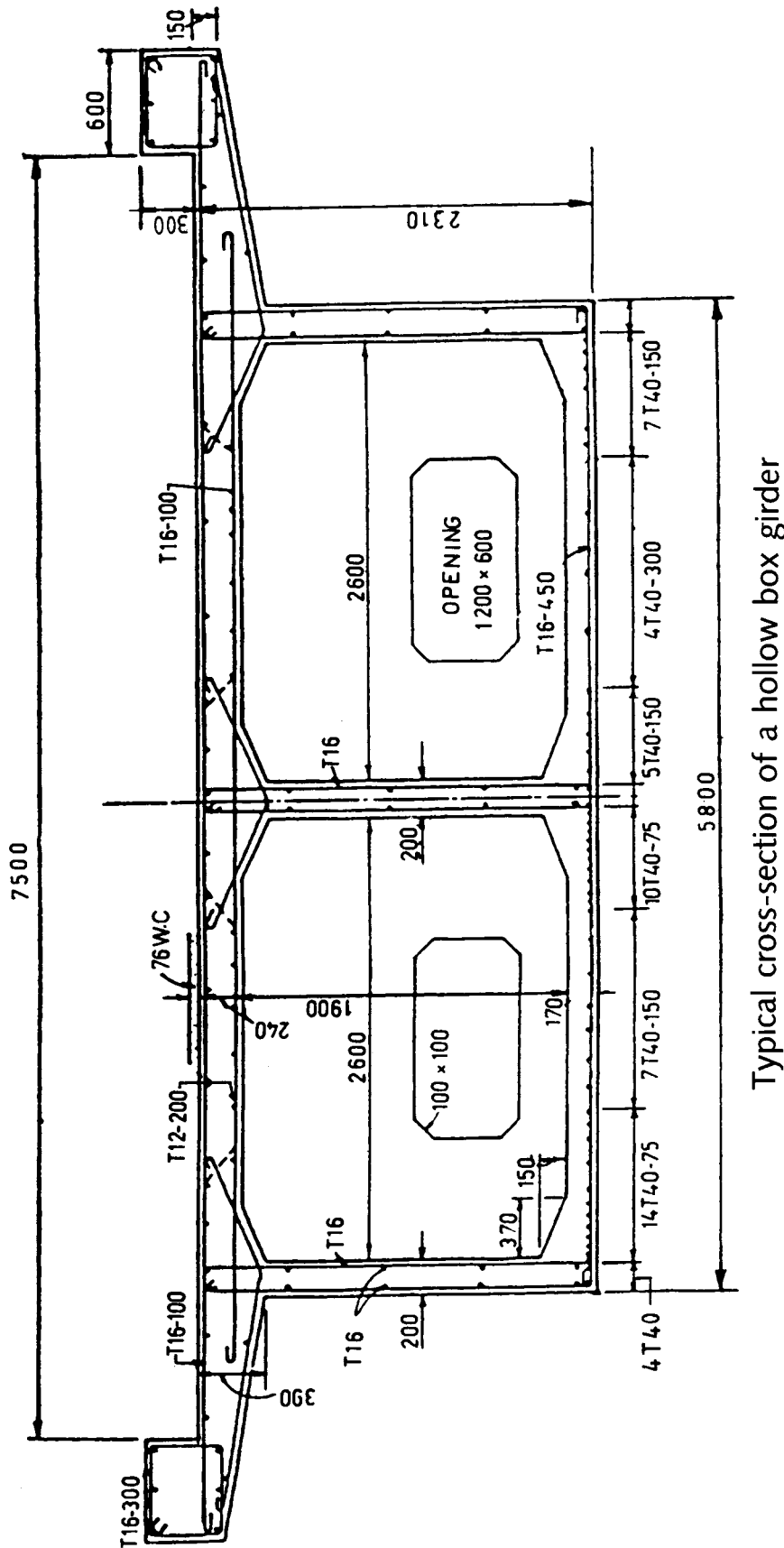
SHEET NO. VIII.1.4



(d) Reinforcement details on both sides of the suspended part



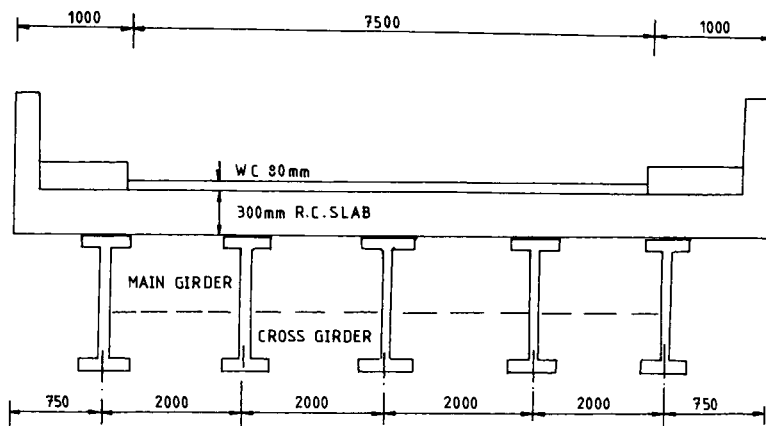
TWIN-BOX BRIDGE (BRITISH PRACTICE)



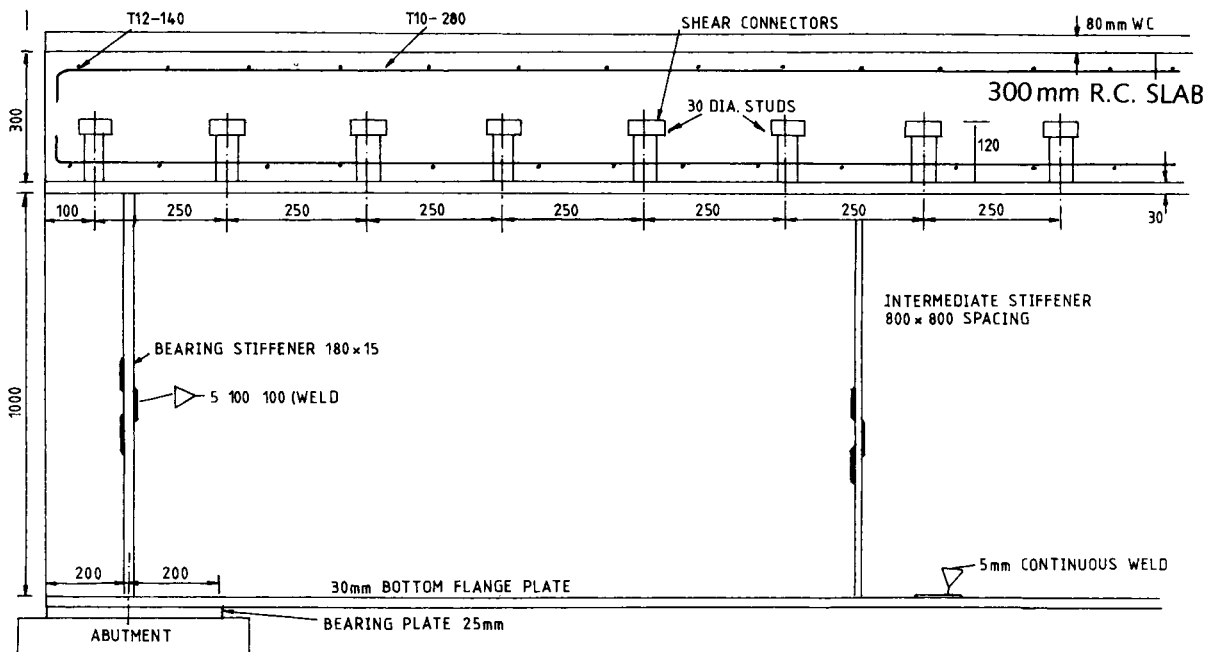
Typical cross-section of a hollow box girder

**COMPOSITE STEEL BEAM CONCRETE DECK BRIDGE
(BRITISH PRACTICE)**

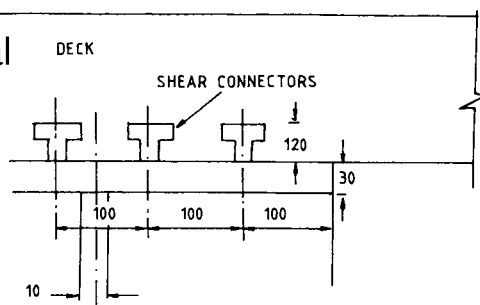
SHEET NO. VIII.1.6



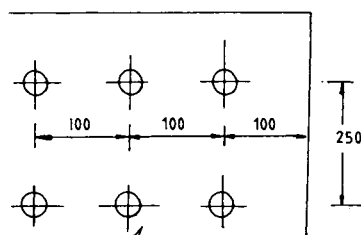
(a) Cross-section of deck slab



(b) Longitudinal elevation



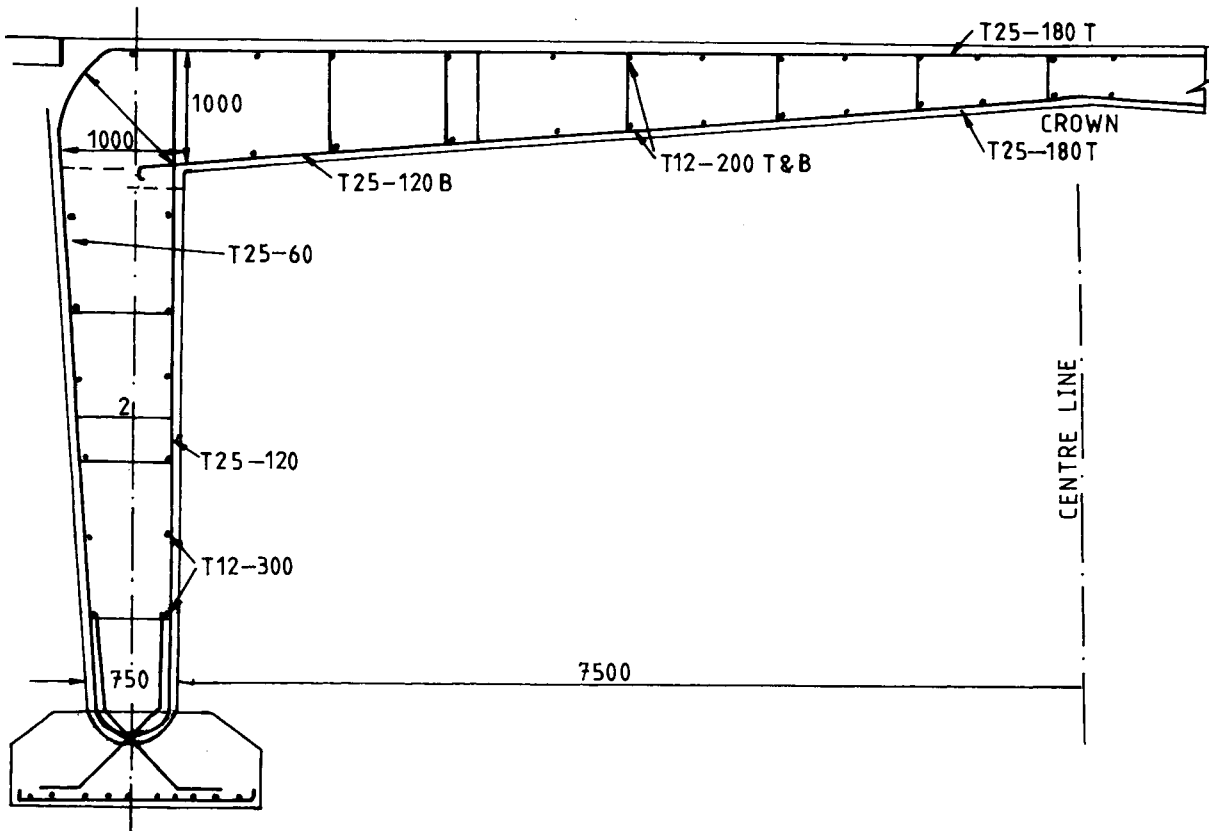
(c) Detail of shear connector



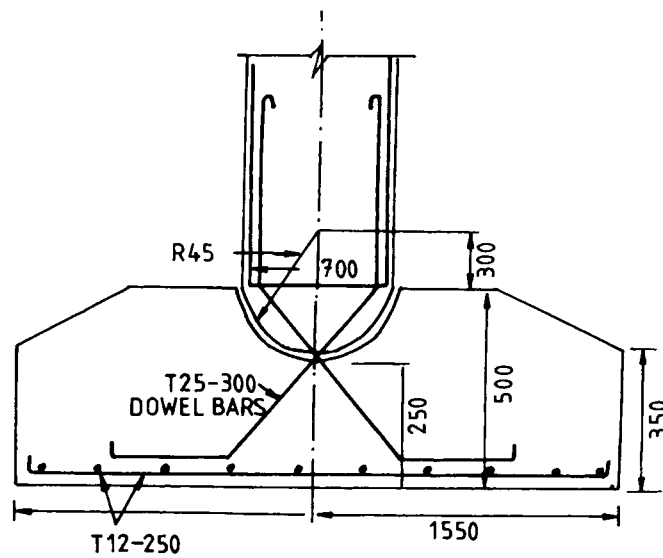
**RC RIGID FRAME BRIDGE
(BRITISH PRACTICE)**

SHEET NO. VIII.1.7

(a) Longitudinal section of a rigid frame

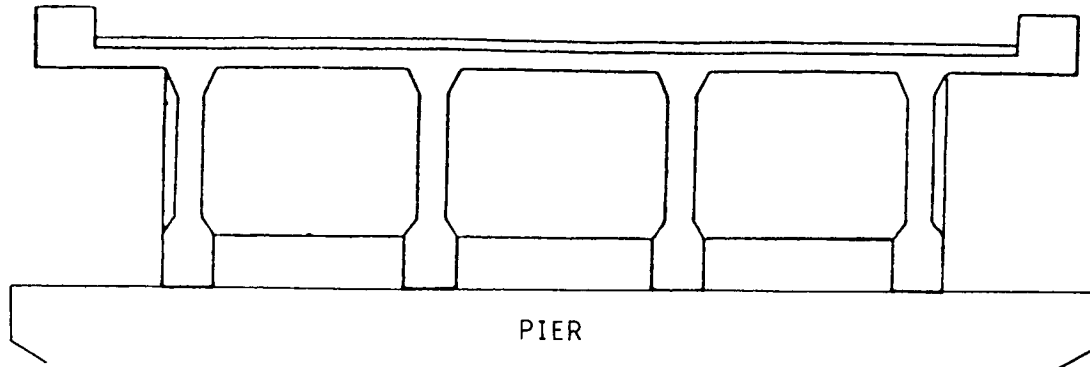


(b) Reinforcement details in footings

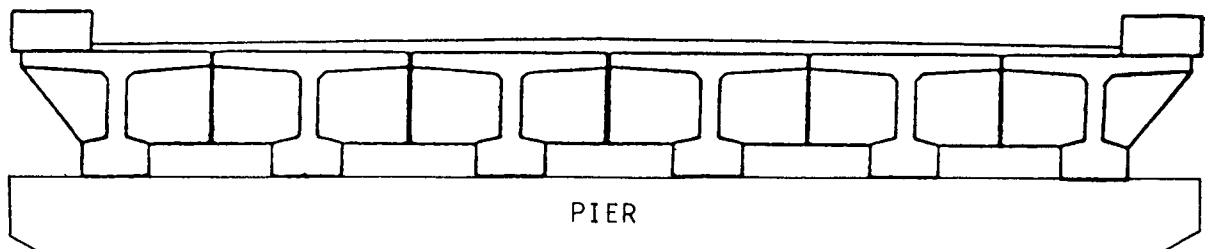


**TYPICAL BRIDGE DECKS WITH POST-TENSIONED GIRDERS SHEET NO. VIII.1.9
(BRITISH PRACTICE)**

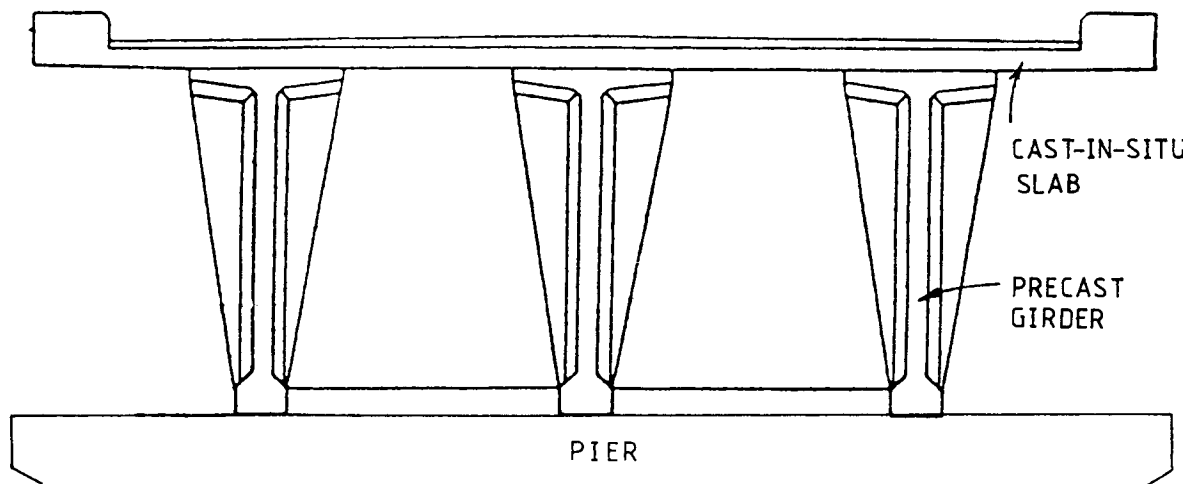
(a) *Cast-in-situ* prestressed concrete deck



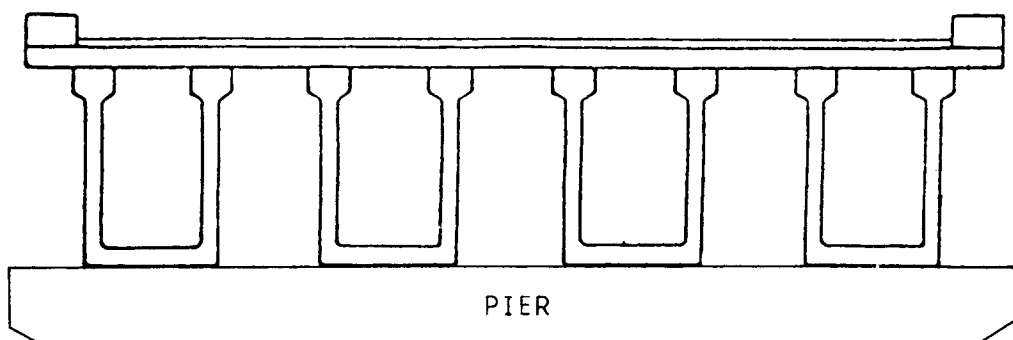
(b) Bridge with precast prestressed girders



(c) Deck with composite construction



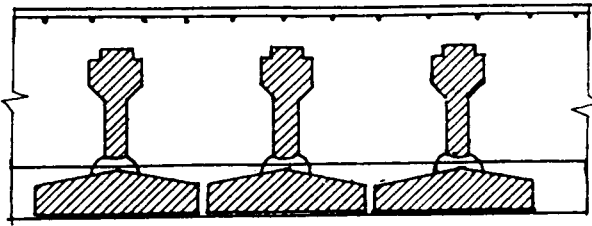
(d) Deck with composite construction



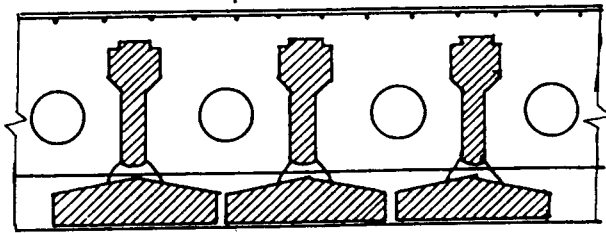
TYPICAL BRIDGE DECKS WITH POST-TENSIONED GIRDERS (CONTD) (BRITISH PRACTICE)

SHEET NO. VIII.1.10

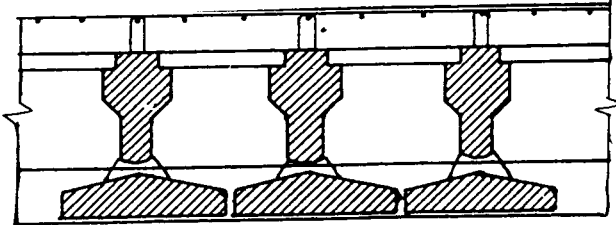
(e) Pretensioned inverted T-beam



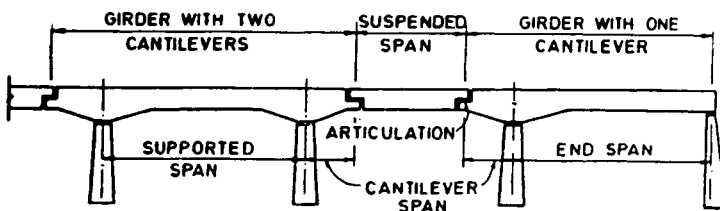
(f) Hollow cored pretensioned deck



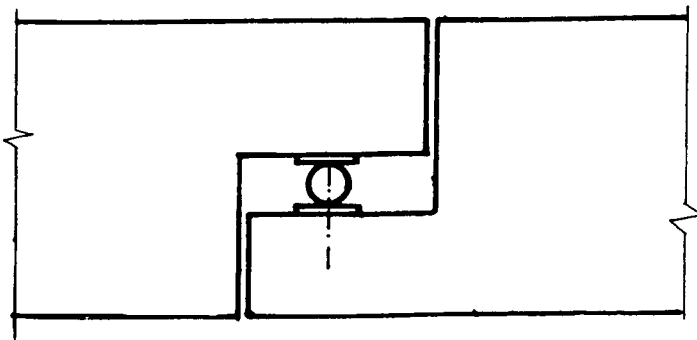
(g) Pretensioned beam with top decking



(h) Schematic diagram of balanced cantilever bridge

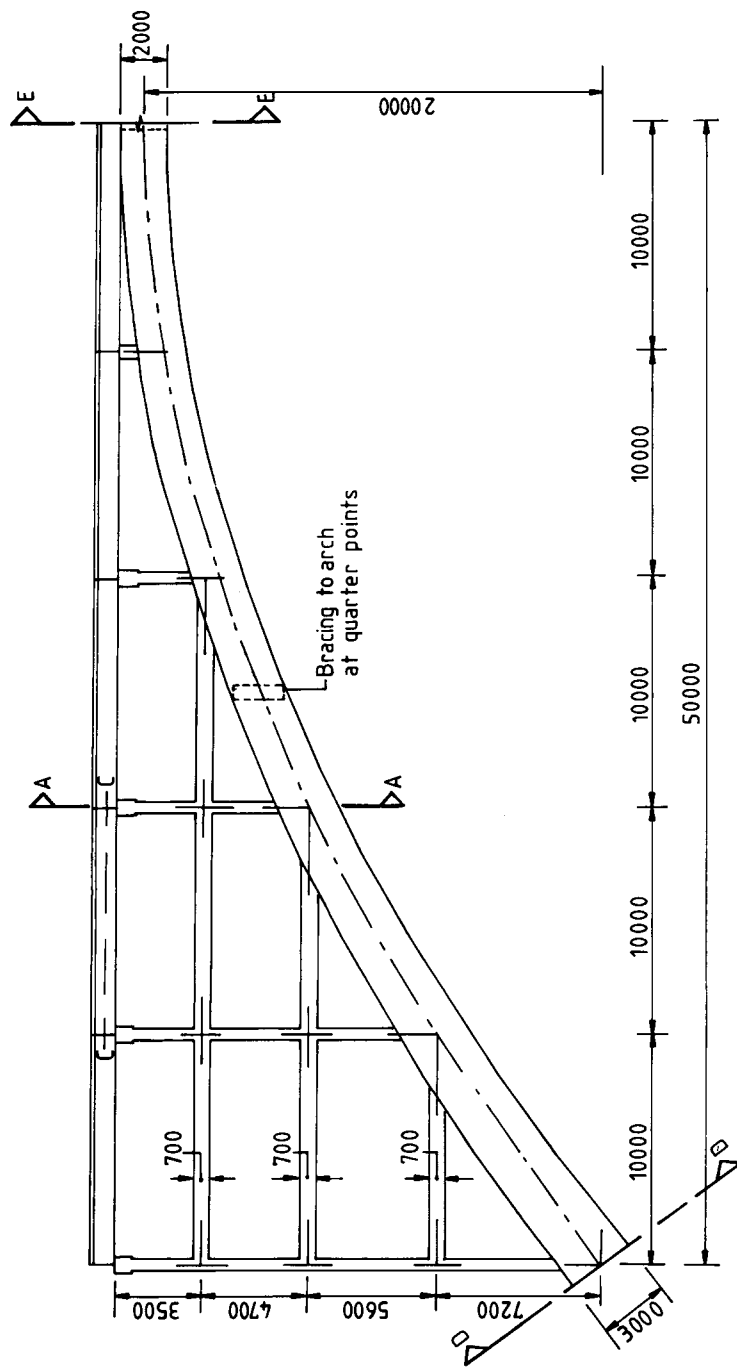


(i) Articulation



**OPEN SPANDREL ARCH BRIDGE
(BRITISH PRACTICE)**

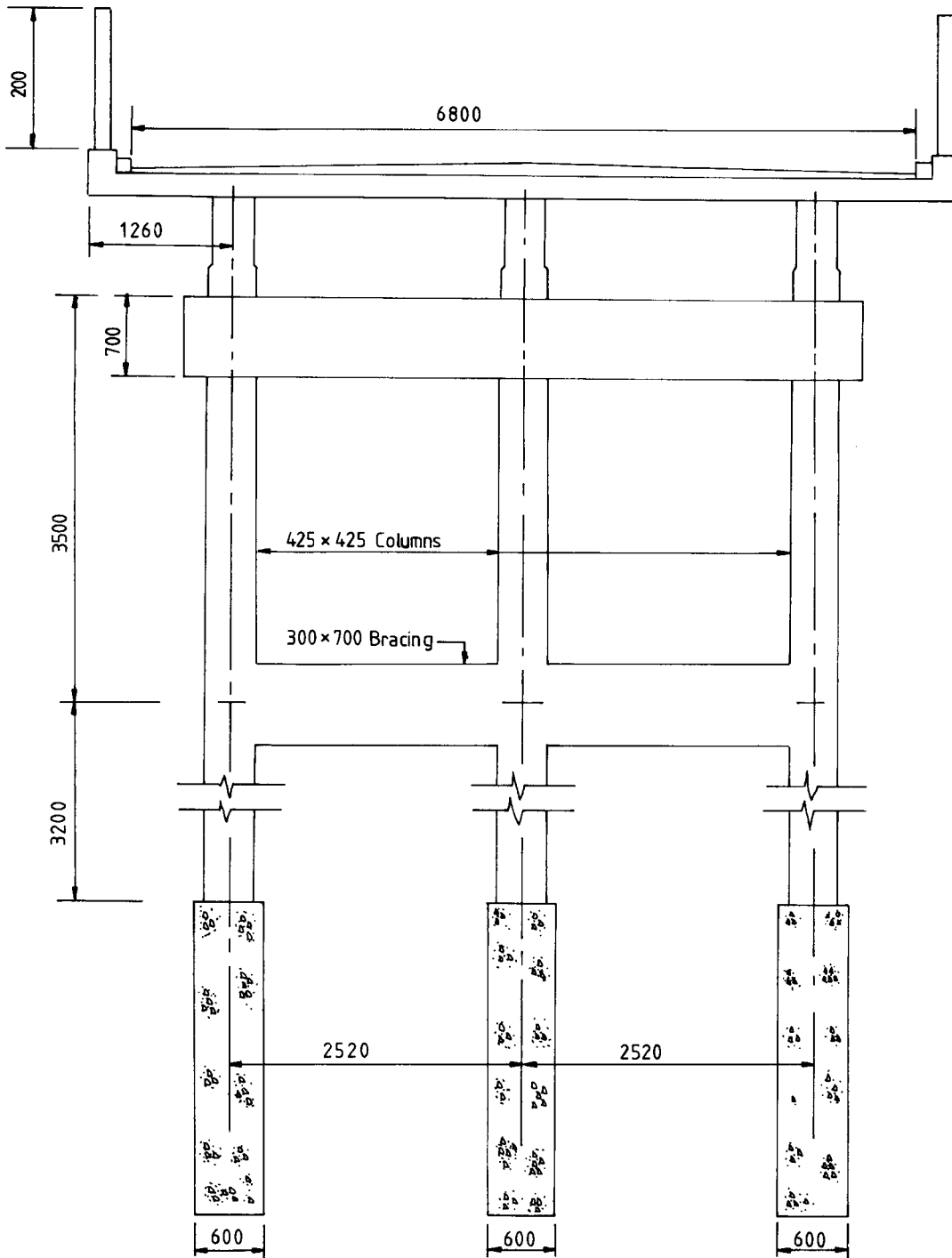
SHEET NO. VIII.1.11



ELEVATION OF HALF SPAN OF BRIDGE
SCALE 1:200

**OPEN SPANDREL ARCH BRIDGE
(BRITISH PRACTICE)**

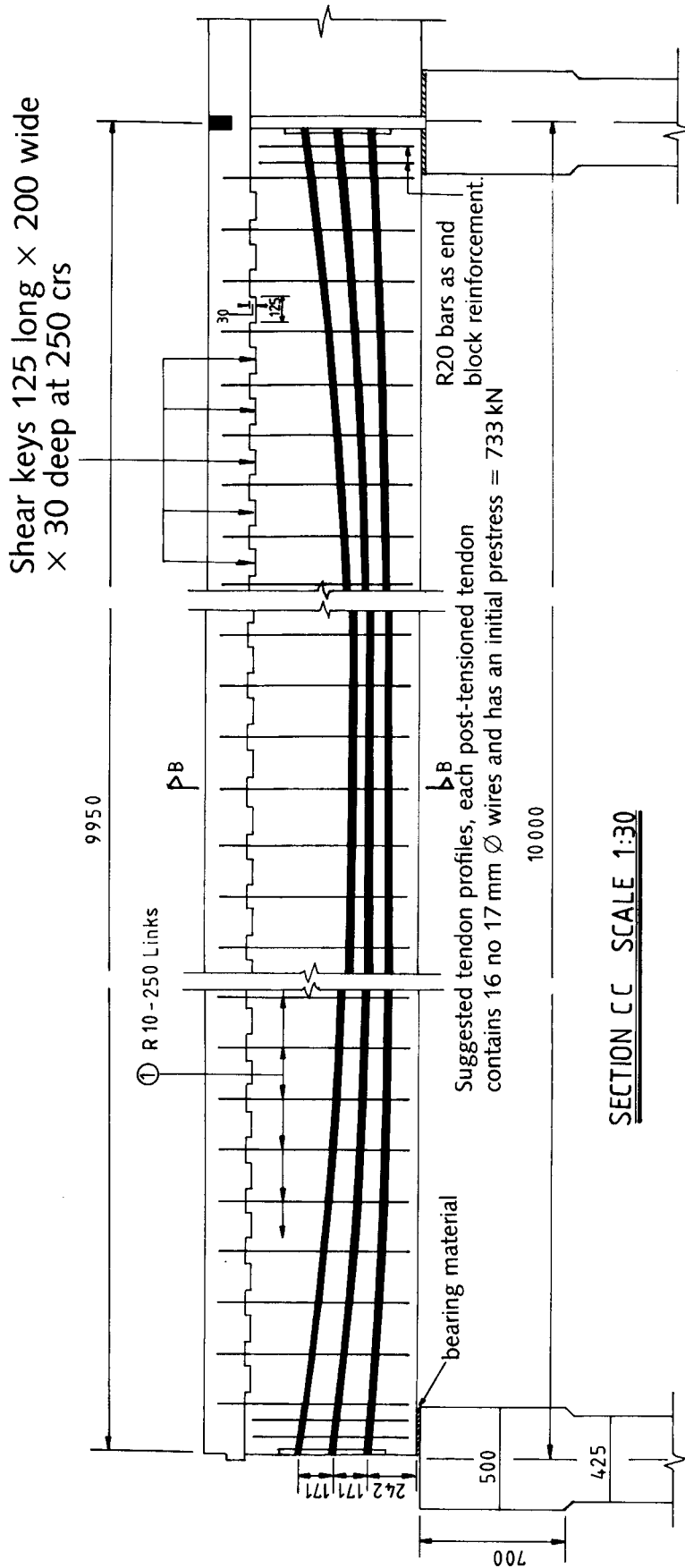
SHEET NO. VIII.1.12



SECTION A-A SCALE 1:50

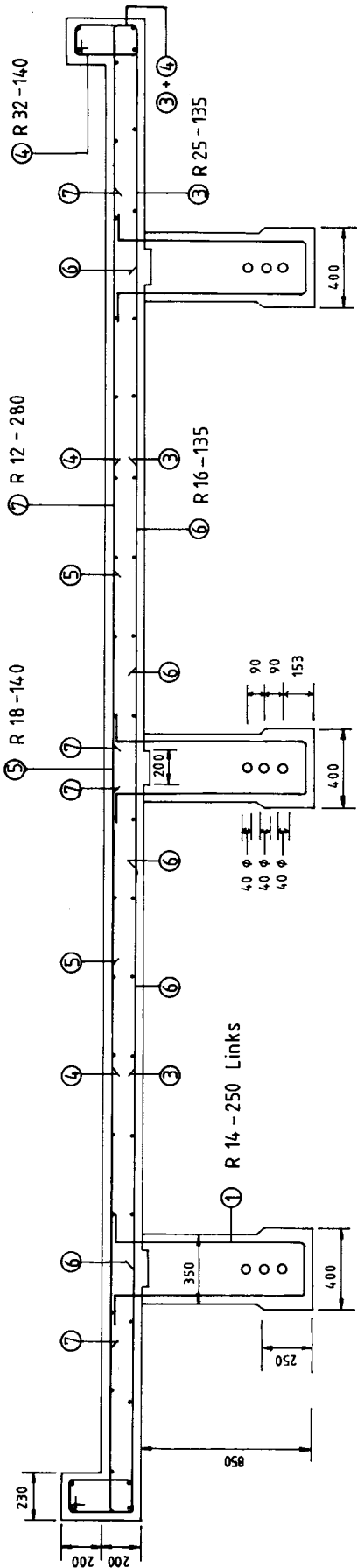
**OPEN SPANDREL ARCH BRIDGE
(BRITISH PRACTICE)**

SHEET NO. VIII.1.13



**OPEN SPANDREL ARCH BRIDGE
(BRITISH PRACTICE)**

SHEET NO. VIII.1.14

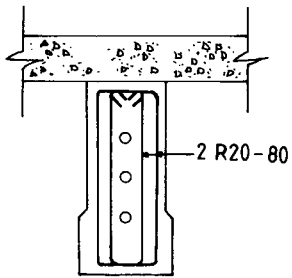


NOTE: All bars shown thus
are 10 dia distribution bars ⑥

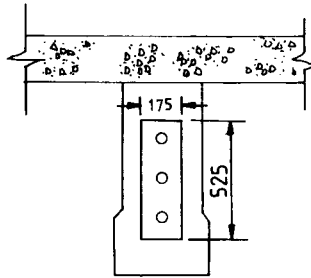
SECTION B-B SCALE 1:30

**OPEN SPANDREL ARCH BRIDGE
(BRITISH PRACTICE)**

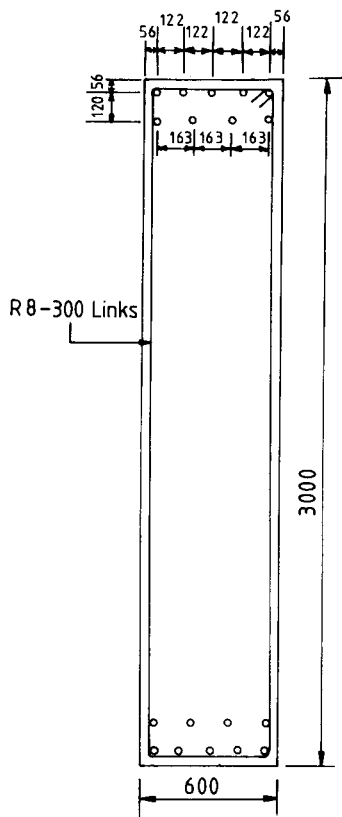
SHEET NO. VIII.1.15



END SECTION OF POST TENSIONED
BEAM SHOWING END BLOCK REINFORCEMENT

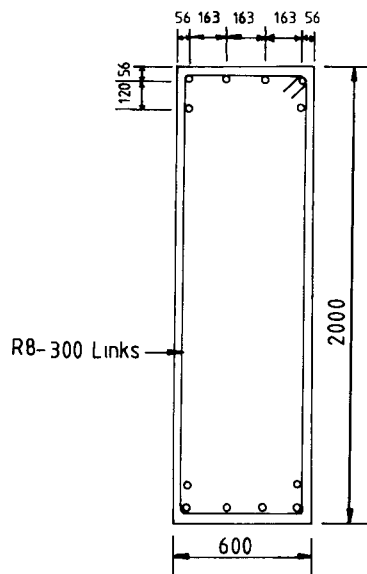


END SECTION OF POST TENSIONED
BEAM SHOWING BEARING PLATE



SECTION D-D

SCALE 1:20



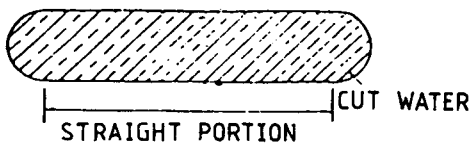
SECTION E-E

NOTE: All longitudinal steel in parabolic arches to be 32 dia.

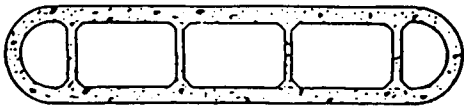
**PIERS AND BED BLOCK
(BRITISH PRACTICE)**

SHEET NO. VIII.1.16

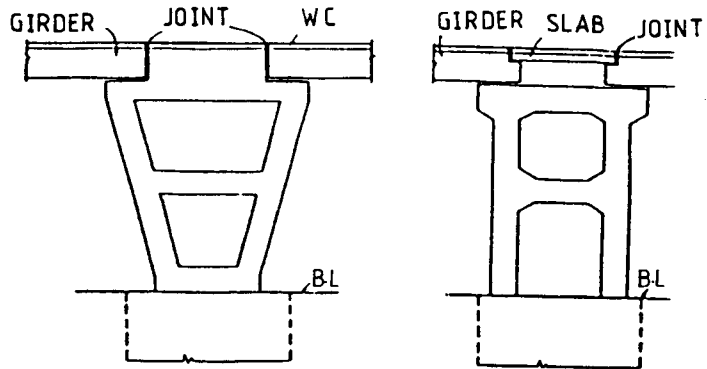
(a) Solid pier of masonry



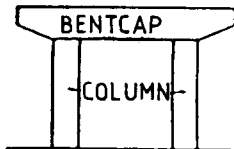
(b) Cellular R.C. pier



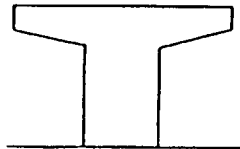
Typical framed piers



(c) Trestle R.C. pier

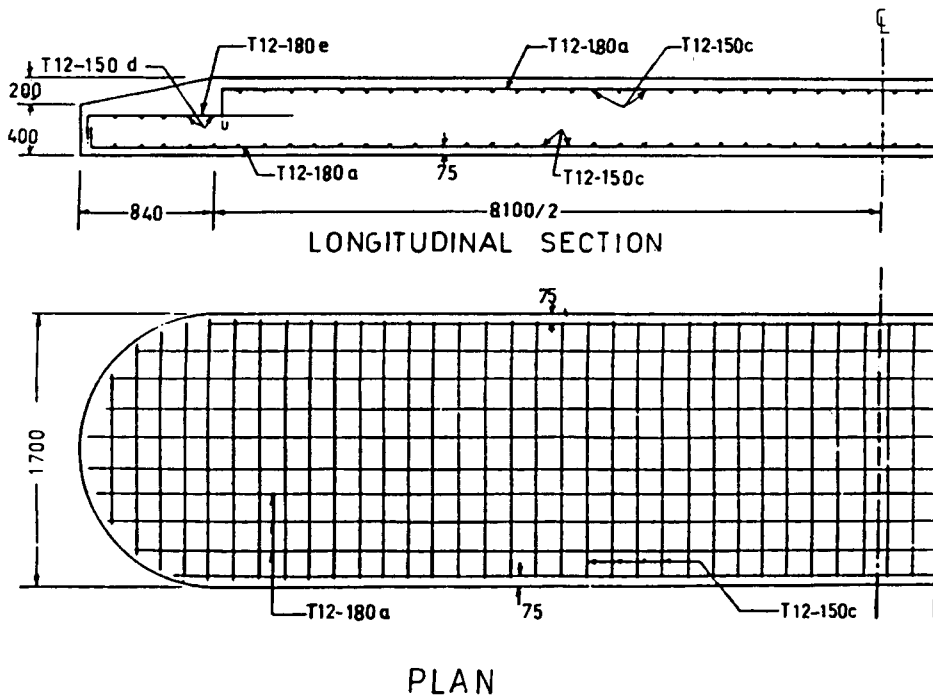


(d) Hammerhead type



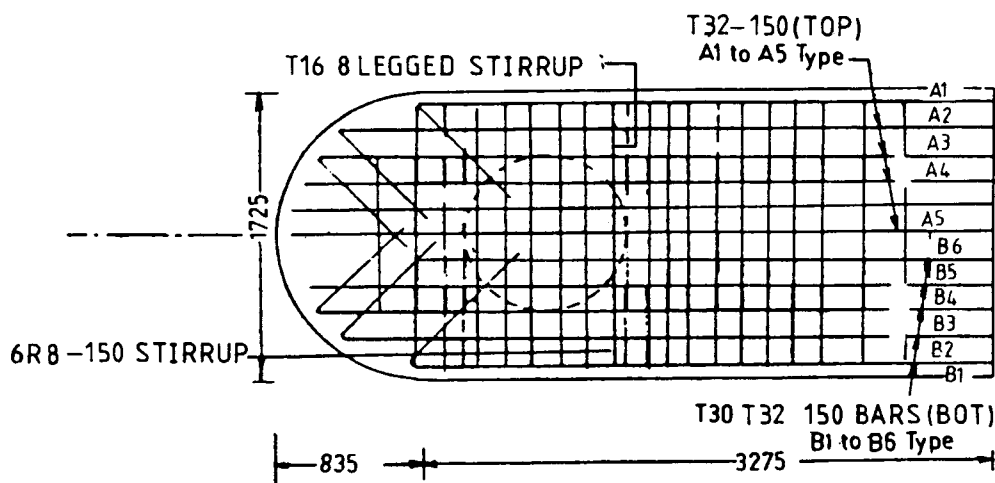
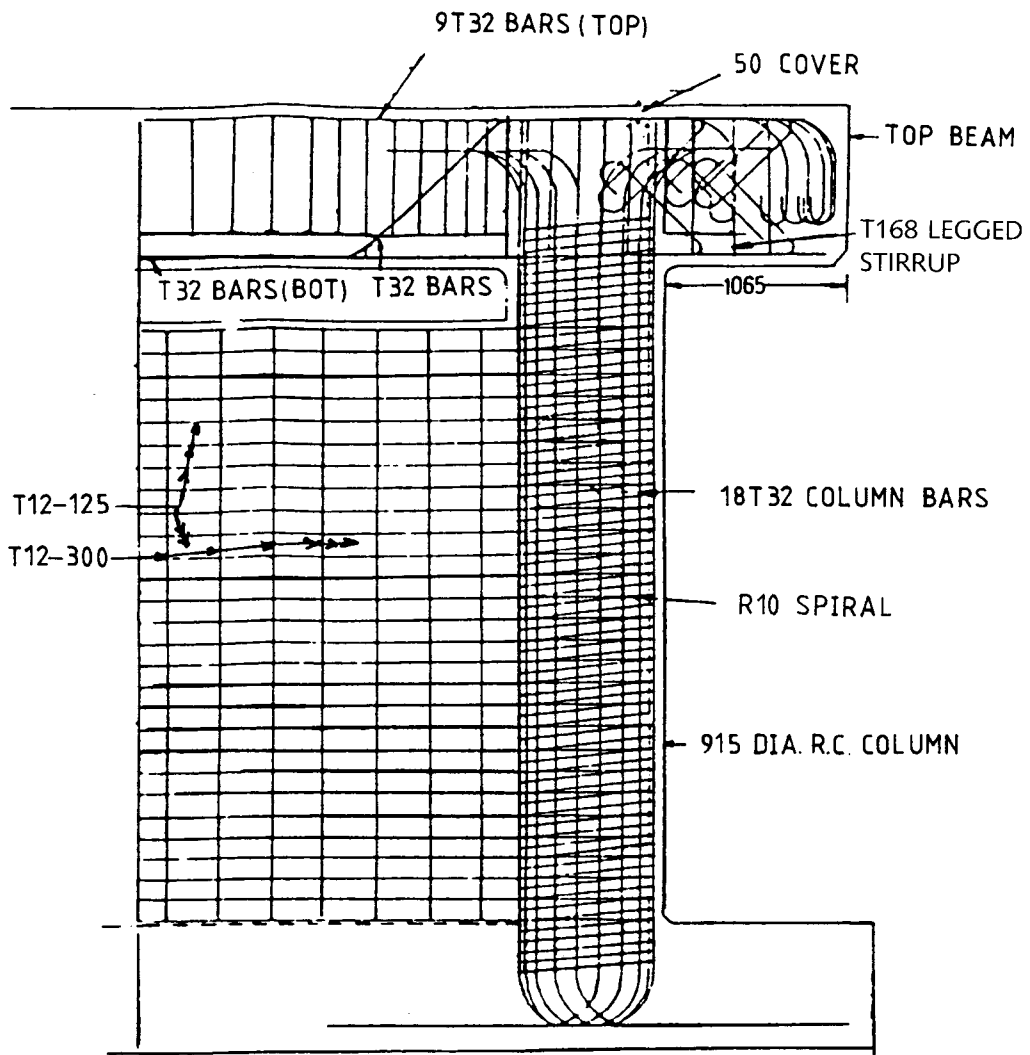
Typical shape of piers

Typical details of bed block over piers



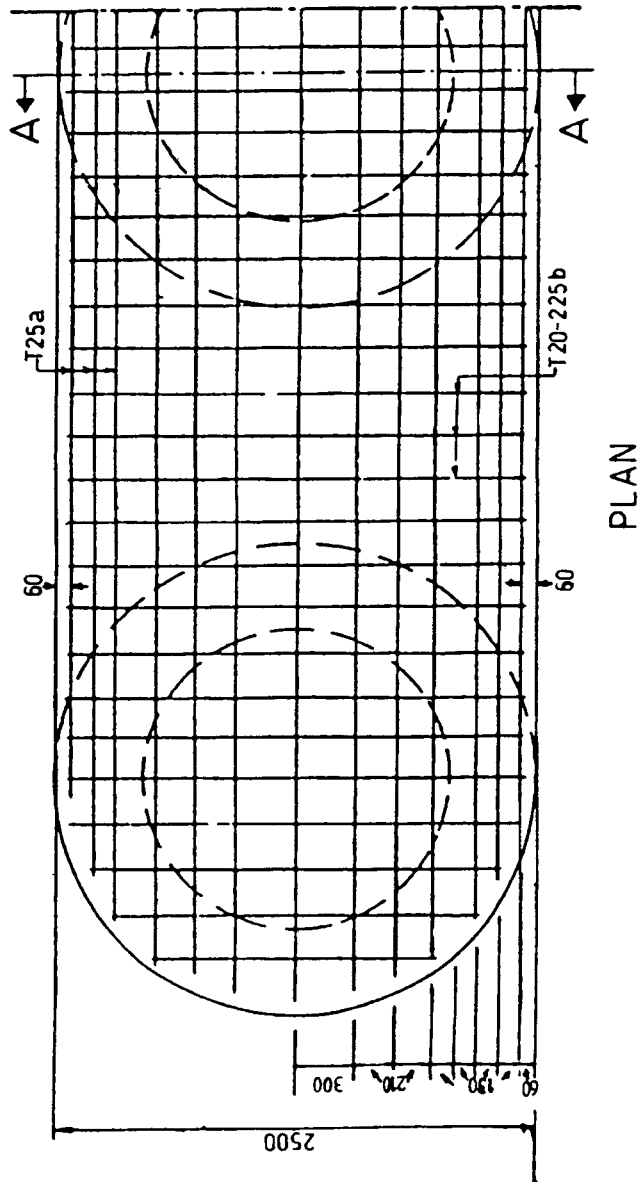
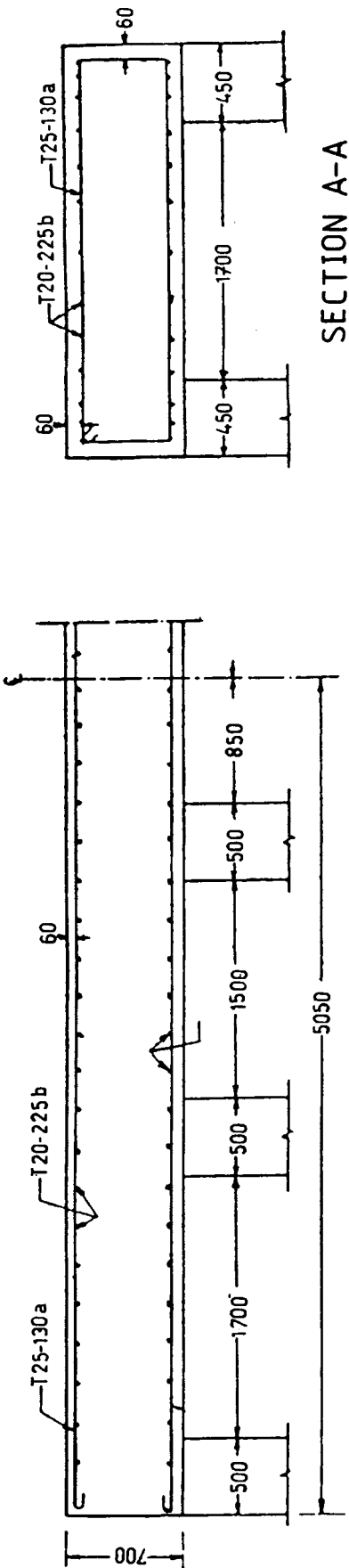
**DETAILS OF PIER BENT
(AMERICAN PRACTICE)**

SHEET NO. VIII.1.17



WELL CAP FOR PIER
(BRITISH PRACTICE)

SHEET NO. VIII.1.18



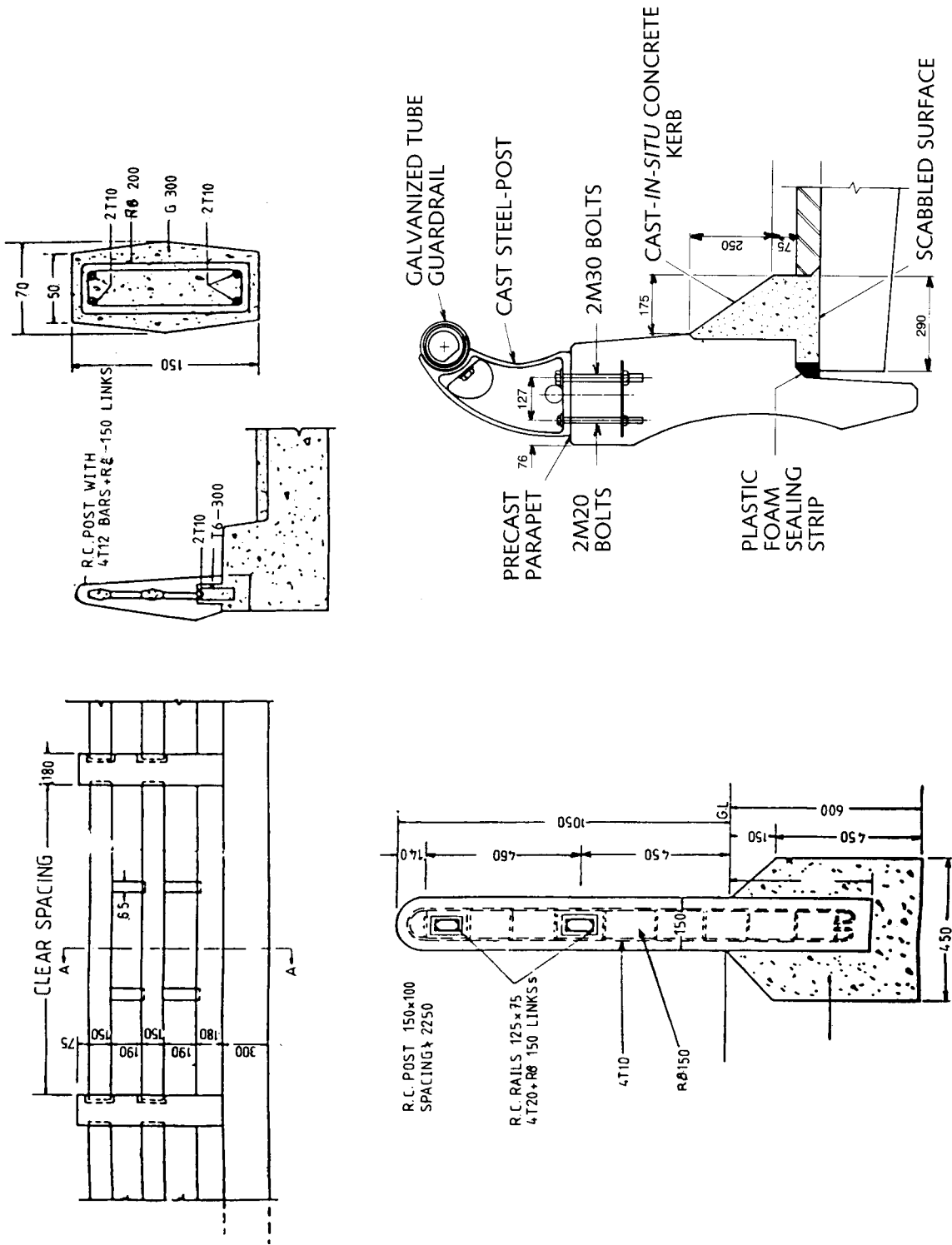
As stated earlier, in order to protect vehicles from accidents, and the pedestrians while crossing the main bridge, typical details are given for hand-rails forming the ballustrades on Sheet No. VIII.1.19.

Sheet Nos. VIII.1.20 and VIII.1.21 give different types of bridge bearings. Structural engineers working on bridges and codes of practice are consulted on the use of any one of these on a specific job. The manufacturers can provide loads and specifications for individual types of bearings.

Sheet Nos. VIII.1.22 to VIII.1.26 show detailing of various elements of a cable-stayed access bridge at the Milan Air Station, and Sheets VIII.1.27 to VIII.1.30 show deck and pier reinforcement for a longer-span cable-stayed bridge.

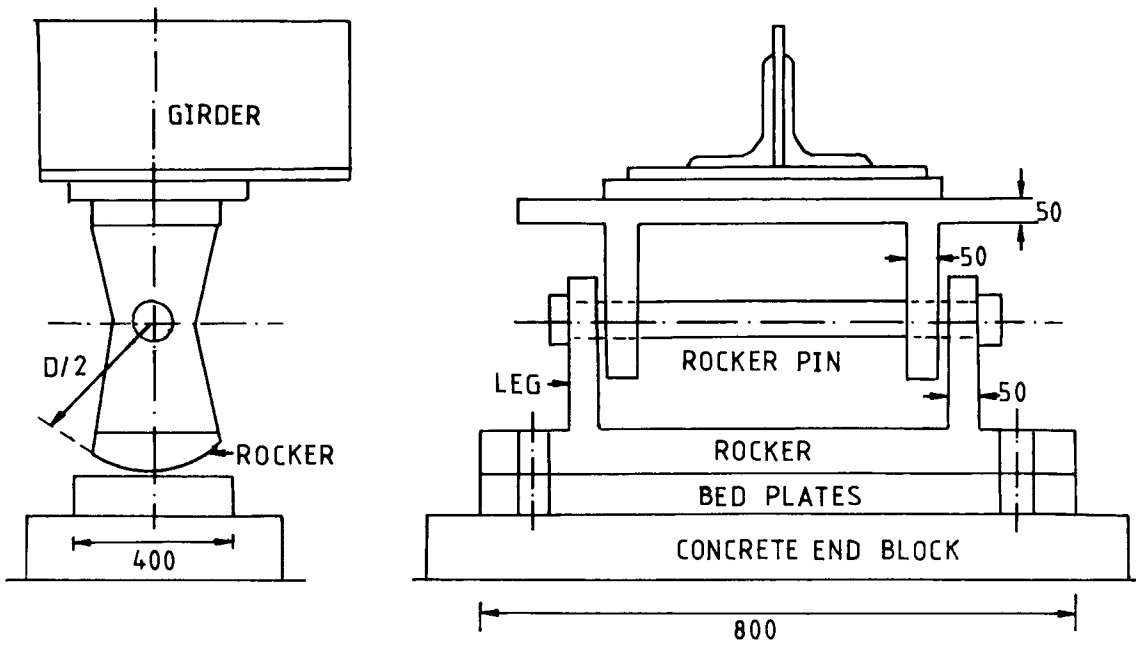
**DETAILS OF HAND-RAILS
(AUSTRALIAN PRACTICE ADOPTED IN UK)**

SHEET NO. VIII.1.19

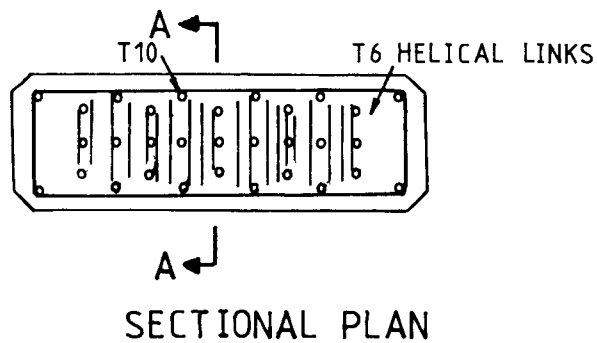
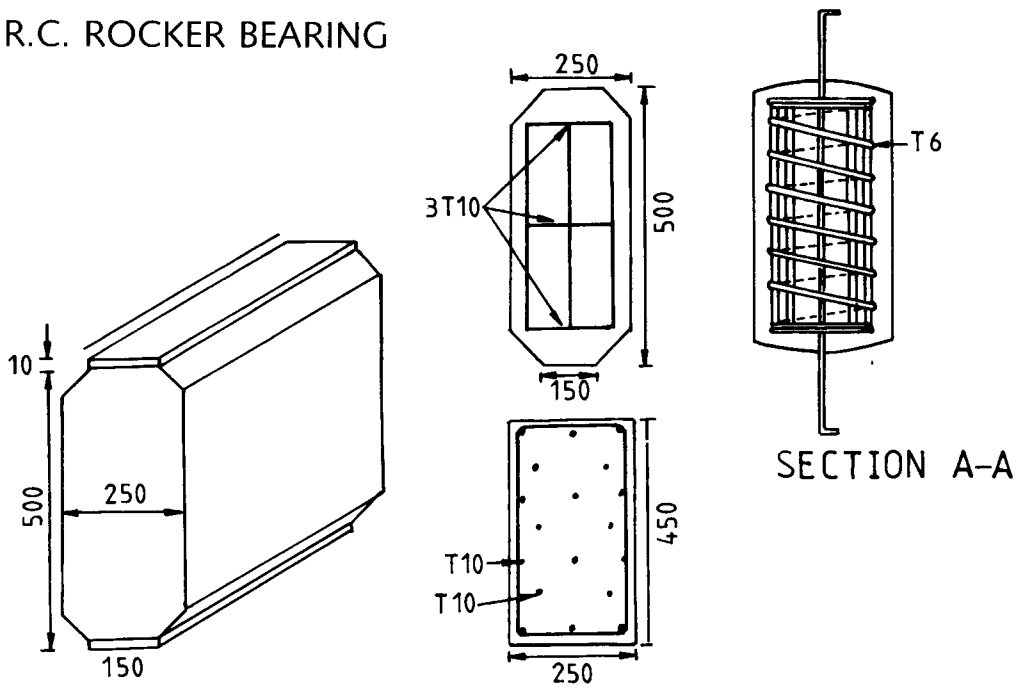


(Courtesy of Overseas Projects Corporation of Victoria, Australia)

TYPICAL ROCKER BEARING FOR COMPOSITE STRUCTURES SHEET NO. VIII.1.20

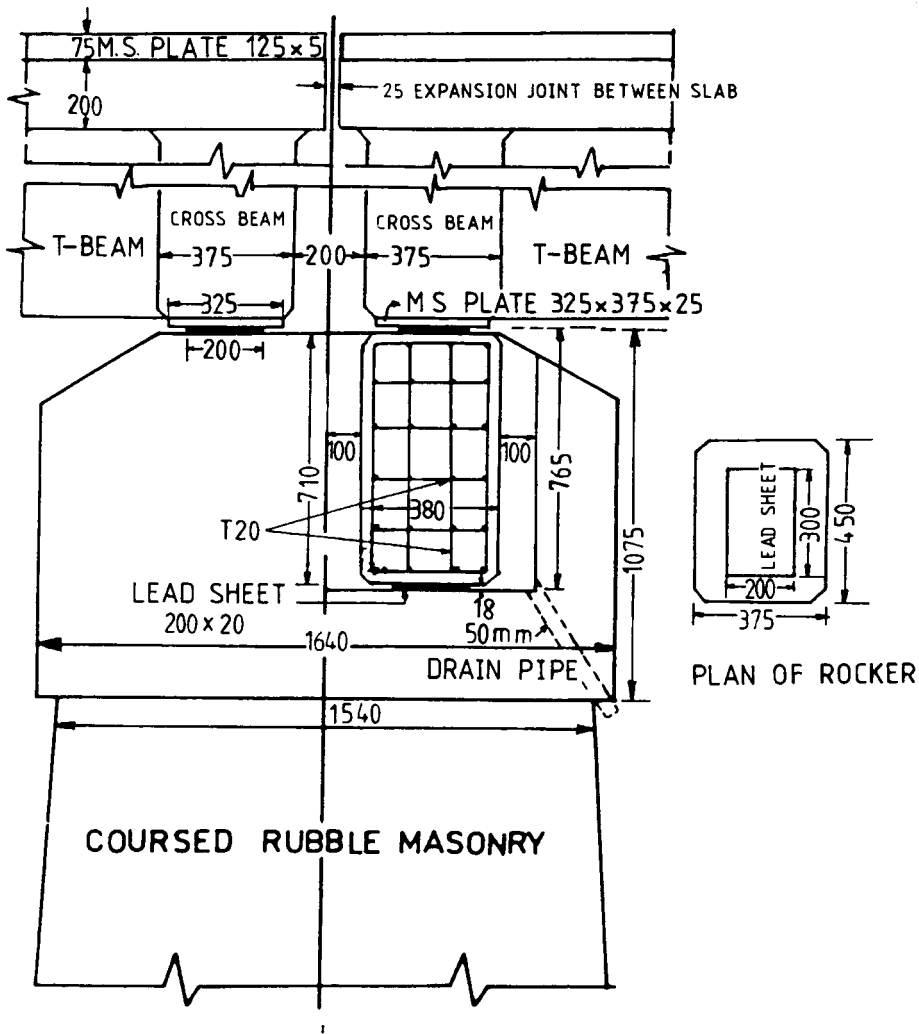


R.C. ROCKER BEARING

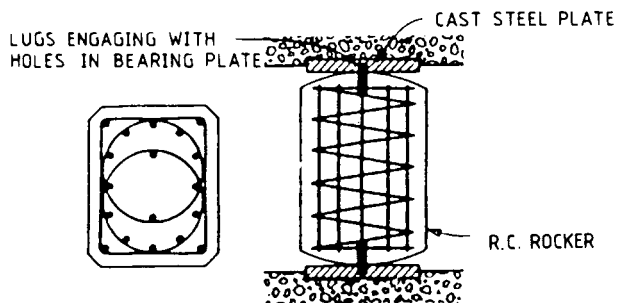


ROCKER BEARING

SHEET NO. VIII.1.21



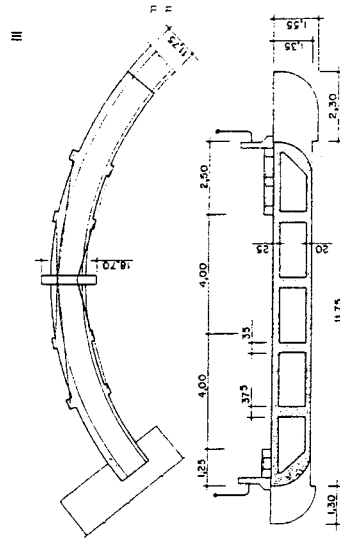
DETAILS OF ROCKER BEARING



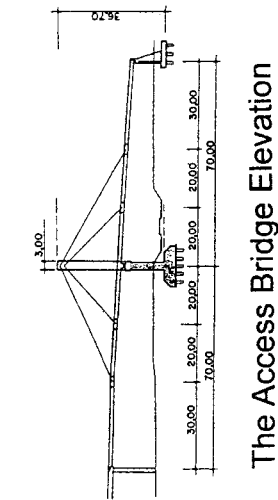
ROCKER BEARING

DECK R.C. AND MPC DETAILS

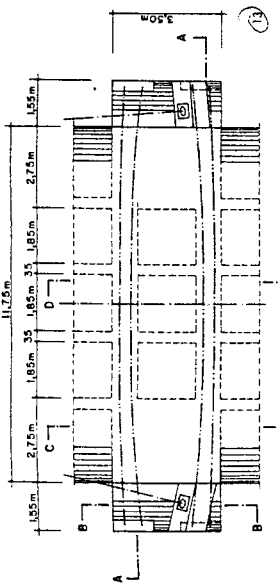
SHEET NO. VIII.1.22



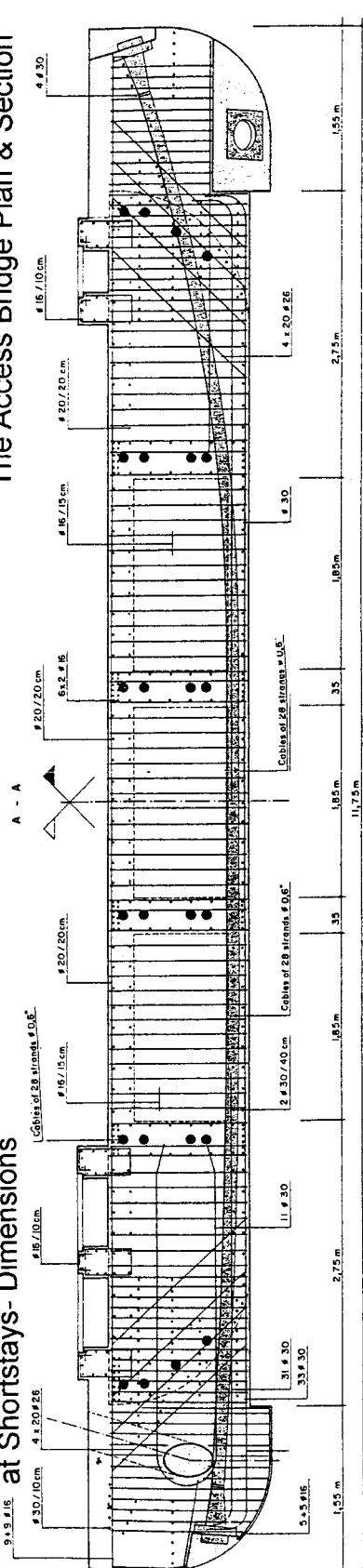
The Access Bridge Plan & Section



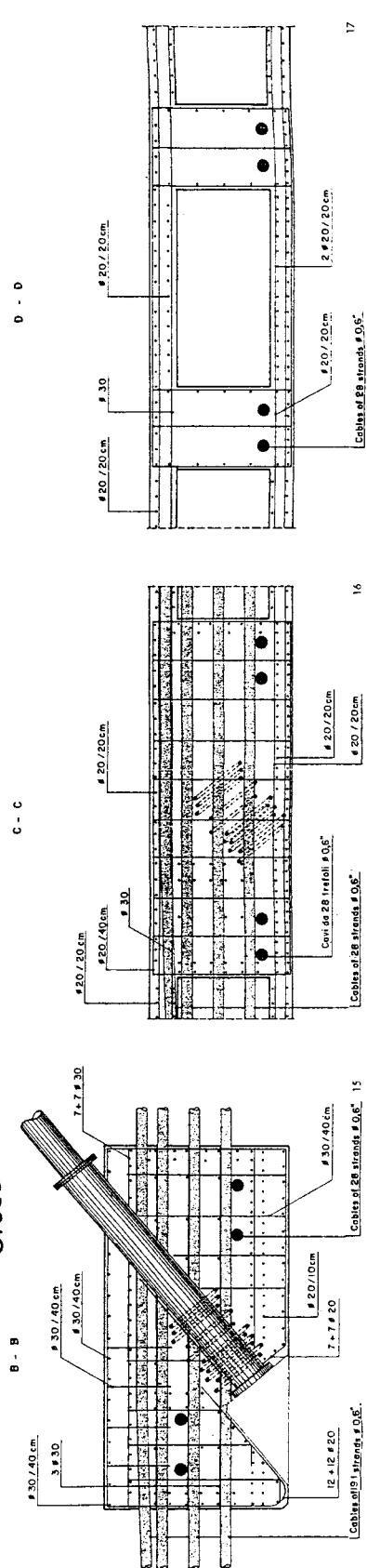
The Access Bridge Elevation



Plan for the Deck Cross Beams at Shortstays - Dimensions



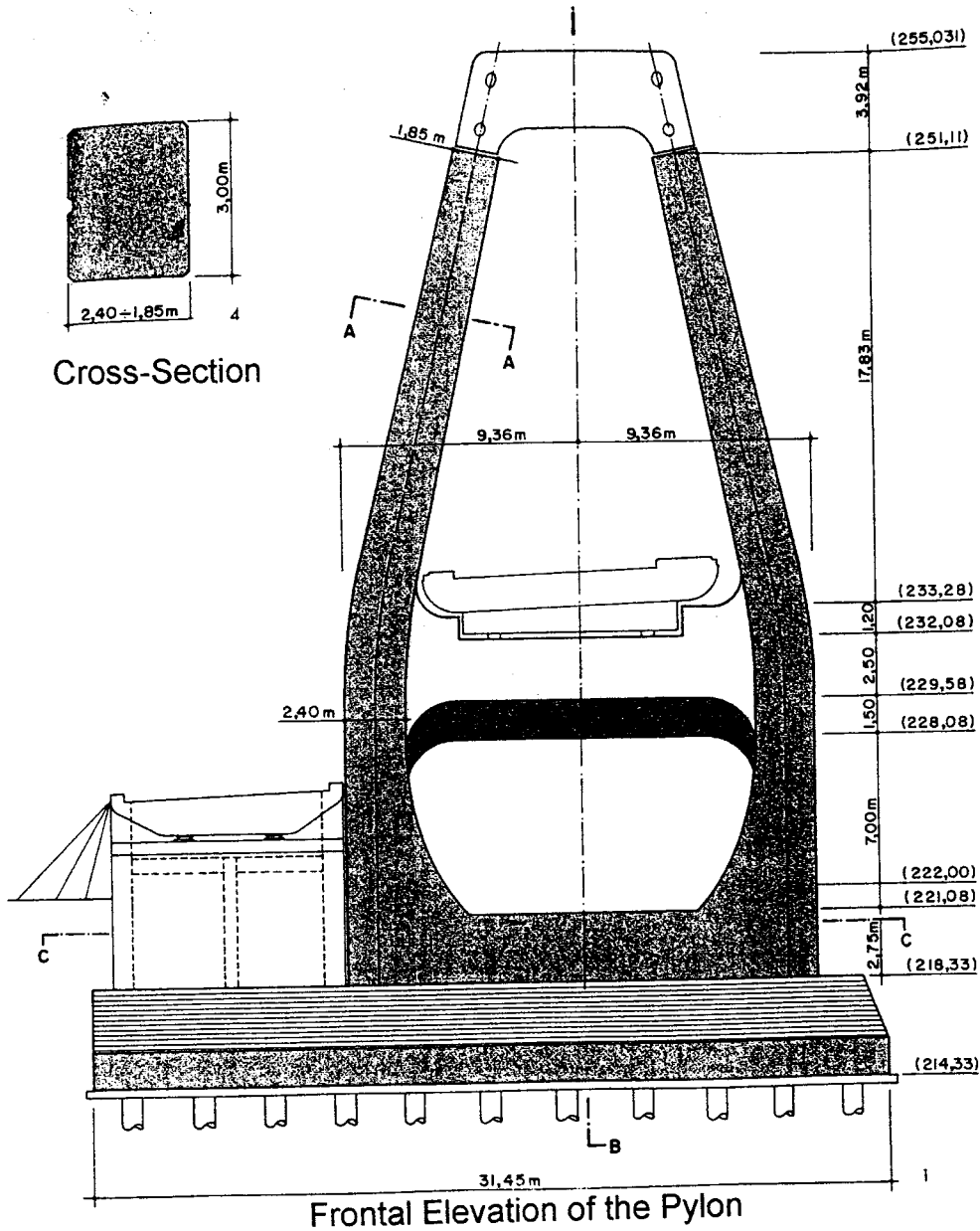
Cross Beam Section Conventional Prestressed Concrete Reinforcement Details



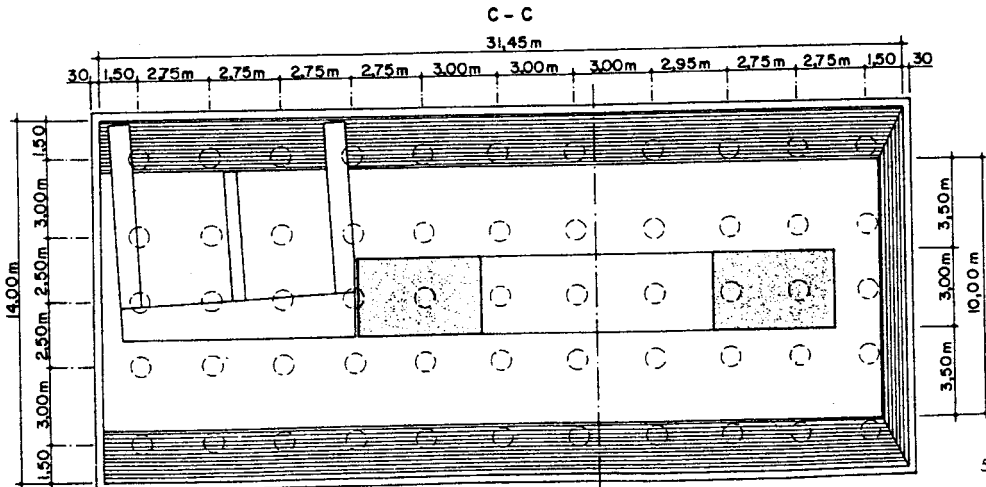
Reinforcements for Beams Chords

PYLON/TOWER SECTION AND PLAN

SHEET NO. VIII.1.23



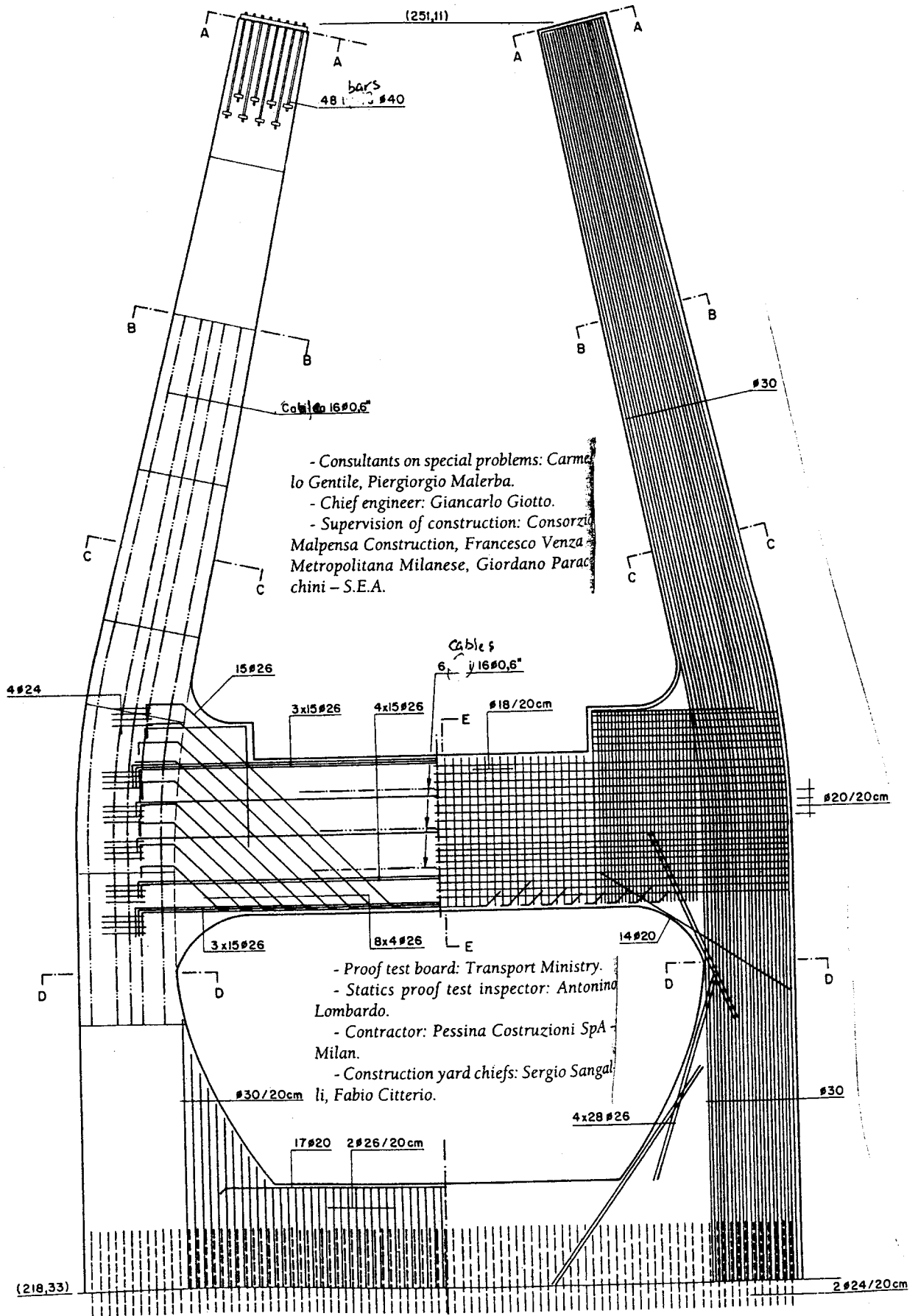
Frontal Elevation of the Pylon



Foundation Plan

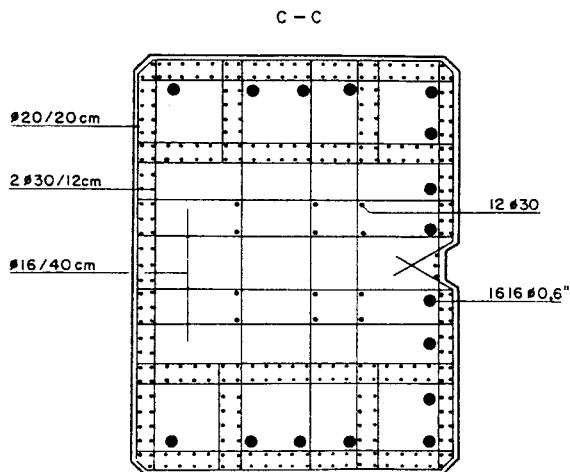
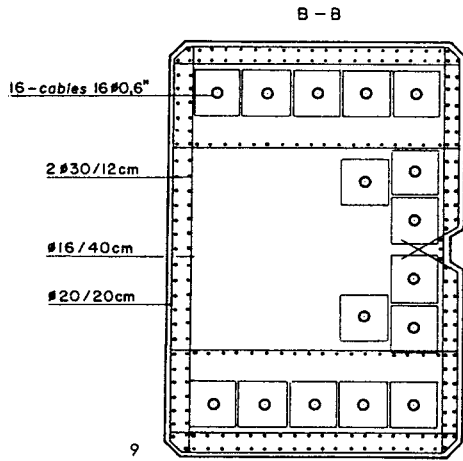
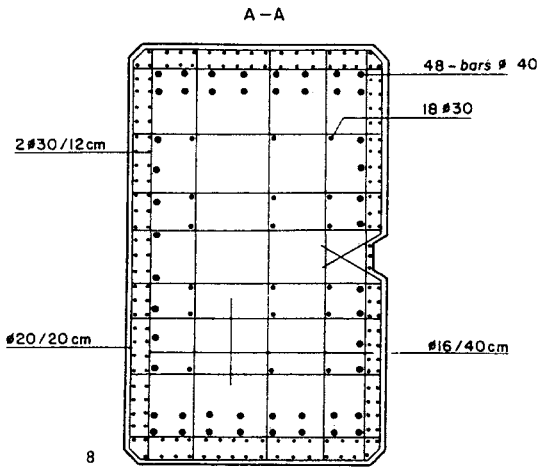
REINFORCEMENT DETAILS FOR A PYLON

SHEET NO. VIII.1.24

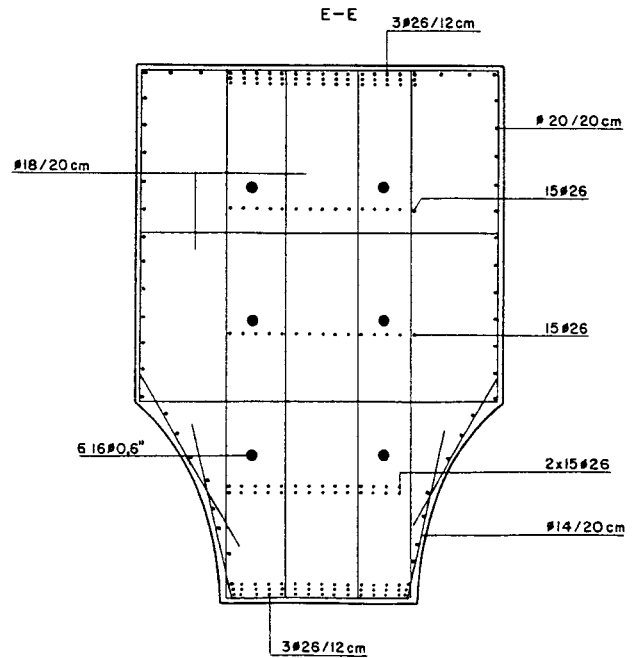
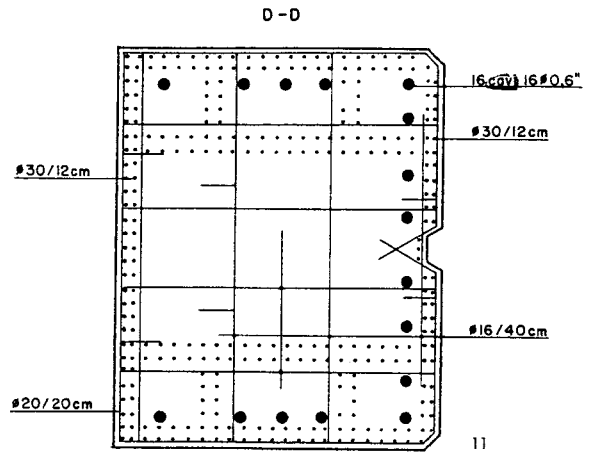


PYLON UPRIGHTS R.C. DETAILS

SHEET NO. VIII.1.25

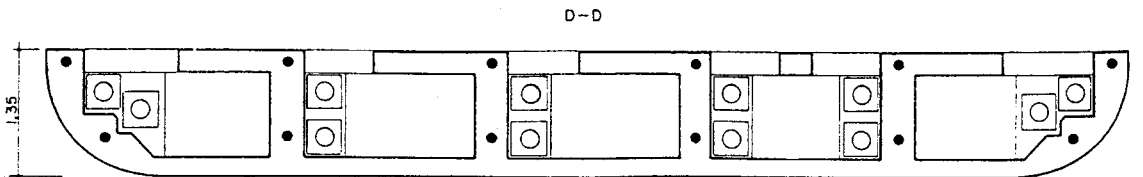
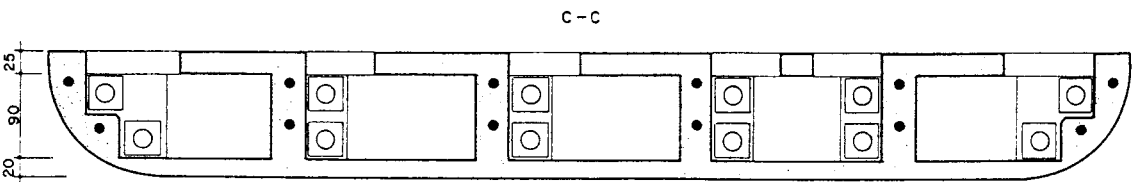
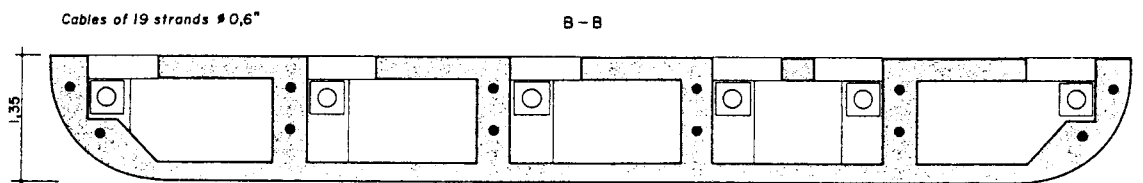
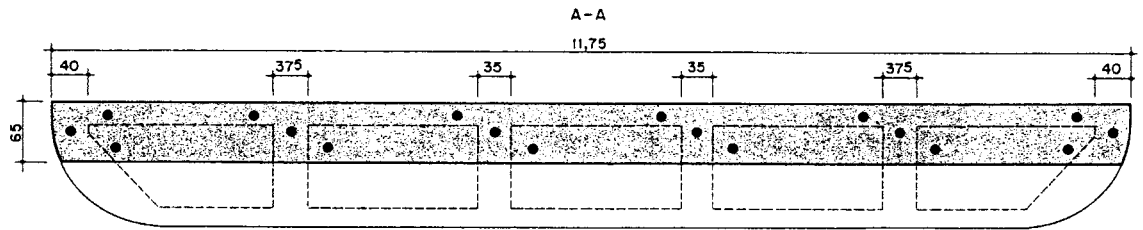
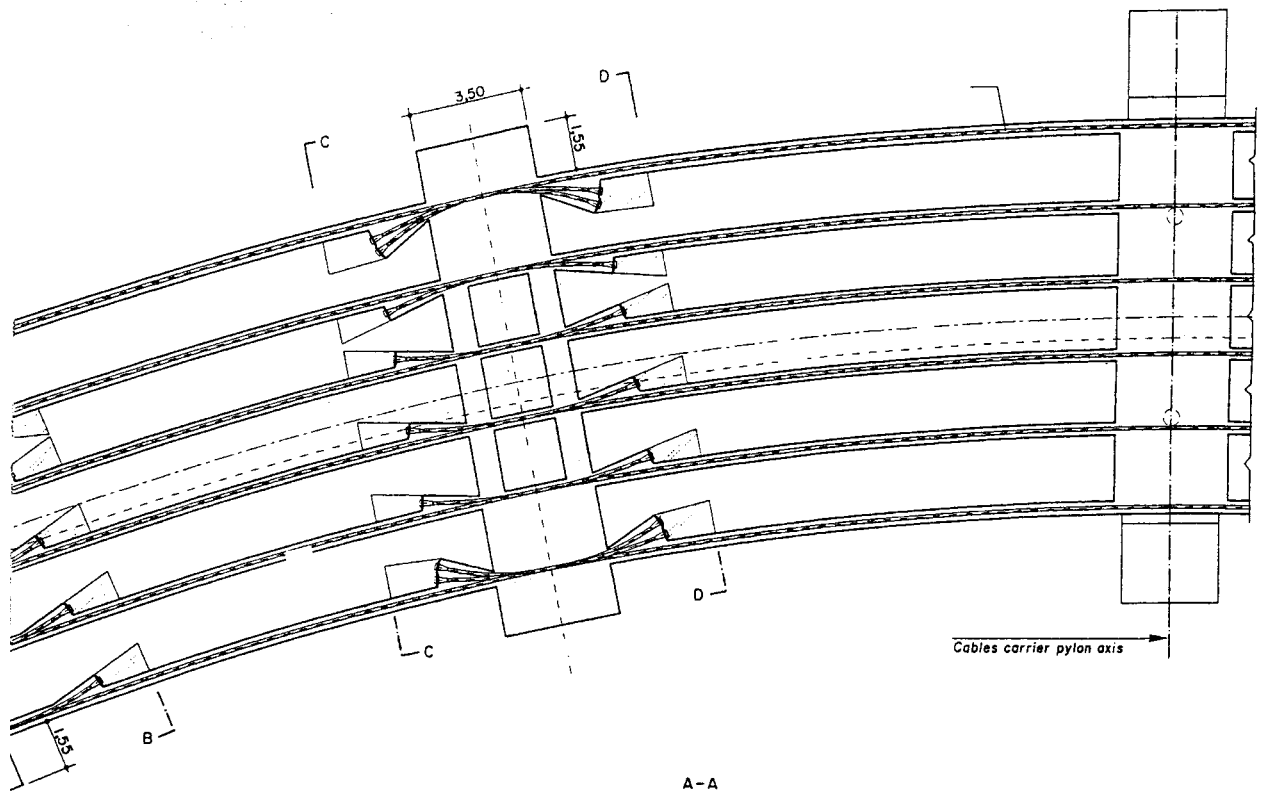


Customer administration: Transport Ministry-E.N.A.C
Customer's agent concessionary for the work:
 S.E.A. Societa esercizi Aeroportuali-Milan
 D.N.C.I. Direzione Nuove Costruzioni e Impianti
Road system design: Giovanni Da Rios
Structure design: Francesco Martinez y Cabrera



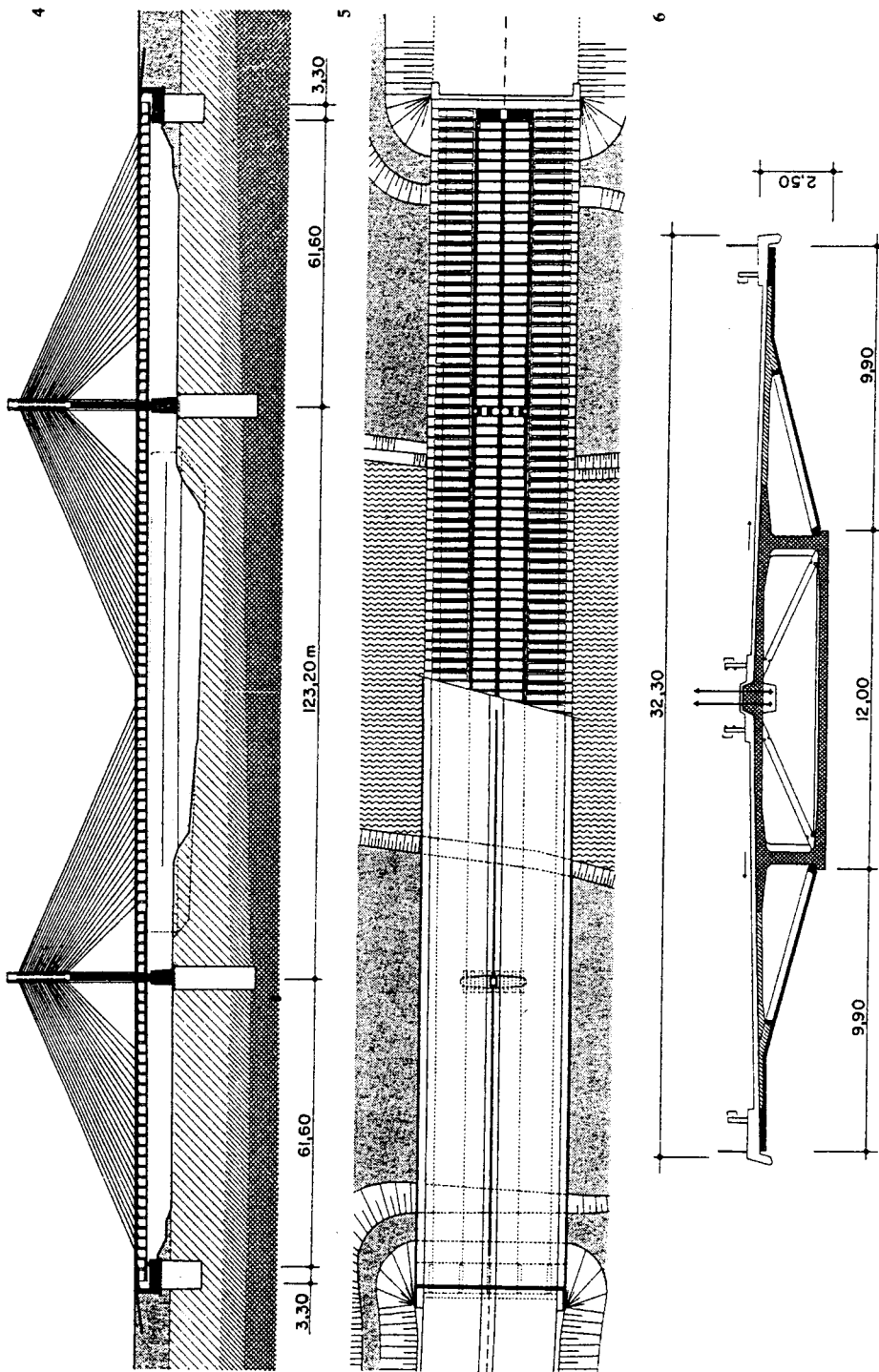
DECK CROSS-SECTION CABLES AND ANCHORAGES

SHEET NO. VIII.1.26



**CABLE STAYED BRIDGE ACROSS THE ELBE RIVER
AT PODEBRADY CZECH**

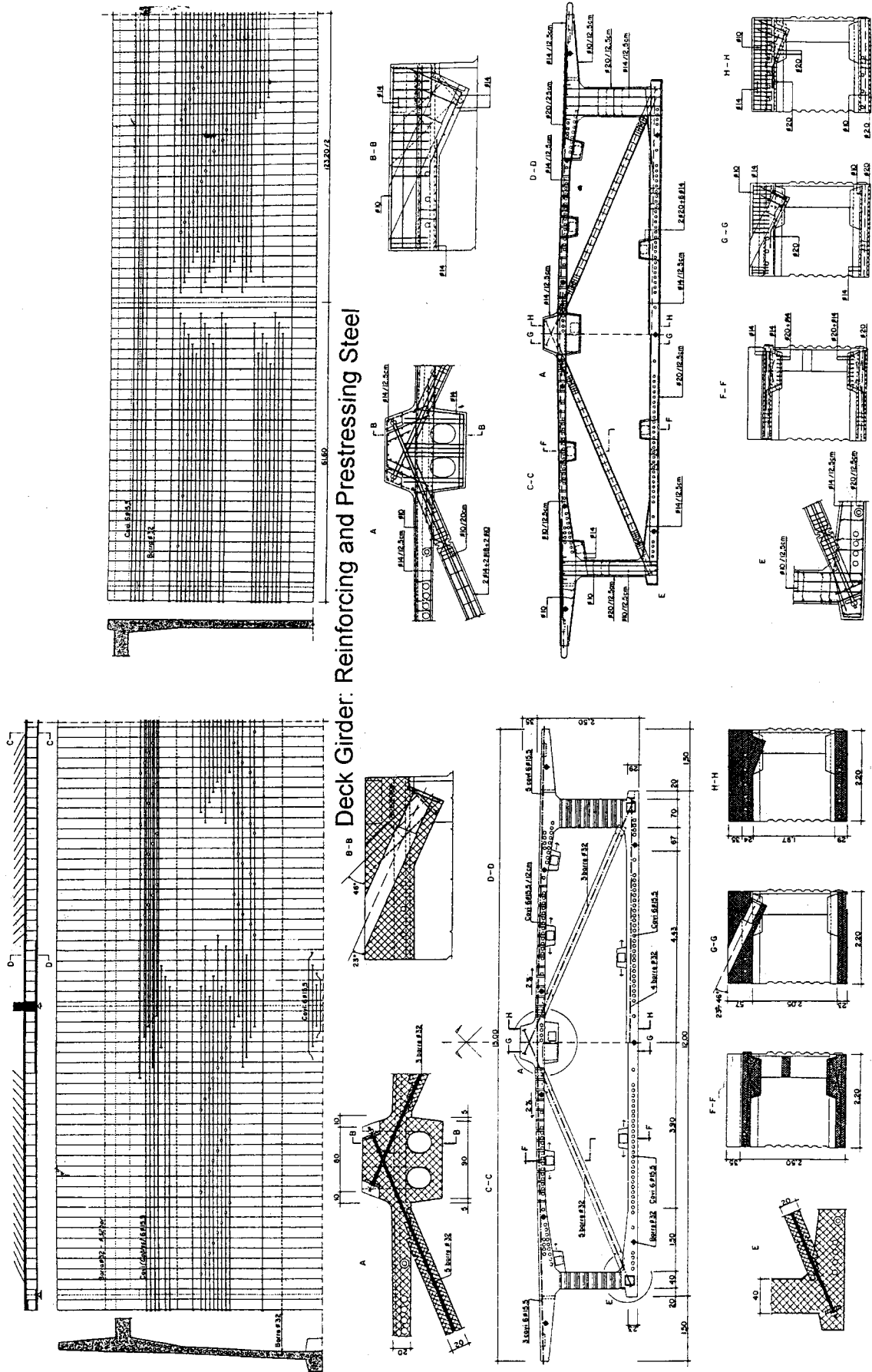
SHEET NO. VIII.1.27



*Chief Design Engineer: Ing. Jiri Strasky
Project Manager: Ing. Miroslav Korenek
Chief Research Engineer: Ing. Vojtech Menci
Design and Construction Engineers :Dopravni Stavby Olomouc, s.p.*

DECK R.C./P.C. DETAILS

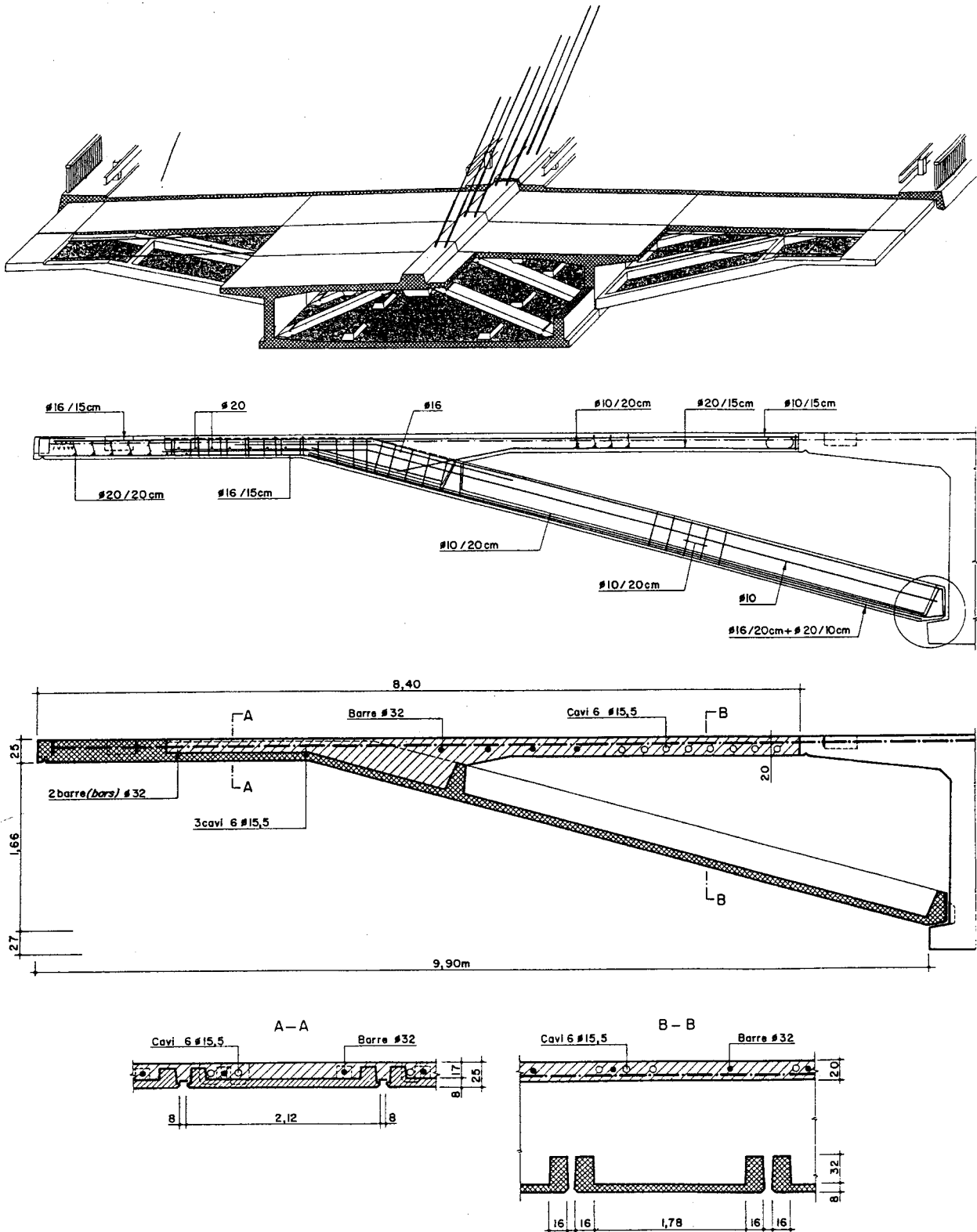
SHEET NO. VIII.1.28



Deck Girder: Reinforcing and Prestressing Steel

R.C. AND P.C. DECK CANTILEVERS – R. DETAILS

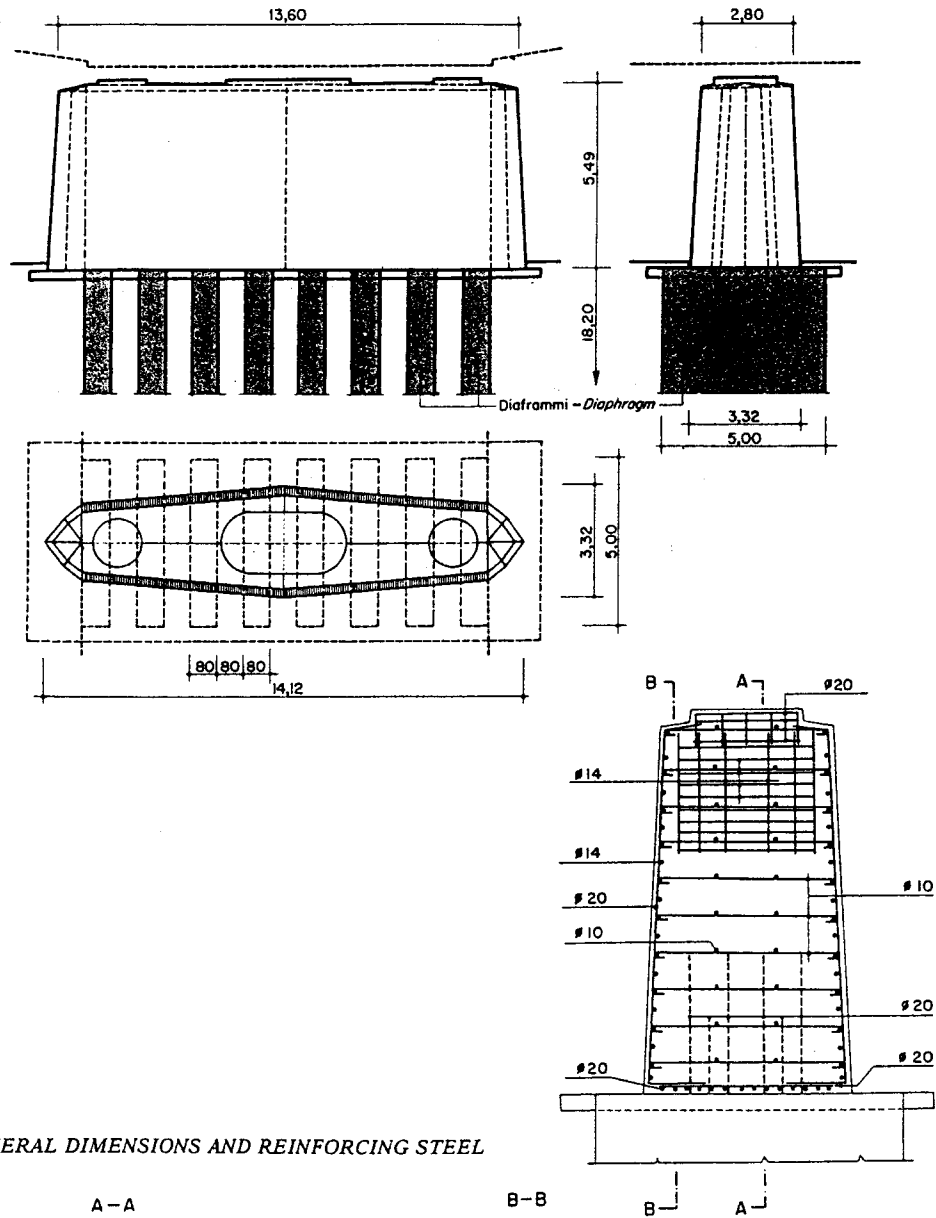
SHEET NO. VIII.1.29



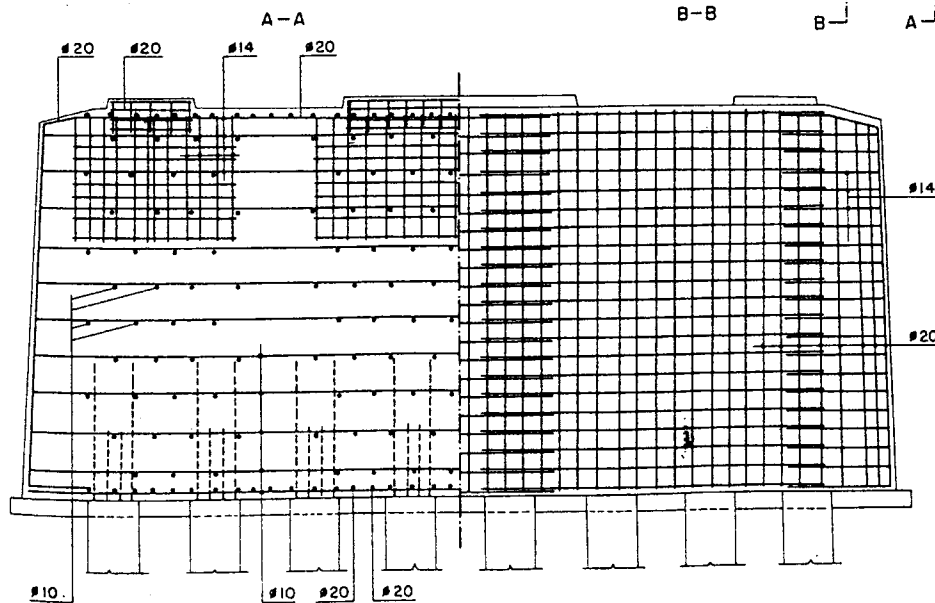
Axonometric View of the Deck and Reinforcing and Prestressing steel of the Cantilevers

PIERS AND SUPPORTS
R.C. DETAILS

SHEET NO. VIII.1.30



PIERS: GENERAL DIMENSIONS AND REINFORCING STEEL



VIII.2. Conventional building details

VIII.2.1. General introduction

This section covers conventional building details based on:

- (a) BS 8110 and related codes and practices
- (b) EC2 and related codes and practices.

Most of these drawings and details are based on computer-aided work. Important drawings are included to give the reader a comparative study of British and European practices. It is emphasized that the British practice or version is very much European oriented as well.

VIII.2.2. Case studies based on British practice

Prepared by Ward & Cole Constructing Engineers, St. Margaret, Middlesex.

VIII.2.2.1. Drawings with job number 30/386

These drawings illustrate steel reinforcement details to selected structural elements for a multi-storey high specification development. The building consists of a primary reinforced concrete frame and utilizes conventional beam, column, slab and wall load-bearing elements. The building includes a full height basement construction for car parking purposes. The foundations to the building take the form of a thick piled basement slab since this form of construction was established to be the most cost economical solution given the prevailing ground conditions on the site.

Drawing No. 300: Sheet No. VIII 2.1 basement raft RC details
Drawing No. 301: Sheet No. VIII 2.2 basement raft RC details
Drawing No. 302: Sheet No. VIII 2.3 basement raft RC details
Drawing No. 303: Sheet No. VIII 2.4 basement raft RC details
Drawing No. 304: Sheet No. VIII 2.5 basement raft RC details
Drawing No. 305: Sheet No. VIII 2.6 basement raft RC details

VIII.2.2.1.1. RC Walls

Drawing No. 440: Sheet No. VIII 2.7 RC details basement to ground slab
Drawing No. 441: Sheet No. VIII 2.8 RC details basement to ground slab
Drawing No. 442: Sheet No. VIII 2.9 RC details basement to ground slab

VIII.2.2.1.2. GR Beams

Drawing No. 450: Sheet No. VIII 2.10 ground floor beams RC details
Drawing No. 451: Sheet No. VIII 2.11 ground floor beams RC details
Drawing No. 452: Sheet No. VIII 2.12 ground floor beams RC details
Drawing No. 453: Sheet No. VIII 2.13 ground floor beams RC details
Drawing No. 454: Sheet No. VIII 2.14 ground floor beams RC details
Drawing No. 455: Sheet No. VIII 2.15 ground floor beams RC details
Drawing No. 456: Sheet No. VIII 2.16 ground floor beams RC details
Drawing No. 457: Sheet No. VIII 2.17 ground floor beams RC details
Drawing No. 458: Sheet No. VIII 2.18 ground floor beams RC details

Drawing No. 459: Sheet No. VIII 2.19 ground floor beams RC details

Drawing No. 460: Sheet No. VIII 2.20 ground floor beams RC details

Drawing No. 461: Sheet No. VIII 2.21 ground floor beams RC details

***VIII.2.2.1.3. RC Details
columns***

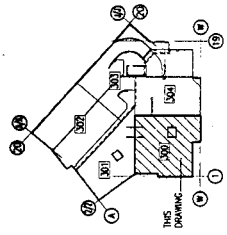
Drawing No. 510: Sheet No. VIII 2.22 ground floor beams RC details

Drawing No. 511: Sheet No. VIII 2.23 ground floor beams RC details

Drawing No. 512: Sheet No. VIII 2.24 ground floor beams RC details

SHEET NO. VIII.2.1

- Notes:
1. For general notes refer to Ward Cole drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 4. High yield steel preferred 'T' on drawing and BS 4449. All rebar to be in accordance with BS 4449. All rebar to be marked 'R' on drawing and delivery logs to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows:
 - 20mm top
 - 20mm bottom
 - 20mm sides
 6. Minimum laps to reinforcement to be as follows:
 - 125 - 1225mm
 - 170 - 800mm
 7. Reinforcement is listed on bending schedules 01-02 incl.



Key Plan

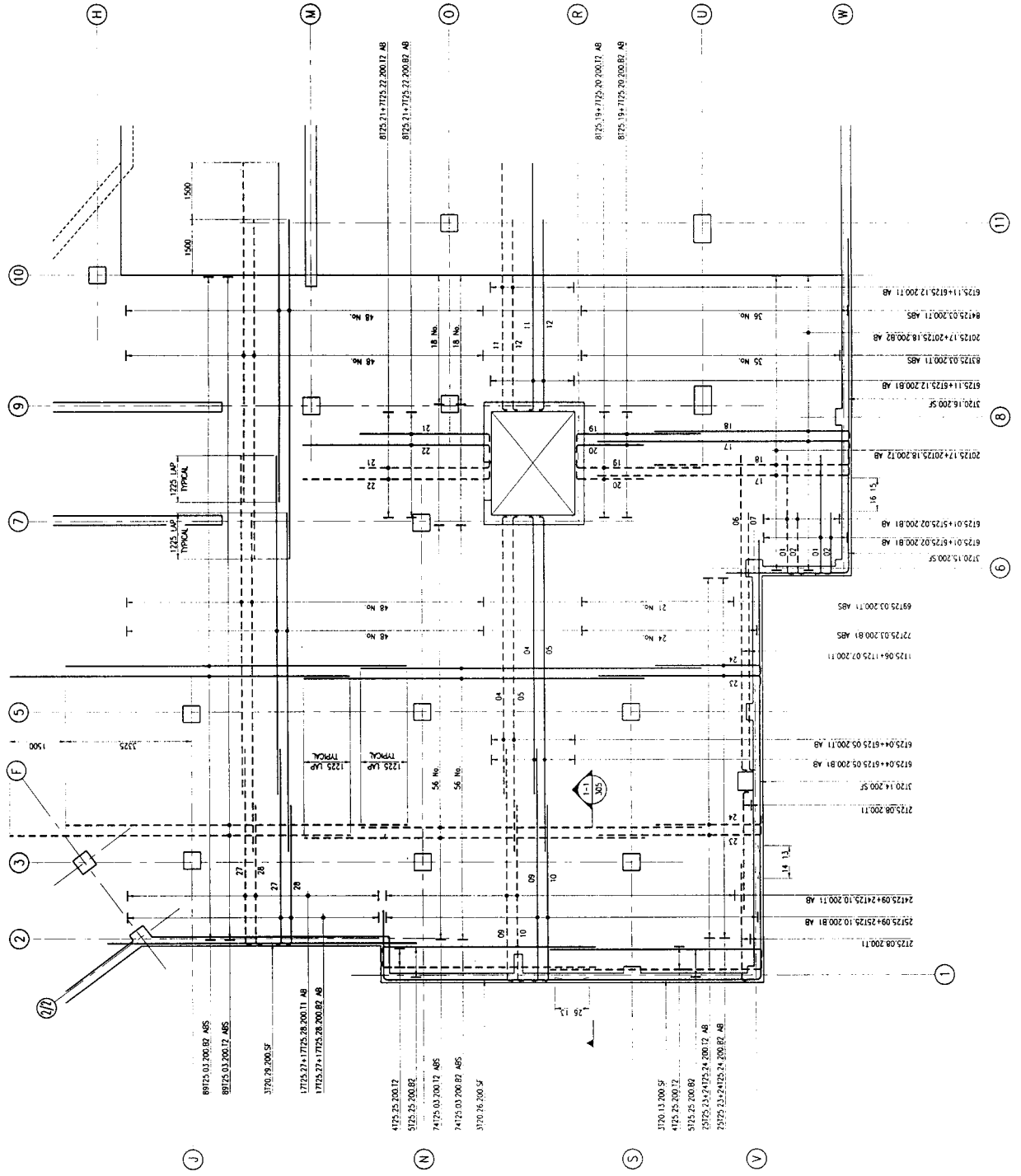
no.	date	description	CHK.
0	19.1.99	ISSUED FOR CONSTRUCTION	

WARD COLE
 CONSULTING ENGINEERS
 101 Oldfield Road, Oldfield, Exeter, Devon, EX4 4JF
 Tel: 01392 744330 Fax: 01392 744332
 Email: info@wardcole.co.uk

Drawing title: **Basement Roof Reinforcement Details**
 Sheet 1 of 6

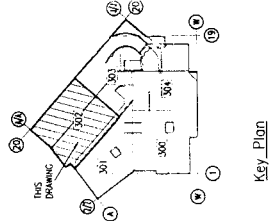
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 Date: 04.1998
 Project: JLN

Job number: 30/386
 Drawing number: 300
 Revision: 0



SHEET NO. VIII.2.3

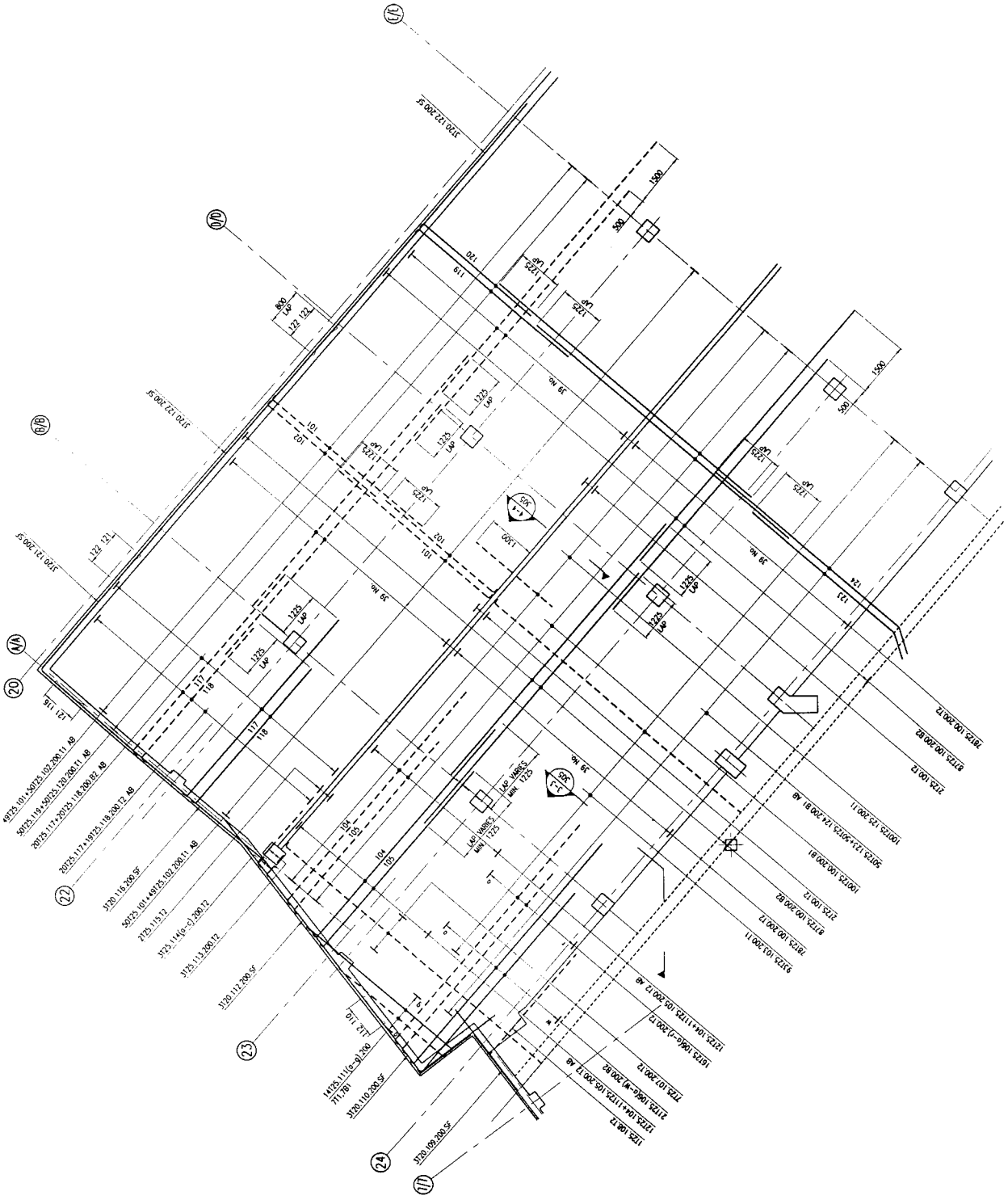
- Notes
1. For general notes refer to Ward Cole drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, and aggregate to be appropriate in accordance with BS 8110.
 4. High yield steel preferred. In an drawing and reinforcement details, reinforcement shall be BS 4443 or BS 4446. All steel specified in drawing and delivery logs to be in accordance with BS 4443.
 5. Concrete cover to reinforcement to be as follows:
 - 100mm top and sides
 - 75mm bottom
 6. Minimum laps to reinforcement to be as follows:
 - 125 - 1275mm
 - 170 - 800mm
 7. Reinforcement is listed on bending schedules 01-02 and.



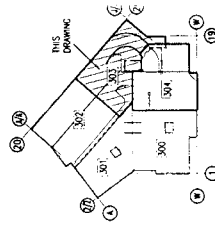
rev.	date	description	auth.
0	19.11.96	ISSUED FOR CONSTRUCTION	

WARD COLE
 structural engineering
 1st & 2nd floors, 67-69, St. James Street, London W1P 8LP
 Tel: 01753 632616 Fax: 01753 632617
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Drawing title		Revision No.	
Basement Raft Reinforcement Details		302	
Sheet 3 of 6		0	
Scale	Date	PC	LN
1:50	Oct 1998		
Job number	Drawing number		
30/386	302		



- Notes
1. General notes refer to Ward Code drawing number 30/386/23100.
 2. This drawing must be read in conjunction with the drawings and details of the structure and architect.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 4. High yield steel, unless noted otherwise, to be in accordance with BS 4449 or BS 4483. High yield steel preferred. Reinforcement bars to be in accordance with BS 4449.
 5. Composite cover to reinforcement to be as follows:
 40mm top and sides
 75mm bottom
 6. Minimum laps to reinforcement to be as follows:
 125 - 1275mm
 120 - 800mm
 7. Reinforcement is listed on bending schedules.

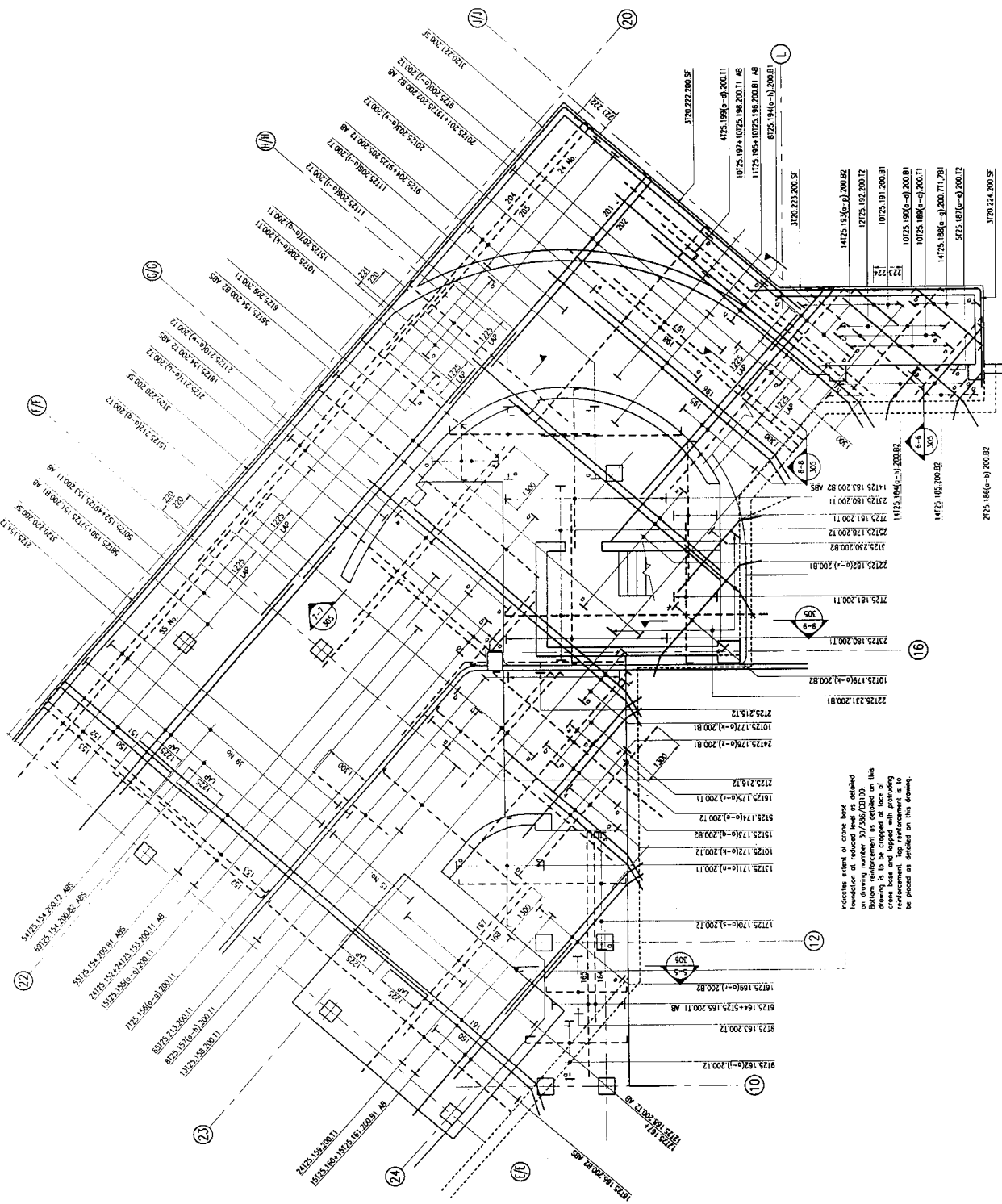


Key Plan

REV	DATE	ISSUED FOR CONSTRUCTION	DESCRIPTION	CHK
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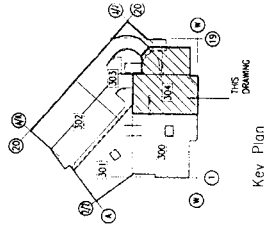
WARD :: COLE
 CONSULTING ENGINEERS
 153 Bridge Road, Singapore, 109551
 Tel: 65-434-3535 Fax: 65-434-3532
 E-mail: ward@wardcole.com.sg

Project Name	Basement Raft Reinforcement Details		
Sheet No.	Sheet 4 of 6		
Scale	1:50	Revision No.	0
Client	Oct 1998	Project No.	303
Drawn By	PC	Checked By	LM
Issue No.	30/368	Revision	0



SHEET NO. VIII.2.5

- NOTES
1. For general notes, refer to Ward Coe drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 4. High yield steel preformed "T" on drawing and delivery tags, to be in accordance with BS 4449. All reinforcement and steel preformed "R" on drawing and delivery tags to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows:
 Column top and sides 40mm
 Column bottom 50mm
 6. Minimum laps to reinforcement to be as follows:
 125 - 175mm
 120 - 800mm
 7. Reinforcement is listed on bending schedules 01-04 incl.



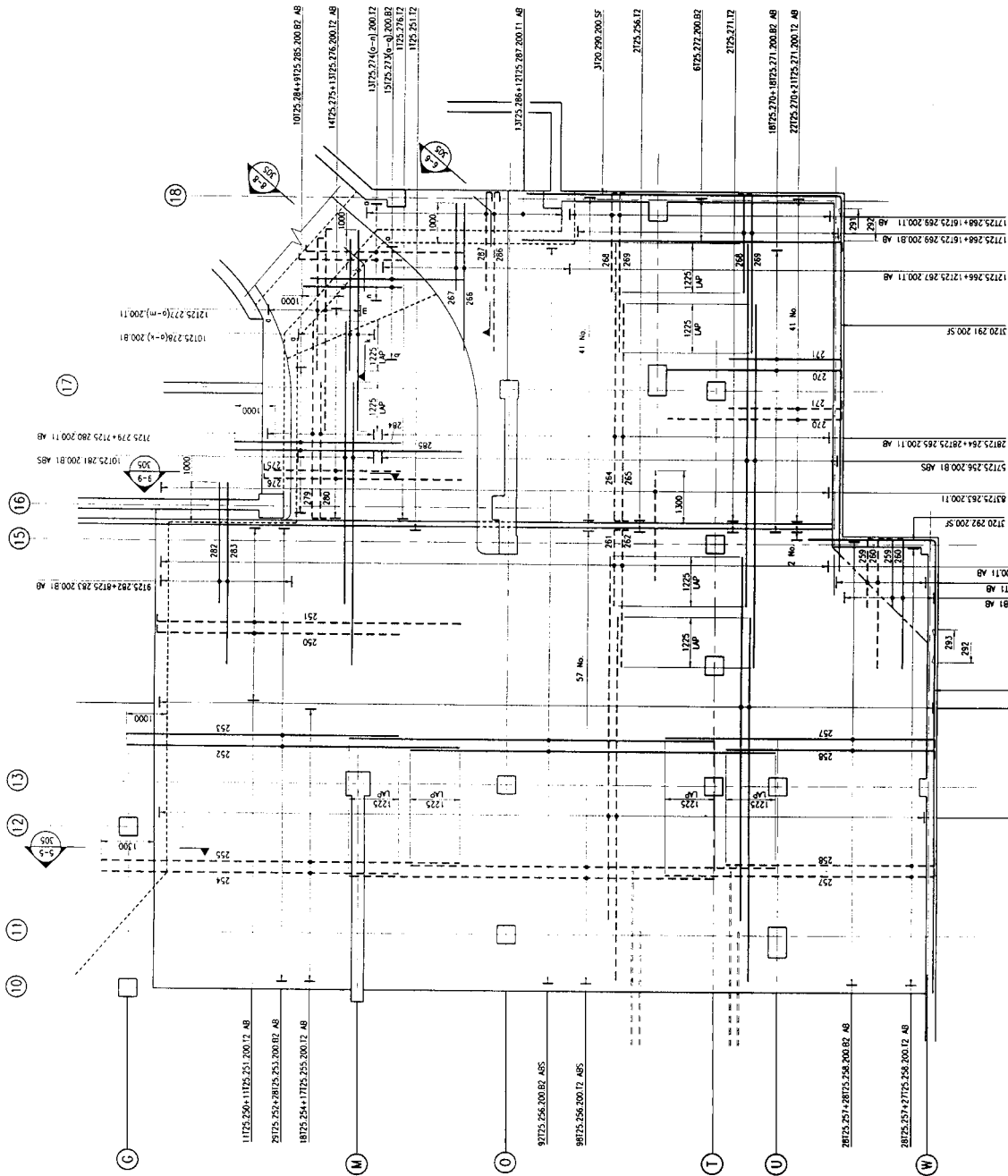
REV	DATE	ISSUED FOR CONSTRUCTION	DESCRIPTION	CHK.
0	18.11.99	ISSUED FOR CONSTRUCTION		

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 CONSULTING ENGINEERS
 150 BRIDGE ROAD, SINGAPORE 105417
 TEL: 3757 4141 FAX: 3757 4432
 E-MAIL: WARD@WARD-COE.COM

Drawing title: **Basement Roof Reinforcement Details**
 Sheet 5 of 6

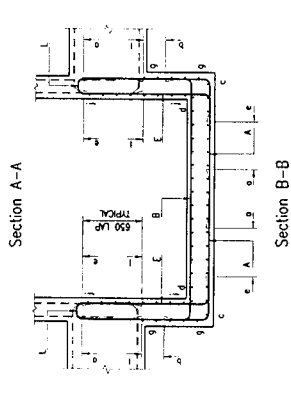
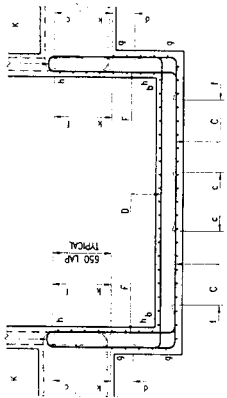
Date: 11/90
 Drawn by: JLM
 Checked by: JLM
 Project: 30/386

REV	DATE	ISSUED FOR CONSTRUCTION	DESCRIPTION	CHK.
0	18.11.99	ISSUED FOR CONSTRUCTION		



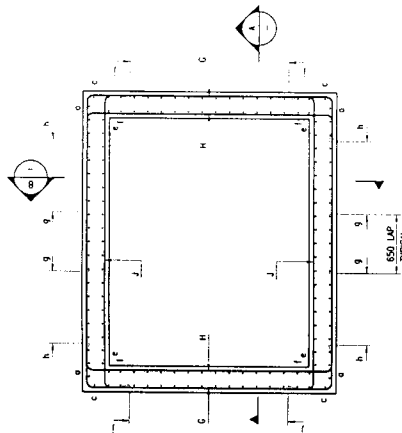
- NOTES:
1. For general notes refer to Ward Cole drawing number 30/386/701
 2. The details must be made in consultation with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40 unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110
 4. High yield steel preferred. 'T' on drawing and delivery tags, to be in accordance with BS 4449. High yield steel preferred 'R' on drawing and delivery tags to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows:
 50mm top and sides
 50mm bottom
 6. Minimum lap to reinforcement to be as follows:
 120 - 300mm
 170 - 300mm
 U.N.C.
 7. Reinforcement is listed on bending schedule D1

	LIFT PIT 1	LIFT PIT 2
A	2-19116.301.150.B1	2-19116.310.150.B1
B	19116.302.150.F1	17116.311.150.F1
C	2-18116.303.150.B2	2-17116.312.150.B2
D	18116.304.150.F2	15116.313.150.F2
E	2-19116.305.150.B1	2-17116.305.150.B1
F	2-18116.305.150.B2	2-15116.305.150.B2
G	2-15116.306.150.F3	2-15116.314.150.F3
H	2-18116.307.150.F3	2-18116.315.150.F3
J	2-18116.308.150.F3	2-18116.316.150.F3
K	2-18116.309.150.B8	2-15116.309.150.B8
L	2-19116.309.150.B8	2-17116.309.150.B8



Section A-A

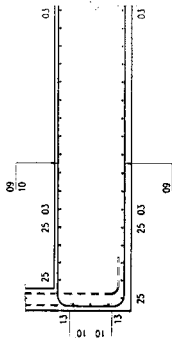
Section B-B



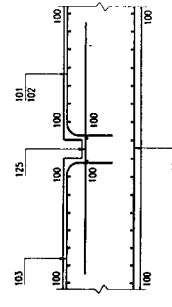
Plan on Lift Pit 1

(Typical - Refer to Key Plan)

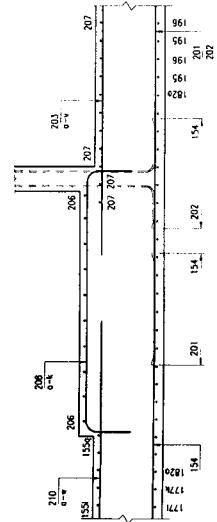
COVER TO REINFORCEMENT TO BE 40mm EXTERNALLY & 30mm INTERNALLY.



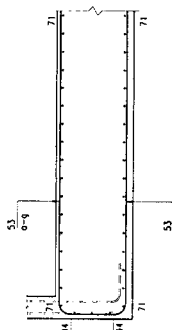
Section 1-1 (Refer to drg. no. 300)



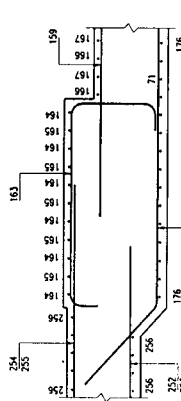
Section 4-4 (Refer to drg. no. 302)



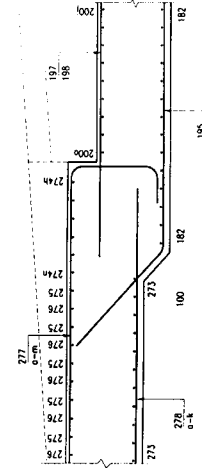
Section 7-7 (Refer to drg. no. 303)



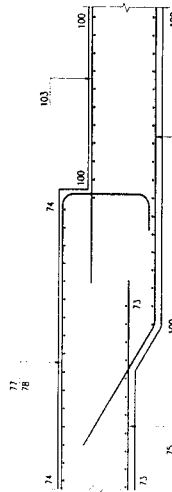
Section 2-2 (Refer to drg. no. 301)



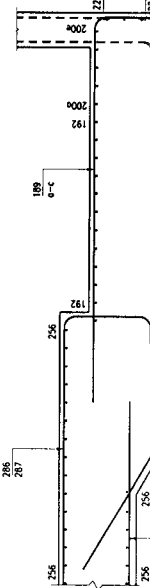
Section 5-5 (Refer to drg. no. 303 & 304)



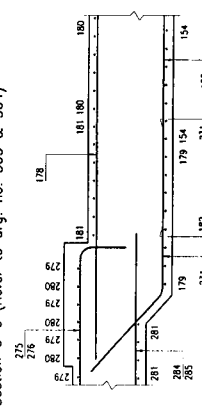
Section 8-8 (Refer to drg. no. 303 & 304)



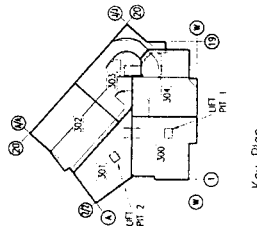
Section 3-3 (Refer to drg. no. 301 & 302)



Section 6-6 (Refer to drg. no. 303 & 304)



Section 9-9 (Refer to drg. no. 303 & 304)



Key Plan

ISSUED FOR CONSTRUCTION

DATE: 01/19/98

DESCRIPTION: BASEMENT RAFT REINFORCEMENT SHEET 6 OF 6

WALD COLE CONSULTING ENGINEERS

100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120, 121, 122, 123, 124, 125, 126, 127, 128, 129, 130, 131, 132, 133, 134, 135, 136, 137, 138, 139, 140, 141, 142, 143, 144, 145, 146, 147, 148, 149, 150, 151, 152, 153, 154, 155, 156, 157, 158, 159, 160, 161, 162, 163, 164, 165, 166, 167, 168, 169, 170, 171, 172, 173, 174, 175, 176, 177, 178, 179, 180, 181, 182, 183, 184, 185, 186, 187, 188, 189, 190, 191, 192, 193, 194, 195, 196, 197, 198, 199, 200, 201, 202, 203, 204, 205, 206, 207, 208, 209, 210, 211, 212, 213, 214, 215, 216, 217, 218, 219, 220, 221, 222, 223, 224, 225, 226, 227, 228, 229, 230, 231, 232, 233, 234, 235, 236, 237, 238, 239, 240, 241, 242, 243, 244, 245, 246, 247, 248, 249, 250, 251, 252, 253, 254, 255, 256, 257, 258, 259, 260, 261, 262, 263, 264, 265, 266, 267, 268, 269, 270, 271, 272, 273, 274, 275, 276, 277, 278, 279, 280, 281, 282, 283, 284, 285, 286, 287, 288, 289, 290, 291, 292, 293, 294, 295, 296, 297, 298, 299, 300, 301, 302, 303, 304, 305, 306, 307, 308, 309, 310, 311, 312, 313, 314, 315, 316, 317, 318, 319, 320, 321, 322, 323, 324, 325, 326, 327, 328, 329, 330, 331, 332, 333, 334, 335, 336, 337, 338, 339, 340, 341, 342, 343, 344, 345, 346, 347, 348, 349, 350, 351, 352, 353, 354, 355, 356, 357, 358, 359, 360, 361, 362, 363, 364, 365, 366, 367, 368, 369, 370, 371, 372, 373, 374, 375, 376, 377, 378, 379, 380, 381, 382, 383, 384, 385, 386, 387, 388, 389, 390, 391, 392, 393, 394, 395, 396, 397, 398, 399, 400, 401, 402, 403, 404, 405, 406, 407, 408, 409, 410, 411, 412, 413, 414, 415, 416, 417, 418, 419, 420, 421, 422, 423, 424, 425, 426, 427, 428, 429, 430, 431, 432, 433, 434, 435, 436, 437, 438, 439, 440, 441, 442, 443, 444, 445, 446, 447, 448, 449, 450, 451, 452, 453, 454, 455, 456, 457, 458, 459, 460, 461, 462, 463, 464, 465, 466, 467, 468, 469, 470, 471, 472, 473, 474, 475, 476, 477, 478, 479, 480, 481, 482, 483, 484, 485, 486, 487, 488, 489, 490, 491, 492, 493, 494, 495, 496, 497, 498, 499, 500, 501, 502, 503, 504, 505, 506, 507, 508, 509, 510, 511, 512, 513, 514, 515, 516, 517, 518, 519, 520, 521, 522, 523, 524, 525, 526, 527, 528, 529, 530, 531, 532, 533, 534, 535, 536, 537, 538, 539, 540, 541, 542, 543, 544, 545, 546, 547, 548, 549, 550, 551, 552, 553, 554, 555, 556, 557, 558, 559, 560, 561, 562, 563, 564, 565, 566, 567, 568, 569, 570, 571, 572, 573, 574, 575, 576, 577, 578, 579, 580, 581, 582, 583, 584, 585, 586, 587, 588, 589, 590, 591, 592, 593, 594, 595, 596, 597, 598, 599, 600, 601, 602, 603, 604, 605, 606, 607, 608, 609, 610, 611, 612, 613, 614, 615, 616, 617, 618, 619, 620, 621, 622, 623, 624, 625, 626, 627, 628, 629, 630, 631, 632, 633, 634, 635, 636, 637, 638, 639, 640, 641, 642, 643, 644, 645, 646, 647, 648, 649, 650, 651, 652, 653, 654, 655, 656, 657, 658, 659, 660, 661, 662, 663, 664, 665, 666, 667, 668, 669, 670, 671, 672, 673, 674, 675, 676, 677, 678, 679, 680, 681, 682, 683, 684, 685, 686, 687, 688, 689, 690, 691, 692, 693, 694, 695, 696, 697, 698, 699, 700, 701, 702, 703, 704, 705, 706, 707, 708, 709, 710, 711, 712, 713, 714, 715, 716, 717, 718, 719, 720, 721, 722, 723, 724, 725, 726, 727, 728, 729, 730, 731, 732, 733, 734, 735, 736, 737, 738, 739, 740, 741, 742, 743, 744, 745, 746, 747, 748, 749, 750, 751, 752, 753, 754, 755, 756, 757, 758, 759, 760, 761, 762, 763, 764, 765, 766, 767, 768, 769, 770, 771, 772, 773, 774, 775, 776, 777, 778, 779, 780, 781, 782, 783, 784, 785, 786, 787, 788, 789, 790, 791, 792, 793, 794, 795, 796, 797, 798, 799, 800, 801, 802, 803, 804, 805, 806, 807, 808, 809, 810, 811, 812, 813, 814, 815, 816, 817, 818, 819, 820, 821, 822, 823, 824, 825, 826, 827, 828, 829, 830, 831, 832, 833, 834, 835, 836, 837, 838, 839, 840, 841, 842, 843, 844, 845, 846, 847, 848, 849, 850, 851, 852, 853, 854, 855, 856, 857, 858, 859, 860, 861, 862, 863, 864, 865, 866, 867, 868, 869, 870, 871, 872, 873, 874, 875, 876, 877, 878, 879, 880, 881, 882, 883, 884, 885, 886, 887, 888, 889, 890, 891, 892, 893, 894, 895, 896, 897, 898, 899, 900, 901, 902, 903, 904, 905, 906, 907, 908, 909, 910, 911, 912, 913, 914, 915, 916, 917, 918, 919, 920, 921, 922, 923, 924, 925, 926, 927, 928, 929, 930, 931, 932, 933, 934, 935, 936, 937, 938, 939, 940, 941, 942, 943, 944, 945, 946, 947, 948, 949, 950, 951, 952, 953, 954, 955, 956, 957, 958, 959, 960, 961, 962, 963, 964, 965, 966, 967, 968, 969, 970, 971, 972, 973, 974, 975, 976, 977, 978, 979, 980, 981, 982, 983, 984, 985, 986, 987, 988, 989, 990, 991, 992, 993, 994, 995, 996, 997, 998, 999, 1000

Basement Raft Reinforcement Details Sheet 6 of 6

DATE: 01/19/98

CHECKED BY: PC

DRAWING NUMBER: 30/386

PROJECT NUMBER: 305

SHEET NO. VIII.2.7

- NOTES**
1. For general notes refer to Ward Cole drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max. size aggregate in accordance with BS 5110
 4. High yield steel rebar: "1" on drawing and BS 4449 or BS 4451. 1000 stress pattern "R" on drawing and delivery tags to be in accordance with BS 4443
 5. Concrete cover to reinforcement to be as follows:
Walls - 30mm to all steel
Floors:
112 - 40mm
117 - 30mm
120 - 30mm
125 - 80mm
 6. Minimum laps to reinforcement to be as follows:
112 - 400mm
117 - 300mm
120 - 300mm
125 - 800mm

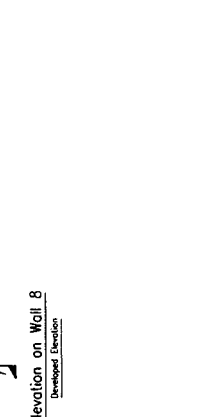
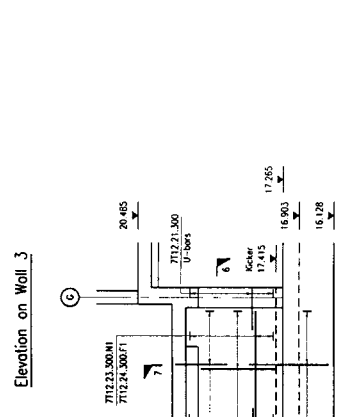
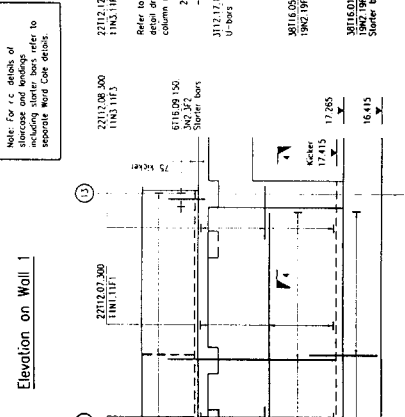
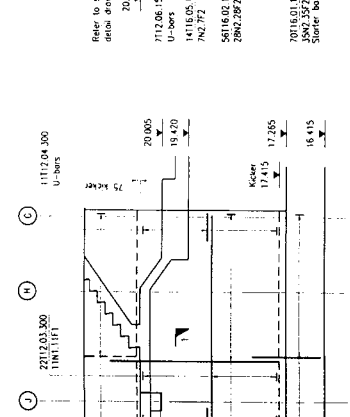
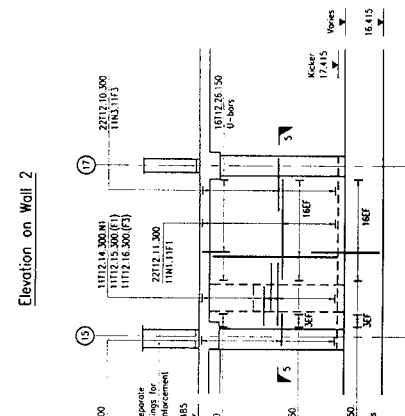
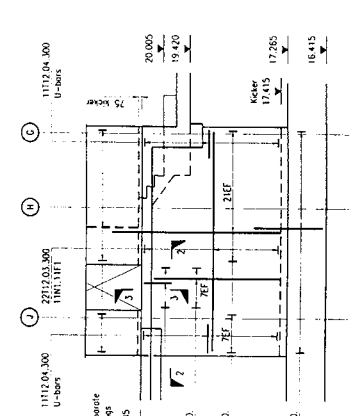
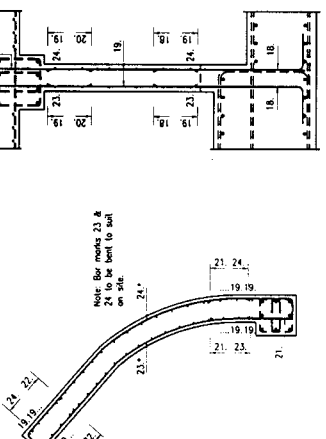
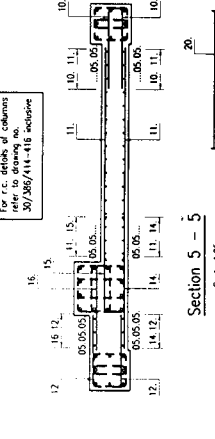
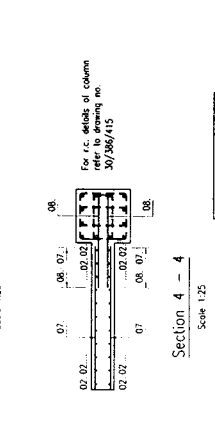
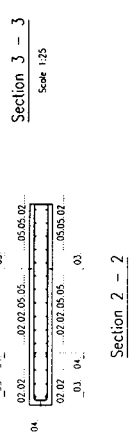
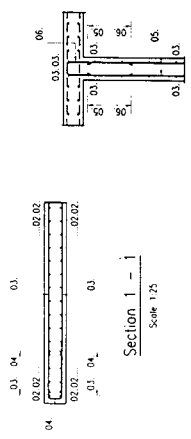
NOTES
Concrete to penetrate basement retaining walls and basement slab to a depth as indicated on relevant drawings shall be in accordance with the requirements of EVERBURE CALITE SYSTEM products including CALITE at the rate of 30 litres per cubic metre of concrete. The use of any other product is recommended. The manufacturer's recommendations shall be followed. EVERBURE CALITE concrete shall be not less than 20kg per cubic metre. The water content shall be controlled to ensure the concrete is workable, however the water/cement ratio must not exceed 0.45. The curing regime shall be prior to construction and cement content is to be stated on the prime concrete docket.

For location of walls refer to wall and column key plan on drawing no. 30/286/212

NO.	DATE	DESCRIPTION	CHKD.
1	05.1.98	Minor revision	
2	26.2.99	Revised DWF CONSTRUCTION	

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 CONSULTING ENGINEERS
 1st Floor, 200, St. Andrew's House,
 11, Cross Street, Manchester, M2 1AB
 Tel: 0161 442 3535 Fax: 0161 442 3537
 E-mail: ward@wardcole.co.uk

Drawing Title	R.C. Details of Walls
Project	Basement to Ground Floor
Sheet	Sheet 1 of 3
Date	15.01.25 Feb 1999
Drawn By	KJ
Checked By	
Scale	
Drawing number	440
Revision	1



- NOTES:**
- For general notes refer to Ward Cole drawing number 30/386/201
 - This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 - Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate as appropriate with BS 5110.
 - High yield steel rebar: T on drawing and BS 4449 or BS 4449-1. Mild steel rebar: R on drawing and delivery logs to be in accordance with BS 4449.
 - Concrete cover to reinforcement to be as follows:
Walls - 20mm to all steel
Minimum laps to reinforcement to be as follows:
T12 - 450mm
T16 - 750mm
T25 - 875mm
 - NOTES:** Details to perimeter basement reflecting walls and basement slab for a depth as indicated on relevant drawings shall be in accordance with the relevant manufacturer's instructions for the EVERBURE CALITE SYSTEM products including CALITE at the rate of 30 litres per cubic metre and SUPERBOND or the manufacturer's EVERBURE CALITE concrete shall be not less than 35kg per cubic metre. The water content shall be controlled to ensure adequate workability, however the water/cement ratio must not exceed 0.5. The water content shall be stated on the construction and cement content is to be stated on the pre-mix concrete tickets.

For location of walls refer to wall and column key plan on drawing no. 30/386/212

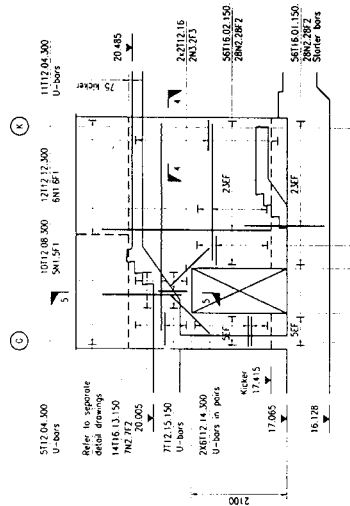
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- Notes:
1. For general notes refer to Ward Cole drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40 unless noted otherwise. Reinforcement to be appropriate in accordance with BS 8110
 4. High yield steel rebar: 7 on drawing and 8 on detail. Minimum lap length shall be BS 4449 or BS 4446. All steel profiles R on drawing and delivery tags to be in accordance with BS 4446
 5. Concrete cover to reinforcement to be as follows:
Walls - 30mm to all filed
Minimum laps to reinforcement to be as follows:
T12 - 450mm
T16 - 500mm
T20 - 700mm
T25 - 875mm
 6. Minimum laps to reinforcement to be as follows:
Walls - 30mm to all filed
Minimum laps to reinforcement to be as follows:
T12 - 450mm
T16 - 500mm
T20 - 700mm
T25 - 875mm
- NOTE: All details to perimeter basement retaining walls and basement slab for a depth as indicated on relevant drawings shall be PERFORMED CALITE SYSTEM dependent on recommended rate. The cement content of PERFORMED CALITE concrete shall be not less than 300 kg per cubic metre. The PERFORMED CALITE content shall be as per the manufacturer's instructions and maintain the required workability. Before the water/cement ratio must be determined by testing. All details prior to construction and cement content is to be stated on the precast concrete dockets.

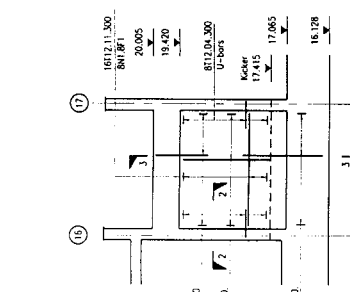
For location of walls refer to wall and column drawings no. 30/386/212

1	05.1.99	Opening Added
0	28.1.99	Bending Schedule revised
ISSUED FOR CONSTRUCTION		
Rev.	Date	Description
<p>WARD :: COLE CONSULTING ENGINEERS 100, CHIFFINDELL ROAD, CHIFFINDELL, WIMBORNE, DORSET, BH20 9JL TEL: 01202 831111 FAX: 01202 831112 E-MAIL: info@wardcole.co.uk</p>		

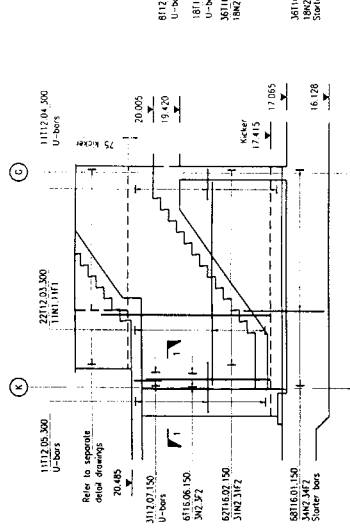
Drawing Title	R.C. Details of Walls Basement to Ground Floor
Sheet	Sheet 3 of 3
Date	1:50 1.25 Feb 1999
Drawn by	KT
Checked by	
Job Number	30/386
Drawing Number	442
Revision	1



Elevation on Wall 7



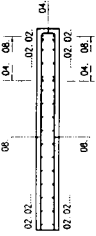
Elevation on Wall 6



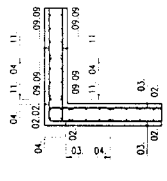
Elevation on Wall 5

For r.c. details of column refer to drawing no. 30/386/213

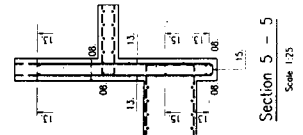
Note: For r.c. details of stirrups and lapings refer to separate Ward Cole details



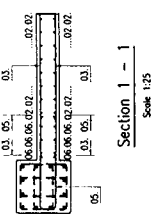
Section 4 - 4
Scale 1:25



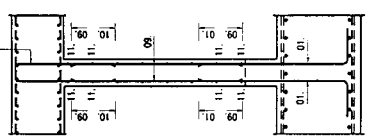
Section 2 - 2
Scale 1:25



Section 5 - 5
Scale 1:25

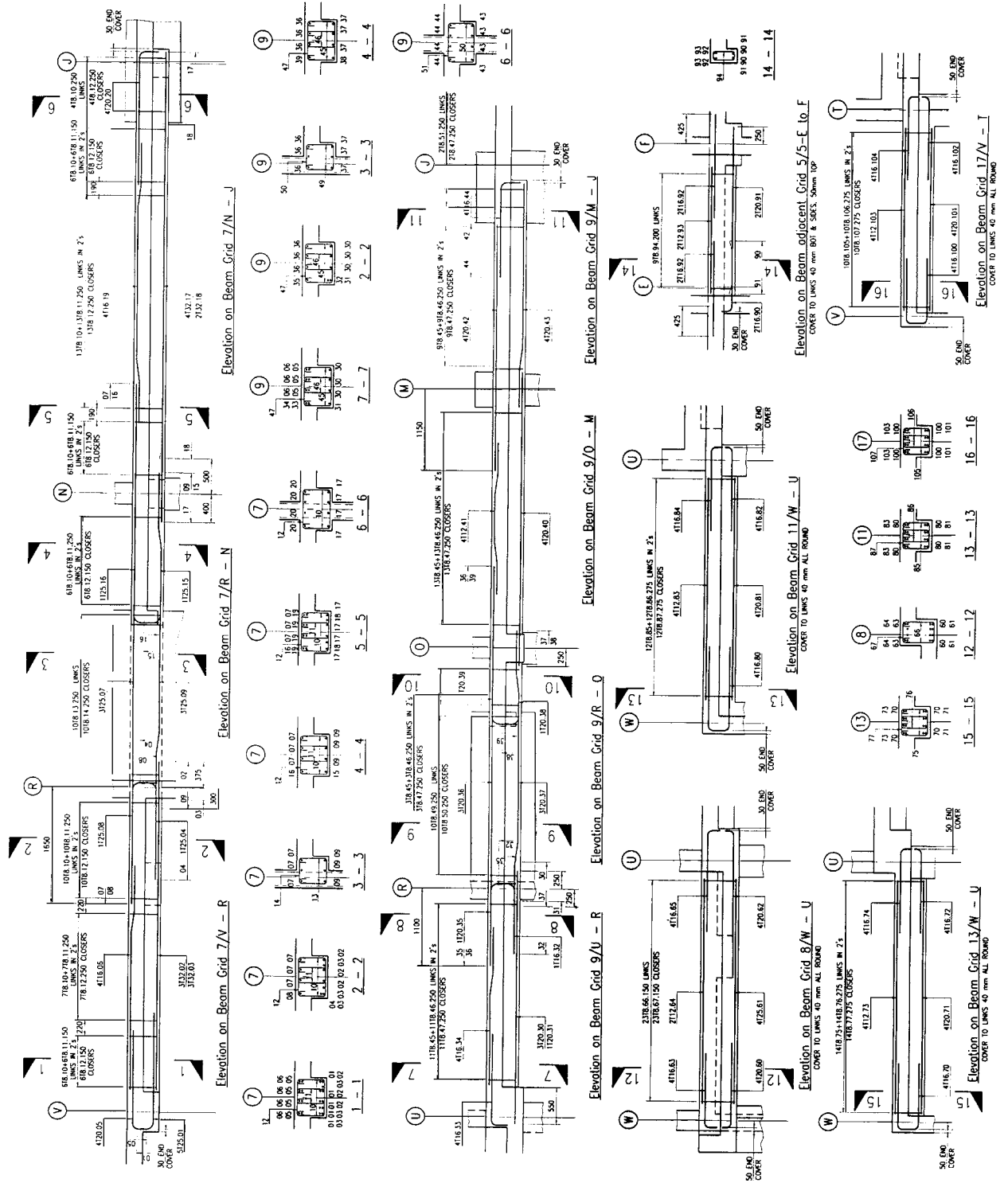


Section 1 - 1
Scale 1:25



Section 3 - 3
Scale 1:25

- Notes:
1. For general notes refer to Word Call drawing number 30/386/702
 2. This drawing must be read in conjunction with the Structural Design and Architects drawings and details.
 3. Concrete to be Grade C40, where noted otherwise in the drawing.
 4. High yield steel reinforcement, in drawing and primary logs, to be in accordance with BS 4449 or BS 4449. Mild steel perforated 'R' reinforcement to be in accordance with BS 4449.
 5. Reinforcement to be as per drawing (L.A.C.)
 6. Reinforcement in listed on bending schedules 01-04 incl.

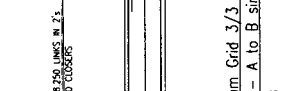
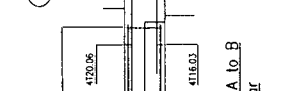
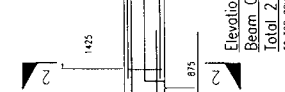
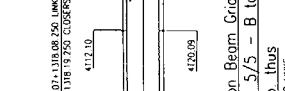
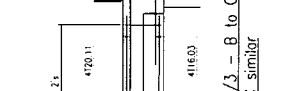
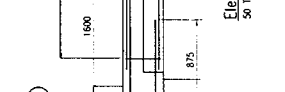
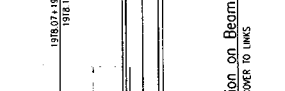
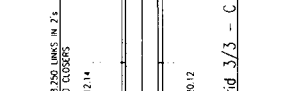
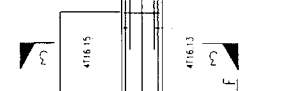
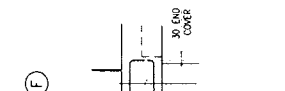
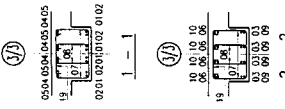


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SHEET NO. VIII.2.11

- Notes:
- 1 For general notes refer to Ward Code drawing number 30/386/201
 - 2 This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 - 3 Concrete to be Grade C40 unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 - 4 High yield steel prestressed T on drawing and delivery logs to be in accordance with BS 5896 and R on drawing and delivery logs to be in accordance with BS 4449.
 - 5 Concrete cover to reinforcement to be as follows (U.N.O.)
 30mm bottom & sides
 40mm top
 - 6 Reinforcement is listed on bending schedules 01-04 etc.

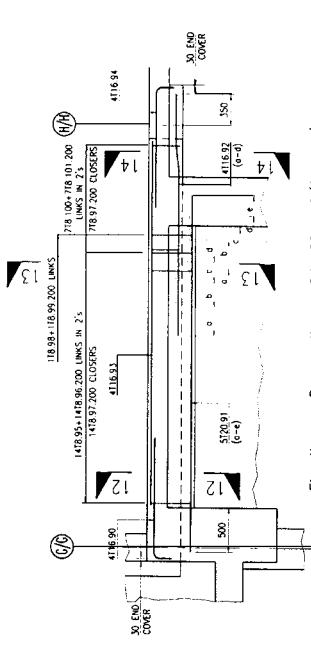


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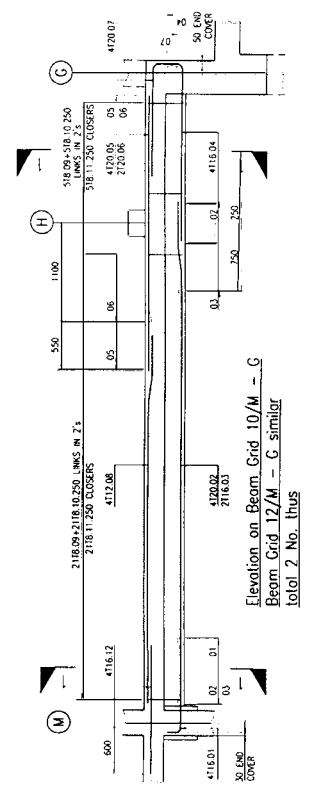
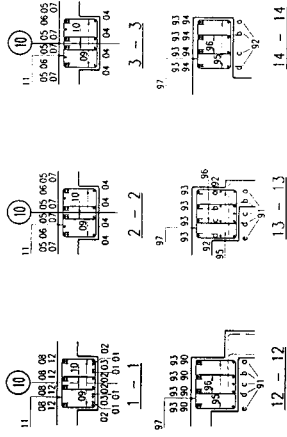
Issued for Construction
 Date: 12/20/2011
 Description: Ground Floor Beams Reinforcement Details

Sheet No.	30/386	Revision	0
Date	Feb 99	Checked by	R.L.
Scale	1:25	Drawn by	P.C.
Project No.	30/386	Quantity	451

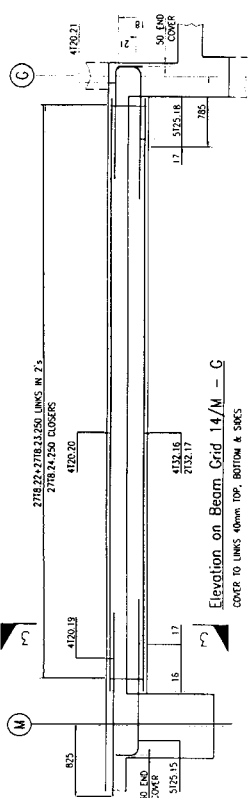
- Notes:**
1. For general notes refer to Work Code drawing number 147/207/201.
 2. This drawing must be read in conjunction with the drawings and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 4. High yield steel reinforced T on ground and delivery logs, to be in accordance with BS 4449 or BS 4449, and high yield steel reinforcement to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows: 40mm for 30mm bottom & sides.
 6. Reinforcement is listed on bending schedules 01-05 incl.



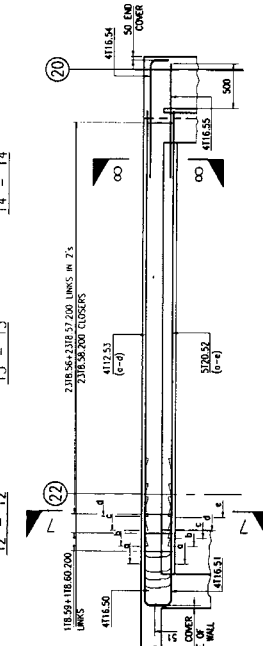
Elevation on Beam adjacent Grid 22 - G/G to H/H
COVER TO LINKS 40mm TOP, BOTTOM & SIDES



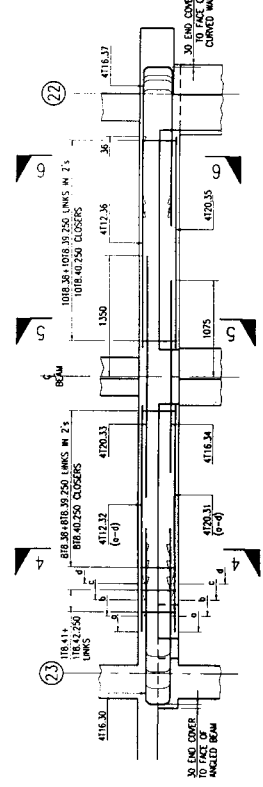
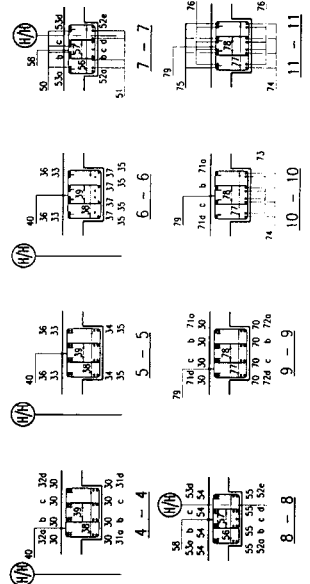
Elevation on Beam Grid 10/M - G
Beam Grid 12/M - G similar
Total 2 No. thus



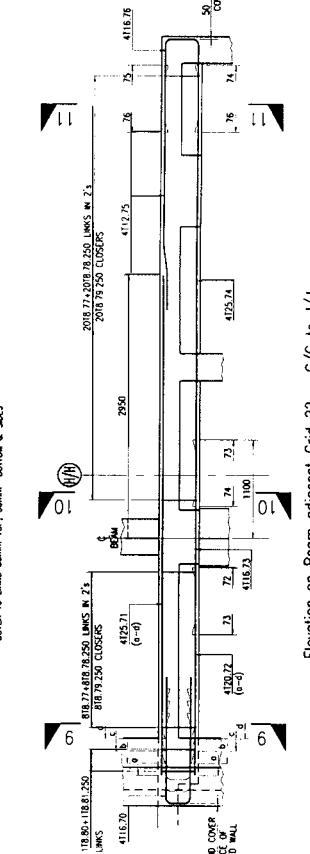
Elevation on Beam Grid 14/M - G
COVER TO LINKS 40mm TOP, BOTTOM & SIDES



Elevation on Beam adjacent Grid H/H - 20 to 22
COVER TO LINKS 30mm TOP, 40mm BOTTOM & SIDES



Elevation on Beam adjacent Grid H/H - 22 to 23
COVER TO LINKS 30mm TOP, 30mm BOTTOM & SIDES



Elevation on Beam adjacent Grid 22 - G/G to J/J

Issued for Construction

WARD COLE

2008-2009

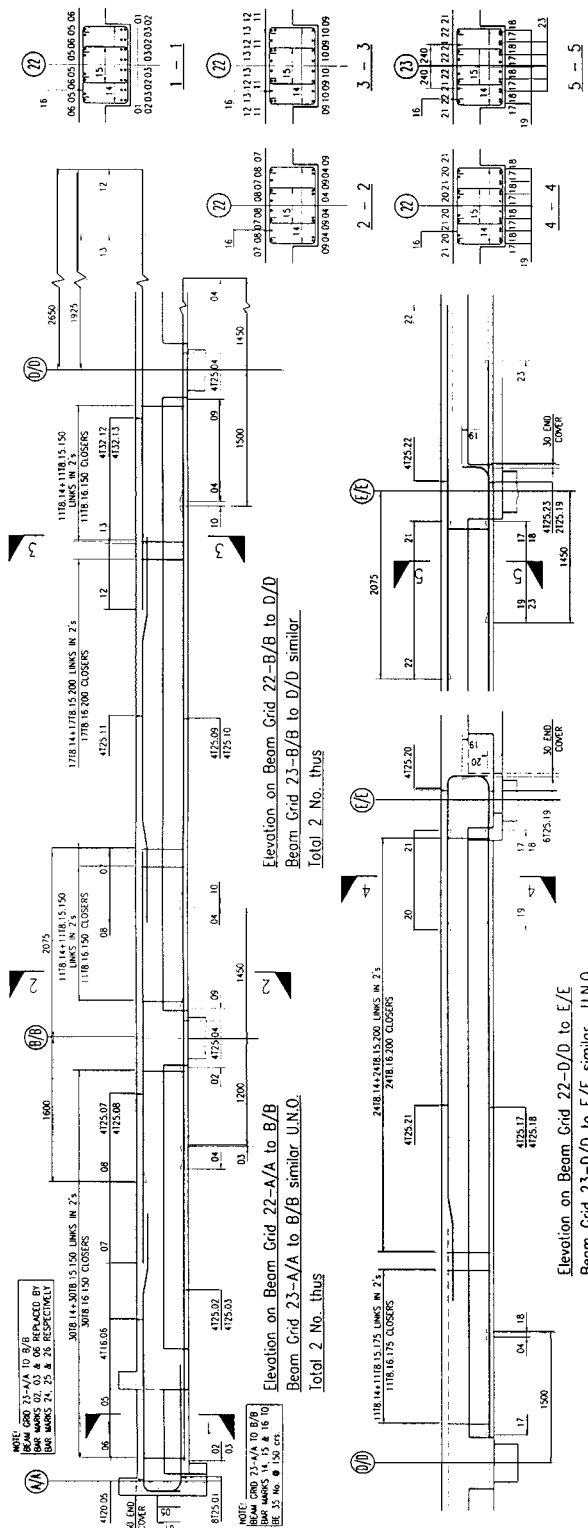
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Rev. date description

Drawing Date	30/3/86	Sheet No.	452
Ground Floor Beams		P.C.	R.T.
Reinforcement Details		Scale	1:25
Sheet 3 of 12		Drawn By	
		Checked By	
		Quantity	0

SHEET NO. VIII.2.13

- Notes:
1. For general notes refer to Word Code drawings number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise. Reinforcement to be appropriate in accordance with BS 8110.
 4. High yield steel preferred. "r" on drawing and "R" on drawings and delivery logs to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as shown (MIN. COV.)
30mm bottom & sides
01-04 and
 6. Reinforcement is listed on bending schedules



REVISIONS AS SHOWN

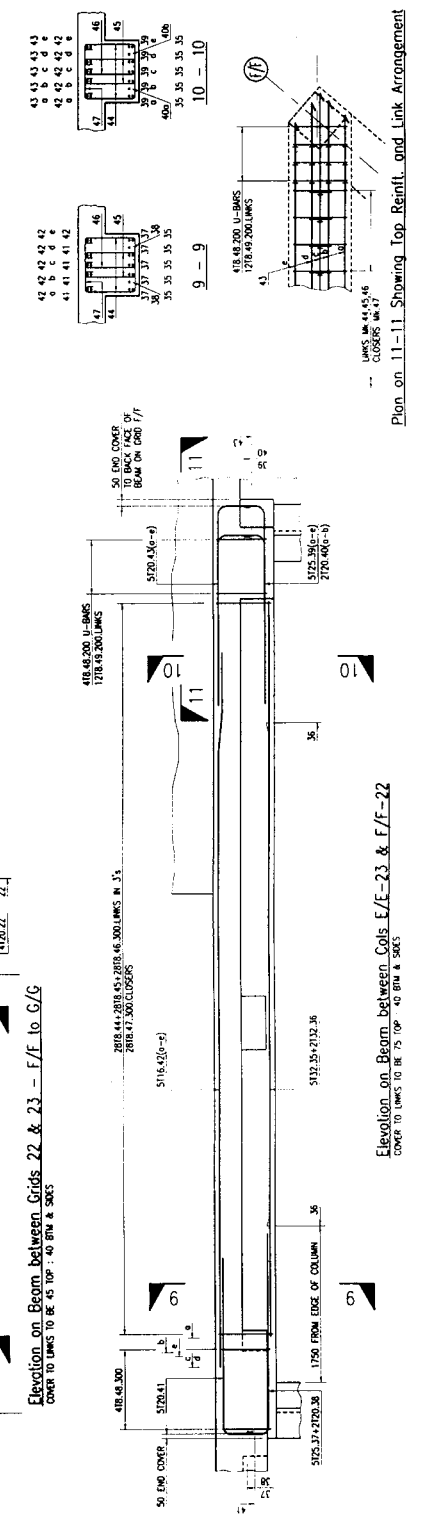
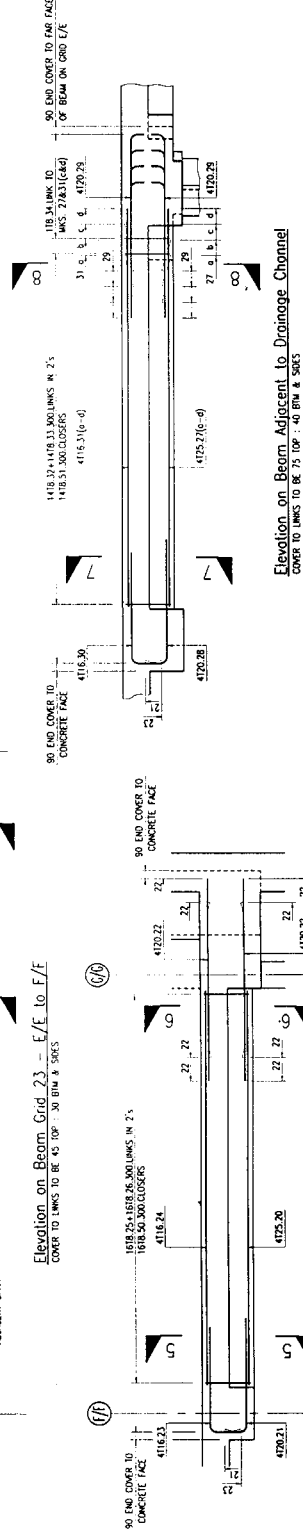
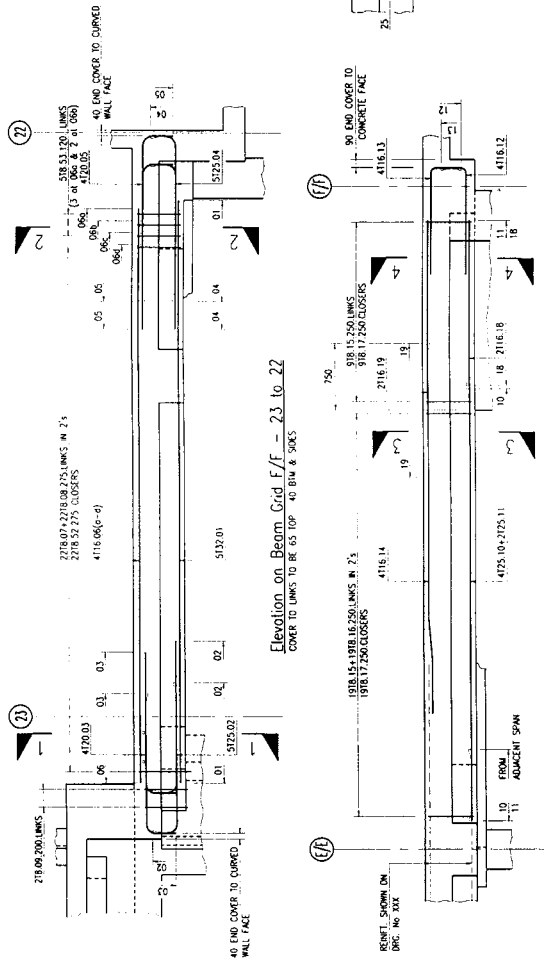
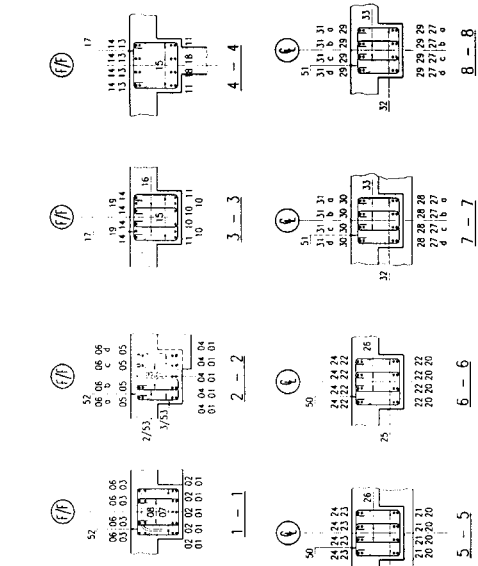
NO.	DATE	DESCRIPTION
1	13.10.19	Revised Schedule Unchanged
2	26.3.19	Issued For Construction

WARD :: COLE
Structural Engineers
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Ground Floor Beams
Reinforcement Details
Sheet 4 of 12

DATE: 1.25
DATE: Feb 99
P.C. D.M.
JOB NUMBER: 30/386
DRAWING NUMBER: 453

- Notes:
1. For general notes refer to Ward Cole drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate at accordance with BS 8110
 4. High yield steel preferred "T" on drawing and BS 4449 S460 in accordance with BS 4449 S460. All reinforcement to be as on drawing and delivery logs to be in accordance with BS 4449
 5. Concrete cover to reinforcement to be as note on the drawing
 6. Reinforcement is listed on bending schedules 01-04 etc.



REV	DATE	ISSUED FOR CONSTRUCTION	DESCRIPTION	CHK
0	26.2.09			

WARD :: COLE
 CONSULTING ENGINEERS
 12, DUNSTON PLACE, STAMFORD
 LEICESTER, LE4 4DQ
 TEL: 0116 274 4000
 FAX: 0116 274 4001
 E-MAIL: LEICESTER@WARD-COLE.CO.UK

Drawing Title: Ground Floor Beams Reinforcement Details
 Sheet 5 of 12

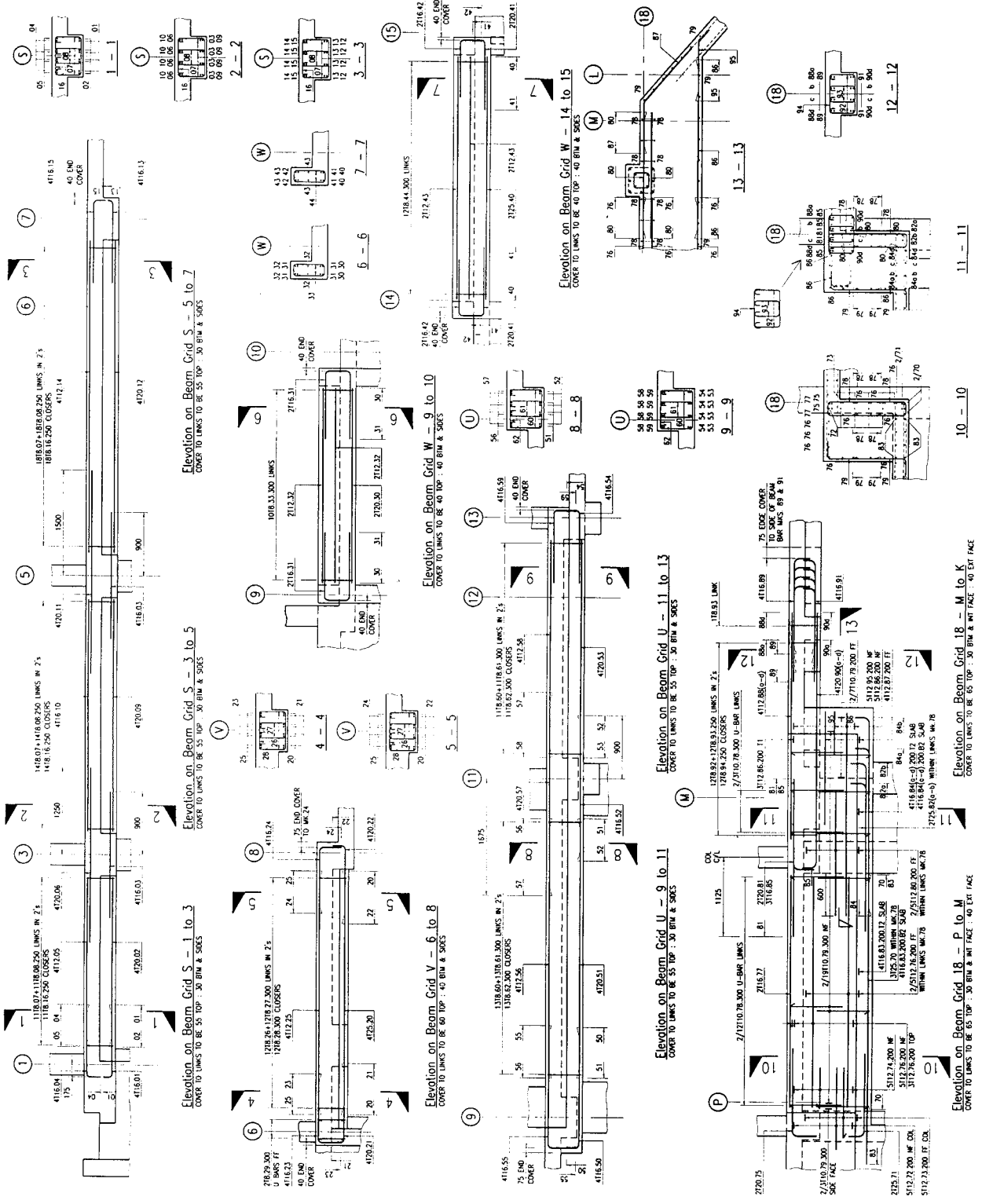
DATE	BY	CHKD BY	NO.
27.2.09	AW	AW	454

- Notes:**
1. For general notes, refer to Ward Code drawing number 37/380/201.
 2. The drawing must be read in conjunction with the drawings and schedules.
 3. Concrete to be Cast, C10, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 4. High yield steel rebar 'T' on drawing and delivery logs, to be in accordance with BS 4449 or BS 4449-1, max size permitted R in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as noted on the drawing.
 6. Reinforcement is listed on bending schedules 01-05-001.

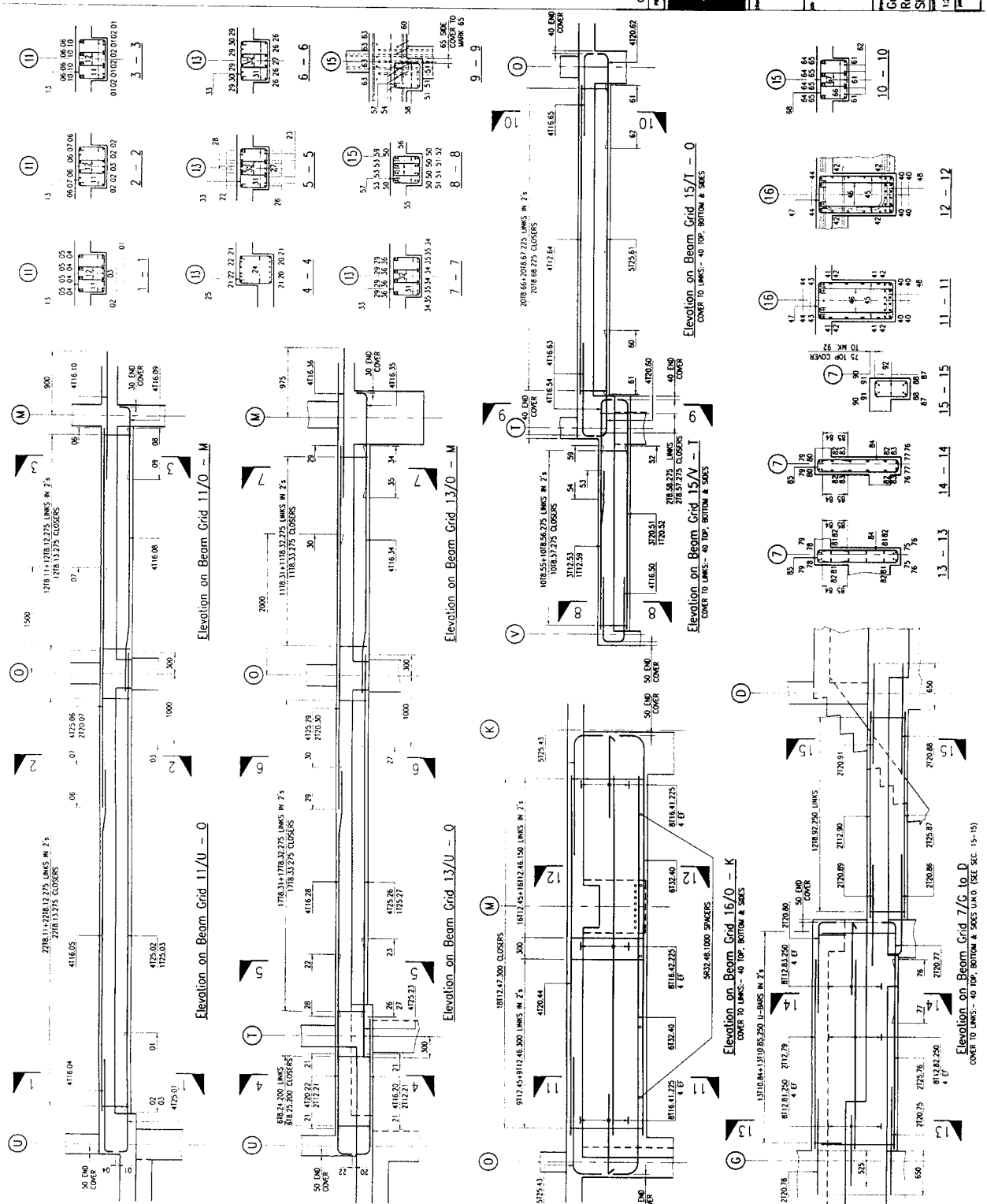
no.	date	description	chk.
1	15.1.99	Beam Details Completed	
2	28.2.99	Bending Schedule Revised	
3	10.10.00	Revised for Construction	

WARD & COLE
 201-110-1300
 201-110-1300
 P. 02003 164114
 F. 015 744 9325 fax 015 744 9322
 6002 19-05 1945566-001

drawing title:		drawn by:	checked by:
Ground Floor Beams		Feb 99	P.C.
Reinforcement Details			A.N.
Sheet 6 of 12			
1:25	30/386	drawing number:	455
		sheet number:	1



- Notes:
1. For general notes refer to Reinforcement Schedule number 30/386/201.
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 4. High yield steel (referred 'T' on drawing and BS 4443) to be used in accordance with BS 8110. All reinforcement to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows (U&O):
 40mm top
 30mm bottom & sides
 6. Reinforcement is listed on bending schedules 01-04-001.



Issued for Construction

Rev.	Date	Description	By
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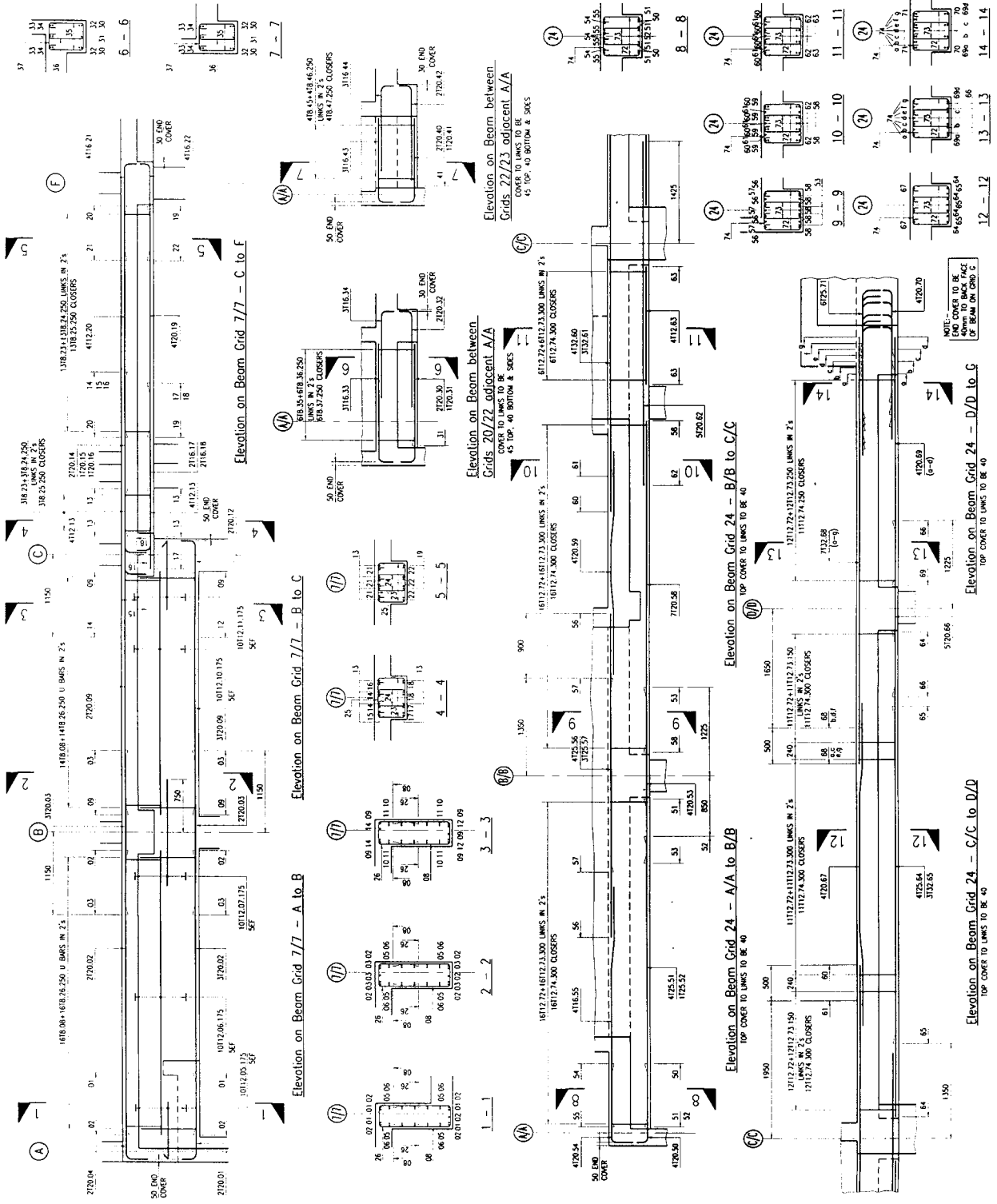
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- Notes:
1. For general notes refer to Main Code drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 5110.
 4. High yield steel rebar 'T' on drawing and drawings to be used in accordance with BS 4449. All bars to be 'T' unless stated 'R' on drawing and delivery logs to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows (U.K.C.):
50mm top
30mm bottom & sides
01-04 end.
 6. Reinforcement is listed on bending schedules 01-04 end.

REV.	DATE	DESCRIPTION	CHK.
1	13.09	Beam Details Completed	
0	26.2.99	Bending Schedules Revised	

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drawing title	Ground Floor Beams Reinforcement Details
sheet	Sheet 9 of 12
date	Feb 99
drawn by	P.C.
checked by	M.W.
proj number	30/386
drawing number	458
revision	1

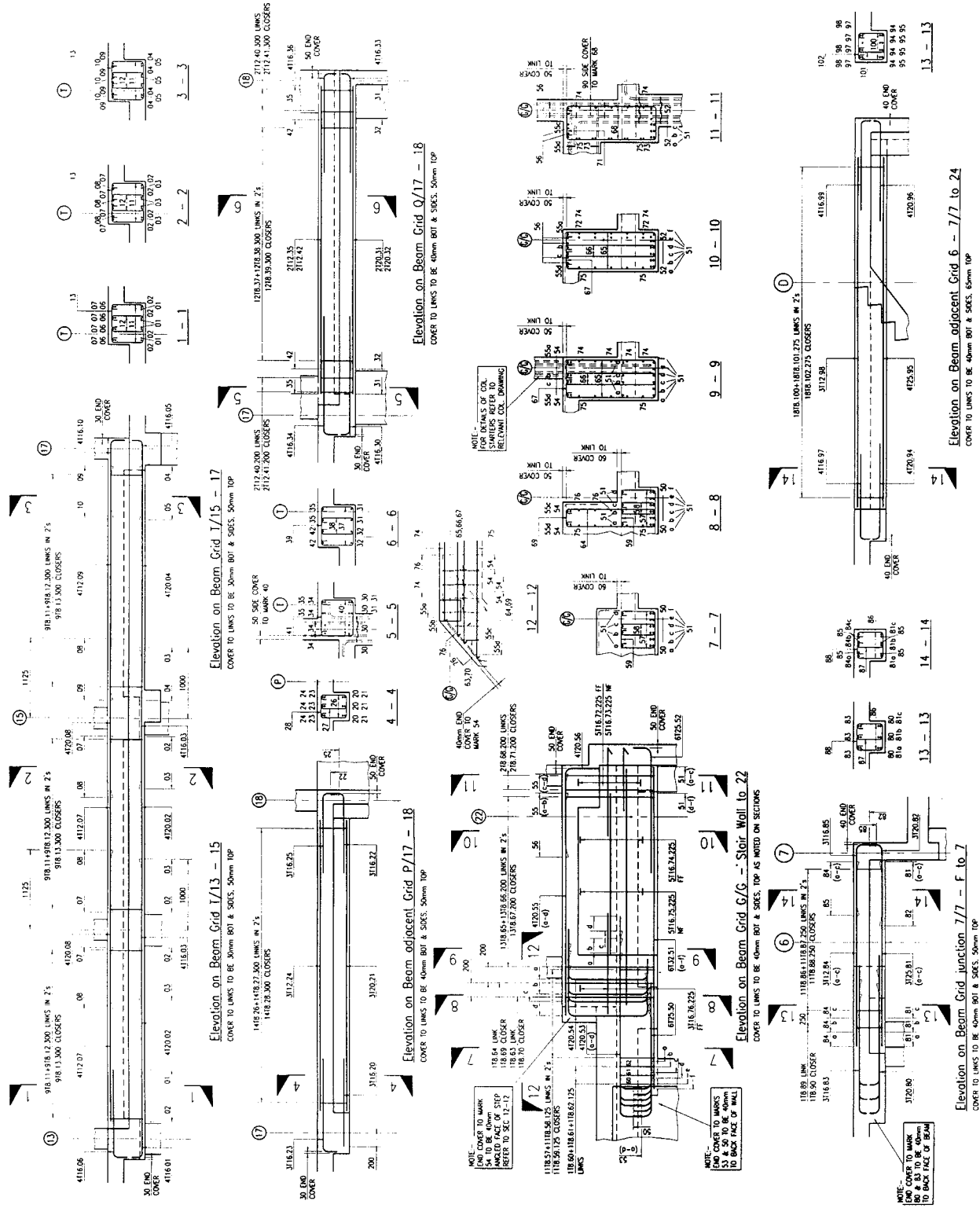


- Notes:
1. For general notes refer to Ward Code drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and sheets.
 3. Concrete to be Code C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8111.
 4. High yield steel preferred. If in drawing and reinforcement is not in accordance with BS 4449 or BS 4446, reinforcement shall be on drawing and delivery logs to fit in accordance with BS 4443.
 5. Concrete cover to reinforcement to be as noted on the drawing.
 6. Reinforcement is listed on bending schedule: 01-04 etc.

0	28.2.98	Issued for Construction
1	10.1.99	Revised
2	10.1.99	Revised
3	10.1.99	Revised
4	10.1.99	Revised
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6	10.1.99	Revised
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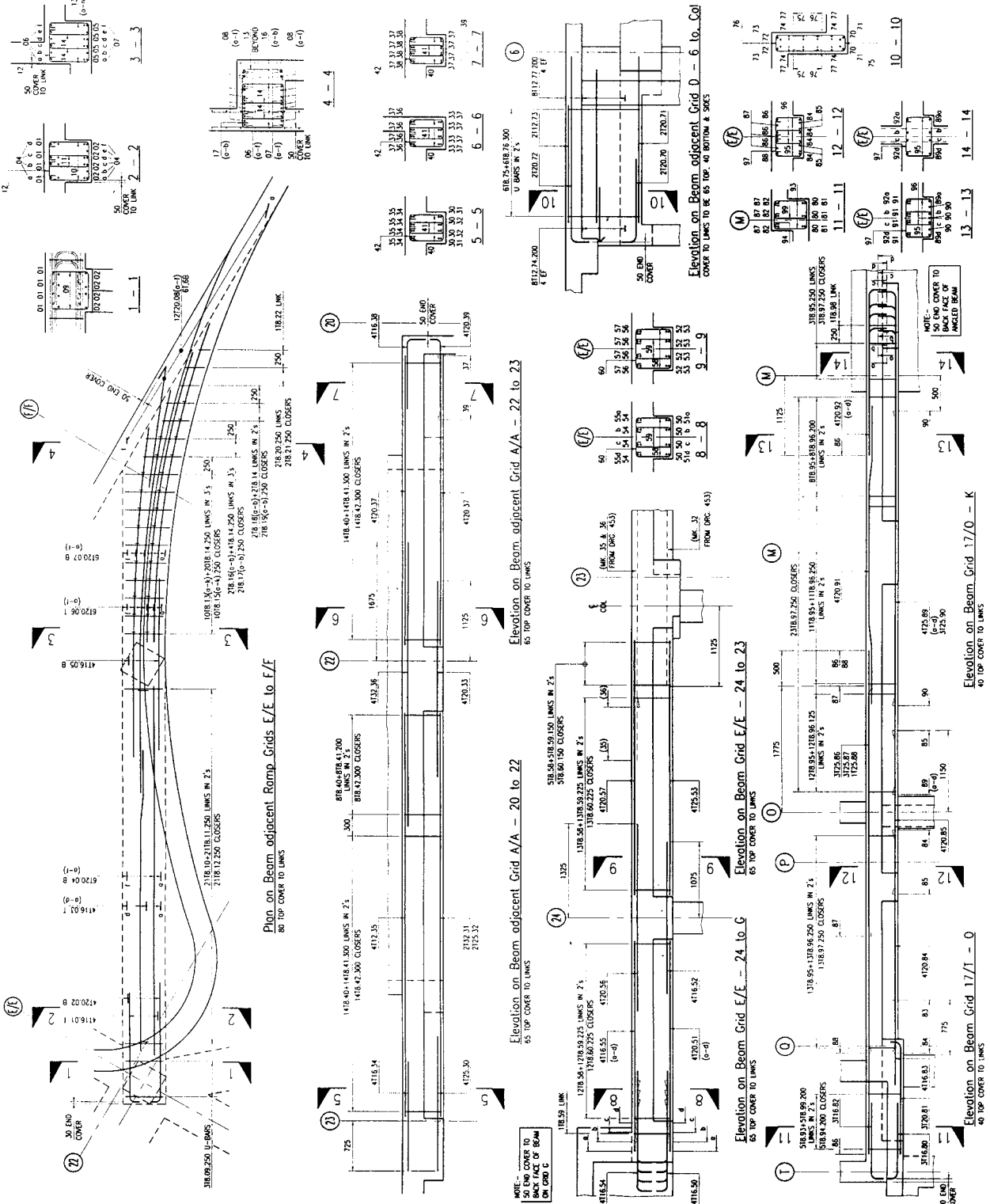
WARD :: COLE
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30/386	460
Ground Floor Beams Reinforcement Details Sheet 11 of 12	
1:25	Feb 99
P.C.	R.T.



SHEET NO. VIII.2.21

- NOTES:**
1. For general notes refer to Base Code drawing number 30/386/201.
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 40mm max size aggregate in accordance with BS 8110.
 4. High yield steel preferred 'T' on drawing and drawings to be in accordance with BS 4449. Reinforcement to be as indicated 'R' on drawing and delivery logs to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows (LINK)
 - a) 50mm
 - b) 25mm
 - c) 20mm
 - d) 15mm
 top as noted on drawing.
 6. Reinforcement is listed on bending schedules BS 4449.



REV.	DATE	DESCRIPTION	CHK.
1	15.1.99	Beam Details Completed	
0	16.2.99	Issued for Construction	

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Project Name: Ground Floor Beams Reinforcement Details
 Sheet 12 of 12
 Date: 1.12.99
 P.C. No: 461
 Drawing Number: 30/386

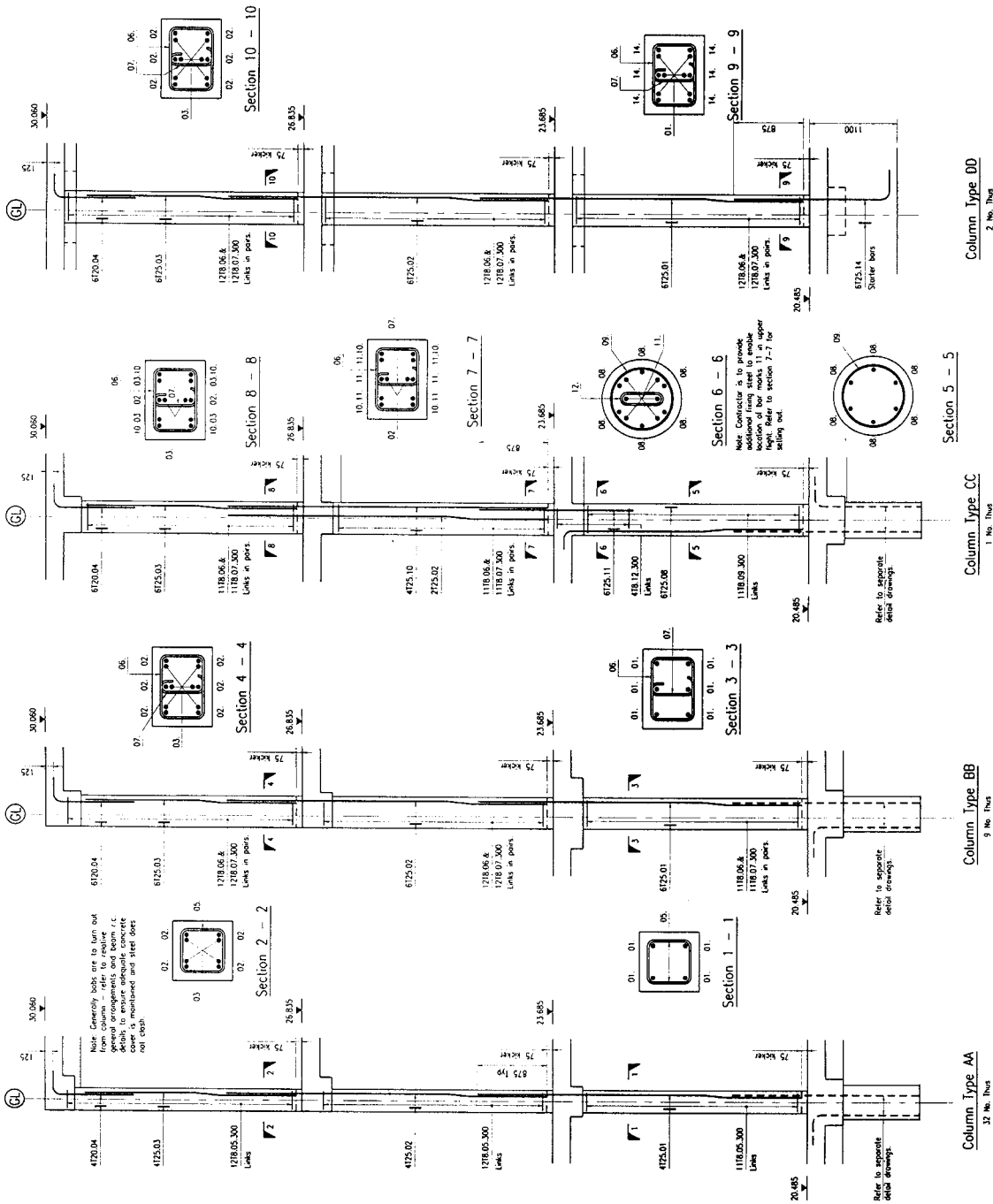
- Notes
- 1 For general notes refer to Ward Cole drawing number 30/386/701
 - 2 This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details
 - 3 Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110
 - 4 High yield steel provided in drawing and reinforcement bars to be BS 4449 or BS 4449 mild steel provided in drawing and delivery lugs to be in accordance with BS 4449
 - 5 Concrete cover to reinforcement to be as follows:
Columns - 50mm to main steel
Minimum laps to reinforcement to be as follows:
112 - 450mm
118 - 300mm
120 - 200mm
125 - 875mm
 - 6 Maximum laps to reinforcement to be as follows:
112 - 450mm
118 - 300mm
120 - 200mm
125 - 875mm

Note For column key plan and column reinforcement details refer to drawing no. 30/386/703

no.	date	ISSUED FOR CONSTRUCTION	description	CHK.
0	14.4.99			

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drawing title		R.C. Details of Columns	
sheet no.		Ground to Third Floor	
sheet of		Sheet 1 of 3	
date	1/75	drawn by	KT
checked by		drawing number	30/386
revision		quantity	510
		0	



Column Type DD
2 No. Thus

Column Type CC
1 No. Thus

Column Type BB
9 No. Thus

Column Type AA
32 No. Thus

SHEET NO. VIII.2.23

- Notes:
1. For general notes refer to Ward Cole drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 4. High yield steel precast T on drawing and BS 4449 or BS 4451 Mild steel precast R on drawing and delivery logs to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows:
Columns - 50 to minimum
Minimum laps to reinforcement to be as follows:
112 - 450mm
118 - 200mm
123 - 200mm
125 - 875mm
 6. Minimum laps to reinforcement to be as follows:

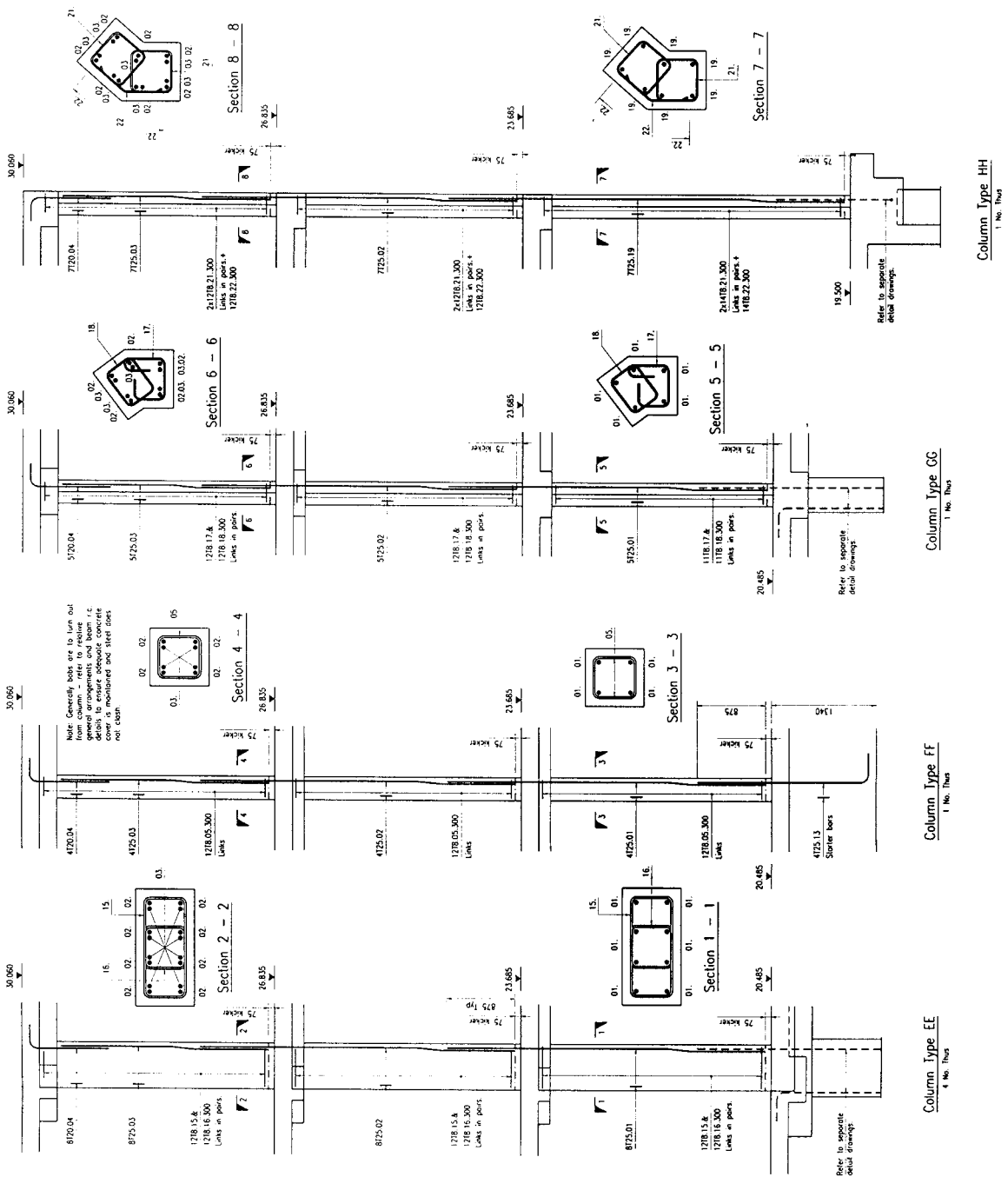
Note: For columns lap pin and column reinforcement refer to drawing no. 30/386/215.

no.	date	description	chk.
0	14.4.99	ISSUED FOR CONSTRUCTION	

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 125, 126 & 127, 2001
 75, 76 & 77, 2001
 78, 79 & 80, 2001
 81, 82 & 83, 2001
 84, 85 & 86, 2001
 87, 88 & 89, 2001
 90, 91 & 92, 2001
 93, 94 & 95, 2001
 96, 97 & 98, 2001
 99, 100 & 101, 2001
 102, 103 & 104, 2001
 105, 106 & 107, 2001
 108, 109 & 110, 2001
 111, 112 & 113, 2001
 114, 115 & 116, 2001
 117, 118 & 119, 2001
 120, 121 & 122, 2001
 123, 124 & 125, 2001
 126, 127 & 128, 2001
 129, 130 & 131, 2001
 132, 133 & 134, 2001
 135, 136 & 137, 2001
 138, 139 & 140, 2001
 141, 142 & 143, 2001
 144, 145 & 146, 2001
 147, 148 & 149, 2001
 150, 151 & 152, 2001

Drawing title:
**R.C. Details of Columns
 Ground to Third Floor
 Sheet 2 of 3**

Scale: 1/25
 Date: Mar 1999
 Checked by: [Signature]
 Drawing number: 30/386
 Revision: 511
 No. of sheets: 0



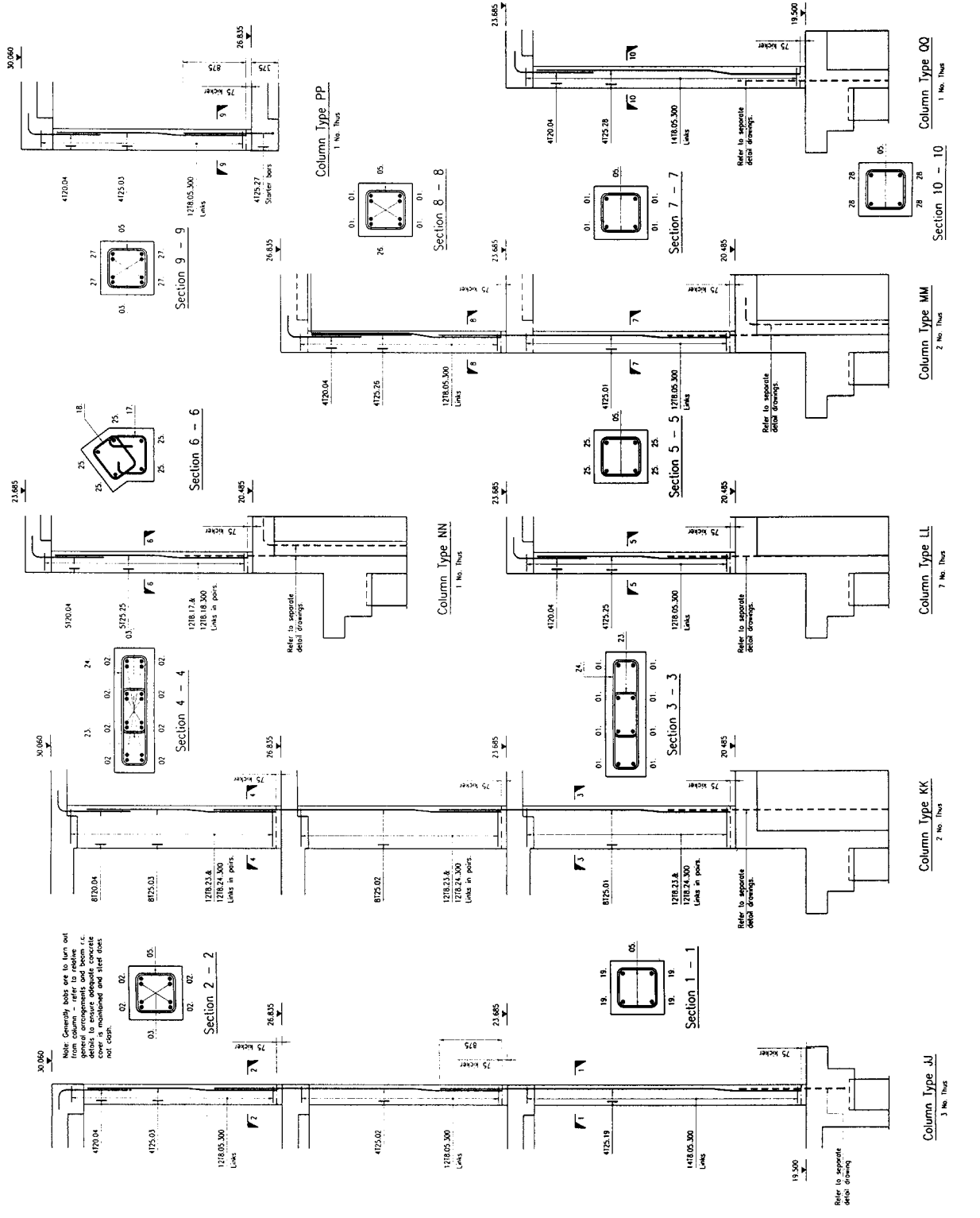
- Notes:
1. For general notes refer to Ward Cole drawing number 30/386/201
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise. All slabs must be cast in place with 40mm max size aggregate and reinforced with BS 50119.
 4. High yield steel rebar: on drawing and BS 4449 or BS 4449-1. All steel printed 'R' on drawing and delivery tags to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows:
 Columns - 50 to main, 30 to stirrups.
 Minimum laps to reinforcement to be as follows:
 112 - 450mm
 114 - 450mm
 120 - 700mm
 125 - 875mm
 6. Minimum laps to reinforcement to be as follows:
 112 - 450mm
 114 - 450mm
 120 - 700mm
 125 - 875mm

Note: For column key plan and section refer to drawing no. 30/386/215

0	14.8.99	ISSUED FOR CONSTRUCTION	
REV.	DATE	DESCRIPTION	BY

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Drawing No.	30/386	Sheet No.	512
Date	1/75	Rev.	0
Project	R.C. Details of Columns	Ground to Third Floor	
Sheet	Sheet 3 of 3		



**VIII.2.2.2. Drawings
with job number
30/427: ramp
structures**

These drawings relate to the construction of a new ramp structure to allow vehicular access to the basement level of an existing building for car parking purposes. The ramp structure consists of reinforced concrete walls and a piled base slab to suit the prevalent ground conditions on the site.

Drawing numbers 100 to 102 (Sheet Nos VIII.2.25 to VIII.2.27) inclusive show the general arrangement including sectional information for the ramp structure. Drawing 103 (Sheet No. VIII.2.28) outlines a suggested sequence of operations to facilitate the construction of the ramp and was prepared to assist the main contractor in the planning of the works. Drawing 104 (Sheet No. VIII.2.29) shows the foundation base slab and pile layout to the ramp structure.

The 200 series of drawings detail the reinforcement requirement to all the various structural elements and include base and capping slabs together with the walls to the sides of the ramp.

Drawing No. 100: Sheet No. VIII 2.25 CA of car park ramp

Drawing No. 101: Sheet No. VIII 2.26 CA of car park ramp

Drawing No. 102: Sheet No. VIII 2.27 CA of car park ramp

Drawing No. 103: Sheet No. VIII 2.28 CA of car park ramp suggested
sequence of operations

Drawing No. 104: Sheet No. VIII 2.29 Car park ramp – pile layout

Drawing No. 204: Sheet No. VIII 2.30 RC details of beam and columns to
ramp

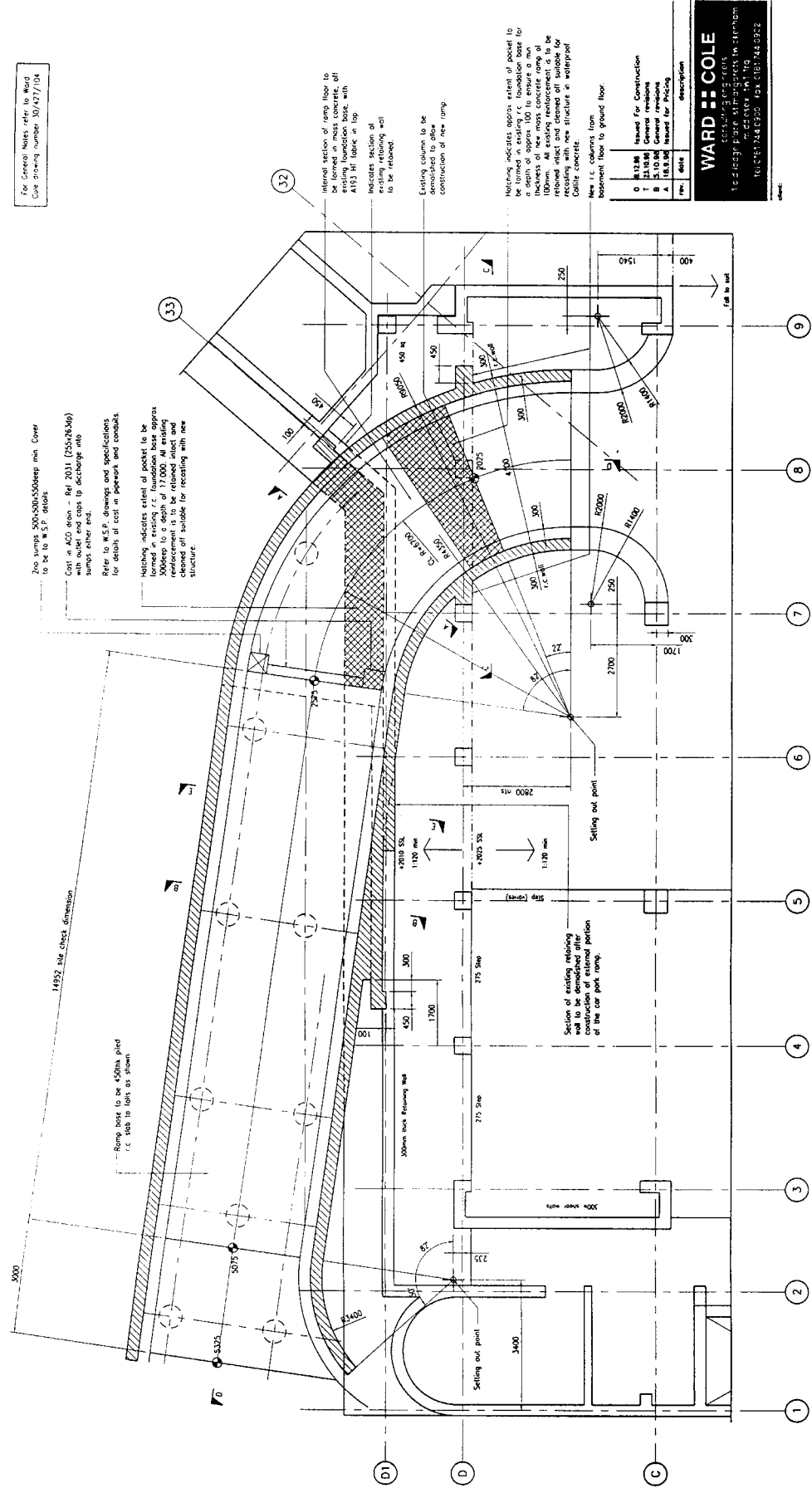
Drawing No. 200: Sheet No. VIII 2.31 RC details of car park ramp walls

Drawing No. 201: Sheet No. VIII 2.32 RC details of car park ramp wall
sections

Drawing No. 202: Sheet No. VIII 2.33 RC details of car park ramp – plan on
ramp base slab

Drawing No. 203: Sheet No. VIII 2.34 RC details of car park plan on ground
floor slab

For General Notes refer to Ward
Core Drawing number 30/427/04



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10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50, 52, 54, 56, 58, 60, 62, 64, 66, 68, 70, 72, 74, 76, 78, 80, 82, 84, 86, 88, 90, 92, 94, 96, 98, 100, 102, 104, 106, 108, 110, 112, 114, 116, 118, 120, 122, 124, 126, 128, 130, 132, 134, 136, 138, 140, 142, 144, 146, 148, 150, 152, 154, 156, 158, 160, 162, 164, 166, 168, 170, 172, 174, 176, 178, 180, 182, 184, 186, 188, 190, 192, 194, 196, 198, 200, 202, 204, 206, 208, 210, 212, 214, 216, 218, 220, 222, 224, 226, 228, 230, 232, 234, 236, 238, 240, 242, 244, 246, 248, 250, 252, 254, 256, 258, 260, 262, 264, 266, 268, 270, 272, 274, 276, 278, 280, 282, 284, 286, 288, 290, 292, 294, 296, 298, 300, 302, 304, 306, 308, 310, 312, 314, 316, 318, 320, 322, 324, 326, 328, 330, 332, 334, 336, 338, 340, 342, 344, 346, 348, 350, 352, 354, 356, 358, 360, 362, 364, 366, 368, 370, 372, 374, 376, 378, 380, 382, 384, 386, 388, 390, 392, 394, 396, 398, 400, 402, 404, 406, 408, 410, 412, 414, 416, 418, 420, 422, 424, 426, 428, 430, 432, 434, 436, 438, 440, 442, 444, 446, 448, 450, 452, 454, 456, 458, 460, 462, 464, 466, 468, 470, 472, 474, 476, 478, 480, 482, 484, 486, 488, 490, 492, 494, 496, 498, 500, 502, 504, 506, 508, 510, 512, 514, 516, 518, 520, 522, 524, 526, 528, 530, 532, 534, 536, 538, 540, 542, 544, 546, 548, 550, 552, 554, 556, 558, 560, 562, 564, 566, 568, 570, 572, 574, 576, 578, 580, 582, 584, 586, 588, 590, 592, 594, 596, 598, 600, 602, 604, 606, 608, 610, 612, 614, 616, 618, 620, 622, 624, 626, 628, 630, 632, 634, 636, 638, 640, 642, 644, 646, 648, 650, 652, 654, 656, 658, 660, 662, 664, 666, 668, 670, 672, 674, 676, 678, 680, 682, 684, 686, 688, 690, 692, 694, 696, 698, 700, 702, 704, 706, 708, 710, 712, 714, 716, 718, 720, 722, 724, 726, 728, 730, 732, 734, 736, 738, 740, 742, 744, 746, 748, 750, 752, 754, 756, 758, 760, 762, 764, 766, 768, 770, 772, 774, 776, 778, 780, 782, 784, 786, 788, 790, 792, 794, 796, 798, 800, 802, 804, 806, 808, 810, 812, 814, 816, 818, 820, 822, 824, 826, 828, 830, 832, 834, 836, 838, 840, 842, 844, 846, 848, 850, 852, 854, 856, 858, 860, 862, 864, 866, 868, 870, 872, 874, 876, 878, 880, 882, 884, 886, 888, 890, 892, 894, 896, 898, 900, 902, 904, 906, 908, 910, 912, 914, 916, 918, 920, 922, 924, 926, 928, 930, 932, 934, 936, 938, 940, 942, 944, 946, 948, 950, 952, 954, 956, 958, 960, 962, 964, 966, 968, 970, 972, 974, 976, 978, 980, 982, 984, 986, 988, 990, 992, 994, 996, 998, 1000

Rev.	Date	Description
0	11/12/08	Issued For Construction
1	21/10/08	General revisions
2	15/10/08	General revisions
3	10/10/08	Issued For Pricing

Note: For suggested sequence of work and for extent of waterproofing refer to W.S.P. Core Drawing number 30/427/03

Part Plan on Basement - Building 2

Drawing Title: C.A. of Car Park Ramp

Date:	Sept 1998	Drawn by:	KT
Scale:	1:50	Checked by:	
Sheet Number:	30/427	Drawing Number:	100
		Revision:	0

SHEET NO. VIII.2.26

For General Notes refer to Ward
Cole drawing number 30/427/104

New slab to be supported on
150x150x10 s.s. angle bolted to
existing concrete using M16 HSJ.
Hanging by 4 nos. 450CRs, along
this edge.
Upstand to be recast with new
slab to original profile.

Existing beams to be carefully
braced and supported, retaining,
retaining all reinforcement,
penetrating from columns,
pipes and conduits for recasting
with new structure.

Upstand to be recast with new
slab to original profile.

Existing beams to be demolished
and recast. New top level of beam
to be 250 above original top level.

Existing top and bottom slab
reinforcement to be retained and
replaced with new slab for recasting in
one section of slab.

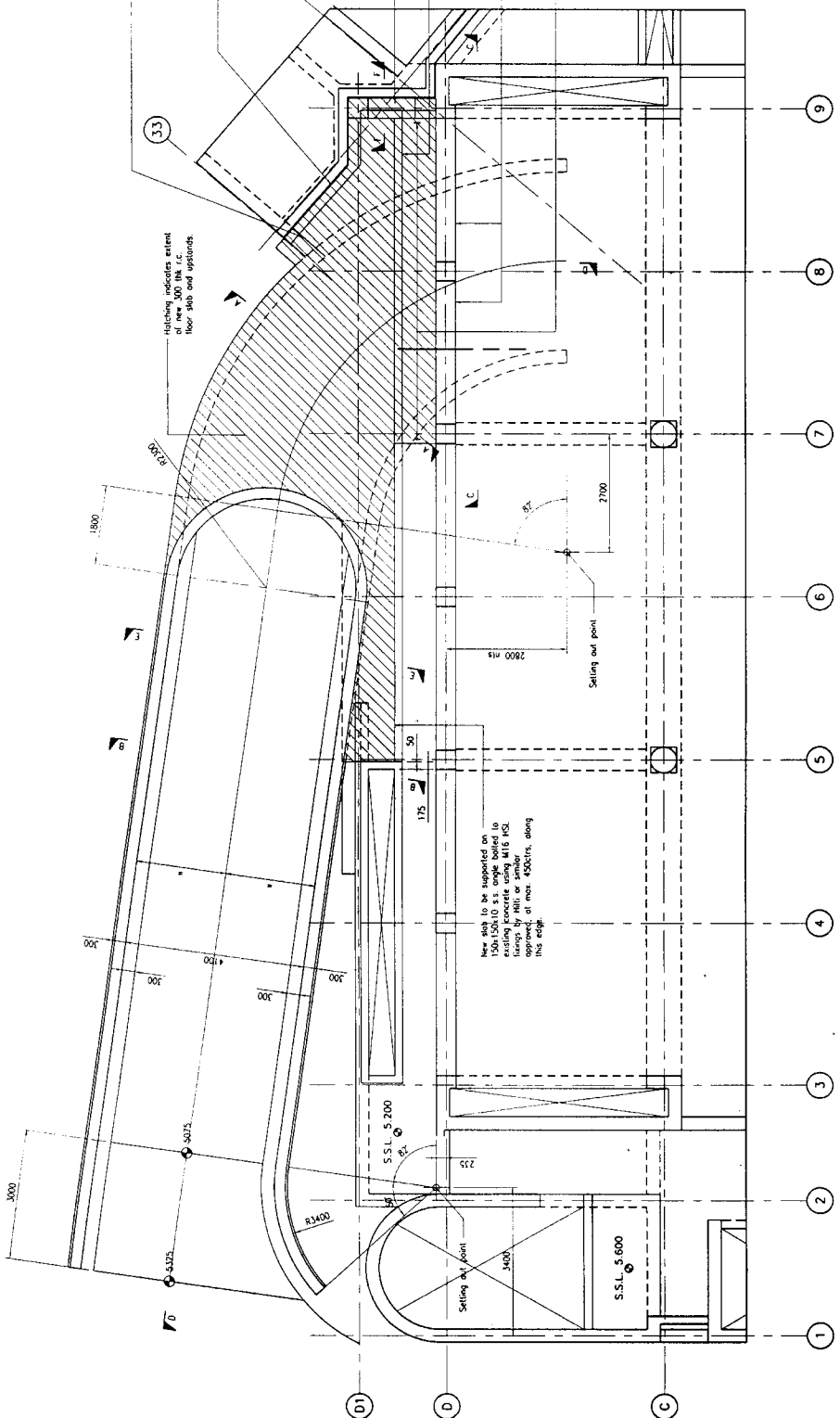
no.	date	description
0	21.12.98	Issued for Construction
1	21.10.98	Issued for Tender
2	11.10.98	General revisions
A	18.12.98	Issued for Pricing

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 DORSET, ENGLAND. TA25 5SDA. UK. Tel:
 01305 744 550 Fax: 0161 744 692

drawing title:
G.A. of Car Park Ramp

Notes: For suggested sequence of
works and for extent of waterproof
concrete refer to Ward Cole drawing
number 30/427/103

scale:	1:50	date:	Sept 1998	drawn by:	KF
file number:	30/427	drawing number:	101	revision:	0



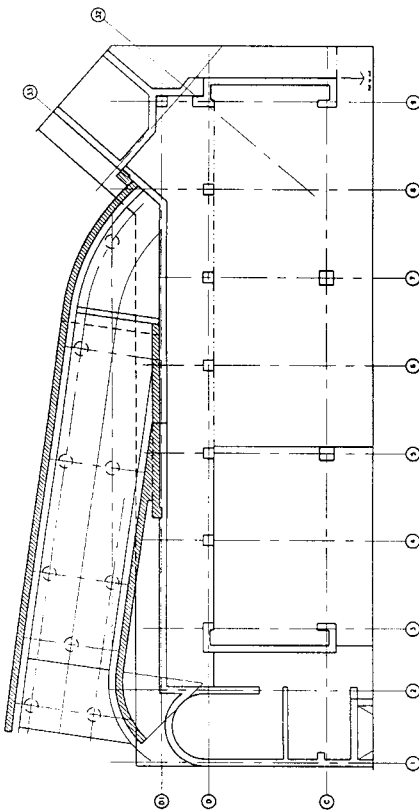
Part Plan on Ground Floor – Building 2

For General Notes refer to Ward
Coke Drawing number 30/427/104

SUGGESTED METHOD STATEMENT AND SEQUENCE OF WORKS

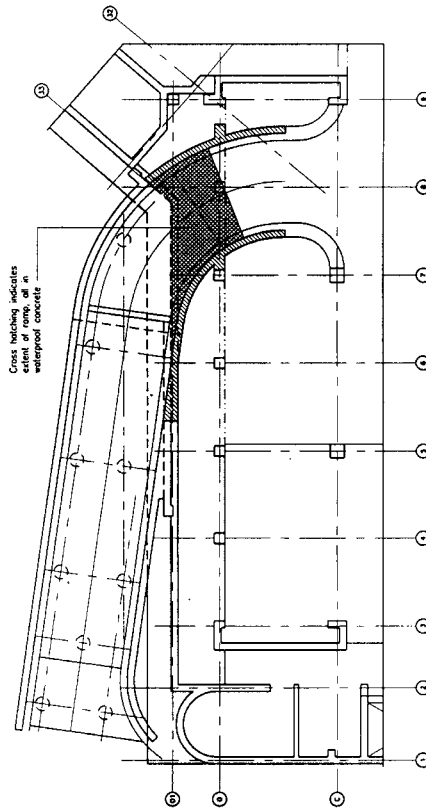
- It is suggested that the ramp construction is carried out in two distinct phases as shown on this drawing in order to maintain the water tightness of the existing basement construction.
- Positively locate and temporarily divert as necessary all existing services below ground within the area that is likely to be affected by the proposed work.
- Establish a suitable piling platform and water piles. Pile installation to be carried out by specialist.
- The contractor should be aware that after legal working to construct the ramp may be necessary given the ground water regime has to exist on site. Consideration may be given to the installation of a temporary pump and sump system or more conventional sump and pump methods.
- Install suitably designed temporary retaining structure to the inside wall of the existing ramp. The structure should facilitate excavation down to ramp slab level. Consideration could be given to battering back and regrading the existing ground to a suitable depth together with the installation of a temporary retaining wall to the existing retaining wall to the rear of building (2) could be considered. The contractor is required to prepare a detailed temporary works scheme and submit this for approval by the relevant authority at least two weeks prior to commencement of works.
- Excavate down to suit level of ramp slab and cut down all piles to the required cut off level.
- Construct the suggested phase 1 of the proposed ramp construction in accordance with details shown on relevant drawings. All construction work should be carried out in accordance with the recommendations of Cementech.
- Install suitably designed temporary works to allow demolition of reinforced concrete beams and columns along grid line (0) for an extent as shown on relevant Ward Coke drawings.
- Construct new R.C. Columns integral with R.C. Wall and beam along grid (0).
- Carrially demolish existing R.C. retaining wall to the rear of building (2) if necessary on the basis of the recommendations of Cementech. Care should be taken to ensure that any voids should be used to make the section of the existing wall to be removed from the existing structure prior to breaking down and removal.
- Carry out the remaining phase 2 works and complete the ramp construction.

THE ABOVE IS A SUGGESTED METHOD STATEMENT AND SEQUENCE OF WORKS. THE CONTRACTOR IS REQUIRED TO PREPARE HIS OWN METHOD STATEMENT TO BE SUBMITTED AND APPROVED BY THE RELEVANT AUTHORITY PRIOR TO THE COMMENCEMENT OF WORKS.



Part Plan on Basement - Phase 1

Note: All new concrete forming part of Phase 1 work is shown in solid lines. Existing concrete is shown in dashed lines. Cementech Excavate Cable System as described in General Notes on drawing number 30/427/103.



Part Plan on Basement - Phase 2

Note: The cross-hatching indicates the extent of ramp, all in waterproof concrete. The remaining new structure (walls, columns, etc.) is shown in solid lines. Existing concrete is shown in dashed lines. Cementech Excavate Cable System as described in General Notes on drawing number 30/427/104.

0	R12.04	Issued for Construction
1	1997	Issued for Tender
no.	date	description

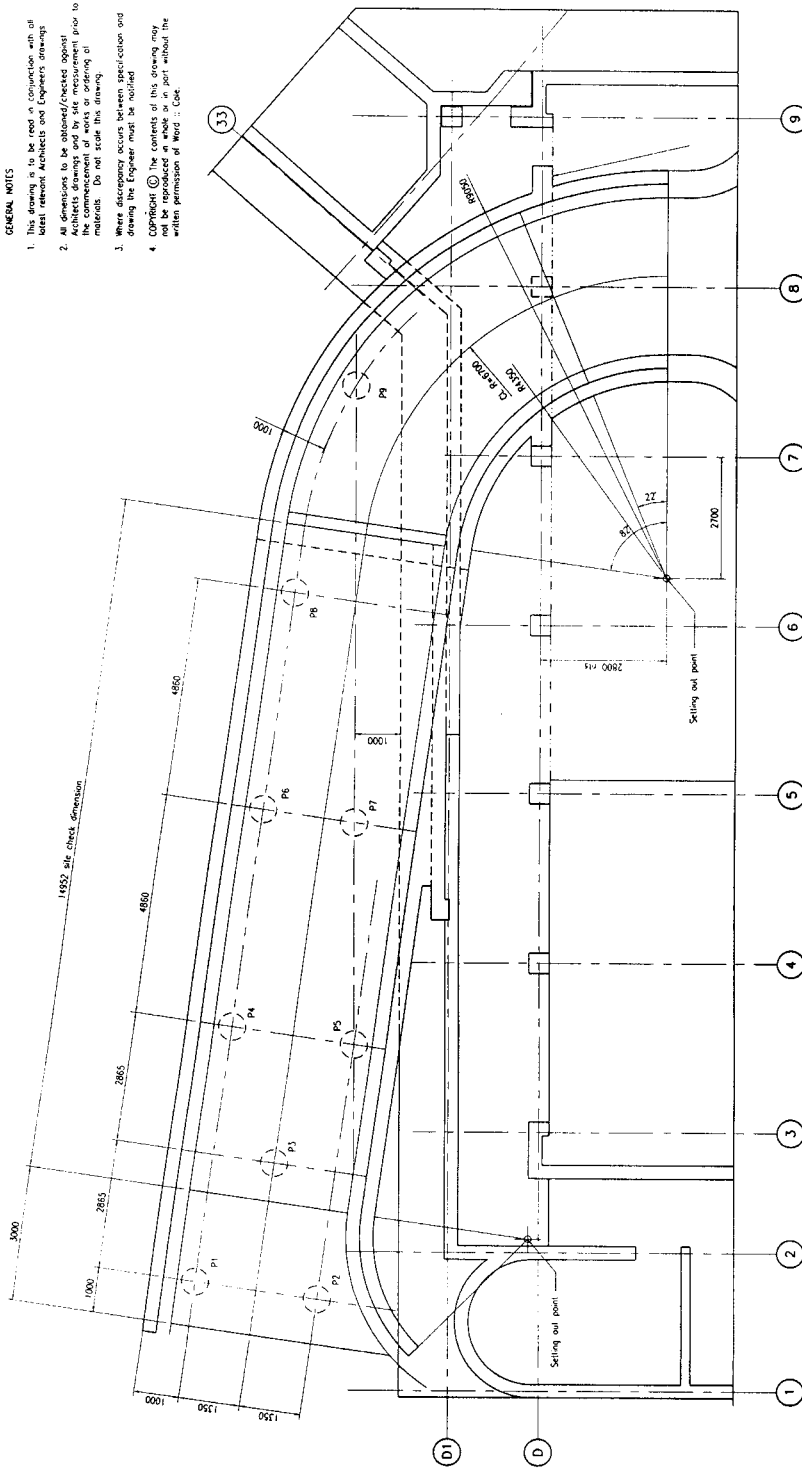
WARD :: COLE
 CONSULTING ENGINEERS
 1401 BRIDGE STREET, BRISTOL, AVON, BRISTOL, ENGLAND
 TEL: 0117 925 2000 FAX: 0117 925 2001
 TEL: 0117 925 2002 FAX: 0117 925 2003

drawing title: Car Park Ramp - Suggested Sequence of Operations

scale:	date:	drawn by:	checked by:
1:100	Oct 1998	RT	
no. number:	drawing number:	revision:	
30/427	103	0	

5. The contractor is responsible for all overall stability of the structure from the commencement of works, including the design and provision of all necessary temporary works. Contractor is to submit full details of temporary works to the Engineer at least 4 weeks prior to commencement of works.
6. Where delay occurs before placing of concrete, a minimum 50mm layer of binding concrete is to be placed over the reinforcement after erection and exposure of the formwork.
7. The contractor is responsible for the location of all underground services that may exist and to ensure them if necessary prior to the commencement of works.
8. All structural works to be carried out strictly in accordance with the details given by Ward :: Cole. The contractor is to ensure that any significant variation prior to its implementation.
9. High yield steel rebar "T" on drawing and delivery logo to be in accordance with B.S. 4449 and 4461. High yield steel rebar "N" on drawing and delivery logo to be in accordance with B.S. 4449 and 4461.
10. All concrete is to be Grade C40 with 30mm max size of aggregate. All concrete is to be in accordance with BS9110. Reinforced concrete as indicated on drawings numbered 30/427/105 is to be cast in accordance with the addition of Cemented Evaporative Gypsum System additives including Cellulose at the rate of 30 litres per cubic metre of concrete. The manufacturer's recommended rate. The cement content shall not be less than 300kg per cubic metre. The water/cement ratio shall be reduced to adjust for workability. However the water/cement ratio must not exceed 0.45. A trial mix must be conducted prior to commencement of concrete to ensure that the content is to be stated on the permit to construct details.
11. All construction details to be approved with the Engineer and are to be prepared in accordance with the Cemented requirements. Provide 'Track System' or similar approved at an construction joint positions. Consultation with manufacturers.
12. The Contractor is to refer to Cemented for the completion of appropriate sequencing of concrete pours.
13. Specific restrictions through Cells are to be provided with the appropriate approved support systems to specialist design and detail.

- GENERAL NOTES**
1. This drawing is to be read in conjunction with all latest relevant Architects and Engineers drawings.
 2. All dimensions to be indicated/checked against Architects drawings and for site measurement prior to the commencement of works or ordering of materials. Do not scale this drawing.
 3. Where discrepancy occurs between specification and drawing the Engineer must be notified.
 4. COPYRIGHT © The contents of this drawing may not be reproduced in whole or in part without the written permission of Ward :: Cole.



Part Plan on Basement – Building 2

- Piling Notes:**
1. All piles to be 675 mm diameter bored piles designed by specialist for a load of 600kN per pile. The piles are to be installed in a vertical position. The load of pile at head of pile applied in any direction. In addition piles to be designed for an uplift tension force of 275kN.
 2. All piles to be taken down to rockhead level and socketed into rock. Actual penetration of pile into rock to suit Specialist design requirements.
 3. The Contractor is to form a suitable piling platform level at corners. Existing ground level to suit existing site conditions. Existing external piling to be removed as required. Pile cut off levels to be as indicated on drawing.
 4. The Contractor is required to take full account of ground conditions on site including local ground water regime in the design of piles. The Contractor is to submit a report on the ground conditions. The method of pile construction should take into account the nature of the ground water in particular between 0.5 AOD and 2.5 AOD approximately.
 5. The Contractor is required to provide a full and detailed method statement in respect of the pile installation. The method statement is to be submitted to the Engineer for comment. The responsibility for the design and installation of all piling to remain fully with the Contractor.
 6. A copy of the site investigation has been previously provided. The site investigation report is for information only. Notwithstanding the above and the Contractor is to conduct a detailed investigation of the nature and location of soils and strata on site before commencing work.

Pile No.	Cut Off Level
P1	4940
P2	4940
P3	4600
P4	4120
P5	4120
P6	3110
P7	3110
P8	2495
P9	1775

Reinforcement Quantities For The Purpose of Bidding

Ramp Slab	250kg/m ²
Columns	1500kg/m ²
Roof slab	100kg/m ²
Reinforced beam	350kg/m ²

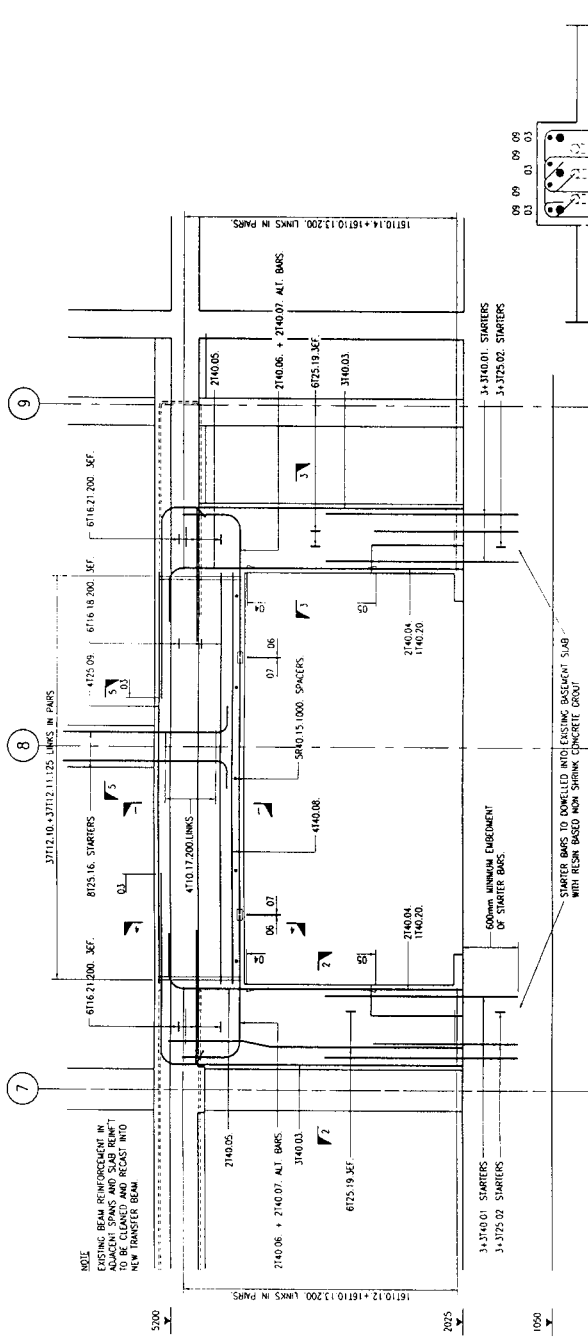
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0	12.08	Issued for Construction
1	23.03.08	Issued for Tender

WARD :: COLE
 CONSULTING ENGINEERS
 11, DUNDEE PLACE, ST. LEONARDS, NEWCASTLE UPON TYNE, NE11 1BA
 TEL: 01665 244455 FAX: 01665 244552

**Car Park Ramp
Pile Layout and General Notes**

DATE	OCT 1998	SCALE	KT
DRAWN BY		CHECKED BY	
JOB NUMBER	30/427	WORKING NUMBER	104
REVISION			0

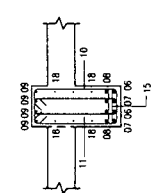
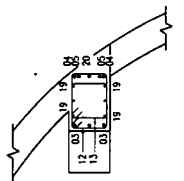
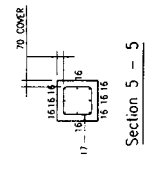
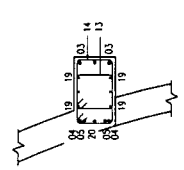
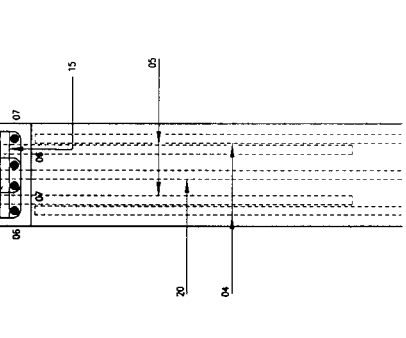
- Notes:
1. For general notes refer to Ward Code drawing number 30/427/194
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110
 4. High yield steel precast 'T' on drawing and delivery logs to be in accordance with BS 4449. All reinforcement bars to be marked 'R' on drawing and delivery logs to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows:
30mm lap, bottom and sides to links



no.	date	issued for pricing	description
0	18.12.98		

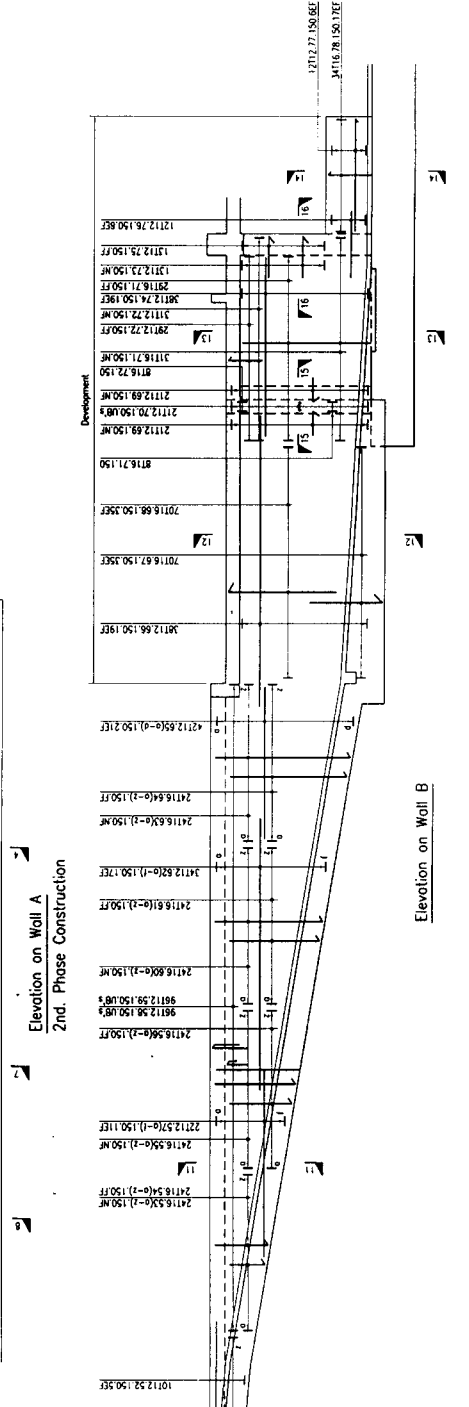
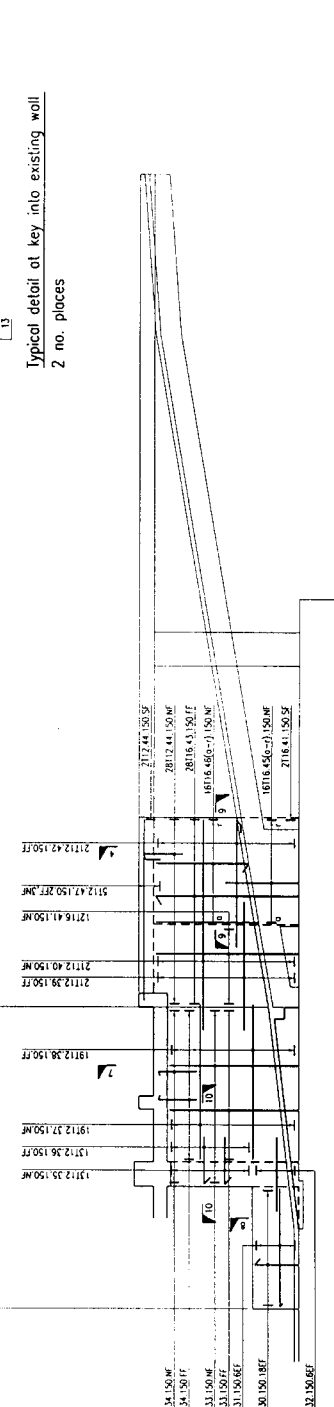
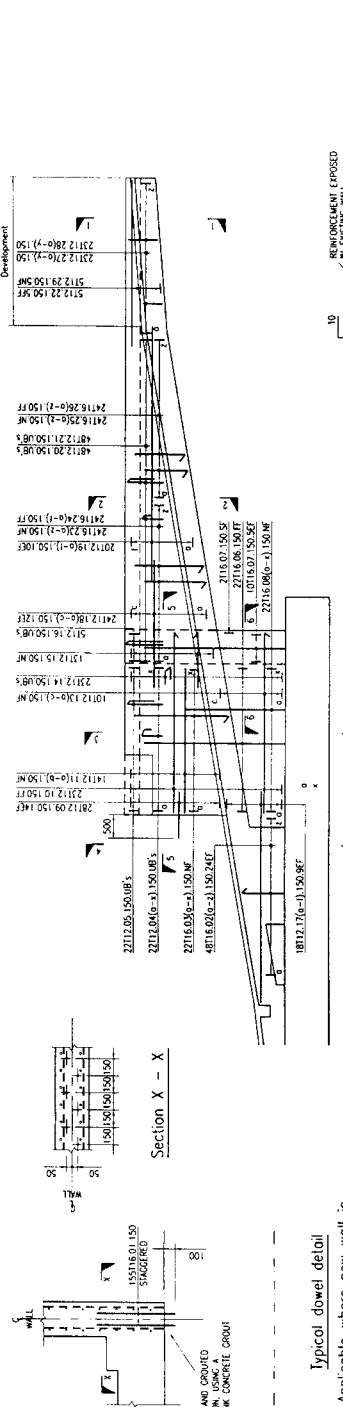
WARD :: COLE
 CIVIL ENGINEER
 14-23 BEECH LANE, WITZEL ROAD, WIMBORNE, DORSET, BH20 3JG
 TEL: 0187 744 5565 FAX: 0187 744 0292

drawing title	R.C. Details of Beam & Column: to Car Park Ramp		
date	1.25.1.10	drawn by	KT ALK
check by		checked by	
drawn number	30/427	drawing number	204
revision		revision	0



Section 4 - 4

- Notes:
1. For general info, refer to Ward Cole drawing number 30/427/104
 2. This drawing shall be read in conjunction with all relevant drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110
 4. High yield steel rebar 'T' on drawing and delivery logs, to be in accordance with BS 4449 or BS 4449. All steel shall be galvanized 'R' as shown on drawing. All rebar to be in accordance with BS 4449.
 5. Concrete cover to reinforcement to be as follows:
50mm top, bottom and sides
 6. Minimum laps to reinforcement to be as follows:
112 - 450mm
116 - 600mm
 7. For Sections See Drawing No. 30/427/201



REV.	DATE	ISSUED FOR CONSTRUCTION	DESCRIPTION
0	11/12/08		

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 222, 115, 62, 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 96, 97, 98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120, 121, 122, 123, 124, 125, 126, 127, 128, 129, 130, 131, 132, 133, 134, 135, 136, 137, 138, 139, 140, 141, 142, 143, 144, 145, 146, 147, 148, 149, 150, 151, 152, 153, 154, 155, 156, 157, 158, 159, 160, 161, 162, 163, 164, 165, 166, 167, 168, 169, 170, 171, 172, 173, 174, 175, 176, 177, 178, 179, 180, 181, 182, 183, 184, 185, 186, 187, 188, 189, 190, 191, 192, 193, 194, 195, 196, 197, 198, 199, 200, 201, 202, 203, 204, 205, 206, 207, 208, 209, 210, 211, 212, 213, 214, 215, 216, 217, 218, 219, 220, 221, 222, 223, 224, 225, 226, 227, 228, 229, 230, 231, 232, 233, 234, 235, 236, 237, 238, 239, 240, 241, 242, 243, 244, 245, 246, 247, 248, 249, 250, 251, 252, 253, 254, 255, 256, 257, 258, 259, 260, 261, 262, 263, 264, 265, 266, 267, 268, 269, 270, 271, 272, 273, 274, 275, 276, 277, 278, 279, 280, 281, 282, 283, 284, 285, 286, 287, 288, 289, 290, 291, 292, 293, 294, 295, 296, 297, 298, 299, 300, 301, 302, 303, 304, 305, 306, 307, 308, 309, 310, 311, 312, 313, 314, 315, 316, 317, 318, 319, 320, 321, 322, 323, 324, 325, 326, 327, 328, 329, 330, 331, 332, 333, 334, 335, 336, 337, 338, 339, 340, 341, 342, 343, 344, 345, 346, 347, 348, 349, 350, 351, 352, 353, 354, 355, 356, 357, 358, 359, 360, 361, 362, 363, 364, 365, 366, 367, 368, 369, 370, 371, 372, 373, 374, 375, 376, 377, 378, 379, 380, 381, 382, 383, 384, 385, 386, 387, 388, 389, 390, 391, 392, 393, 394, 395, 396, 397, 398, 399, 400, 401, 402, 403, 404, 405, 406, 407, 408, 409, 410, 411, 412, 413, 414, 415, 416, 417, 418, 419, 420, 421, 422, 423, 424, 425, 426, 427, 428, 429, 430, 431, 432, 433, 434, 435, 436, 437, 438, 439, 440, 441, 442, 443, 444, 445, 446, 447, 448, 449, 450, 451, 452, 453, 454, 455, 456, 457, 458, 459, 460, 461, 462, 463, 464, 465, 466, 467, 468, 469, 470, 471, 472, 473, 474, 475, 476, 477, 478, 479, 480, 481, 482, 483, 484, 485, 486, 487, 488, 489, 490, 491, 492, 493, 494, 495, 496, 497, 498, 499, 500, 501, 502, 503, 504, 505, 506, 507, 508, 509, 510, 511, 512, 513, 514, 515, 516, 517, 518, 519, 520, 521, 522, 523, 524, 525, 526, 527, 528, 529, 530, 531, 532, 533, 534, 535, 536, 537, 538, 539, 540, 541, 542, 543, 544, 545, 546, 547, 548, 549, 550, 551, 552, 553, 554, 555, 556, 557, 558, 559, 560, 561, 562, 563, 564, 565, 566, 567, 568, 569, 570, 571, 572, 573, 574, 575, 576, 577, 578, 579, 580, 581, 582, 583, 584, 585, 586, 587, 588, 589, 590, 591, 592, 593, 594, 595, 596, 597, 598, 599, 600, 601, 602, 603, 604, 605, 606, 607, 608, 609, 610, 611, 612, 613, 614, 615, 616, 617, 618, 619, 620, 621, 622, 623, 624, 625, 626, 627, 628, 629, 630, 631, 632, 633, 634, 635, 636, 637, 638, 639, 640, 641, 642, 643, 644, 645, 646, 647, 648, 649, 650, 651, 652, 653, 654, 655, 656, 657, 658, 659, 660, 661, 662, 663, 664, 665, 666, 667, 668, 669, 670, 671, 672, 673, 674, 675, 676, 677, 678, 679, 680, 681, 682, 683, 684, 685, 686, 687, 688, 689, 690, 691, 692, 693, 694, 695, 696, 697, 698, 699, 700, 701, 702, 703, 704, 705, 706, 707, 708, 709, 710, 711, 712, 713, 714, 715, 716, 717, 718, 719, 720, 721, 722, 723, 724, 725, 726, 727, 728, 729, 730, 731, 732, 733, 734, 735, 736, 737, 738, 739, 740, 741, 742, 743, 744, 745, 746, 747, 748, 749, 750, 751, 752, 753, 754, 755, 756, 757, 758, 759, 760, 761, 762, 763, 764, 765, 766, 767, 768, 769, 770, 771, 772, 773, 774, 775, 776, 777, 778, 779, 780, 781, 782, 783, 784, 785, 786, 787, 788, 789, 790, 791, 792, 793, 794, 795, 796, 797, 798, 799, 800, 801, 802, 803, 804, 805, 806, 807, 808, 809, 810, 811, 812, 813, 814, 815, 816, 817, 818, 819, 820, 821, 822, 823, 824, 825, 826, 827, 828, 829, 830, 831, 832, 833, 834, 835, 836, 837, 838, 839, 840, 841, 842, 843, 844, 845, 846, 847, 848, 849, 850, 851, 852, 853, 854, 855, 856, 857, 858, 859, 860, 861, 862, 863, 864, 865, 866, 867, 868, 869, 870, 871, 872, 873, 874, 875, 876, 877, 878, 879, 880, 881, 882, 883, 884, 885, 886, 887, 888, 889, 890, 891, 892, 893, 894, 895, 896, 897, 898, 899, 900, 901, 902, 903, 904, 905, 906, 907, 908, 909, 910, 911, 912, 913, 914, 915, 916, 917, 918, 919, 920, 921, 922, 923, 924, 925, 926, 927, 928, 929, 930, 931, 932, 933, 934, 935, 936, 937, 938, 939, 940, 941, 942, 943, 944, 945, 946, 947, 948, 949, 950, 951, 952, 953, 954, 955, 956, 957, 958, 959, 960, 961, 962, 963, 964, 965, 966, 967, 968, 969, 970, 971, 972, 973, 974, 975, 976, 977, 978, 979, 980, 981, 982, 983, 984, 985, 986, 987, 988, 989, 990, 991, 992, 993, 994, 995, 996, 997, 998, 999, 1000.

SHEET NO. VIII.2.32

- Notes:
1. For general notes refer to Ward Code drawing number 30/427/104
 2. This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 3. Concrete to be Grade C40, unless noted otherwise, with 20mm max size aggregate in accordance with BS 8110.
 4. High yield steel precast 'T' on drawing and delivery logs to be in accordance with BS 4443. Reinforcement bars to be in accordance with BS 4443 on drawing and delivery logs to be in accordance with BS 4443.
 5. Concrete cover to reinforcement to be as follows:
50mm top, bottom and sides
 6. Minimum laps to reinforcement to be as follows:
112 - 450mm
116 - 600mm
 7. For Elevations See Drawing No. 30/427/200

0. 30.12.06 Issued for Construction	
rev.	date description
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Drawing Date: R.C. Details of Car Park Ramp Wall Sections

Scale: 1:20

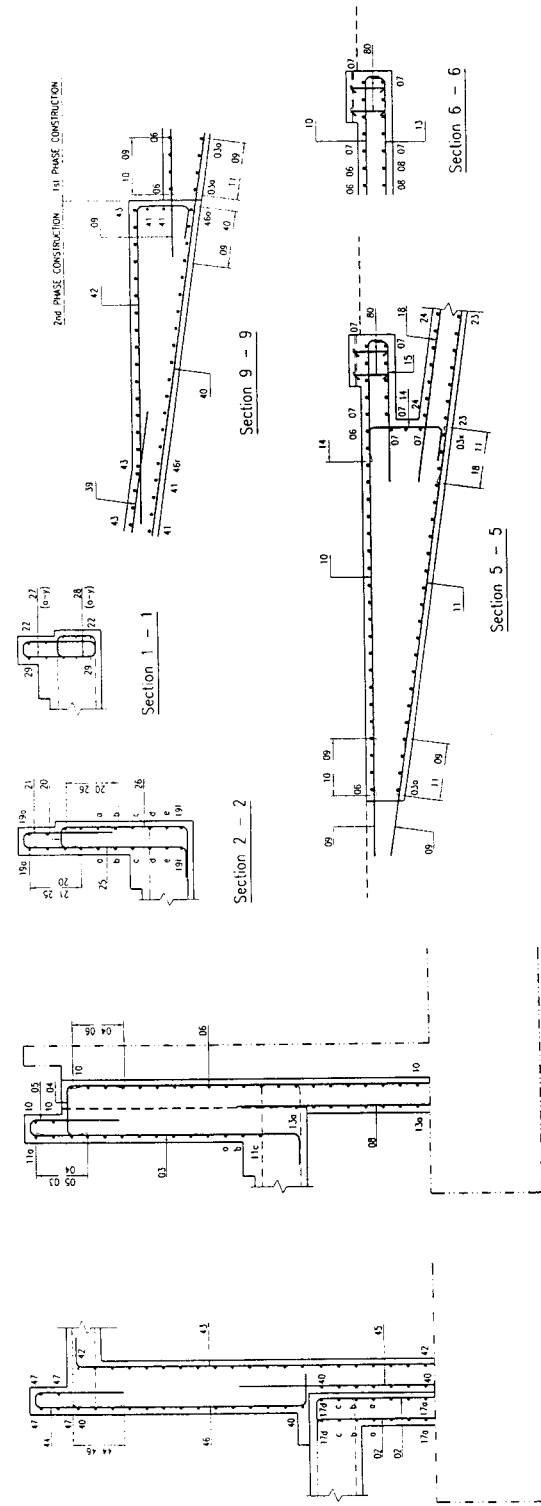
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Checked By: R.V.

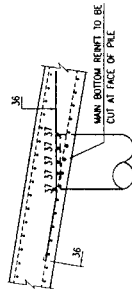
Project Number: 30/427

Revision Number: 201

Revision: 0



- Notes
- For general notes refer to Ward Cole drawing number 30/427/134
 - This drawing must be read in conjunction with all relevant Engineers and Architects drawings and details.
 - Concrete to be Grade C40, unless noted otherwise, with 20mm max. size aggregate in accordance with BS 8110.
 - High yield steel preferred. If on drawing and delivery logs to be in accordance with BS 4449 on drawing and delivery logs to be in accordance with BS 4449.
 - Concrete cover to reinforcement to be as follows:
20mm bottom
20mm sides
50mm ends
 - Minimum laps to reinforcement to be as follows:
112 - 600mm
112 - 800mm
120 - 700mm

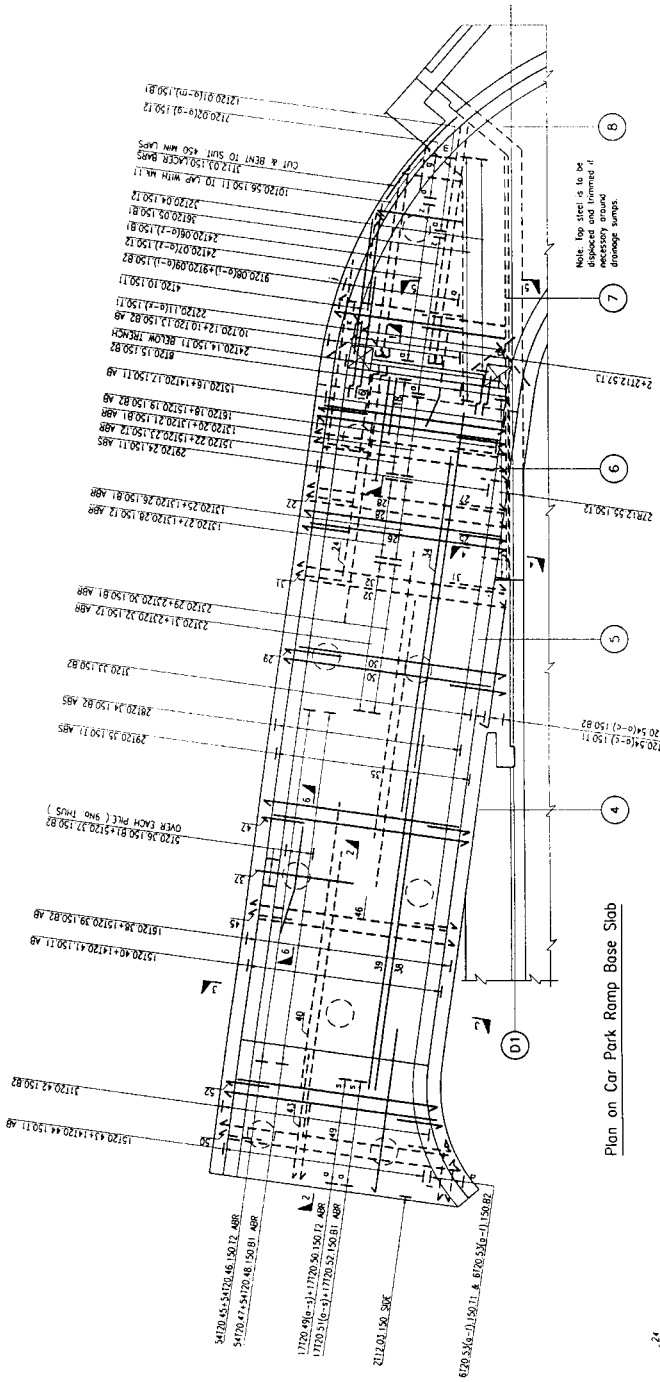


Section 6 - 6
(9 No. PILES)
SHOWING ADDITIONAL REINFT OVER PILES

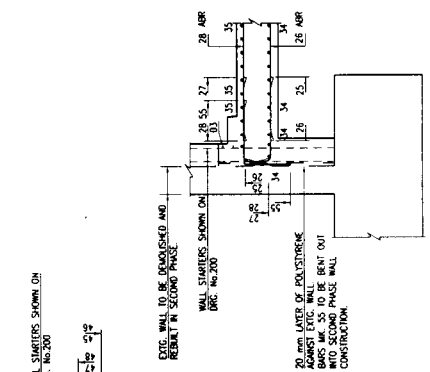
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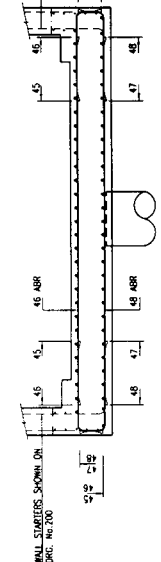
Project Name	R.C. Details of Car Park Ramp Plan on Ramp Base Slab		
Date	1:50, 1:25	Drawn By	DAI
Scale	30/427	Checked By	WJC
Sheet No.	202	Revision	0



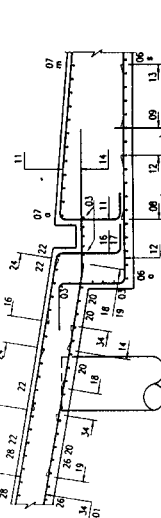
Plan on Car Park Ramp Base Slab



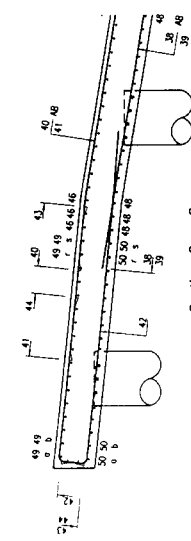
Section 3 - 3



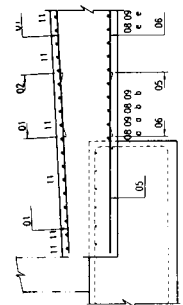
Section 4 - 4



Section 1 - 1



Section 2 - 2



Section 5 - 5

VIII.2.3. Case studies based on EC2 and European practices

VIII.2.3.1. Roof beams with suspended floors

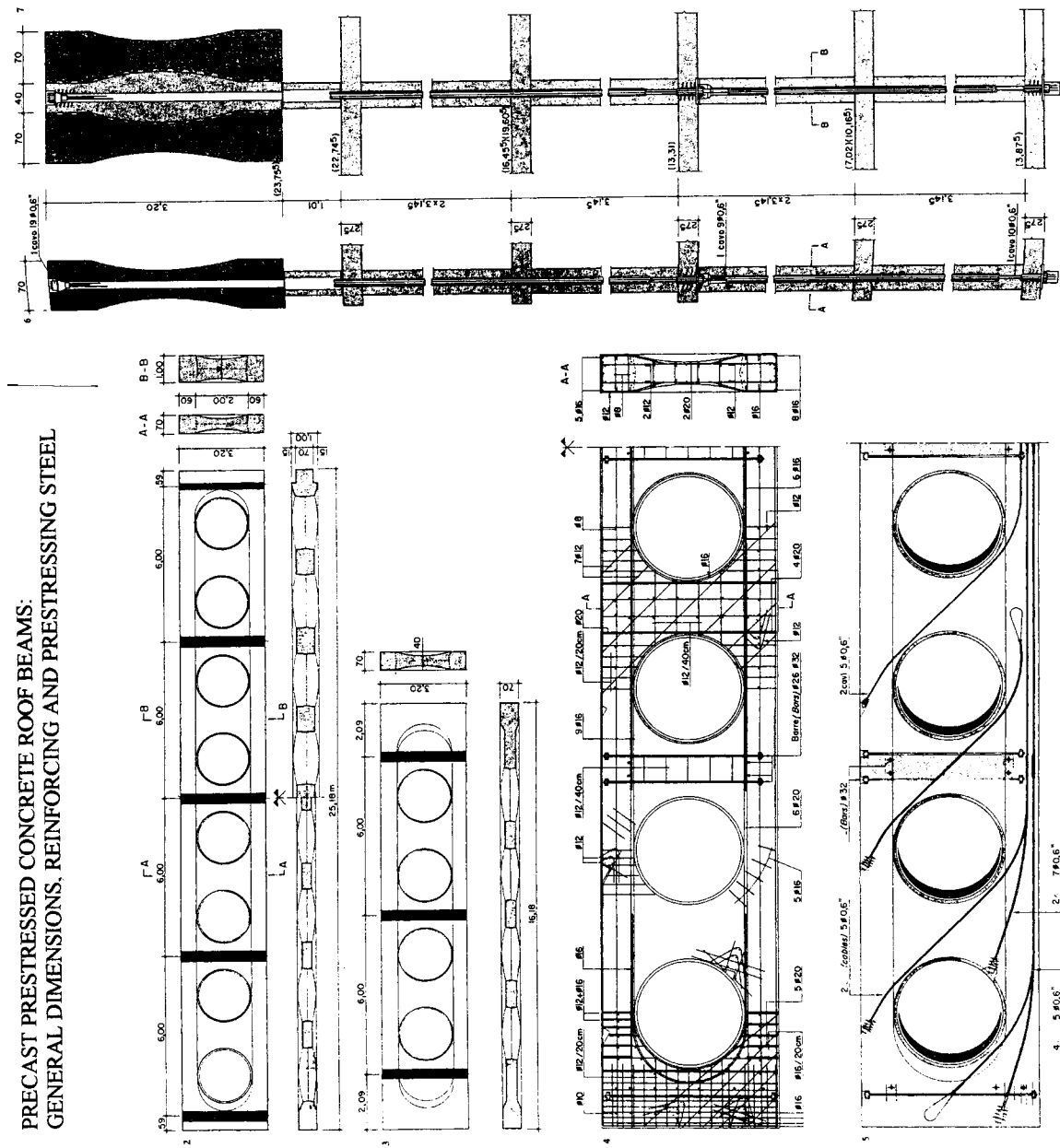
Sheet No. 2.35 shows a typical structural plan indicating reinforced concrete shafts resting on a prestressed concrete roof beams and waffle (waffle) slab floor. Positions of suspensions, i.e. suspenders, are marked. Reinforcement details are clearly given on the partial plan of waffle floor slabs at top and at bottom using EC2 detailing phenomenon. Sheet No. VIII.2.36 gives the main dimensions and prestressed and reinforced concrete details of precast and prestressed concrete roof beams together with suspension elements of the floors. The bearings of the precast concrete beams on shafts were fixed using mobile cranes. The in-situ pour of the ribbed decks is held up by the suspension fastened to the beams. The following operations have been considered.

1. In-situ pour of the reinforced concrete columns around the suspension tendons for all floors and up to the beam soffits.
2. Post-tensioning of the suspensions on completion of the pours, and their grouting.
3. General excavation and construction of the basement floors.

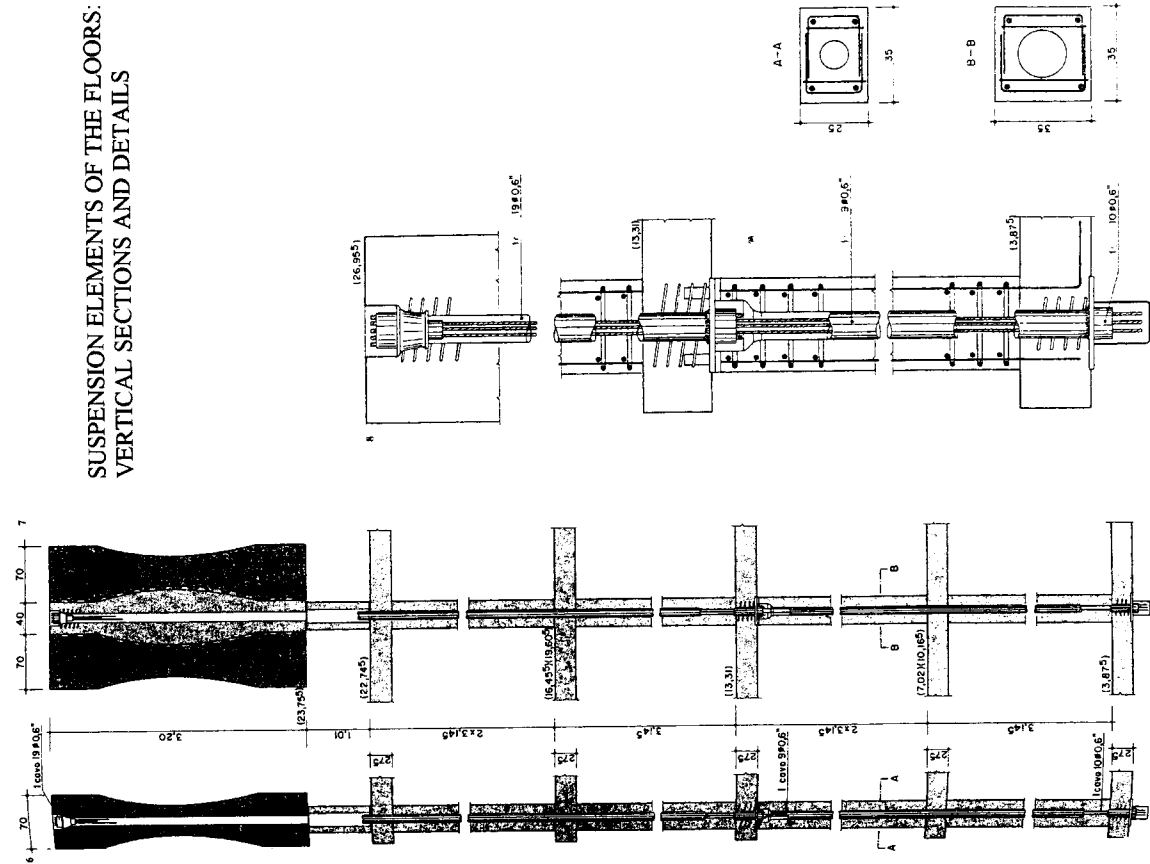
This construction method had the following advantages as well: the tower foundations worked out much of their settlement during construction, making it possible to use the tower structures for support of the basement floor decks.

A credit is given to the following for the wonderful system in which innovative technologies have been created: Customer: Seat-Leasing, division of STET, Turin; architectural design: Arch. Plinio Danieli, Arch. Giovanni Trevisan, Mestre; structural design: Ing. Gian Carlo Giuliani of Redesco Srl, Milan; equipment design: Ariatta and Harasser Engineers, of Copresit Srl, Milan; contracts management: Edilpro (Italstat), Rome; general contractors: Ediltransappennina, Rome; Adanti e Solazzi, Bologna; Italedil Rome; Pontello, Florence; structure construction: Edim Spa, Milan, Edilfornacai, Bologna; Frabbroni, Bologna; general supervision of construction: Ing. Raffaello Rizzo, Rome; structures sup. const.: Ing. Gian Carlo Giuliani, Milan; artistic direction: Arch. Giovanni Trevisan, Mestre.

PRECAST PRESTRESSED CONCRETE ROOF BEAMS:
GENERAL DIMENSIONS, REINFORCING AND PRESTRESSING STEEL



SUSPENSION ELEMENTS OF THE FLOORS:
VERTICAL SECTIONS AND DETAILS



VIII.2.3.2. The headquarter building at Bologna

VIII.2.3.2.1. *Universo Assicurazioni Building*

The new 'Universo Assicurazioni' headquarters is located in the 'Pilastro' residential complex nearby the San Donato exit of the Bologna inner ring road. The structure chosen for the 'high' main building, characterized by considerable symmetry and repetitiveness on finite levels, as well as by very fine finishings of its façades, was wholly precast, and shearbraced by the elevator–stair cores and by vertical systems conduits placed at the building ends, the only in-situ cast elements.

A reference is made to the building elevations shown on Sheet No. VIII.2.37. The plan sections and reinforced concrete details of a typical floor are shown on Sheet No. VIII.2.38. The floor deck is connected by shear bracings. The beam is supported by five reinforced concrete columns. The beam is fitted with a set of steel devices in its soffit and on its sides that would receive the cladding panels and the upper bearing structure.

The precast façade panels, bearing vertical loads exclusively, are hinged one to the other so as to form a 'pendular columns' scheme. On the long span precast prestressed-concrete floor structures (simply supported on the façade panels) a working reinforced concrete slab was poured for several reasons: to hold the chain-type reinforcing for the façades; to see to the vertical-load shears being transmitted between adjacent panels; and to form a stiff diaphragm by means of which the horizontal forces due to wind blowing on the façades would be transferred to the shear-bracing head-end cores.

To do all this the working slab is reinforced within its thickness like a wall beam 'bearing' on the head-end cores: stairwells and elevator shafts on the one side, systems conduits on the other.

The head-end shearbracing, in situ poured in steel forms ahead of the mounting of the precastings, are in fact complex brackets designed by means of Rosman-Beck's analysis, to take the wind thrust or, alternatively, to take a conventional horizontal force of 2% of the floor masses, by which account is probabilistically taken of precasting assembly errors.

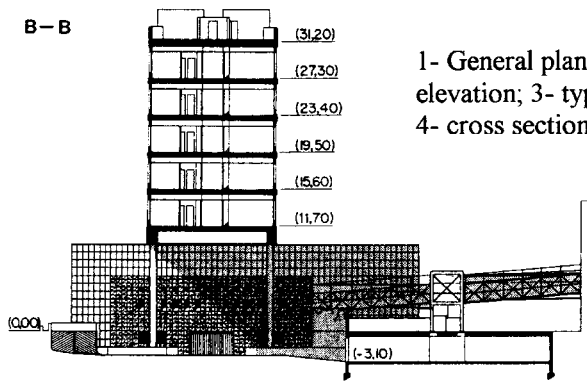
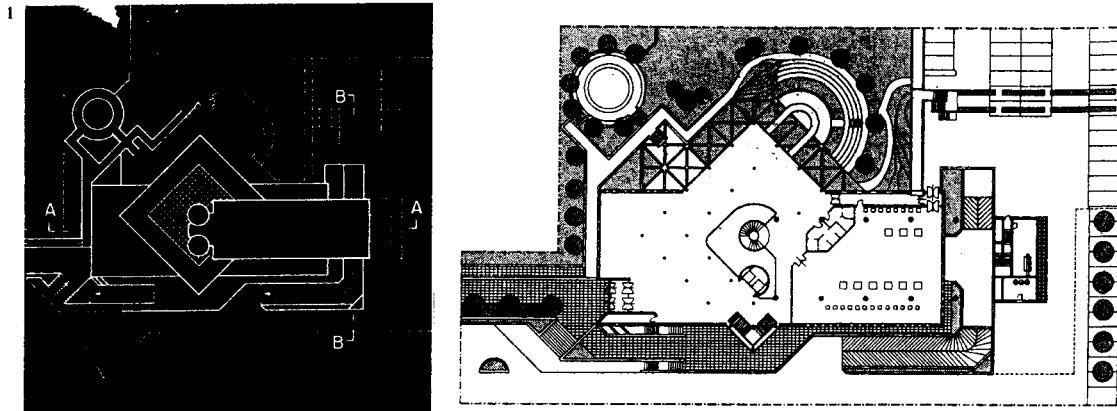
These complex brackets comprise pairs of cylindrical or box elements connected by working crosspieces that make them brackets subject to mixed bending and shear strains.

The foundations are footings bearing on augur-drilled piles, this being required by statics and systems reasons.

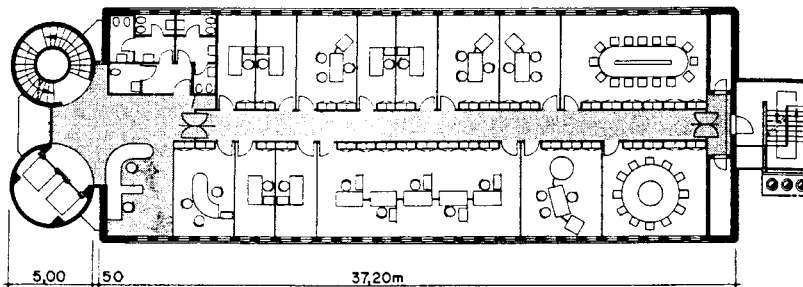
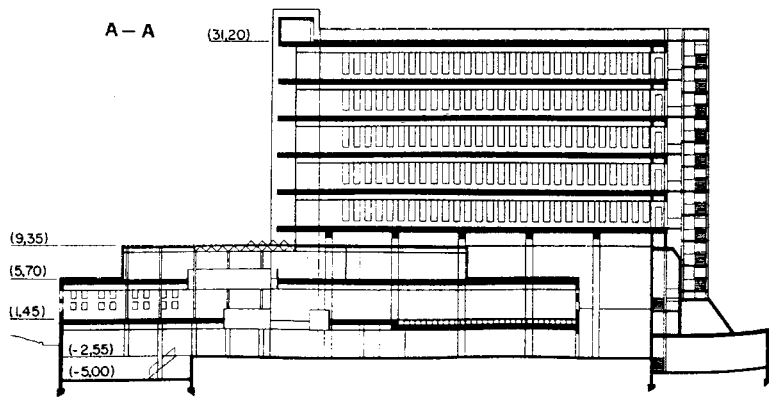
To build these members of fundamental importance to the stability of the whole precast building, a structure of 40 cm-deep plane prestressed hollow core panels of the extruded type, self-bearing over a 12.75 m span, was chosen, their considerable span being taken into account. The head ends of the deck plates, which bear on the panels over 10 cm with an interposed layer of felt and levelling mortar, were so shaped as to create the space needed to insert the reinforcing bars for the longitudinal chaining. These bars, with their stirrups that create the floor curbing, are also the reinforcing needed to take the bending stresses owing to the floor structure's wall-beam behaviour when loaded in its own plane by the wind.

PLANS AND ELEVATIONS FOR THE
UNIVERSO ASSICURAZIONI BUILDING

SHEET NO. VIII.2.37



1- General plan; 2- Ground floor plan, at 1.45 m elevation; 3- typical offices-floor plan; 4- cross section; 5- longitudinal section.

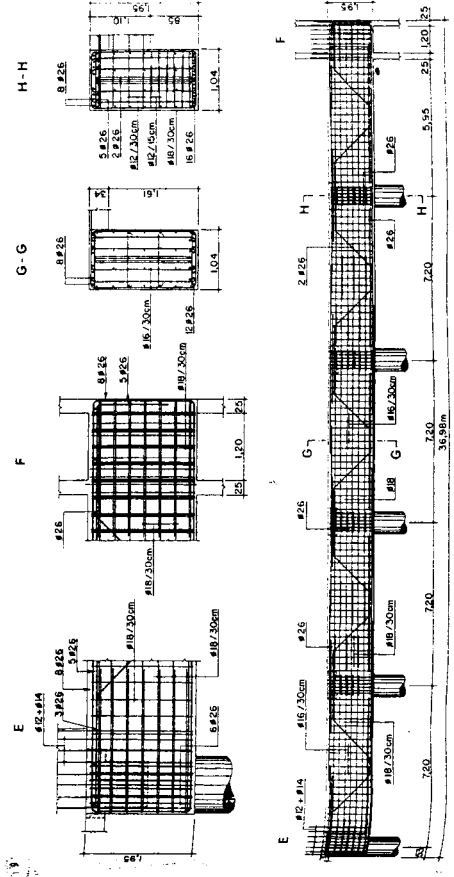
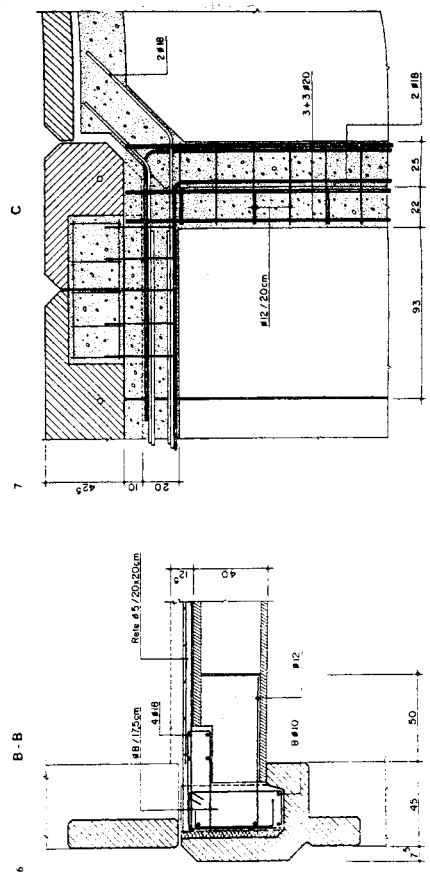
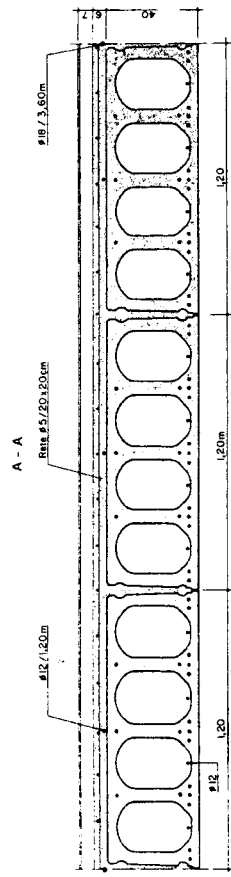
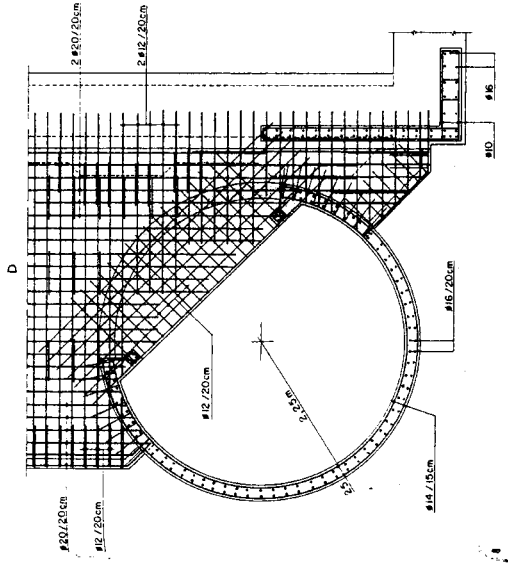
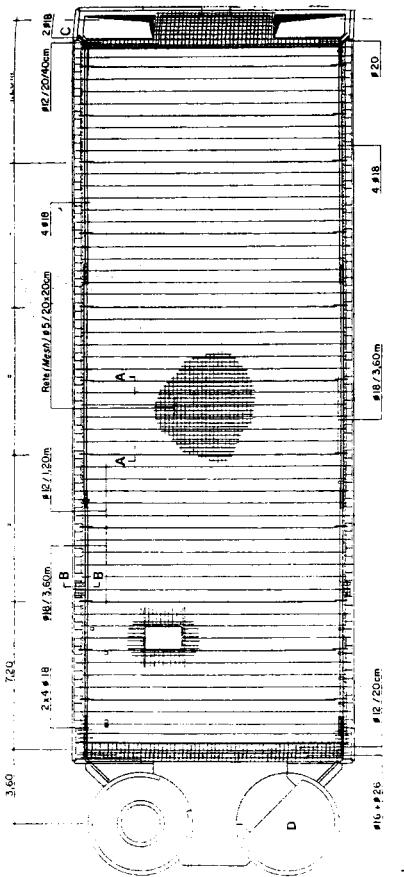


Customer: « Universo Assicurazioni SpA » Insurance company, of Bologna. Architectural design: Ettore Masi arch. and Dante Bersani arch., both of Thesis Srl, and Stefano Silvagni, of Technicoop, Bologna. Structural design: Ing. Stefano Tampieri. Final design and structural calculation: Ing. Sandro Zecchinelli and Ing. Stefano Tampieri, with Prof. Ing. Claudio Ceccoli consulting. Geotechnical consulting: Ing. Maurizio Merli and Ing. Gianfranco Marchi. Supervision of construction on behalf of customer: Ing. Mario Ciammitti, aided by Mirko Sughi jr. Contractor: Edilter S.c. a r.l., under the direction of Ing. Antonio Rignanese. The precast facade panels were manufactured by SPAV S.p.A. of Martignacco, Udine. The prestressed concrete floor elements were manufactured by Generale Prefabbricati S.p.A. of Città della Pieve, Perugia.

PLANS, SECTIONS AND RC DETAILS OF A TYPICAL FLOOR

SHEET NO. VIII.2.38

THE FIRST OFFICES FLOOR: GENERAL DIMENSIONS AND REINFORCING STEEL
 4-5-6-7- Plan, sections and detail of the floor;
 8- Reinforcing steel at the deck's connection to a cylindrical shearbase; 9- Reinforcing steel of the edge main beam.



VIII.2.3.2.2. The façade load-bearing panels (Sheet No. VIII.2.39)

The typical panel is made by combining three constant cross-section (40 cm × 45 cm) columns 3·80 m high by means of two horizontal members.

The upper of these members acts as lintel and sheet strut for the windowing (of dimensions 80 cm × 220 cm), while also creating the seat for the curbing for the floor, and the lower takes over the function of low spandrel, creating a volume between the columns and making it possible to insert healing A/C elements.

The panels are connected together above and below by a dry system in such fashion as to bring the construction system into line with the static system (pendular column).

VIII.2.3.2.3. Quantities and materials used

The complex, which also features a single-floor underground garage on surface foundations and a network of pedestrian ways connecting to adjacent existing buildings, is characterized by the following numbers:

Lot area	7861 m ²
Total offices area	4660 m ²
Central systems area	1196 m ²
Total building volume	37 500 m ³

The following material quantities were used in building the complex:

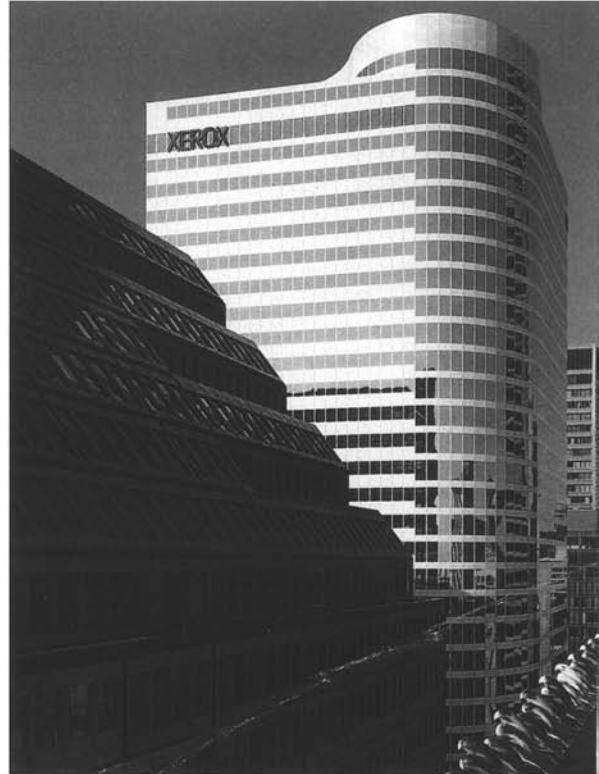
Excavation	20 000 m ³
Piles	2000 m ³
Reinforcing for concrete pours	450 000 kg
In-situ-cast concrete	4300 m ³
Prestressed concrete hollow core floor structures	2200 m ²
Predalles-type floor structures	2440 m ²
Floor structures using hollow clay brick and reinforced concrete	1255 m ²
Precast façade panels	2200 m ²

VIII.2.4. Case studies based on American practices

Sheet No. VIII.2.40(a) shows the Xerox Center, in Chicago, Illinois, USA, which has integrated concrete elements. The distinctive curved façade, which is clad in white-painted aluminium and reflective glass, is intended to represent a piece of paper tumbling out of a photocopier. The entire design is based on American codified methods and states' practices.

Sheet No. VIII.2.40(b) shows the tall Citicorp Center, also in Chicago. This building also has integrated concrete elements and white aluminium cladding. Because of its height, it has a tuned mass damper – a 400 ton block of concrete that moves on a thin layer of oil and which is hydraulically activated to limit the sway of the building during extreme loads and pressures. The building was designed by Hugh Stubbins and engineered by Le Messurier Consultants. The entire design is based on American practice.

CONCRETE BUILDINGS BASED ON AMERICAN PRACTICES SHEET NO. VIII.2.40



(a) Xerox Center, Chicago.

(b) Citicorp Center, Chicago.

VIII.3. Stadia, arenas and grandstands

VIII.3.1. Introduction

The market for specialist and multi-functional sports and leisure facilities to host international and regional sports, conferences and leisure events is growing fast on all continents. This section represents the structural detailing of such structures. Emphasis is placed on safety to spectators and customers. The glossary is given to aid the reader. Loads and stresses adopted in the constructed facilities are summarized. Two case studies are introduced. Structural details of the constructed facilities are given based on EC2 and European practices.

VIII.3.2. Glossary

Barrier	Any element of a sports ground, permanent or temporary, intended to prevent people from falling, and to retain, stop or guide people. Types of barriers used at sports grounds are further defined in Section 10.1 of the code.
Certifying authority	The local authority responsible for issuing a safety certificate under the Safety of Sports Grounds Act 1975 or the Fire Safety and Safety of Places of Sport Act 1987.
Circulation	Free movement of spectators within a sports ground.
Combustible	Able to burn.
Concourse	A circulation area providing direct access to and from spectator accommodation, via stairways, ramps, vomitories or level passageways, and serve as a milling area for spectators for the purposes of refreshment and entertainment, it may also provide direct access to toilet facilities.
Contingency plan	A contingency plan is prepared by the ground management setting out the action to be taken in response to incidents occurring at the venue which might prejudice public safety or disrupt normal operations (for example, the loss of power to CCTV or PA systems).
Control point	A designated room or area within the sports ground from which the safety management structure is controlled and operated. Also known as a 'match control', 'event control' or 'stadium control' room.
Crush barrier	A barrier which protects spectators from crushing, positioned in areas of standing accommodation.

Datum	The finished level of the floor, seat row, terrace, ramp, landing, pitch line of stairs, or, in the case of barriers behind seats, the seat level.
Emergency plan	An emergency plan is prepared and owned by the emergency services for dealing with an event at the venue or in the vicinity (for example, a major fire or bomb alert). Also known as an emergency procedure plan, or major incident plan.
Exit	A stairway, gangway, passageway, ramp, gateway, door, and all other means of passage used to leave the sports ground and its accommodation.
Exit system	A set of different types of exits, linked to form a means of passage of spectators.
Fire resistance	Ability of a component of a building to resist fire for a stated period of time, when subjected to an appropriate test in accordance with the current relevant British Standard.
First aid fire-fighting equipment	Equipment, such as portable extinguishers, fire blankets, buckets of sand and hose reel equipment, intended for use by safety staff or employees prior to the arrival of the fire brigade.
First aider	A person who holds the standard certificate of first aid issued by the voluntary aid societies to people working as first aiders under the Health and Safety (First Aid) Regulations 1981.
Handrail	A rail normally grasped by hand for guidance or support.
Horizontal imposed load	The load assumed to be produced by the intended use (usually of a barrier).
Landing	A level surface at the head, foot, or between flights of stairways or ramps.
Lateral gangway	Channel for the passage of spectators through viewing accommodation running parallel with terrace steps or seat rows.
(P) factor	The term used for the assessment of the physical condition of an area of viewing accommodation.
Pitch perimeter barrier	A barrier which separates spectators from the pitch or area of activity.
Pitch perimeter fence	A barrier higher than 1.1 m, which separates spectators from the playing area or area of activity.
Place of comparative safety	A place where people can be safe from the effects of fire for 30 minutes or more, thus allowing extra time for them to move directly to a place of safety.
Place of safety	A place where a person is no longer in danger from fire or other emergencies.

Radial gangway	Channel for the passage of spectators through viewing accommodation, running with the slope between terrace steps or seat rows. For the purposes of design and assessment, the criteria applying to radial gangways may be different from those pertaining to stairways.
Ramp	An inclined, surface linking two areas at different elevations.
Rate of passage	The number of persons per metre width per minute passing through an element of an exit system.
Robustness	The capability of a structure to withstand some misuse and to tolerate accidental damage without catastrophic consequences.
(S) factor	The term used for the assessment of the safety management of an area of viewing accommodation.
Side gangway	Channel for the forward passage of spectators between an end row of seats and a protective barrier at the edge of a structure.
Sightline	The ability of a spectator to see a predetermined point of focus (such as the nearest touchline or outside lane of a running track) over the top of the head of the spectators sitting immediately in front.
Spectator accommodation	The area of a ground or structure in the ground provided for the use of spectators; including all circulation areas, concourses and the viewing accommodation.
Spectator gallery	A gallery, usually attached to a hospitality area, from which spectators can view the event.
Sports ground	Any place where sports or other competitive activities take place in the open air and where accommodation has been provided for spectators, consisting of artificial structures or of natural structures artificially modified for the purpose.
Stairway	That part of a structure which is not a radial gangway but which comprises at least one flight of steps, including the landings at the head and foot of steps and any landing in-between flights.
Stand	A structure providing viewing accommodation for spectators.
Temporary demountable structure	Any temporary structure erected on a temporary basis at a ground, including stands, standing area, marquees and media installations.
Terrace	An area of steps providing standing accommodation for spectators.

View slope	A non-stepped, sloping area providing standing accommodation for spectators.
Viewing standard	The quality of view available to spectators, consisting of three elements: the sightlines, the presence of any restrictions to viewing, and the distance between spectators and the pitch or area of activity.
Vomitory	An access route built into the gradient of a stand which directly links spectator accommodation to concourses, and/or routes for ingress, egress or emergency evacuation.

VIII.3.3. Introduction to loads

Gravity loads include both dead and live loads, and lateral loads include wind and/or seismic loads, depending upon building site. In some cases, especially for buildings considered in the preliminary design phase.

Live loads are defined as fixed, non-movable loads of a permanent nature which can be divided into two categories: (1) self-weight of the structure and (2) superimposed dead loads.

Superimposed dead loads consist of partitions, hung ceilings, hung mechanical/electrical loads (e.g. sprinklers, lights, etc.), special floor fills and finishes, façade weight, and any other dead load which acts in addition to the weight of the structural elements. Many building codes stipulate that an allowance for partition loads equal to 1 kN/m^2 over the floor area must be considered.

Suspended ceiling weights and mechanical/electrical loadings vary from project to project, depending upon the type of occupancy and the type of structural system utilized, and usually range between 2 and 10 psi (100 and 500 N/m^2). Floor fills or other special floor finishes can result in significant dead loads.

The façade weight can vary significantly, depending upon the type of façade to be used, for example, glass curtain wall, precast concrete, masonry or stone. The weight of glass curtain wall systems usually ranges between 400 and 600 N/m^2 but the weight of precast or masonry façades can be 2 to 4 kN/m^2 or more.

Live loads are non-permanent in nature and vary depending upon the usage of the building floor area in question. For example, most building codes specify a minimum design live load of 2.4 kN for typical office areas. Increased live loads for special usage areas that are known at the time of preliminary design should be taken into account, such as lobbies, restaurants, mechanical equipment rooms, cooling towers, landscaped planting areas, computer rooms, and places of assembly.

Localized areas to be used for storage or heavy filing loads are often unknown at the time of preliminary design and, therefore, must be taken into account during final design or, as sometimes is necessary, during or after construction. Roof live loads, which will be a very small portion of total gravity load, include snow loads with due consideration given to drifting, for example, at vertical surfaces of parapets, penthouses, setbacks and adjacent structures.

Allowable reductions of live load in accordance with applicable building code provisions should be applied during the preliminary design phase.

For wind loads, seismic loads and other abnormal loads, including spectator loads, respective codes are consulted in specific countries.

VIII.3.3.1. Dynamic loads for dancing and jumping

In dynamic analysis it is often convenient to express the dancing loads as a Fourier series representing the variation of load with time as a series of sine functions. Any periodic loading can be decomposed into a combination of a constant load and several harmonics.

VIII.3.3.2. Synchronized dynamic loading

It is caused by activities, such as jumping and dancing, which are periodic and mainly dependent upon:

- (a) the static weight of the dancer(s) (G)
- (b) the period of the dancing load(s) (T_p)
- (c) the contact ratio (α), i.e. the ratio of the duration within each cycle when the load is in contact with the floor and the period of the dancing.

The load at any time (t) may be expressed as

$$F(t) = G \left[1 + \sum_{n=1}^{\infty} r_n \sin \left(\frac{2n\pi}{T_p} t + \varphi_n \right) \right]$$

where n is the harmonic being considered 1, 2, 3, . . . , γ_n is the dynamic load factor for the n th harmonic, and φ_n is the phase angle of n th harmonic.

The values of γ_n and φ_n are functions of the value of the contact ratio α , which is given in Table VIII.3.1.

Table VIII.3.1. Typical values of contact ratio for various activities

Activity	Contact ratio α
Pedestrian movement	0.667
Low-impact aerobics	
Rhythmic exercises	0.500
High-impact aerobics	
Normal jumping	0.333
High jumping	0.250

In practice for the evaluation of displacement and stresses, only the first few harmonics need be considered, as the structural response at higher values is generally not significant. It is generally sufficient to consider the first three harmonics for vertical loads and the first harmonic for horizontal loads. For the calculation of acceleration, additional harmonics need to be considered.

The resultant values of r_n and φ_n for a given period of dancing T_p or a jumping frequency ($1/T_p$) may be obtained from experiments. For individual loads the frequency range that should be considered is 1.5 Hz to 3.5 Hz and for larger groups 1.5 Hz to 2.8 Hz.

For a large group the load $F(t)$ calculated may be multiplied by 0.67 to allow for lack of perfect synchronization.

Vertical jumping also generates a horizontal load, which may be critical for some structures, e.g. temporary grandstands. A horizontal load of 10% of the vertical load should be considered.

The average walking rate is 2 Hz with a standard deviation of 0.175 Hz.

In the absence of available statistical values on human action, the following criteria shall be adopted for spatial structures.

VIII.3.3.2.1. Human actions (Table VIII.3.2)

- (a) 6 people average: 45 kN/m² (max. code value 5 kN/m²)
- (b) Curved seated for 6 people: 4 kN/m²
- (c) Guarding: 3.4 kN/m width of all stairs and loadings and crash barriers (3.4 to 6 kN/m)

Table VIII.3.2.

Activity	Dynamic load factor
Redistribution moment (normal)	1.3
High-impact jump	1.56
Normal jump	1.80
Highest jump	1.89

In addition to the above loadings, all fixings must carry a maximum force of 120 N for a frequency of 1 Hz. The maximum component of the vertical load must not be less than 5.3 kN/m² and the maximum horizontal component of the live loadings should not be less than 175 N/m².

The average walking rate is 2 Hz with a standard deviation of 0.175 Hz. The fundamental frequency associated with the above activity is related as:

$$f = KL^\alpha$$

where for:

Concrete	$K = 39$	$\alpha = -0.77$	$*\xi = -0.02$
Steel	$K = 35$	$\alpha = -0.73$	$*\xi = -0.0045$
Composite	$K = 42$	$\alpha = -0.84$	$*\xi = -0.006$

* ξ = the associated damping ratio.

VIII.3.4. Statistical data on loads on constructed facilities

This section deals with loads actually used on some of the prestigious spatial structures in the world. It now becomes necessary to acquaint the reader about the magnitudes of these loads and to compare them with the currently suggested loads.

Table VIII.3.3 gives a comparative study of loads for well-known spatial structures such as stadia or arena, etc. This table indicates a summary of various loads of constructed facilities.

Table VIII.3.3. Existing constructed facilities, loads and other parameters

Spacial structures, loads and parameters	Sports hall, Billingham, England	Transit store, Schipol, Holland	Ice stadium, Stockholm, Sweden	Madison Square Garden, New York, USA	Forum arena, California USA	Whorley reservoir, Durham, UK	The Palasport, Genova, Italy	Osaka Dome, Japan
1. Spacing (m)	3.35	4.70	5.25	12.3 with ring beam 9.75, deck 0.037	123 dia.	3.50	72 dia.	134 dia.
2. Length (m)	40	140	120	123 dia.	123 dia.	56.3	-	-
3. Design loads								
(a) Dead	0.40	0.5	0.42	0.72 and 1.73 with and without equipment	1.44	0.77	1.105	3.43
(b) Live/action	0.75	1.0	1.0	3.67	0.960	0.53	0.98	0.73
(c) Snow	0.75	0.5	1.0	1.44	0.960	0.77	0.98	0.985
(d) Wind	-0.6	-0.42	-0.40	-0.96	-0.75	-0.385	-1.470	-1.835
(e) Thermal (°C)	-20 to +20	-10 to +25	-20 to +20	-20 to +20	-20 to +20	-20 to +20	-20 to +20	-20 to +20
4. Natural period (s)	0.30	0.5	0.37	0.36	0.35	0.31	0.5	-

Load Combinations

- (1) I + II + III Long term
- (2) I + II + III + VI Long term
- (3) I + II + IV Short term
- (4) I + II + III + V Short term
- (5) I + II + III + V + VI Short term

where:

- I Vertical load (dead weight only)
- II Vertical load (Live, imposed, other actions)
- III Thermal load $\pm 15^{\circ}\text{C}$
- IV Thermal load $\pm 30^{\circ}\text{C}$
- V Earthquake
- VI Wind

VIII.3.5. Case study 1—Olympic stadium at San Pedrosula, Honduras, based on EC2, and Italian codified methods

Honduras' new Olympic stadium, (Sheet No. VIII.3.1) built in the suburbs on the outskirts of the city of San Pedro Sula, was expressly conceived to be the theatre for the latest Central American Olympic games. Built to hold the principal soccer matches, it can, as need dictates, also hold road and track athletic events or other team competitions.

The stadium can seat 40 000 spectators (Sheet No. VIII.3.2) of which 35 000 are in the grandstands and along the curves, while 5000 are ensconced in the so-called skyboxes, which are a sort of roofed gallery placed above the grandstands, where room is made for small private boxes. Owing to the building's special nature, great attention was paid to the definition of the visibility curve, it being so formed as to assure the greatest visual comfort.

More than 7000 prestressed-concrete precastings were used in the construction of this structure, among the most important of its kind in Central America. An enormous variety of forms and dimensions were cast in the production plants: columns, A and H members, inclined rack beams, main girders, I beams and brackets.

The structure is broken down into six blocks, separated by as many 'cuts' that are in substance six two-metre-wide expansion joints. The length of the straight-line segments is 87·60 m while the longer of the two curved segments measures 172·40 m.

The objective of the design approach was to dilate as far as possible the length of the structural spans, while keeping precasting weights within the capacities of the machines available for hoisting and mounting them in place.

The grandstand is supported by two main girders arranged longitudinally at an inclination $\Delta Y/\Delta X$ of 1:2. Radially, the tiers are supported by beams having the characteristic 'rack' profile. These beams are structurally connected by hinges, both to the frame that holds up the four floor structures of the so-called skyboxes and to the foundation shelf beam—see Sheet No. VIII.3.3.

The building makes use of a wide variety of special beams, needed to deal with the special nature of the stresses induced. For example, the partial stresses directed in the horizontal plane and acting on the frame of the skyboxes are transferred to the ground in the radial direction by the rack beams, while in the longitudinal direction horizontal stability is assured by a shearbracing system created by two crossed steel members and by a definite distribution of the stairbodies.

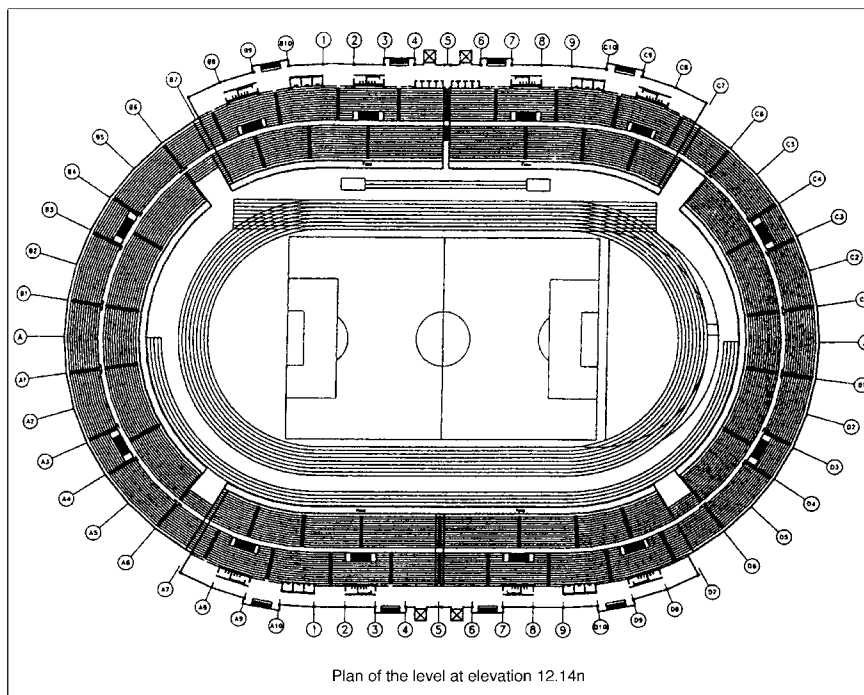
The structural frame that supports the skyboxes consists of a series of pairs of rectangular-section columns that run from deck to deck. The last of these is hollow to reduce the weight of that column from the 4·8 t of the full ones to 3·3 t.

The skybox decks consist of a series of precastings having a typical section made up of two Ts juxtaposed and solidized with a working pour 5 cm deep, which brings total deck depth to 51 cm.

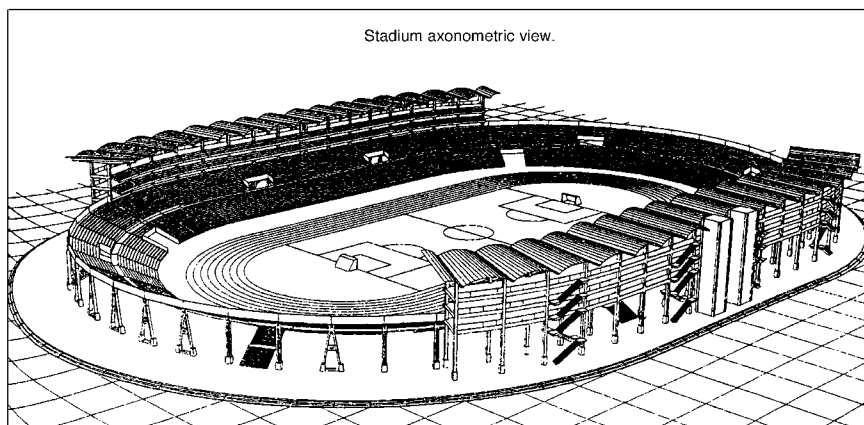
The elements constituting the deck bear on partly prestressed radial inverted-T-section beams, connected to the rectangular section columns mentioned earlier. These single-span beams are rigidly connected to the columns through a complex system carefully concealed from view. To stiffen the frame in the other direction, the outside and the inside perimeters of each span are underscored with a slender beam of length between 10·30 and 11·60 m (to which are then anchored the post-tensioned cables of the shelf girders that hold up the tiers).

**OLYMPIC STADIUM
SAN PEDROSULA — HONDURAS**

SHEET NO. VIII.3.1



2



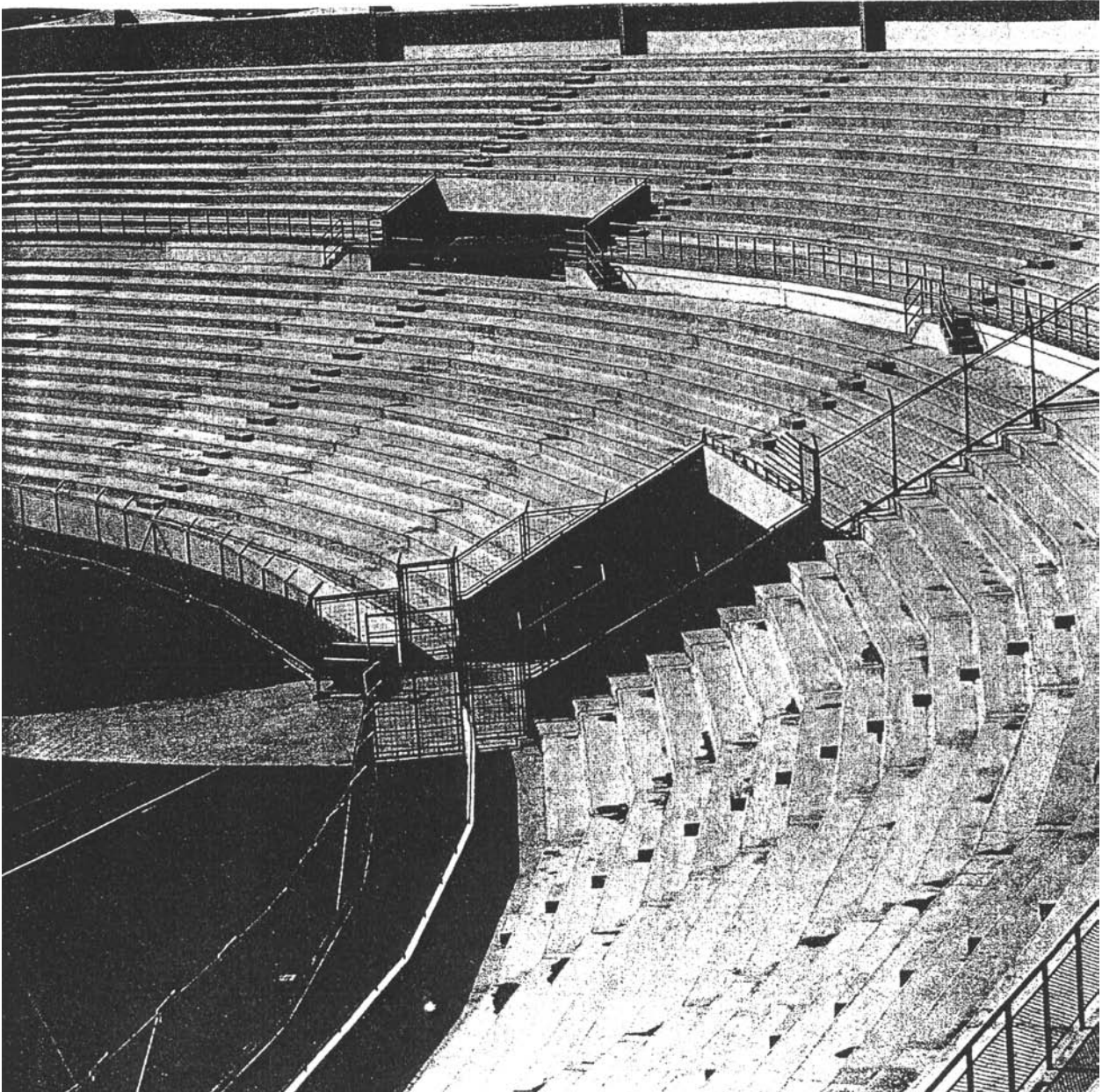
3

Design: Li Zhenqiang, Chief Engineer of CONHSA-PAYSA Group, San Pedro Sula, Honduras; Rigoberto Ramirez C., Structural Engineer of CONHSA-PAYSA Group.

Collaborators: Ricardo Cardenas, Arch. (Project Installation), Otto Umana Crespo, C.E. (Prefabrication Plant), Hector Bustillo, C.E. (Quality Control Department), Jerry Moran, C.E. (Structural Engineer), Elio Brizzio, C.E. (Costs, Supervision), David Leiva, C.E. (Computer Drafting).

**STADIUM GRANDSTAND
(SEATING ARRANGEMENTS)**

SHEET NO. VIII.3.2



A total of 72 precast shelf girders, shaped to receive the pre-castings composing the tiers, are connected to the second and third decks of the skyboxes. These are an inverted T-section and composite section members whose upper ends are in-situ poured and post-tensioned, to prevent surface cracking and to lengthen the structure's life.

The post-tensioning cables were assembled on strands and sheathed in lubricated polyethylene pipes. Then they were anchored by means of dead-end anchorages to the upper flanges of the frame beams of the skyboxes set along the CC perimeter. After the strands were tensioned, a Dywidag-type cable-blocking system was used. The system of the grandstands, which run along the entire ring of the immense sports area, comprises individual straight-line elements that are connected to the rack beams, in fashion similar to the connection of the brackets to the frame of the skyboxes, described above.

Each of the 12 stairbodies of the first group is made up of wholly precast elements: three rectangular columns, six inclined beams, and landings and steps, as too are the foundation structures themselves, consisting of precast prestressed-concrete piles. The design adopted a system of inclined beams, on which the precast steps bear, and a system of rectangular section beams, arranged in the other direction and completed in its upper part by an in-situ pour. The rectangular beams are connected to the columns that compose the structural frame, while the completion pour, made in-situ, offers the advantage of reducing the beam dimensions and of obviating the need to allow for dimensional tolerances, a certainly more sensitive problem for the stairbodies.

VIII.3.6. Case study 2

The project for the construction of the new Helsinki Hall of Sport got its start when, during the second half of the 1990s, the possibility was discerned of being able to host the world championship ice hockey playoffs. With such an objective, a strong will developed among the people of Finland to achieve this aim one way or another.

The vertical structures were the reinforced concrete: columns and bearing septums for the stairwells, while the horizontal structures included lightened decks 32 cm deep with their interiors emptied, and full-section slabs 6–8 cm deep, as well as prestressed concrete plates, all held up by prestressed concrete beams bearing on traditional reinforced concrete brackets provided at the column heads.

The tiers for the stands are made up of L-shaped prestressed concrete elements; each piece is 6–12 m long and is shaped to hold the seats.

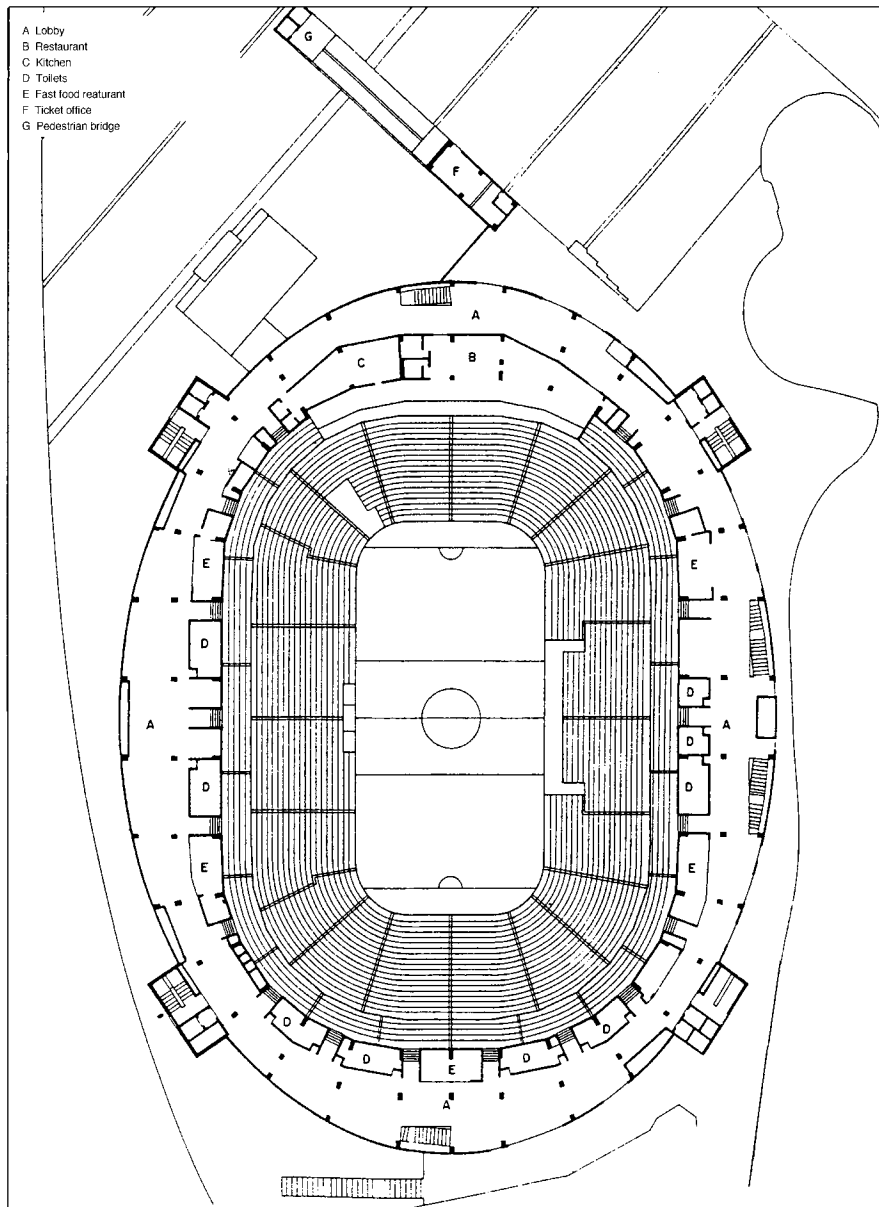
The façades were clad with glazing filtered by perforated steel panels, while the roof, built using wooden panels and insulation material, bears on a steel frame sustained by reinforced concrete beams and columns.

Since the arena is of ellipsoidal shape (Sheet No. VIII.3.4) it was necessary for the design team to perform the design of the precastings and the parts needed to stiffen the structure. No cast-in-situ operations have been carried out. Everywhere precasting was adopted. At each quarter of the arena a large torsion resisting box is provided with a reinforcing frame along each expansion joint in order to provide additional stiffening to the structure.

Rack type beams are provided to sustain the grandstands. Slender columns are provided as shown in Sheet No. VIII.3.5 with brackets to support the longitudinal and transverse beams. A lightweight frame is positioned for the construction of the arena roof (Sheet No. VIII.3.4) on the perimetral columns. The reinforced concrete supporting septum bodies (Sheet No. VIII.3.6) are placed symmetrically along the outside perimeter of the stands

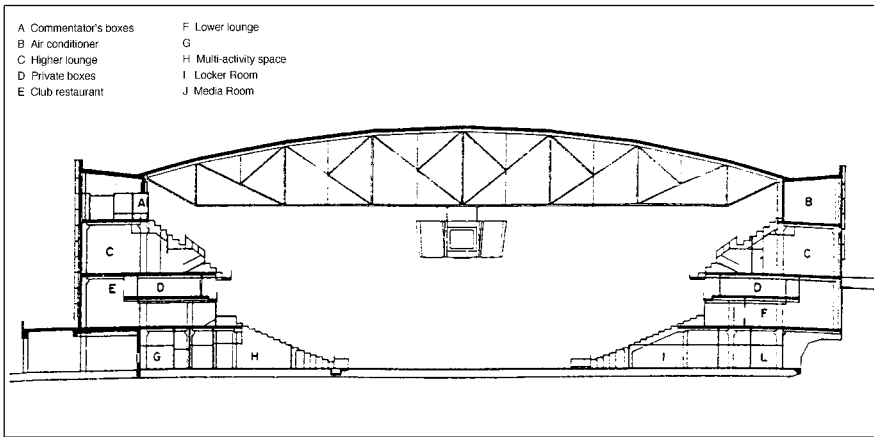
PLAN AND SECTION OF THE ARENA

SHEET NO. VIII.3.4



PLAN AND SECTION OF THE ARENA

SHEET NO. VIII.3.4 (Cont.)



Architects: Arkkitehtitoimisto
 Kontio-Kilpiä Valjento Oy.

Structural Engineers:
 HN-Suunnittelu Oy.
 Addtek Research & Development:
 Oy AB.

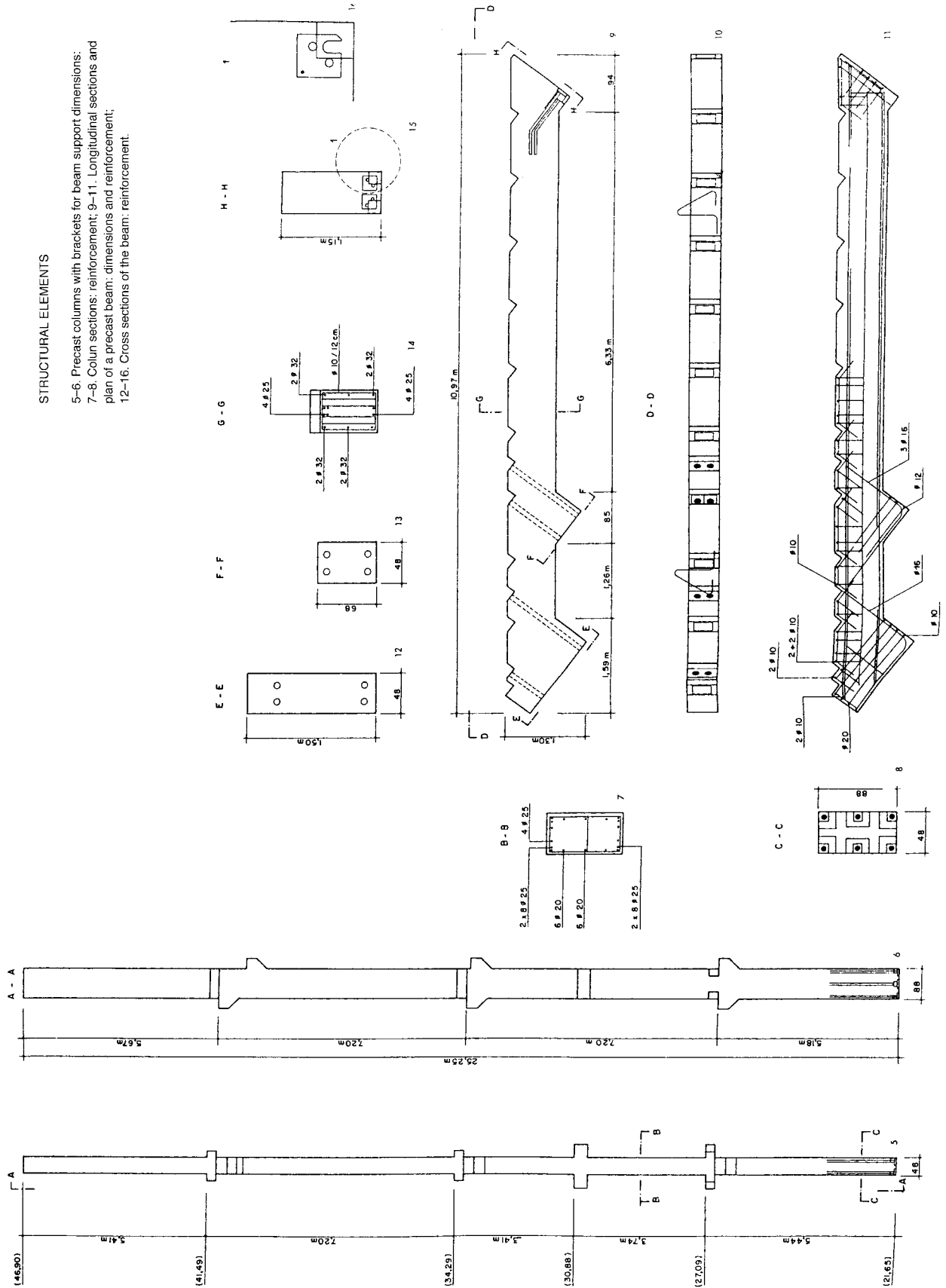
1. Plan of the facility
 2. Section through the arena

REINFORCEMENT DETAILS OF STRUCTURAL ELEMENTS

SHEET NO. VIII.3.5

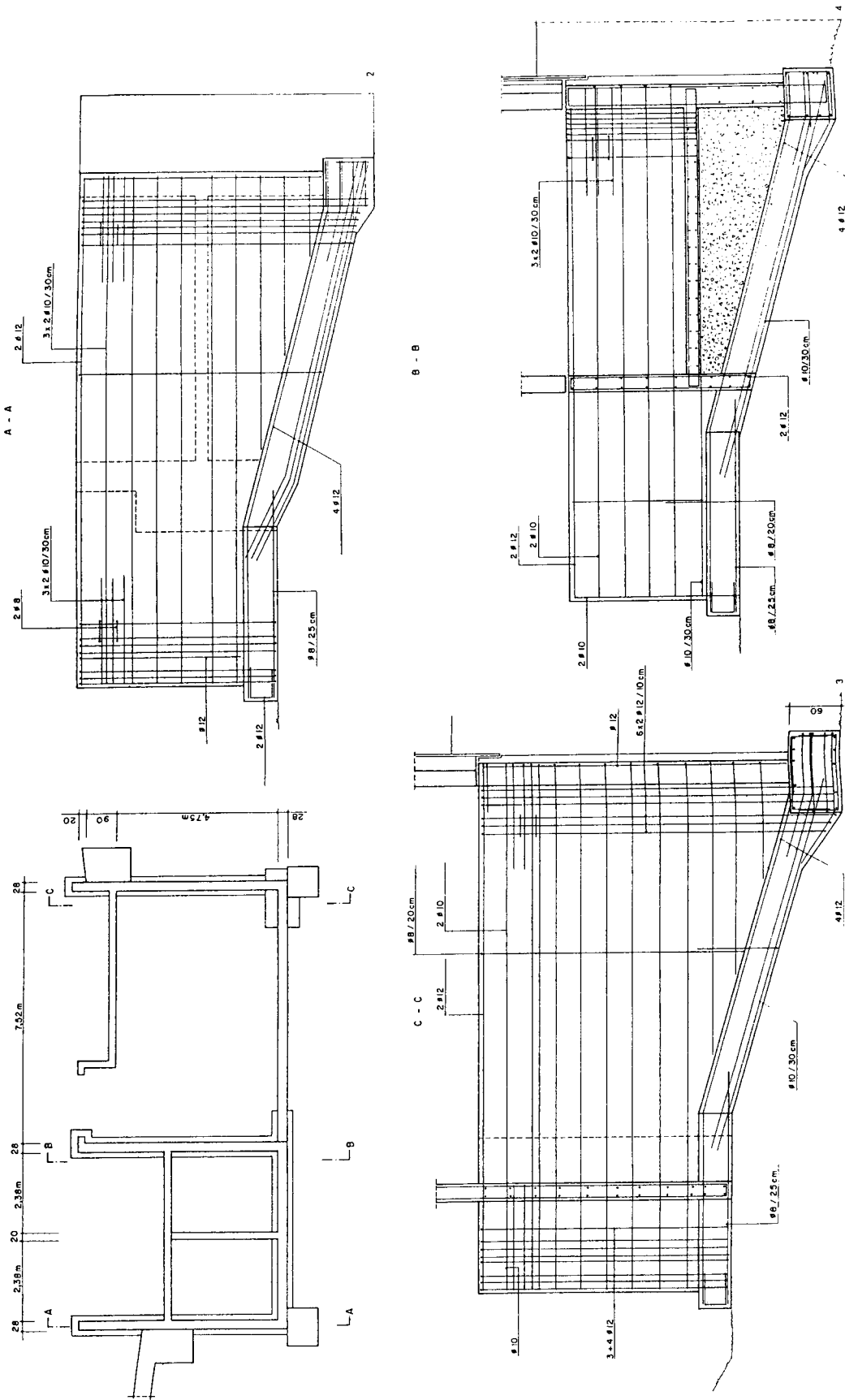
STRUCTURAL ELEMENTS

5-6. Precast columns with brackets for beam support dimensions:
 7-8. Column sections: reinforcement: 9-11. Longitudinal sections and
 plan of a precast beam: dimensions and reinforcement;
 12-16. Cross sections of the beam: reinforcement.



STRUCTURAL DETAILS OF SUPPORTING STRUCTURES

SHEET NO. VIII.3.6



which hold the staircases, the elevator and other services. During the design and drafting, a typical CAD system is used throughout for the Hartwell Arena. The frame and the roof were built at the same time. The entire precast structure was assembled in 14 weeks. The entire building was completed in 14 months.

VIII.3.6.1. Additional details

- (a) Occupying area: 34 500 m²
- (b) Total no. seats: 14 000
- (c) Basement parking capacity: 1500 cars
- (d) Station for the public means of transport

Data provided by the following.

Customer: Helsinki-Halli Oy.

Architectural design: Arkkitehtitoimisto Kontio-Kilpiä-Valjento Oy, Finland.

Structural design: HN-Suunnittelu Oy.

Design and construction of the reinforced concrete structural frame: ADDTEK

Research and development: Oy Ab, Finland.

Reinforced concrete precasting: Parma Betonila Oy.

VIII.3.6.2. Special references

- Allen, D. E. (1990). Building vibrations from human activities. *Concrete Int.*, **12**, No. 6, 66–73.
- Embrahimpour, A. and Sack, R. L. (1992). Design live loads for coherent crowd harmonic movements. *J. Struct. Engrg., ASCE*, **118**, No. 4, 1121–1136.
- Greimann, L. F. and Klaiber, F. W. (1978). Dynamic forces induced by spectators. *J. Struct. Div., ASCE*, **104**, No. 2, 348–351.
- National Research Council of Canada (1990). *Commentary A: serviceability criteria for deflections and vibrations*. Supplement to the National Building Code of Canada, National Research Council of Canada, Ottawa, Canada.
- Tuan, C. Y. and Saul, W. E. (1985). Loads due to spectator movement. *J. Struct. Engrg., ASCE*, **111**, No. 2, 418–434.

VIII.3.7. Case study 3

The Croke Park stadium in Dublin, Ireland, was designed to accommodate in excess of 84,000 spectators, in a modern environment and with a particular emphasis on safety. The three fundamental elements that influenced the stadium design were; crowd movement, viewing distance and sight lines.

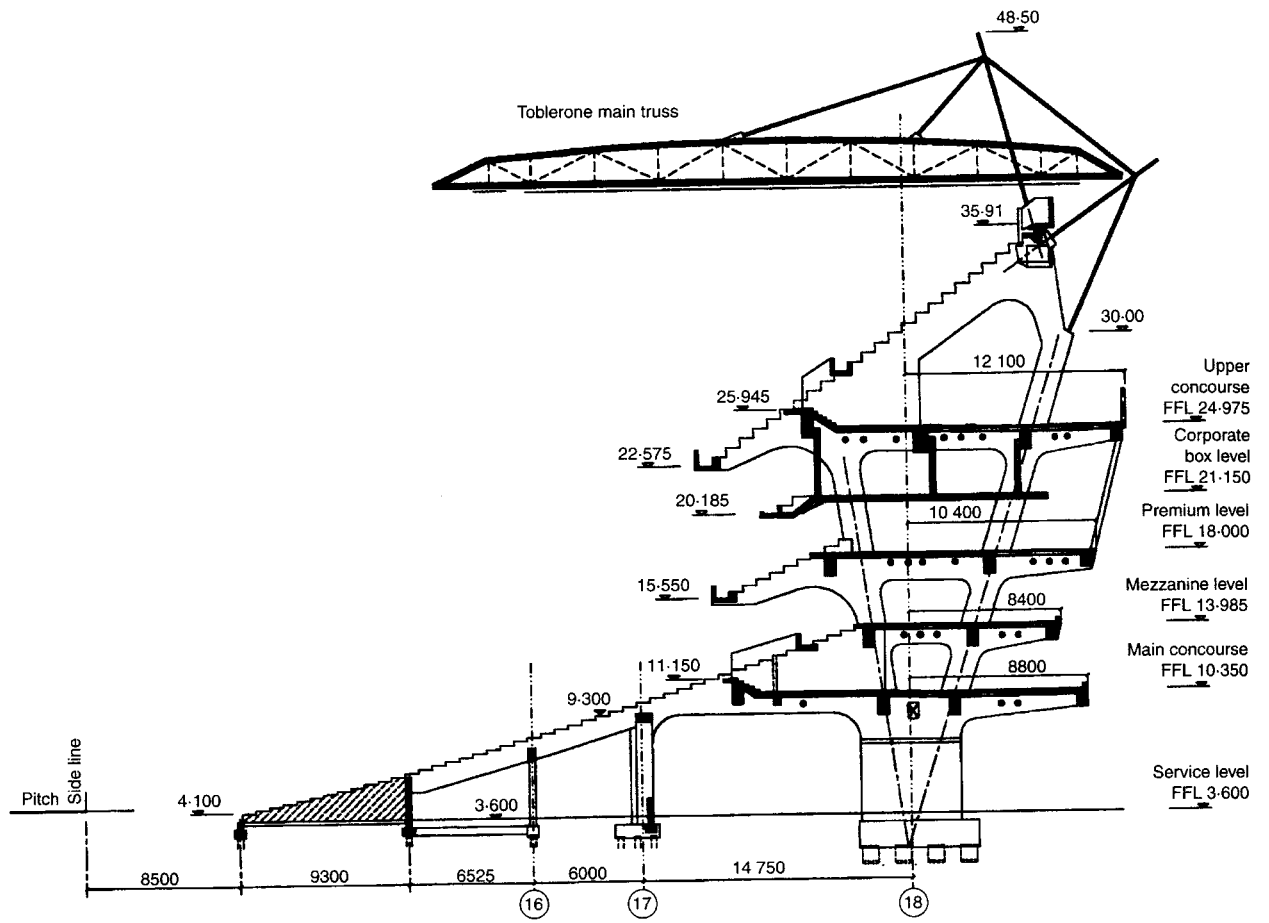
The structural solution adopted was termed the Y-frame (Sheet No VIII.3.7). The frame, set on a 14.35 m grid, consists of a single stalk main column at ground level, which carries substantial loads of approximately 3000 t. Its stability is assisted by a secondary column, on the pitch side of the stand, linking back by beams at main concourse level. The main column then splits at main concourse level and branches to form a finely balanced V-structure, with two legs tied at each concourse level. Tie beams serve to support structure at the spectator levels and act as longitudinal stabilizing elements. The beams are continuous over four spans and have sufficient fixity at the frames to take moments due to horizontal effects.

Precast seating units are approximately 7 m and 14 m in length and vary in depth between 450 mm and 900 mm. The tread is a propped cantilever, with the unit in front providing the support mechanism. The roof structure is fabricated from circular hollow sections with a clear coverage to the spectator area. The main trusses are each supported by twin compression posts fixed to a complex concrete knuckle at the rear of the top deck of the stand. Tension members are tied to the rear leg of the Y-frame just above the upper concourse level.

Source: Murray, F. V. (2000). Croke Park redevelopment – stadium design in an urban context. *Proc. Instn Civ. Engrs Structs & Bldgs*, **140**, Nov, 345–353.

CROKE PARK STADIUM – Y-FRAME SECTION

SHEET NO. VIII.3.7



VIII.4. Water-retaining structures and silos

VIII.4.1. Water-retaining structures

VIII.4.1.1. General introduction

Water-retaining structures must be designed for serviceability, stability, flotation and settlement. For these structures, watertightness and durability are especially required. Cracking due to external loads and early thermal and shrinkage effects must be assumed as a major design criterion.

Tanks as water-retaining structures are situated on the ground, underground and above ground supported on towers. Ground tanks and reservoirs can be circular or rectangular in shape. They must withstand pressure from the water they contain. If such structures are constructed on or below ground they must withstand external forces due to lateral earth pressure, uplift due to water in the surrounding ground and the weight of the earth cover if provided. The roof may be of the beam and slab or the flat slab type. In the case of large tanks, domical or truncated conical roofs are used. The walls can be cantilevered, pinned at the top or fixed at the top. Where floor slabs are involved, a polythene sheeting or similar may be used between the ground slab and the sub-base and a neoprene or rubber strip or similar between the roof and the top of the wall.

In-serviceability requirements for cracking have been seen to depend on the exposure conditions. Some basic data are given below.

1. Class A: exposure (exposed to moist or to corrosive atmosphere or subject to wetting and drying), the crack width $w \leq 0.1$ mm.
2. Class B: exposed to continuous contact with liquid, the crack width $w \leq 0.2$ mm.
3. Class C: not exposed as severely as for either Class A or B, the crack width $w < 0.3$ mm.

The slab thickness in millimetres ranges between 200 and 500 plus (Table VIII.4.1).

Table VIII.4.1.

Wall thickness: mm	Layers	Bar type: mm	Partial joint spacing: m*		
			(1)	(2)	(3)
200	Single	MS 12–16	2.3–2.44	2.86–3.5	3.41–4.05
250	Double	MS 12	1.73	2.29	2.84
500	Double	MS 12, 16, 20	2.6–3.5	3–3.92	4.33
Over 500	Double	20	1.92	2.34	2.75

* (1) = no cracks; (2) = 0.1 mm cracks; (3) = 0.2 mm cracks

Where construction joints are used for:

all thickness less than 400 mm: (1) = 4.8 m, (2) = 5.3 m; (3) = 5.9 m

all thickness equal to or greater than 500 mm: (1) = 4.8 m; (2) = 5.3 m; (3) = 5.6 m

The recommended concrete grades are 25 and 30.

Jointing material may be joint fillers, water stops, joint sealants and joint stress inducers. All jointed material must be chosen by the structural engineer and must accommodate repeated movements in all weathers. It must never be soft in summer or brittle in winter. A reference is made in Section VI to types of watertight joints.

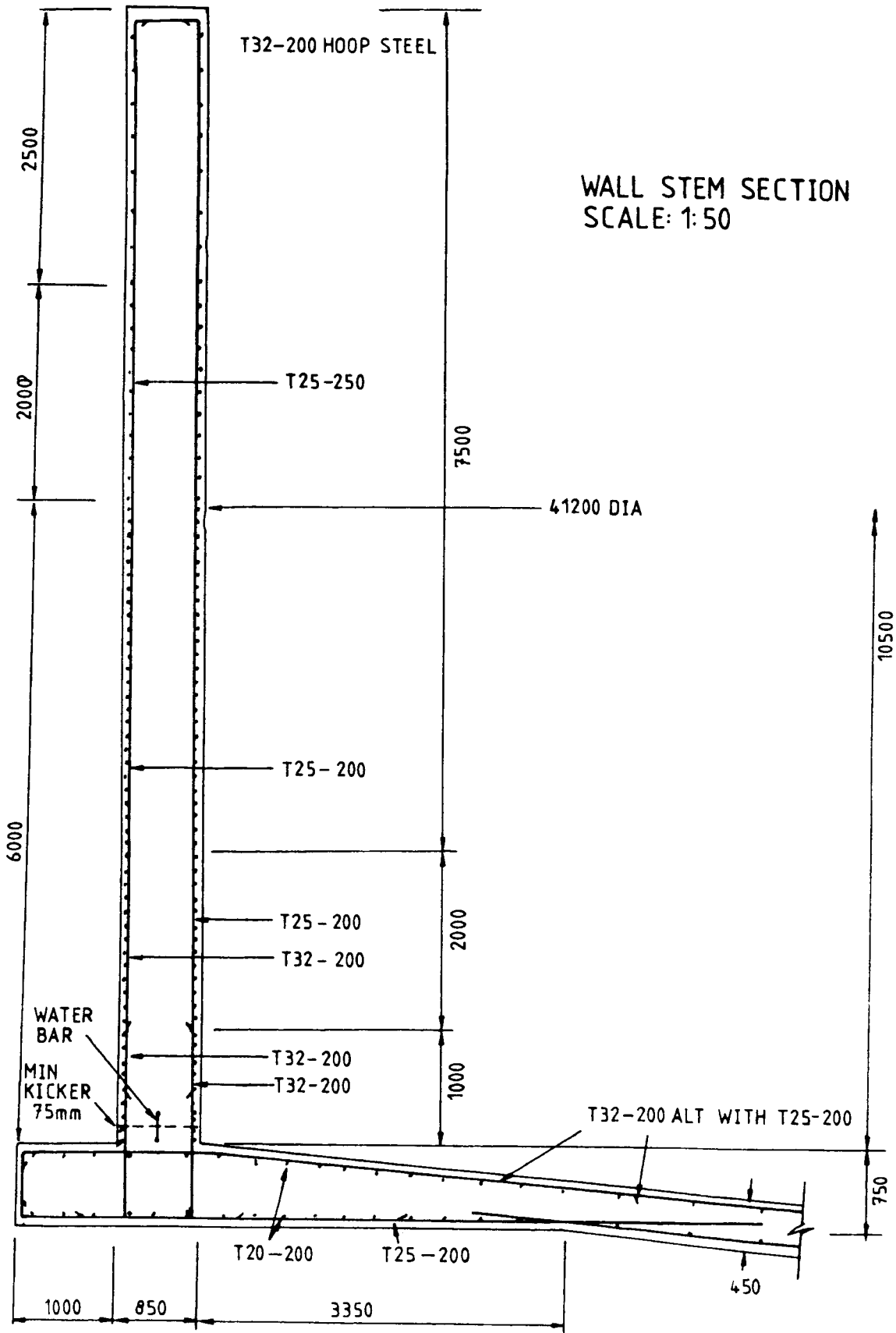
VIII.4.1.2. Typical structural detailing of water-retaining structures

Section VIII.4.2 gives some examples of water-retaining structures. They have been included in that section owing to the importance given to the shell surfaces and reinforcement layouts. Sheet No. VIII.4.1 shows the structural detailing of the wall stem section of a reinforced concrete tank of internal diameter 41.2 m and internal height 10.5 m. The floor slab is sloping towards the centre. It is placed underground. Sheet No. VIII.4.2 shows the sectional elevation of a reinforced concrete rectangular-shaped tank. A double domed INTZE tank is given on Sheet No. VIII.4.3. Two types of reinforcement details are given for two different INTZE tanks. These tanks can be supported on elevated towers. A typical substructure for such towers is given on Sheet No. VIII.4.4. Sometimes a circular tank is supported on towers with columns. These columns can be arranged in a number of ways in plan. Some layouts of the supporting towers are given on Sheet No. VIII.4.5.

Many other tanks with novel shapes have been designed. A bibliography is given for the reader to study such tanks and to follow the rules established in this book.

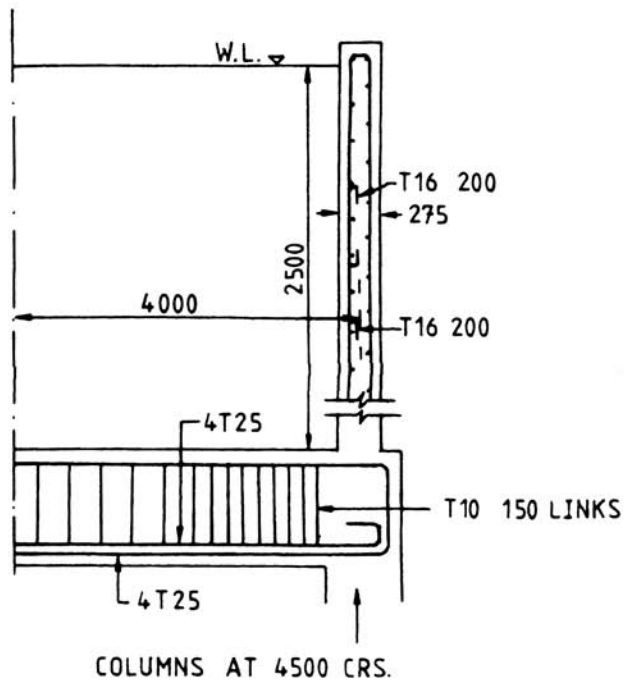
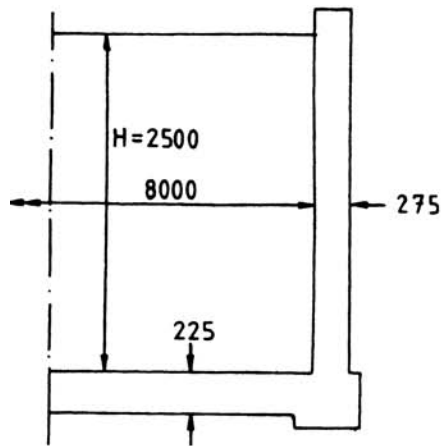
CIRCULAR TANK

SHEET NO. VIII.4.1



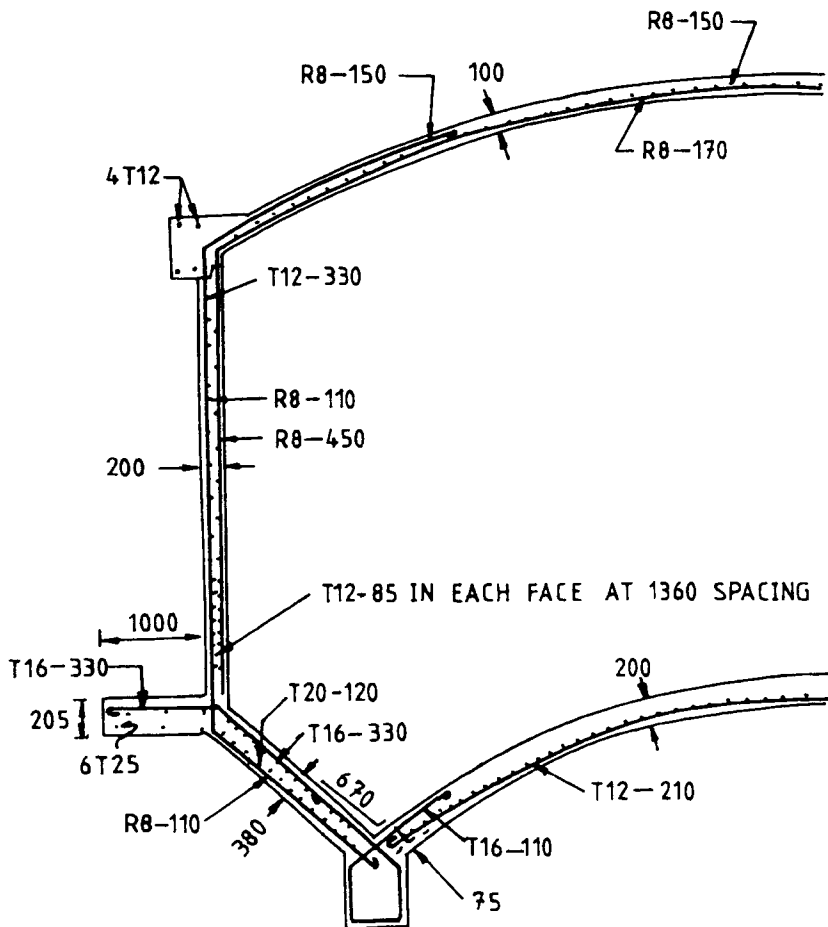
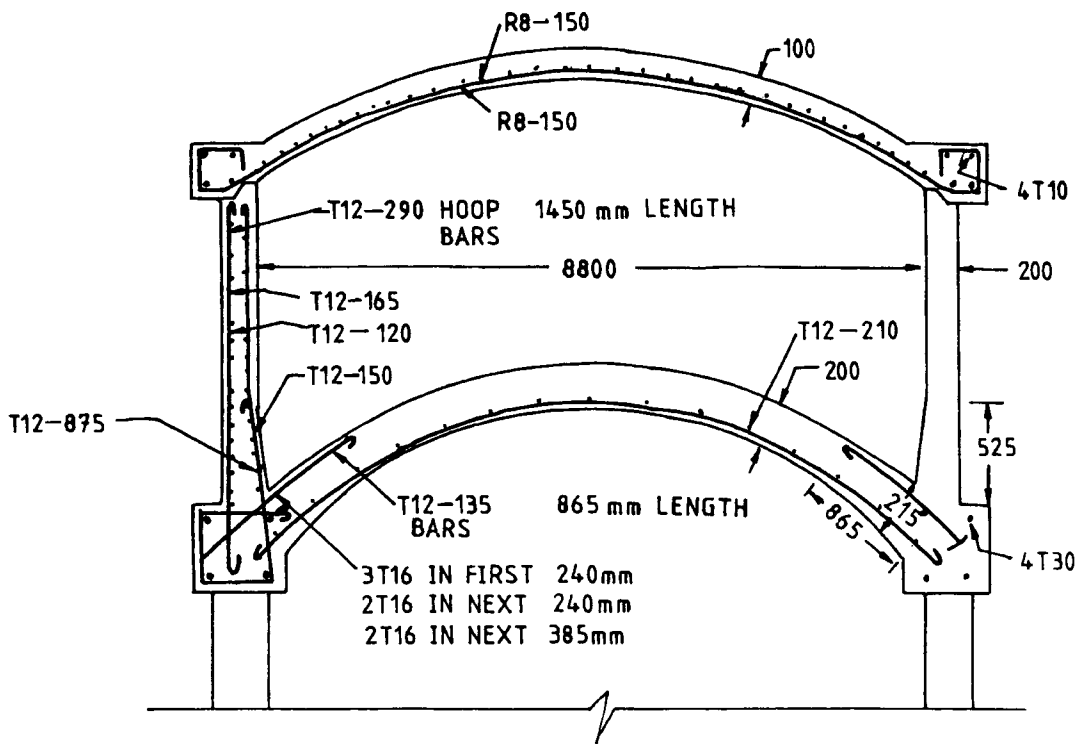
ELEVATED RECTANGULAR WATER TANK

SHEET NO. VIII.4.2



INTZE TANK

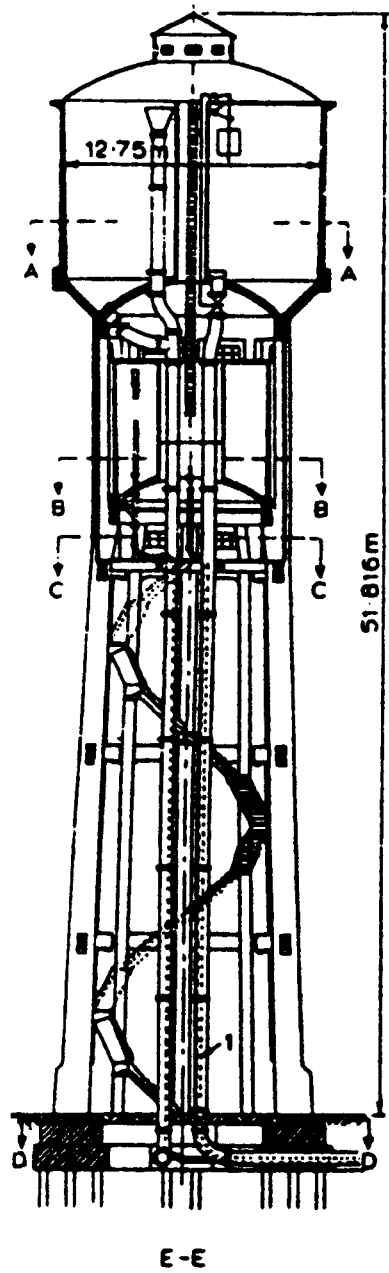
SHEET NO. VIII.4.3



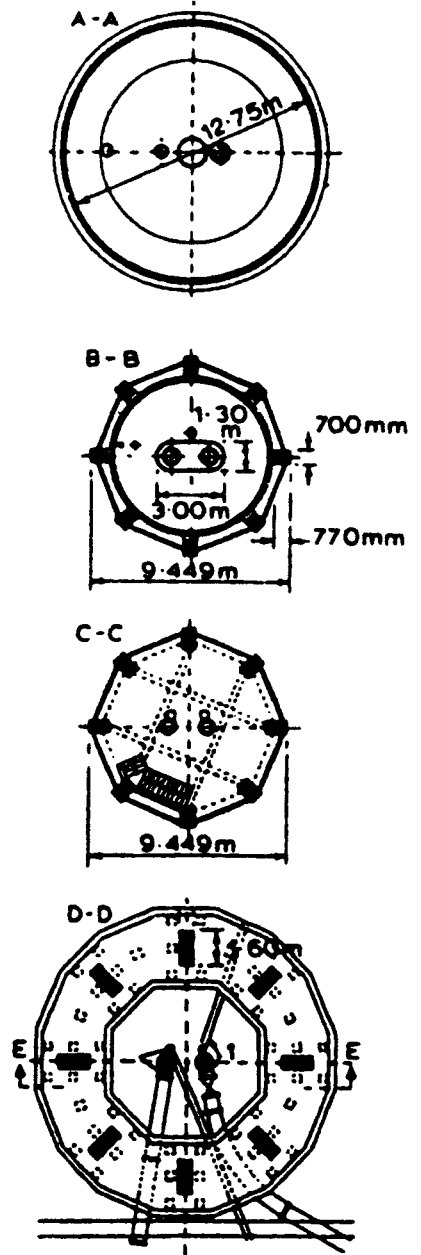
ELEVATED WATER TOWERS

SHEET NO. VIII.4.4

(a) Typical sectional elevation



(b) Cross-section

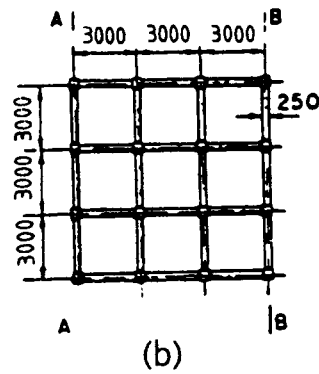
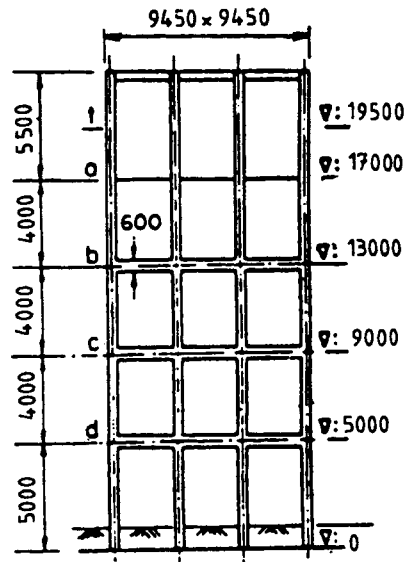
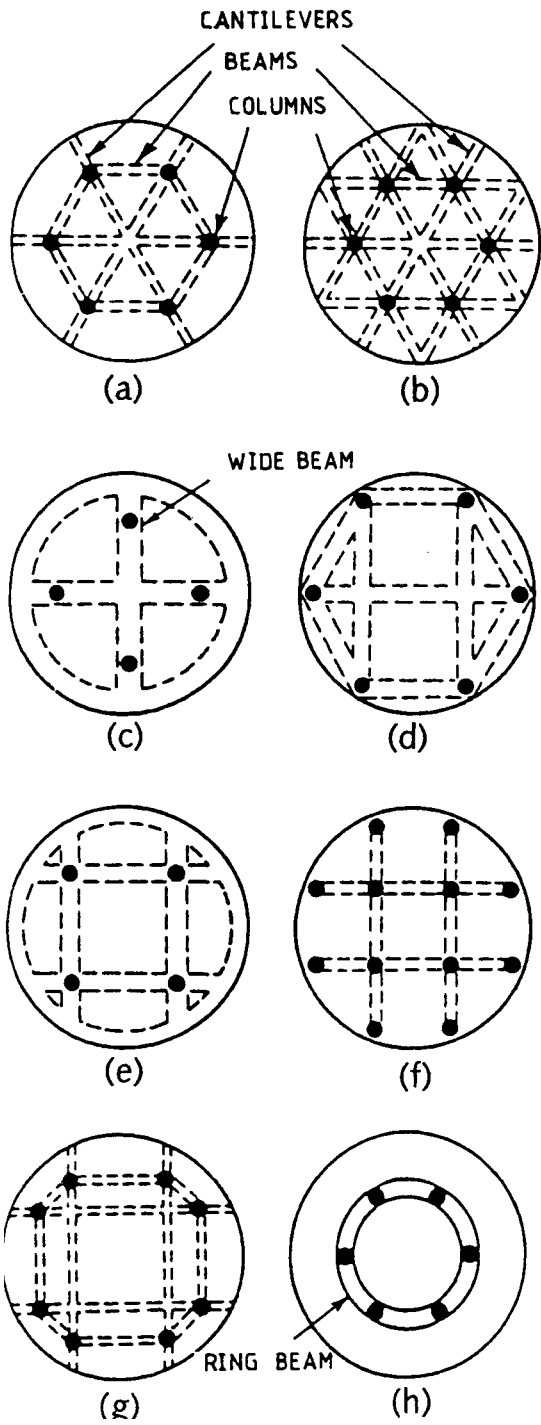


SUPPORTS FOR OVERHEAD TANKS

SHEET NO. VIII.4.5

(a) Plans

(b) Typical elevations



VIII.4.2. Silos**VIII.4.2.1. General introduction**

The irregularity of the crop yield and the uneven distribution of harvests can lead to a building-up of stocks which are best held in silos made in reinforced and prestressed concrete. They comprise tall cells of various cross-sections placed side by side. At the bottom they have discharge hoppers and at the top they are enclosed by a floor carrying the silo-filling equipment.

The silos have triangular, square, cylindrical or polygonal shapes. The walls of these silos are subject to maximum pressure due to the infilled or ensiled materials. The accurate values of these pressures depend on:

- (a) internal friction: grain upon grain
- (b) friction on smooth or rough walls
- (c) natural angle of repose of the material and the angle of material wall friction
- (d) bulk density of the material and the unconfined compressive strength of concrete.

A number of expressions have been developed to evaluate pressure on side walls and in the hopper area.

Hoppers are generally lined with stainless steel, mild steel and other types of abrasion-resistant materials. Various types of silo bottoms are used which include an elevated floor on columns, and a conical steel hopper attached to a concrete ring girder, integral with the silo wall or independently supported on columns or supported on pilasters.

VIII.4.2.2. Typical structural detailing of silos

A number of textbooks and proceedings have been published on the design of silos. In this section two examples are given for the structural detailing of silos. Sheet No. VIII.4.6 shows a sectional elevation of a reinforced concrete silo. This silo is supported on columns and is a part of a group of silos. The walls are 200 mm thick. The internal height (above the hopper line) and the internal height respectively are 35 m and 6 m. The hopper angle is 45°.

Horizontal reinforcement is provided to take the tension in each wall due to the pull of walls perpendicular to it. Owing to the continuity at corners, a horizontal bending moment is induced. For this bending moment it is sufficient to provide horizontal reinforcement equal to the vertical reinforcement of the vertical bending moment.

Generally this reinforcement is limited to the vertical reinforcement at one-third of the wall height. Horizontal reinforcement is also provided at the bottom of the wall to resist direct tension. In the case of walls spanning horizontally, direct tension is considered and the section is designed for combined axial load and bending. The following additional data will be considered in the design of this silo:

Coefficient of friction	0.35
Angle of repose	28°
Roof load	30 t
Density of material	890 kg/m ³ .

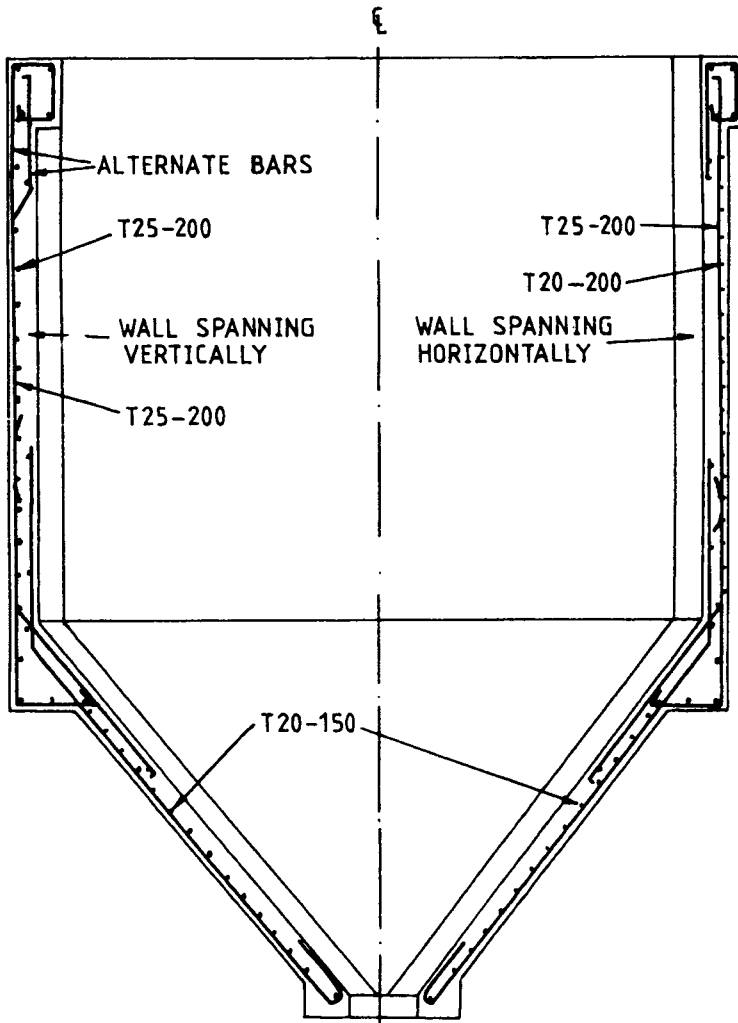
Sheet No. VIII.4.7 shows a prestressed concrete silo with its major dimensions. Circumferential tendons are introduced between buttresses to resist the hoop tension. The cables are anchored at buttresses using the VSL multi-strand prestressing system having 19 prestressing anchorages. A guaranteed minimum breaking force of 184 kN was adopted for each cable.

This silo has been designed on a wharf in Birkinhead, South Australia. Sheet No. VIII.4.8 shows a plan of a group of silos of the type shown on Sheet No. VIII.4.7. Two silos are interconnected using a 'cross' reinforcement. The connection is detailed on the lines suggested in the given section on Sheet No. VIII.4.8.

REINFORCED CONCRETE HOPPER

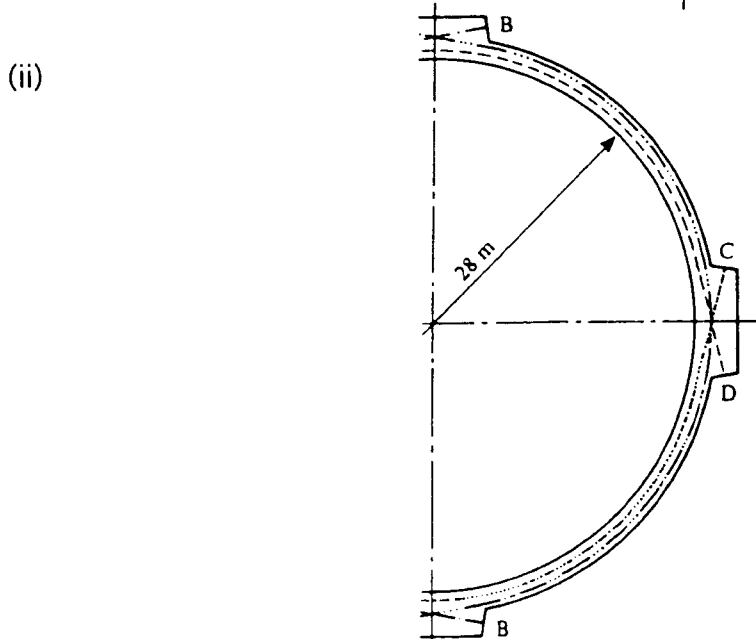
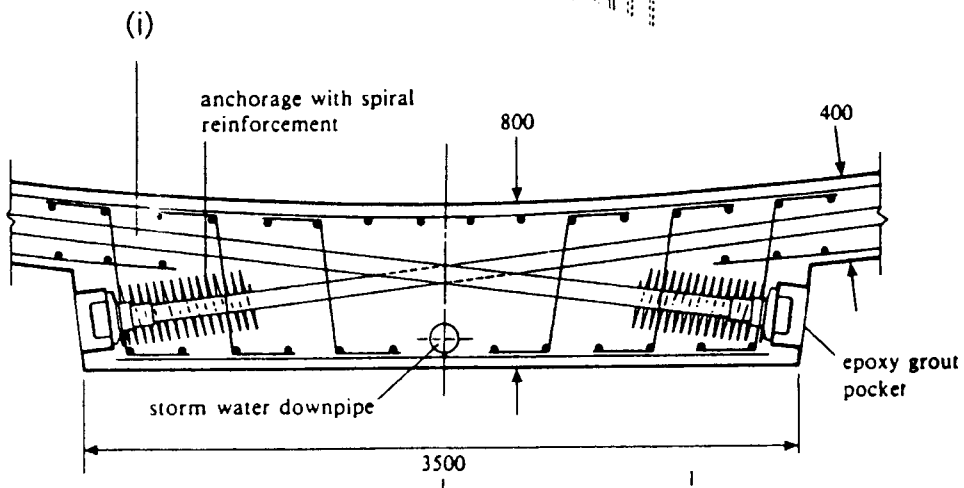
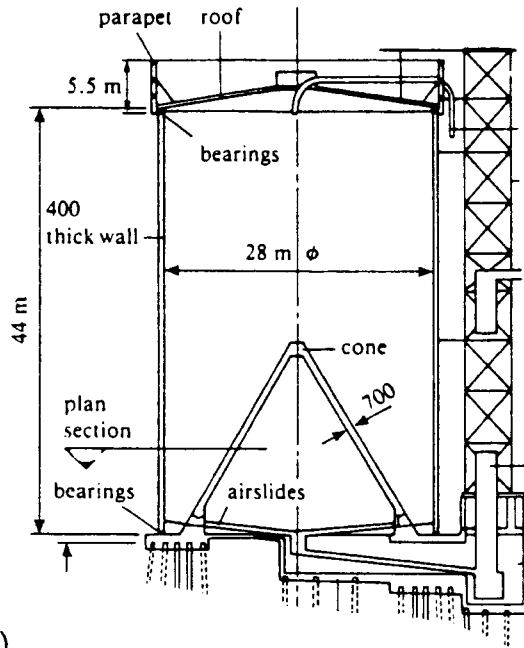
SHEET NO. VIII.4.6

Section through hopper



PRESTRESSED CONCRETE SILO

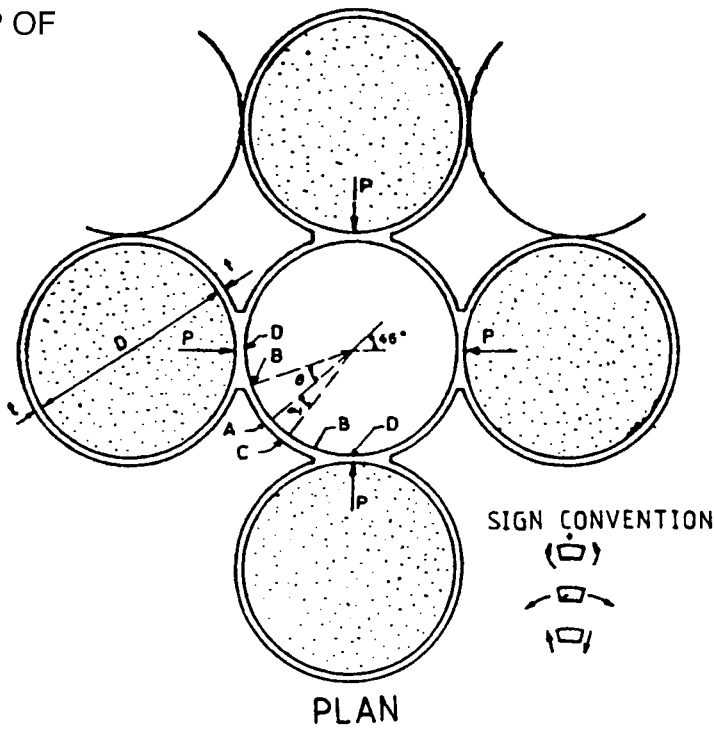
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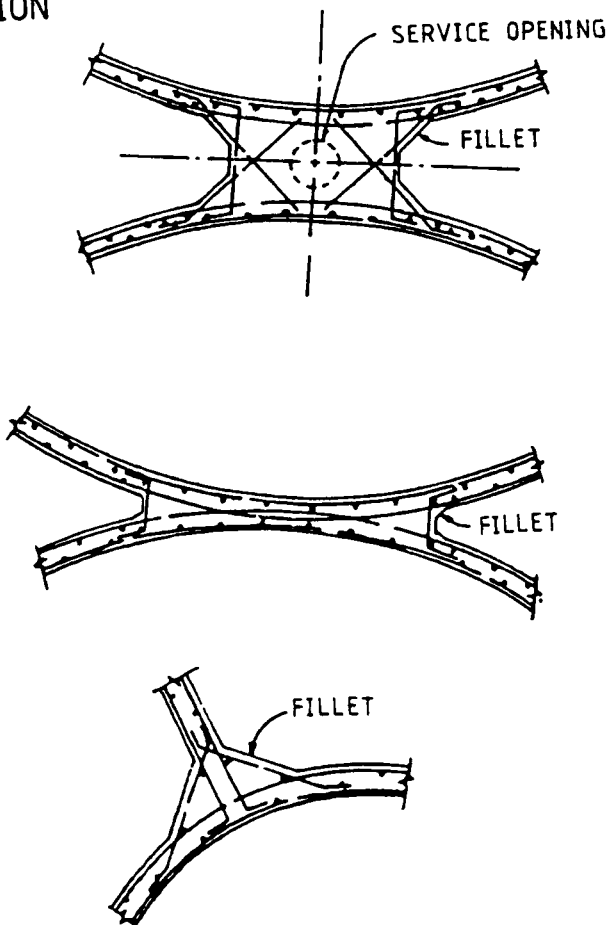
SILOS

SHEET NO. VIII.4.8

GROUP OF
SILOS



SECTION



VIII.5. Bomb protective structures

VIII.5.1. General introduction

Protecting buildings against vehicle bomb attacks has become a priority. The issue of the structural integrity of existing and future buildings is now a burning one. The structural detailing procedure must be such that it covers the effects of bomb explosions in and around structures. Since the requirements of the important buildings do vary, hence prior to structural detailing these requirements have to be identified.

Explosives are capable of exerting sudden pressures and they do generate shock waves. For detailed analysis and design, a reference is made to the author's following publications:

1. *Impact and explosion—analysis and design*, 1st edition, Blackwells, 1993.
2. *Prototype building structures—analysis and design*, Thomas Telford Publishing, 1999, Section 4.

The classification of explosives identified so far is given below:

- (a) small explosives—up to 5 kg TNT
- (b) medium explosives—up to 20 kg TNT
- (c) Large explosives—up to 100 kg TNT
- (d) Very large explosives—up to 2000 + kg TNT

The use of TNT (Trinitrofluorene) is generally considered as a reference. When the high explosive is other than TNT, the equivalent energy is obtained by using the 'charge factor', which is a ratio of the actual mass of the charge and mass of the TNT equivalent.

For RDX with mass specific energy (KJ/kg) of 5360, the TNT equivalent is 4520 and the factor is $5360/4520 = 1.185$. Hence for 100 kg of RDX, the conversion to TNT is 118.5 kg. Similarly, explosives with SEMTEX, the factor is 1.25 and for 100 kg, the conversion to TNT will be 125 kg.

VIII.5.2. Data on bomb explosion on structures

A reinforced concrete wall is loaded by a blast from a vehicle bomb of 100 kg actual mass. The wall is rigidly connected at the foundation and free at the top. Using the following data, calculate the required reinforcement for the wall:

Wall heights: 4 m

Vertical reinforcement, $\rho_{SU} = 0.5\%$ at each face

$R = R' = \text{range} = 4.0 \text{ m}$

Hemispherical charge factor = 1.8

$f_y = \text{static yield stress of the reinforcement} = 460 \text{ N/mm}^2$

$f_{dy} = \text{dynamic yield stress of the reinforcement} = 1.2 f_y$

Concrete grade = 40

Type III category rotation of the base allowed during explosion, $\theta = 12^\circ$

Justify the impulsive load analysis.

If the same wall is fixed at the top as well as at the bottom and is subjected to a quasi-static load of a triangular shape shown in Fig 4.78 of the code and adopting BS 8110 and other relevant criteria, check the reinforcement and the wall thickness while behaving as a single-degree elastoplastic system.

If the wall is subject to vehicle bombs of 50, 100, 150, 200, 250 and 300 kg of TNT at a random range R' of 1 m to 10 m from the explosion to the centre of the wall, draw curves for:

$$\frac{x}{L} \left(\frac{\text{wall deflection}}{\text{wall span}} \right) \text{ versus } R'$$

for various vehicle bombs given. The wall is assumed damaged when:

$$\frac{x}{L} = \frac{1}{60}$$

VIII.5.2.1. Calculations

The vehicle bomb mass produces a hemispherical charge of mass $1.8 \times 100 \text{ kg} = 180 \text{ kg}$, where 100 kg is the actual mass.

$R' = \text{range} = 4.0 \text{ m}$

$$Z = \text{the scaled distance} = \frac{40}{180^{1/3}} = 0.705 \text{ m/kg}^{1/3}$$

$i_r = I_r = \text{reflected overpressure}$

Impulse = 5095 kPa-ms.

For category type III behavior, the material properties are:

$f_y = \text{reinforcement yield stress} = 460 \text{ N/mm}^2$ (static value)

$f_{dy} = 1.2 f_y = 1.2 \times 460 = 552 \text{ N/mm}^2 \approx f_{ds}$ (dynamic value)

$$\frac{I_r^2}{2K_{LM} \rho d_c} = \left(\frac{2}{H} \rho_v f_{ds} d_c^2 \right) \tan \theta$$

$\theta = \text{rotation of the base of the wall} = 12^\circ$ (type III category)

$\tan \theta = 0.2126$

$\rho_v = 0.5\%$, i.e. reinforcement on both sides is equal

$K_{LM} = \text{load-mass factor} = 0.66$

$$\frac{(5090)^2}{2 \times 0.67 \times 2400 d_c} = \frac{2}{4.0} \times \frac{0.5}{100} \times 552 \times 10^6 d_c^2 \times 0.2126$$

$d_c = 0.302 \text{ m} = 302 \text{ mm}$

$T_c = d = \text{overall thickness}$

$= d_c + 2 \times \text{cover of } 40 + 2 \text{ assumed-size bar halves}$

$= 302 + 80 + 25$

$= 407 \text{ mm}$

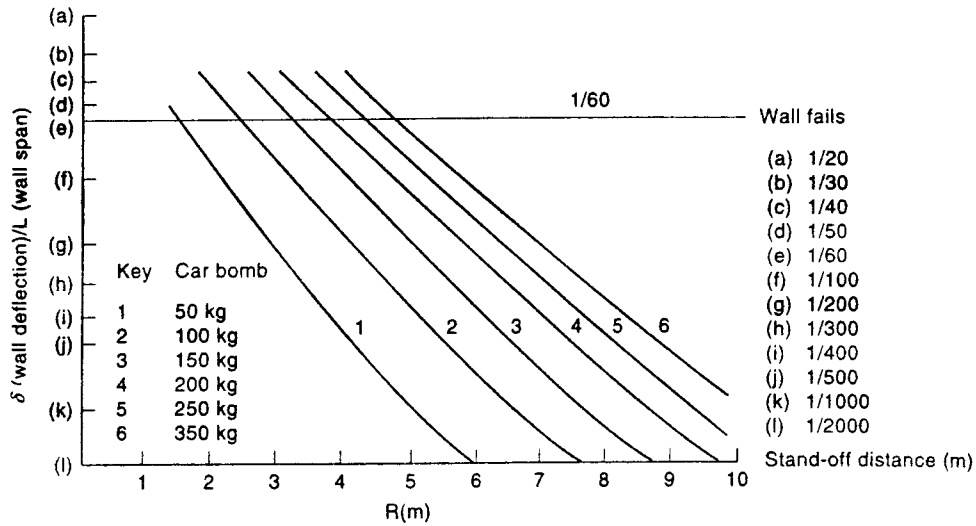
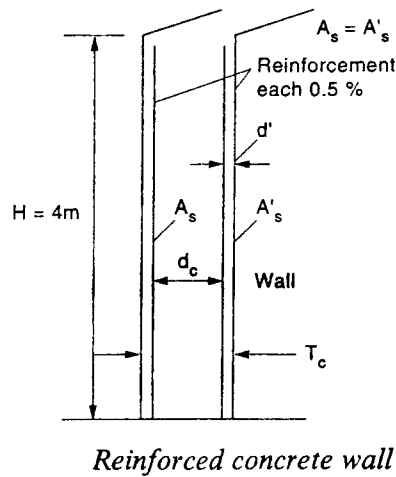
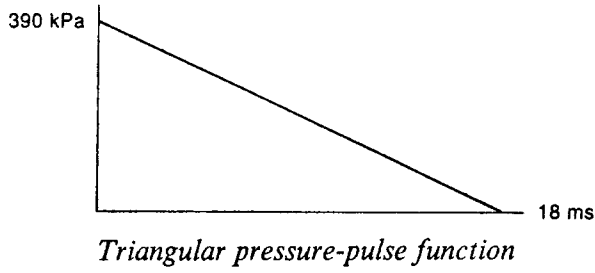
Adopt 425 mm or 17 in. reinforced concrete wall (T_c) (see Fig. 4.79 of the code):

$A_s = 0.005 \times 302 \times 1000 = 1510 \text{ mm}^2/\text{m} = \text{width of wall.}$

Similarly for other cases that are examined. Sheet No. VIII.5.1 gives a relationship between δ/L versus stand off distances $R(m)$.

PULSE-PRESSURE: CAR BOMBS—DEFLECTION AND STAND-OFF

SHEET NO. VIII.5.1



δ/L versus stand-off damage assessment

VIII.5.2.2. American practice

Calculate scaled distance Z for the following stand-off distances R , for 27 kg, 100 kg and 500 kg spherical TNT explosive charges:

$$Z = \frac{R}{W^{1/3}} \left(\frac{m}{\text{kg}^{1/3}} \right)$$

Stand-off distances R :

$$R = 10 \text{ m}; R = 6 \text{ m}; R = 4 \text{ m}; R = 2 \text{ m}$$

Example:

$$R = 10 \text{ m } W = 100 \text{ kg}$$

$$Z = \frac{10}{100^{1/3}} = 2.155 \frac{\text{m}}{\text{kg}_{\text{TNT}}^{1/3}}$$

$$R = 10 \text{ m } W = 500 \text{ kg}$$

$$Z = \frac{10}{500^{1/3}} = 1.26 \frac{\text{m}}{\text{kg}_{\text{TNT}}^{1/3}}$$

For other stand-off distances R , values of Z are computed and given on Sheet No. VIII.5.2.

BOMB CHARGES: PRESSURES AND STAND-OFF

AMERICAN PRACTICE

$P_s =$ overpressure
 $i_s =$ impulse
 $T_s =$ positive phase duration
 $t_a =$ arrival time
 $P_{suc} =$ pressure below ambient
 $T_{suc} =$ negative phase duration
 $i_{suc} =$ suction phase i for impulse
 $P_r =$ the incidence overpressure = 1.649×10^5 Pa
 $P_r = 7.291 \times 10^5$ Pa reflected pressure P_r

$P_{suc} = \frac{-0.35}{Z}$
 $Z > 1.6$
 $T_{suc} = 1.25W^{1/3}$
 $i_s =$ area of triangle

$P_s = 100$ kg of TNT
 $P_r = 1 \times 10^5$ Pa
 $i_s = 5030 \times 10^3$ Pa · ms
 $T_s = 4.8$ ms

RDX is to be converted to TNT.
 The TNT equivalence factor = 1.185.

The bomb charge
 25 kg : 1.185 × 25 = 29.63 kg TNT
 27 kg : 1.185 × 27 = 32.0 kg TNT

A 10 kg explosive with a spherical charge contains the following mixture of ingredients:

- 6.5 kg of PETN
- 3.5 kg of Torpex

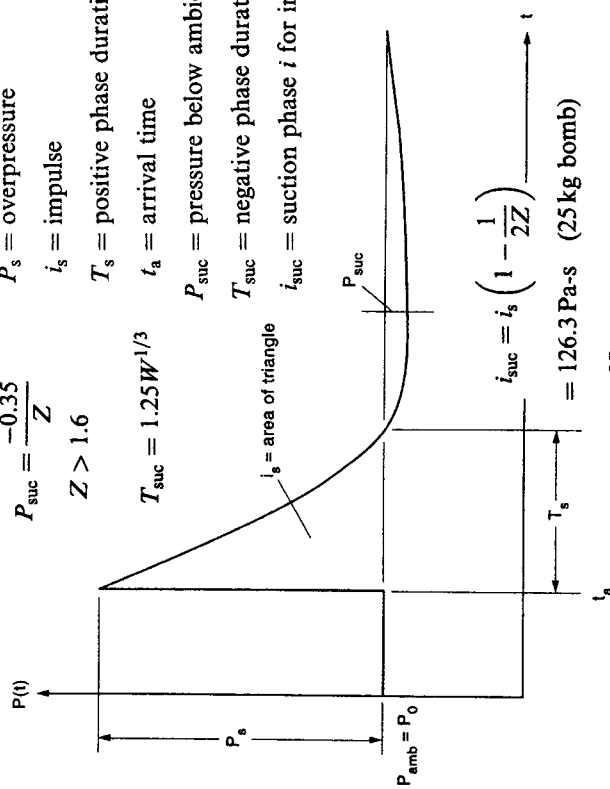
A building wall of 3.5 m is subject to the 10 kg explosive placed at 3.50 m range. Determine the incident overpressure P_s and the reflected pressure P_r .

PETN 6.5 kg → 6.5 × 5800 = 37 700 kJ
 Torpex 3.5 kg → 3.5 × 7540 = 26 390 kJ
 total charge energy = 64 090 kJ

The equivalent value in TNT = $\frac{\text{total energy of the 10 kg explosive}}{4520}$
 $= \frac{64\,090}{4520} = 14.18$ kg TNT

At a range $R = 3.5$ m, the scaled distance is:

$Z = \frac{3.5}{(14.18)^{1/3}}$
 $= 1.67$
 $P_s = 2.39 \times 10^6$ Pa
 $P_r = 1.08 \times 10^6$ Pa



Pressure-time profile = 176.5 Pa-s (27 kg bomb)

Scaled distances Z for three explosive charges at various stand-off distances

Charge (kg)	$R = 10$ m	$R = 6$ m	$R = 4$ m	$R = 2$ m
27	3.33	1.998	1.332	0.666
100	2.155	1.293	0.862	0.431
500	1.26	0.756	0.504	0.252

VIII.5.3. Generalized data for a domestic nuclear shelter

A shelter for a family of six has been designed by the author. The following data are considered.

VIII.5.3.1. Basic data (Home Office code)

For a 1 megatonne burst at a distance of 3 km from ground zero:

Velocity of the shock front = 500 m/s

Ductility ratio $\mu = 5$

Drag coefficient: roof = -0.4; wall = +0.9

Yield strength of reinforcement = 425 N/mm²

f_{cu} (dynamic) = 1.25 f_{cu} (static)

f_y (dynamic) = 1.10 f_y (static)

Main reinforcement $\leq 0.25\% bd$ (b = width, d = effective depth)

Secondary reinforcement $\leq 0.15bd$

The ultimate shear stress $\geq 0.04 f_{cu}$

The dynamic shear stress for mild steel ≥ 172 N/mm²

For a 1 megatonne ground burst at a distance of 1.6 km from ground zero:

Ductility ratio $\mu = 5$

Main reinforcement: $\leq 0.25\% bd$

Secondary reinforcement: $\leq 0.15\% bd$

Ultimate shear stress: $\geq 0.04 f_{cu}$

Dynamic shear stress (mild steel): ≥ 172 N/mm²

Protective factor: 4000

Concrete f_{cu} (static): 30 N/mm² (grade 30)

Concrete f_{cu} (dynamic): 1.5 $f_{cu} = 37.5$ N/mm²

Reinforcement, y (static): 420 N/mm²

Reinforcement, y_d (dynamic): 1.10 $f_y = 462$ N/mm²

Young's modulus, E_c : 20 GN/m²

Young's modulus, E_s : 200 GN/m²

Clear span: 3 m

Slab thickness: 300 mm (with minimum cover 50)

Blast load: 0.17 N/mm², $F_1(t) = P_{do}$

VIII.5.3.2. Additional data for designs based on US codes

Dynamic increase factors (DIF)

Concrete: compression 1.25

diagonal tension 1.00

direct shear 1.10

Reinforcement: bending 1.10

shear 1.00

Dynamic stresses:

Concrete f_c (cylindrical strength) = 0.87/cu

= 3000 lb/in² (psi)

Concrete f_y (static)

= 60 000 lb/in² (psi)

$$R_m = r_u = \left(\frac{1}{1 - \frac{1}{2\mu}} \right) F_1(t) = 1.1 F_1(t) = 0.187 \text{ N/mm}^2$$

Deadload of concrete plus soil = 0.014 N/mm²

$r_u = 0.187 + 0.014 = 0.201$ N/mm²

U.S. Recommended Reinforcement cover $\nless 38$ mm

$$E_c \text{ for concrete} = D^{1.5} 33 \sqrt{f'_c}$$

$$= (150 \text{ lb/in}^3)^{1.5} \times 33(3000)^2$$

$$= 3.32 \times 10^6 \text{ psi}$$

$\rho = D =$ density of concrete = 150 lb/in³ (23.6 kN/m³)

E_s for steel = 30×10^6 psi (200 GN/m²)

Dynamic stresses for concrete:

Compression $1.25(3000) = 3750$ psi

Diagonal tension $1.00(3000) = 3000$ psi

Direct shear $1.10(0.18)(3000) = 600$ psi

Reinforcement:

Bending $1.10(60\ 000) = 66\ 000$ psi

Shear $1.10(60\ 000) = 60\ 000$ psi

Since $f'_c = 3000$ psi and f_y (static) = 60 000 psi

ϕ , Strength reduction factor = 0.85

Common specifications

All slabs $\nless 300$ mm thick

All walls $\nless 300$ mm thick, $\nless 2700$ mm \times 3400 mm

Area of the roof $\nless 9$ m²

Weight of the overhead material = 1340 kg/m²

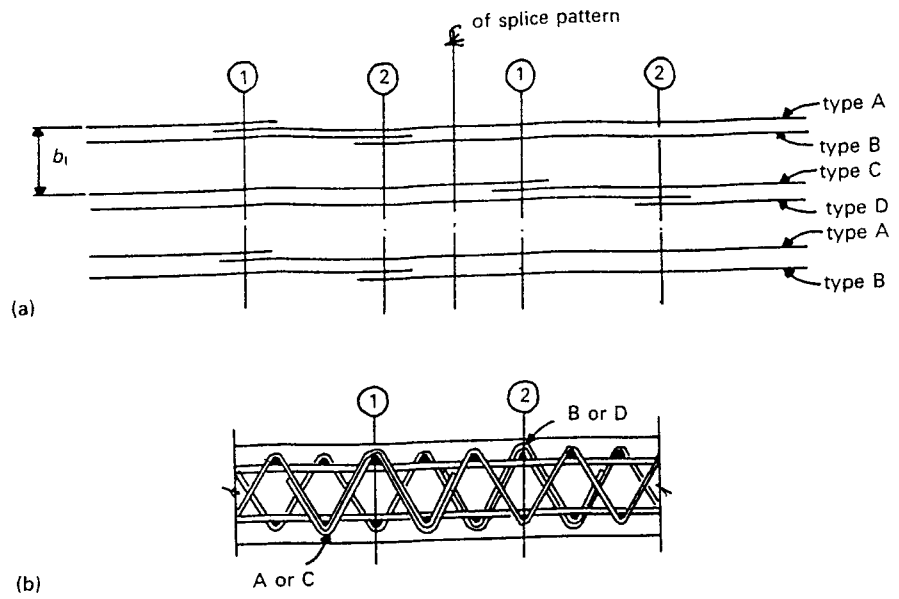
Steel blast doors – clear opening = 800 mm \times 1200 mm $\nless 25$ mm thick

VIII.5.3.3. Shelter details

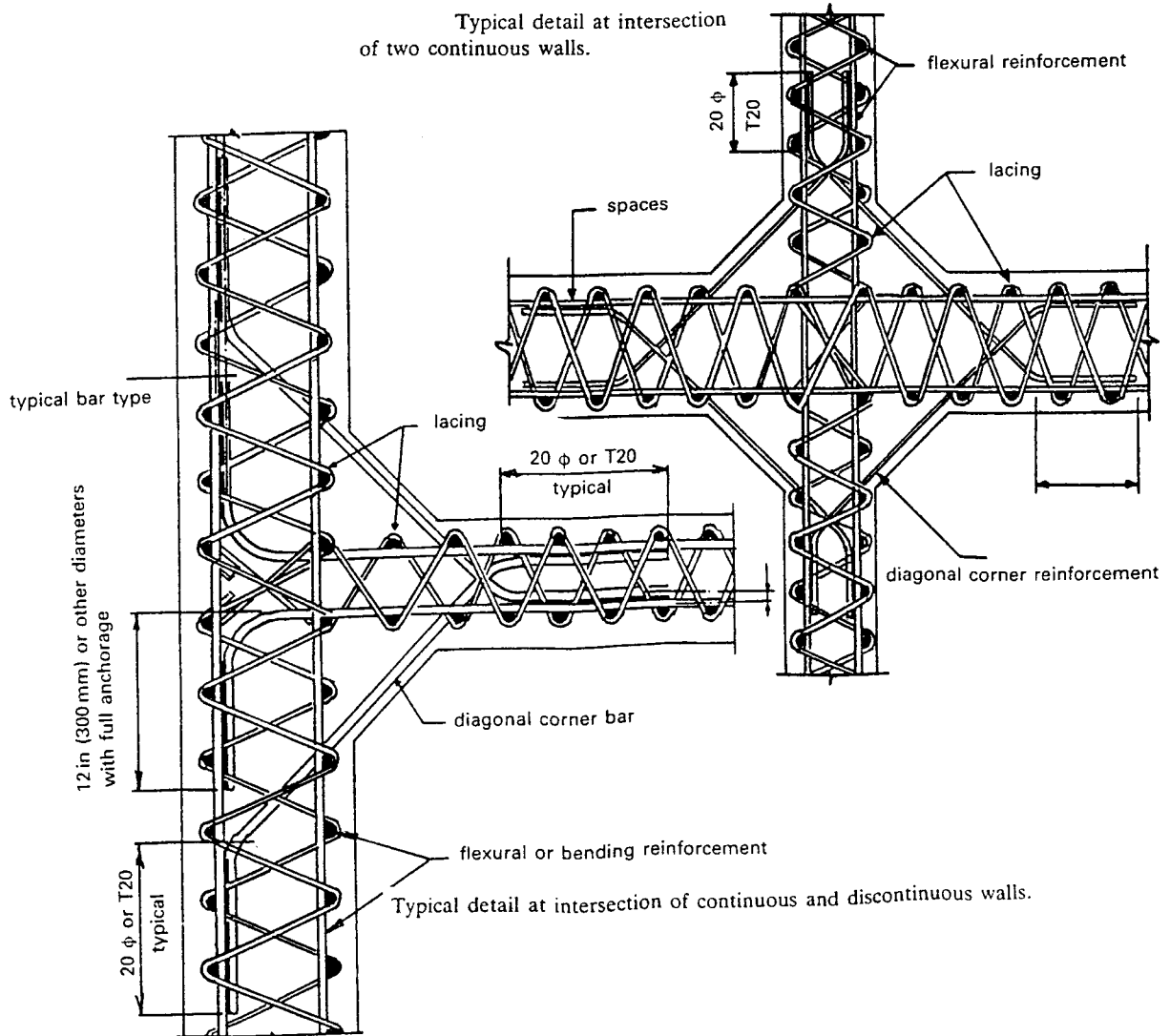
Some typical reinforcement detailing for nuclear protective shelters are shown in Sheet Nos VIII.5.3 and VIII.5.4.

TYPICAL REINFORCEMENT DETAILS OF INTERSECTION WALLS (WITH LACING BARS)

SHEET NO. VIII.5.4



Typical details for splicing of lacing bars: (a) splice pattern; (b) lacing splice.



VIII.6. Nuclear, oil and gas containments

VIII.6.1. Nuclear power and containment vessels

Nuclear reactors are generally housed in large prestressed concrete pressure and containment vessels. Some of the pressure vessels containing gas-cooled reactors with their respective parameters are listed in Table VIII.6.1.

Table VIII.6.1. Prestressed concrete vessels

Plant	H_i : m	D_i : m	d_T : m	d_B : m	d_w : m
Oldbury	18.3	23.45	6.40	6.71	4.58
Dungeness B	17.70	19.95	3.66	5.95	3.81
Hinkley Point B } Hunterston B }	19.40	18.90	6.33	7.51	5.03
St Laurent 1, 2	36.30	19.00	5.70	6.00	4.75
Bugey 1	38.25	17.08	7.46	7.46	5.49
Fort St Vrain	22.85	9.45	4.73	4.73	2.74
VentropTHTR	5.75	15.90	5.10	5.10	4.45

H_i = internal height; D_i = internal diameter; d_T , d_B = top and bottom cap thickness; d_w = wall thickness.

In all the constructed vessels listed in Table VIII.6.1 the gas boilers or circulators are either inside the vessel or placed elsewhere along the periphery of the vessel in an annular space. In addition, multi-cavity vessels are proposed in which boiler and circulators are placed vertically inside the thickness of the vessel. The internal surface of these vessels is covered by a steel liner about 20 to 40 mm thick, the latter is for certain critical areas.

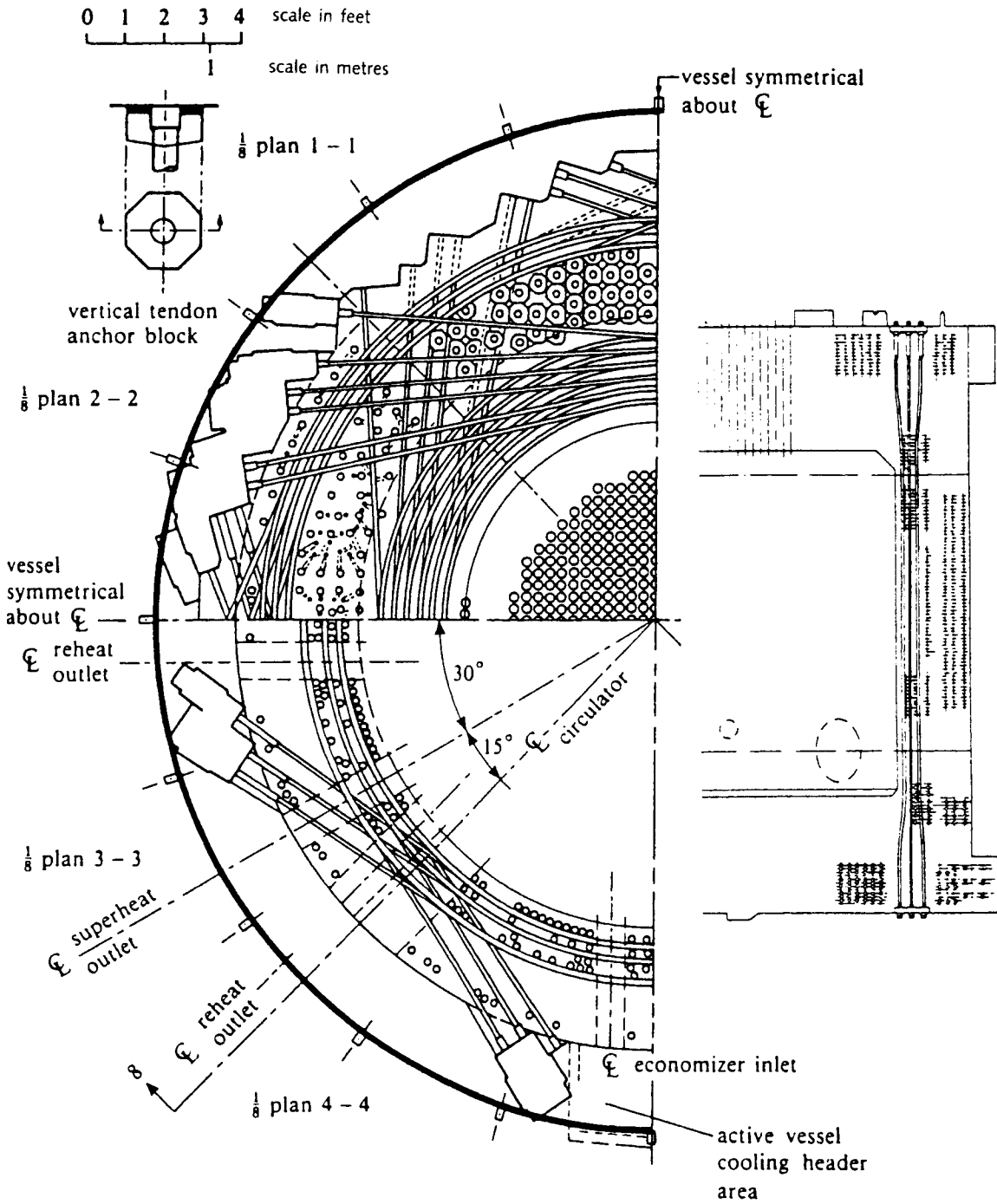
These vessels are designed for an internal design pressure of 2.66 MN/m² to 7 MN/m². The prestressing tendons are arranged in vertical and circumferential directions to withstand external loads and internal gas pressure. Vertical tendons are slightly curved to offset extreme stresses at the corners. The circumferential tendons are anchored at buttresses. Sheet No. VIII.6.1 shows the prestressed tendon layout for the Dungeness B vessel. In the case of the Oldbury vessel, a single helical prestressing system is used to replace vertical and hoop tendons and their effects are similar to the Dungeness B vessel.

In the case of multi-cavity vessels, owing to the placement of boilers and circulators in vertical directions, the circumferential tendons are replaced by a wire strand winding system as shown on Sheet No. VIII.6.2.

The main purposes of the containment structures are (a) to prevent the escape of radioactive materials, (b) to protect the reactor system from damage due to external hazards, e.g. aircraft crashes, missile impact, tornadoes and hurricanes and explosion, and (c) to provide biological shielding against nuclear radiation. A number of these vessels have been designed for boiling water reactors and pressurised water reactors. Some of them are listed in Table VIII.6.2.

PRESSURE VESSEL—DUNGENESS B TYPE

SHEET NO. VIII.6.1



HTGCR VESSEL

SHEET NO. VIII.6.2

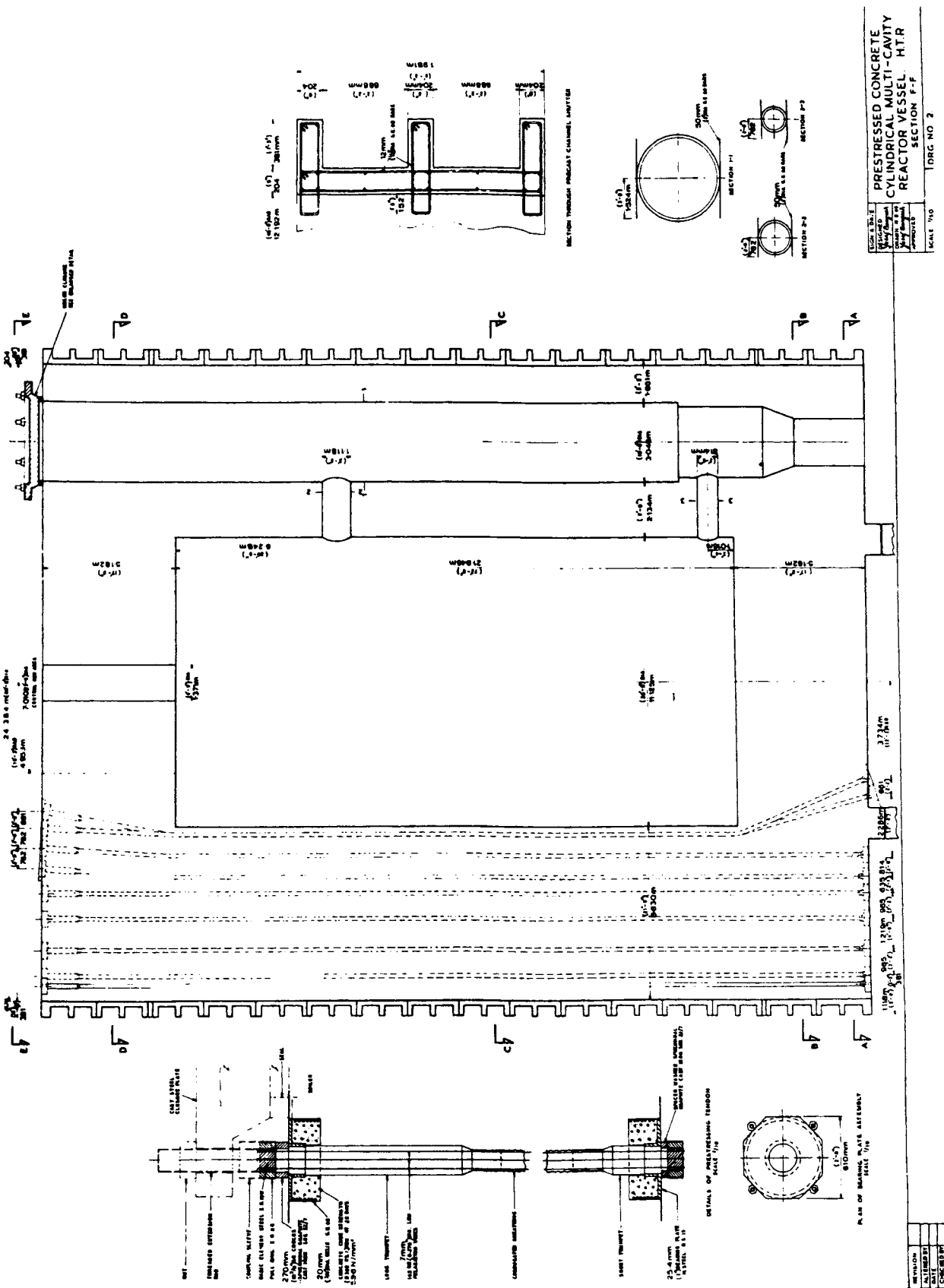


Table VIII.6.2. Prestressed concrete reactor containment vessels

Plant	H_i : m	D_i : m	d_w : m	Description
Palisades Wall	57.90	35.40	1.07	Spherical dome on circular wall
Monts d'Arree	56.00	46.00	0.60	Spherical dome on circular wall
Fessenheim	51.30	37.00	1.00	Spherical dome on circular wall
Super Phenix*	90.00	64	1.00	Spherical dome on circular wall
TVA			0.915	Elliptical dome on circular wall
Sizewell B	63	41.88	1.00	Elliptical dome on circular wall

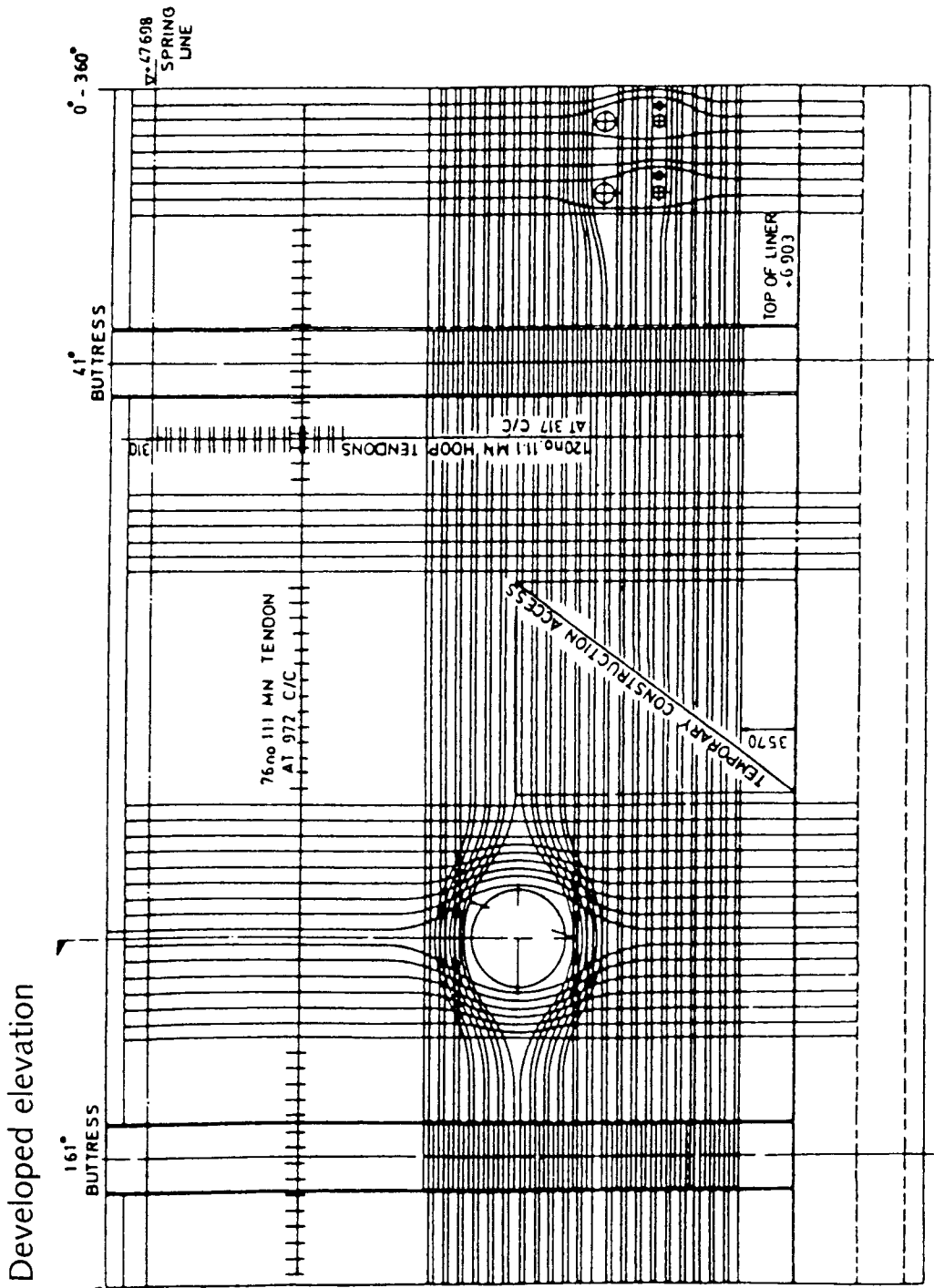
* = total height H : ($H_i = H - D_i - \text{thickness}$)

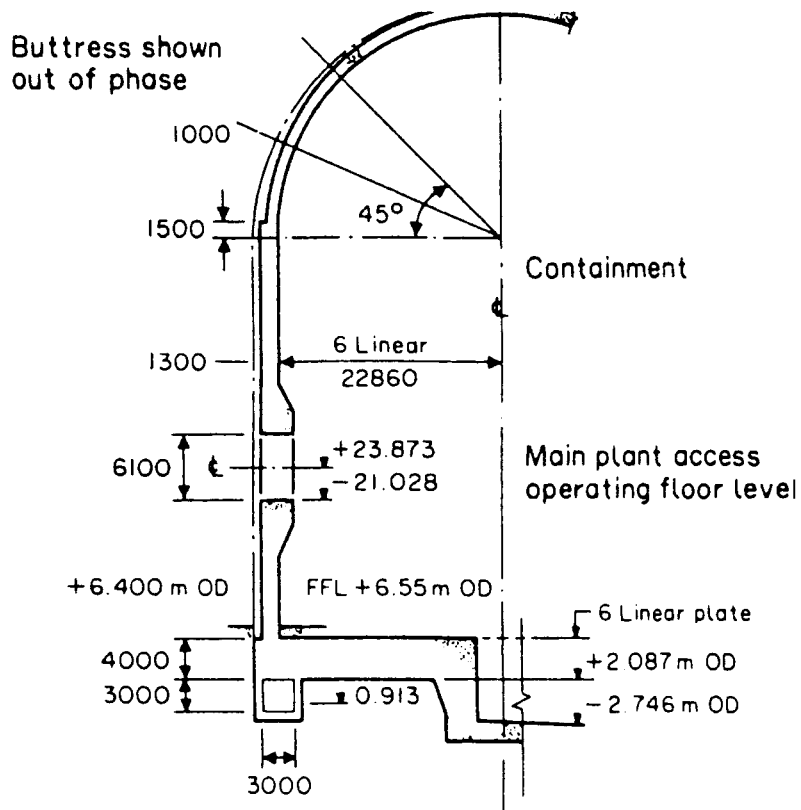
The internal pressure is about 344.75 kN/m² for Sizewell B, as shown on Sheet No. VIII.6.3. The number of vertical tendons are about 76 at 0.97 m centres in the wall. The number of circumferential tendons in the wall are about 120 at 0.317 m centres. In the dome latitude, the circumferential tendons are 38 No. and along the longitude 24 No. with 16 No. in the rings.

A sectional elevation for the Tennessee Valley Authority (TVA) containment building with prestressing tendons in the elliptical dome and the wall is shown on Sheet No. VIII.6.4. The Sizewell containment has 11 MN tendons. Almost all containments are protected by a steel liner lugged to concrete and the thickness of the line varies from 6 mm to 12 mm.

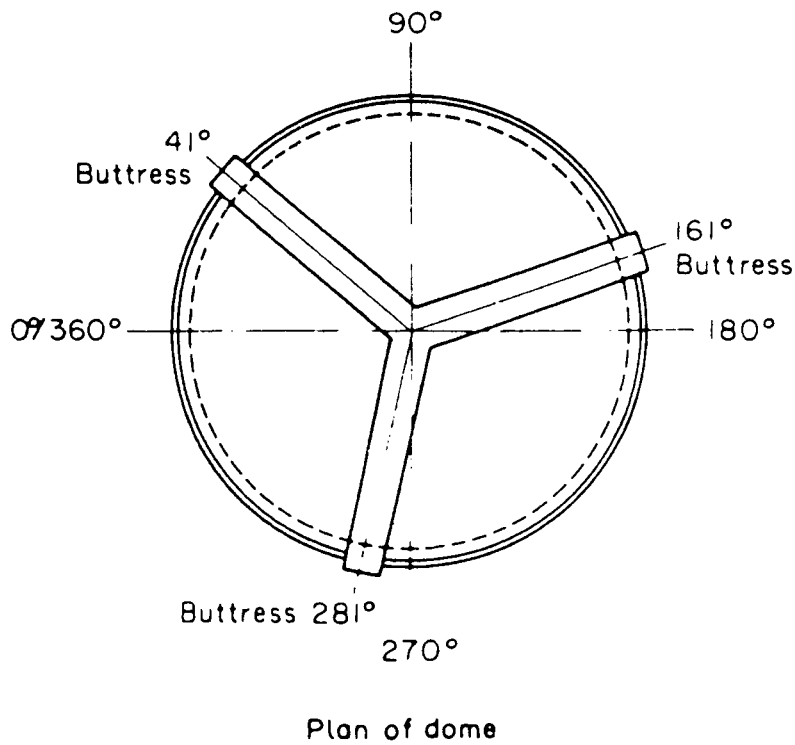
**SIZEWELL B CONTAINMENT WITH SPHERICAL DOME —
MAIN DIMENSIONS AND PRESTRESSING LAYOUT**

SHEET NO. VIII.6.3

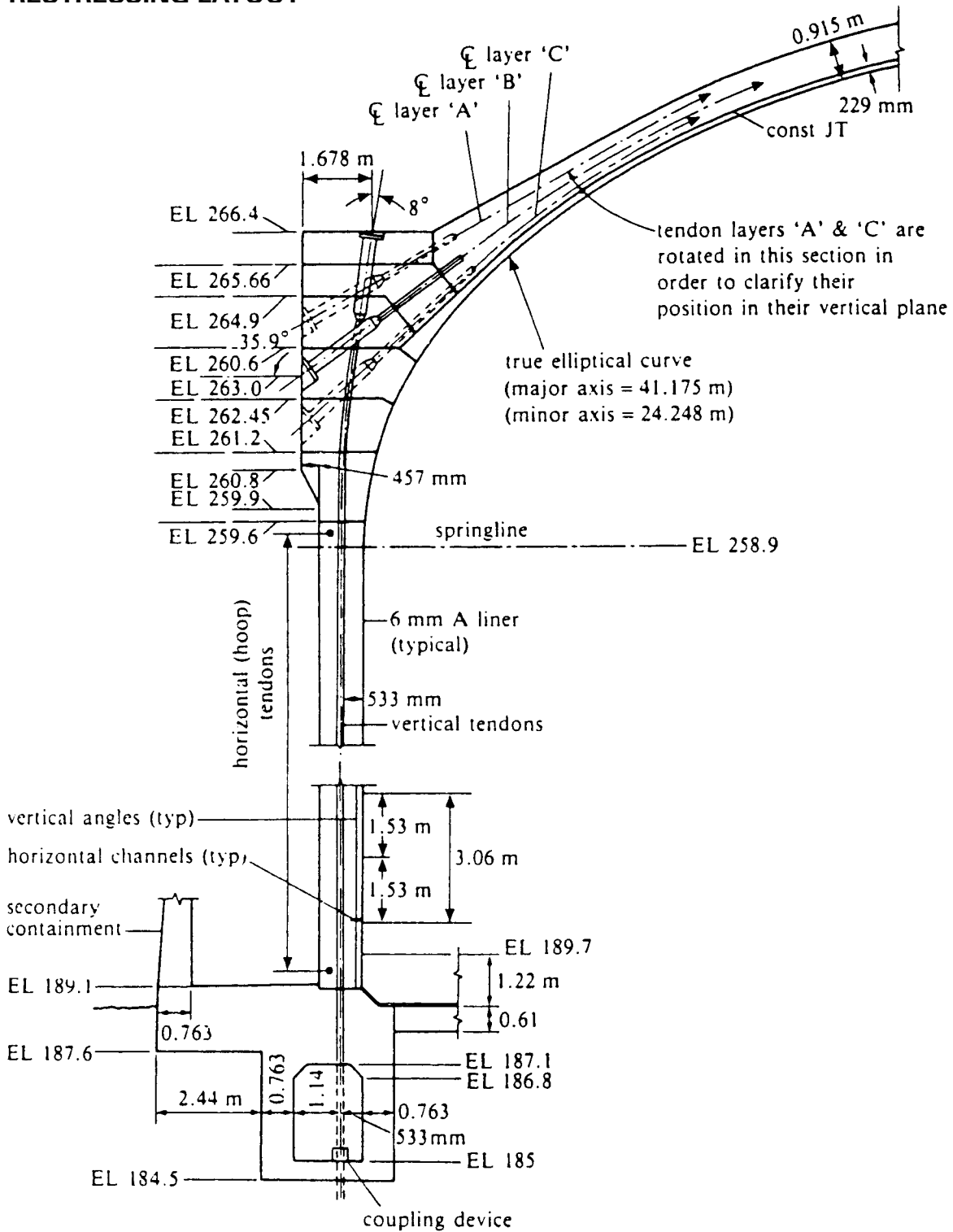




Vertical section through main plant access



**TVA CONTAINMENT BUILDING WITH ELLIPTICAL DOME — SHEET NO. VIII.6.4
PRESTRESSING LAYOUT**



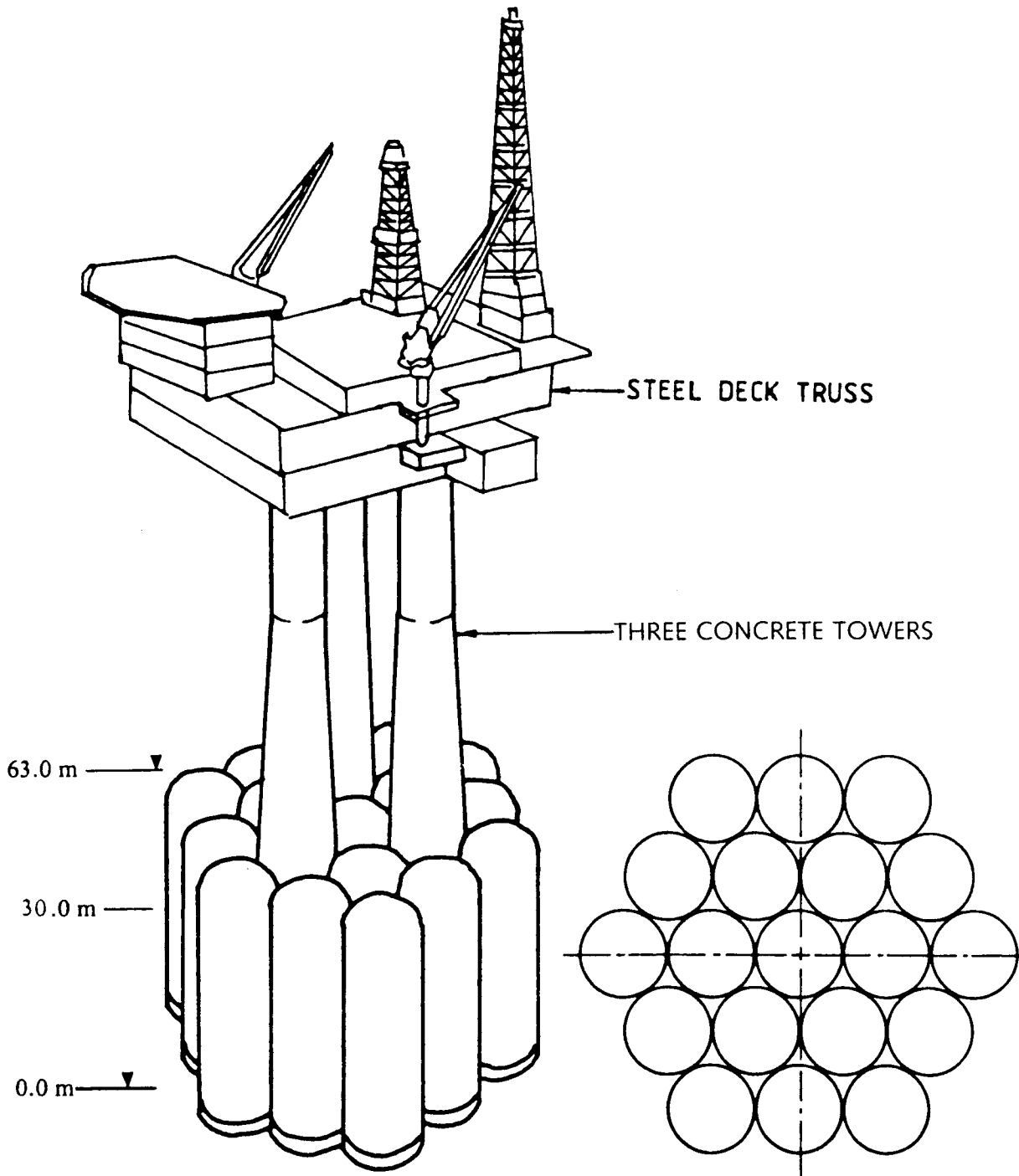
VIII.6.2. Oil containment structures

Most concrete platforms for offshore facilities have been provided with hollow caissons for oil storage (approximately one million barrels) to allow continuous production over an emergency period such as the stoppage of tanker loading in bad weather or pipeline shutdown during the operation of the platform. In the design, apart from the environmental loads, tanker collision, drop weights on the deck and explosion, significant thermal stresses must be included which can be caused by the difference between the ambient sea water temperature (about 5°C) and the stored oil in the cell (about 40°C).

Sheet No. VIII.6.5 shows a sectional elevation and plan of a group of cells designed in reinforced concrete. Alternatively, these cells can be designed and constructed using prestressed tendons on the lines suggested for the TVA and Sizewell B containments as shown on Sheet Nos VIII.6.3 and VIII.6.4 respectively. The Dungeness B vessel layout can be adopted with approximate concrete thickness of barrel walls and caps for storing oil. These cells have domes resting on cylindrical walls with drill shafts and decks at the centre. They are constructed in a dry dock and are towed to the installation site. A number of these platforms have been designed, detailed and constructed, the most well known are Condeep, Ninean, Staffjord, Murchison and Tor.

OIL CONTAINMENT

SHEET NO. VIII.6.5



VIII.6.2.1. Condeep Platform (Table VIII.6.3)

Water depth (in the area) 145 m to 188 m.
 Cells (hollow 16 No.) 20.1 m diameter \times 50 m high.
 Cell walls, etc. 0.61 m thick, 109 m high with three main shafts above cells with outside diameter tapered from 20.1 m to 11.9 m.
 Deck weight with equipment 22.68 million kg.
 Weight of the support structure 2.72 million kg.
 Reinforcement bar size 12 and 25.

Table VIII.6.3.

Platform	Water depth: m	Caisson plan (area/height)	No. of towers	Ext./int. dia.: m	Steel: t
Frigg	104	72 m ² /42 m	2	14/13.4	5800
Brent	140	91 m ² /57 m	4	15/14.4	11 400
Cormorant A	152	100 m ² /56 m	4	16/15.4	13 930

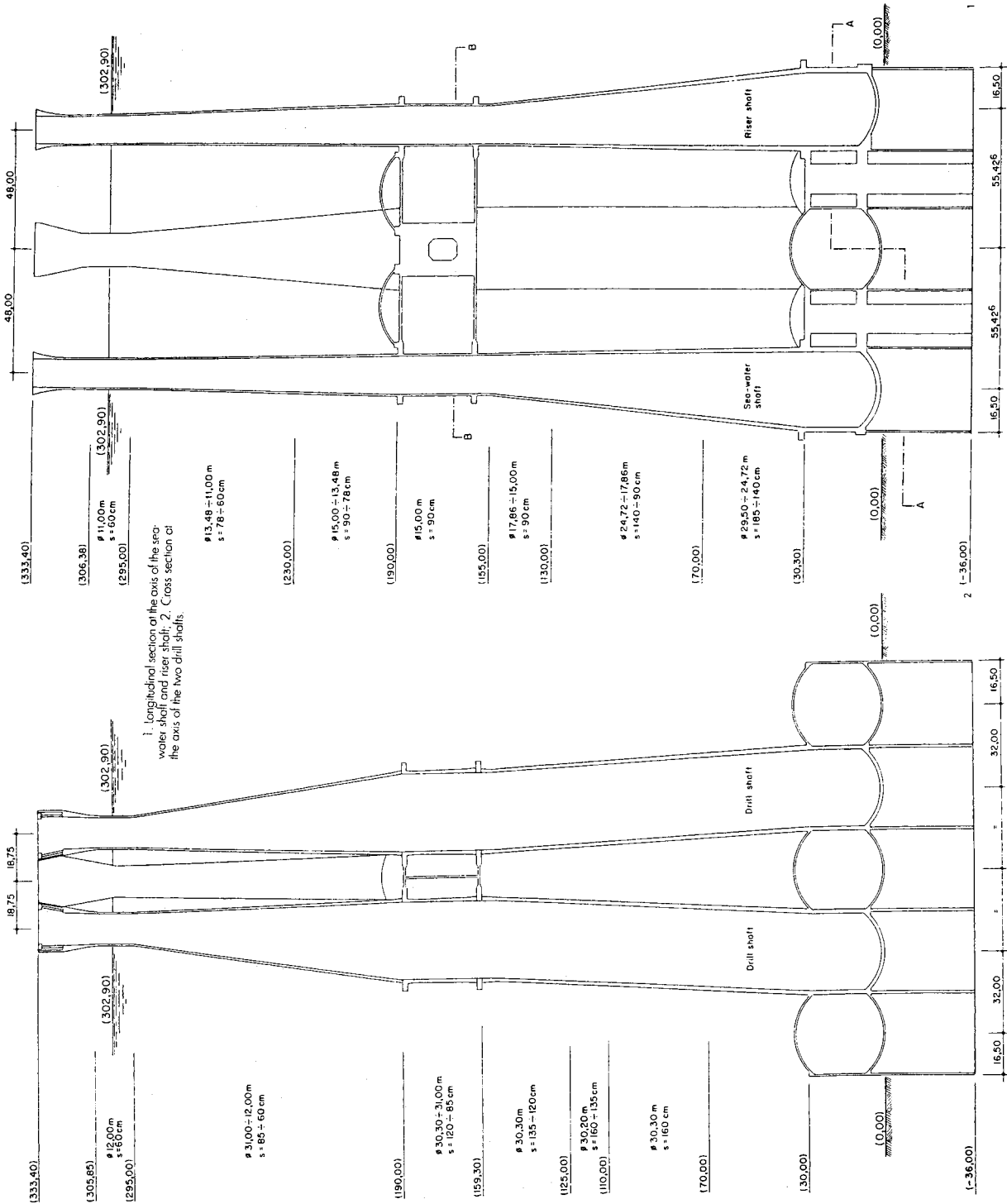
Concrete grade 50 is always recommended.

VIII.6.2.2. Troll platform

The Troll offshore platform, a cyclops in the North Sea, was finally constructed in 1997. The platform has four towers with 19 cells. Sheet No. VIII.6.6 shows the longitudinal section at the axis of the sea water shaft and riser shaft and the cross-section of the axis of the two drill shafts. Various structural dimensions of sea water, riser and drill shafts and water levels are indicated on the drawing. Sheet No. VIII.6.7 shows component elements and it shows the basement plan composed of 19 cylindrical caissons, and the horizontal section of the four shafts at the connecting structure composed by five reinforced concrete beams. The cells have variable radii. The information was provided by the following — customer: Den Norske Stats Oljeselskap a.s., A/S Norsk Hydro Produksjon a.s., SagaPetroleum a.s., Norske Conoco A/S, Elf Aquitaine Norge A/S, Total Norge AIS; design, transport and installation: Norwegian Contractors, Oslo, Norway; forms: Gleitbau Ges.mbH, Salzburg, Austria.

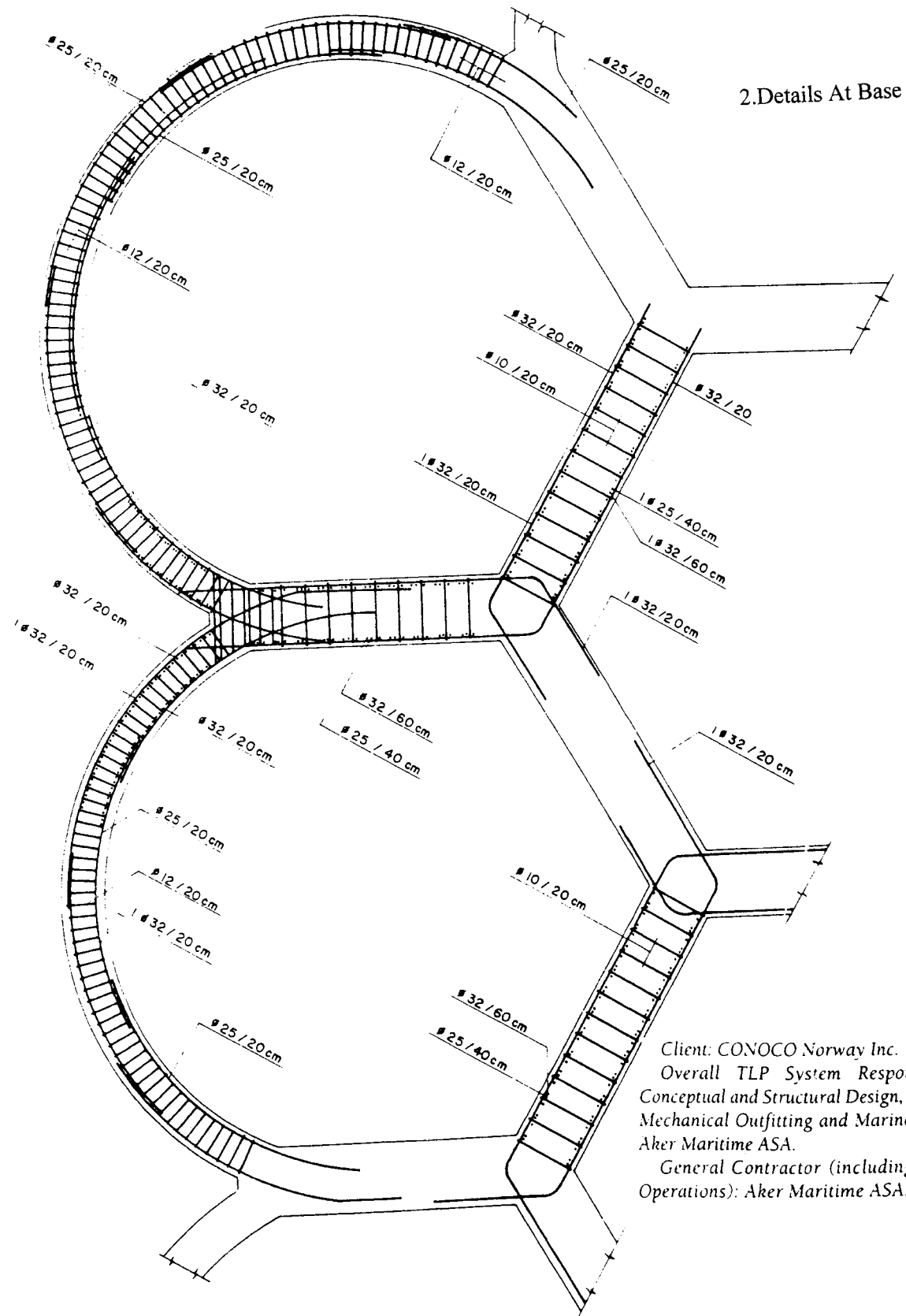
LONGITUDINAL AND SECTIONAL ELEVATION OF TROLL PLATFORM (BASED ON EUROPEAN PRACTICE)

SHEET NO. VIII.6.6



**REINFORCEMENT DETAILS FOR CELLS/SHAFTS
(Heidrun Project Platform)
(BASED ON EUROPEAN PRACTICE)**

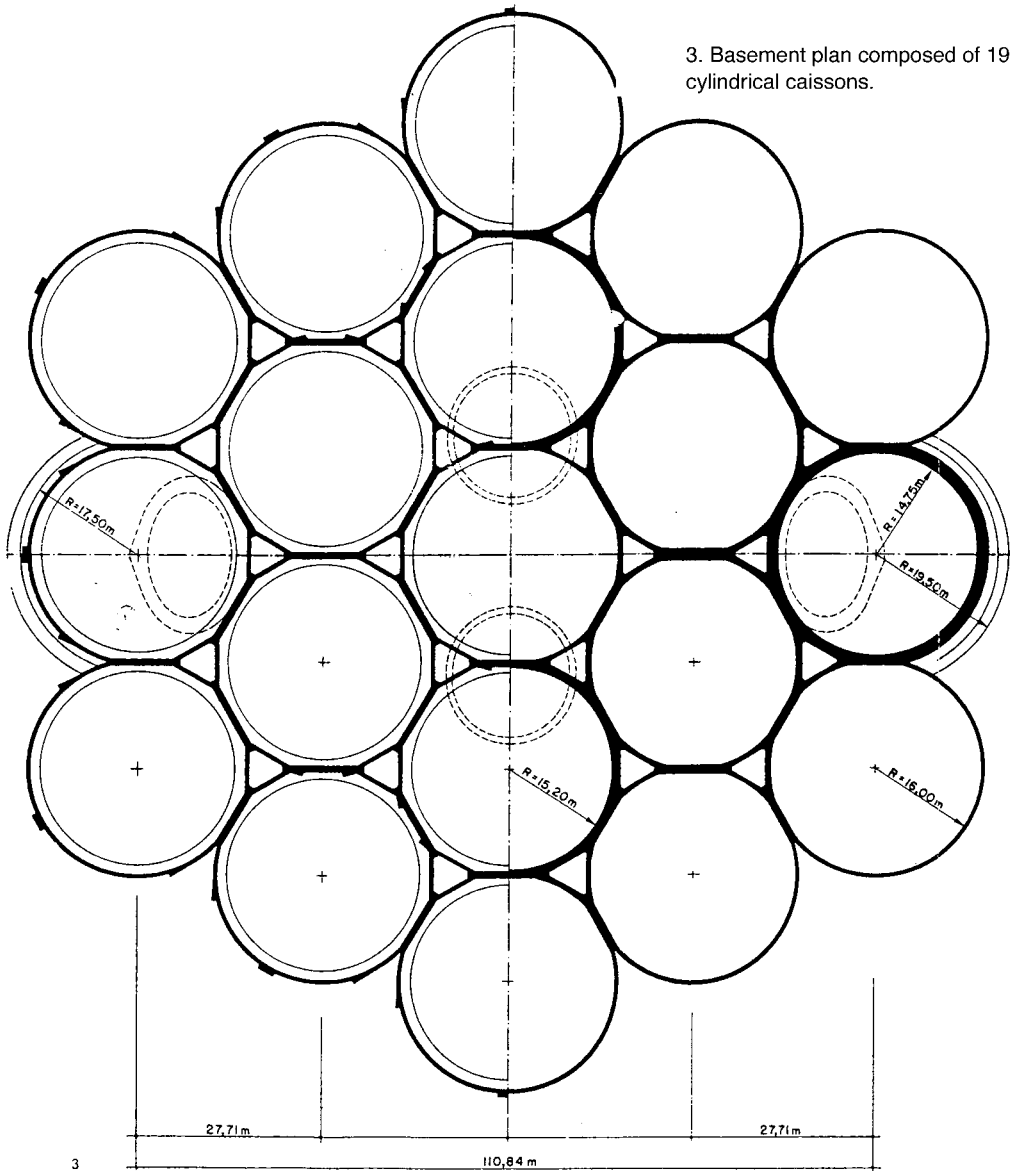
SHEET NO. VIII.6.6 (contd)



**BASEMENT PLAN—CELLS AND SHAFTS WITH INTERCONNECTED BEAMS
(BASED ON EUROPEAN PRACTICE)**

SHEET NO. VIII.6.7

3. Basement plan composed of 19 cylindrical caissons.



3

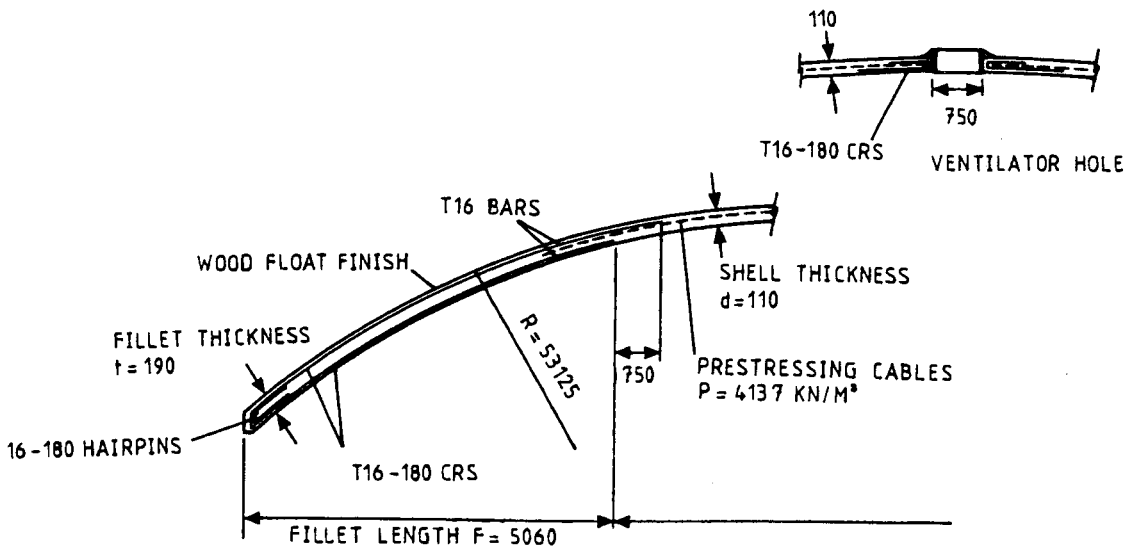
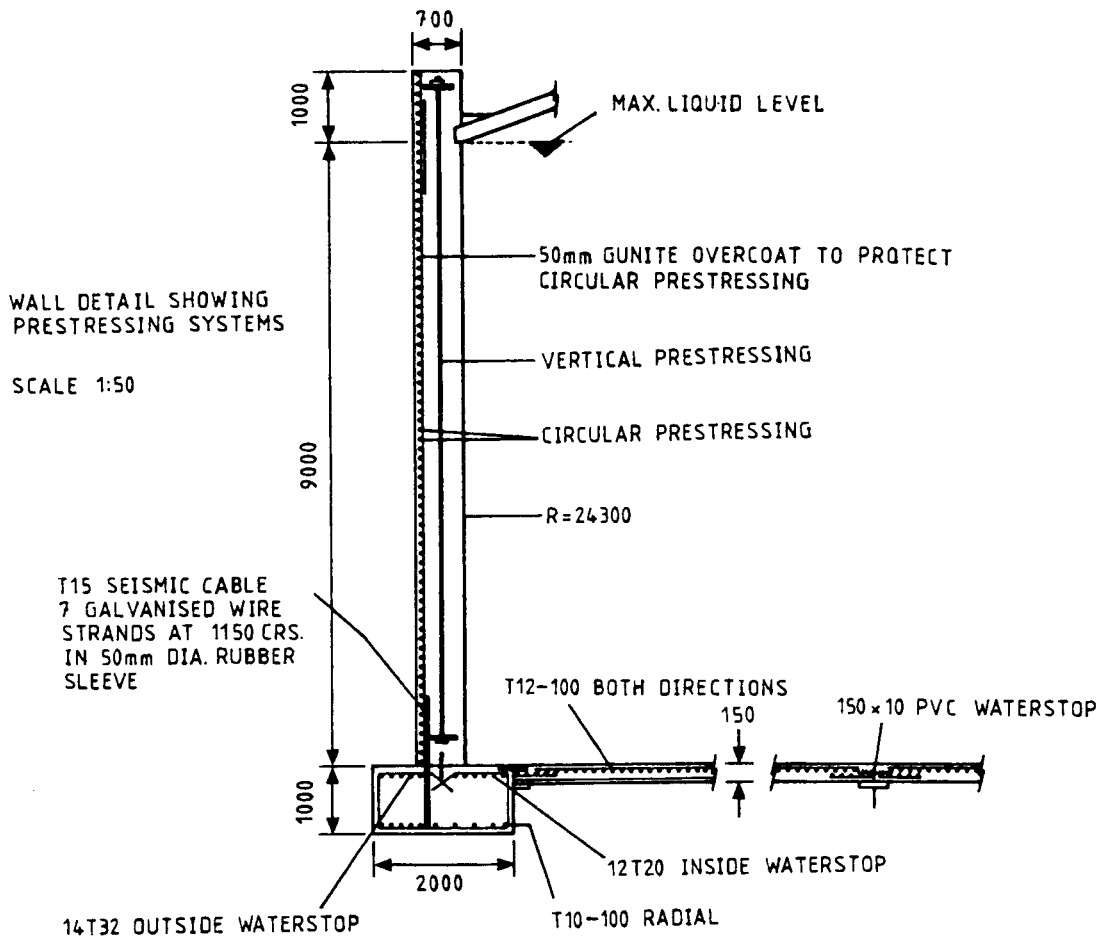
VIII.6.2.3. Liquefied natural gas containments

Liquefied natural gas (LNG) can be contained in a fixed concrete storage structure in the form of a vessel on shore or can have a series of mobile marine structures for its storage offshore. The most common way to store large quantities of gas in preparation for marine transportation is to liquefy it. It is well known that liquefied gases are stored and shipped at low temperatures, e.g. propane at -46°C and LNG at -160°C . The need for special materials, insulation, etc., imposes constraints on the mobile structure design: sea conditions, wind, current directions, seismic conditions and soil conditions. For on-shore conditions, wind, temperatures, seismic and soil conditions must be included in the design of these containments or storage structures. Dykman's BBR of San Diego, California, USA, are the designers and constructors of a large number of such structures worldwide. These containments can be of a single walled or doubled walled type, reinforced or prestressed.

Sheet No. VIII.6.8 shows typical details of the LNG concrete containments of 50 m diameter with 10 m wall height and an elliptical dome of maximum height at the centre of 5.5 m. The maximum liquid height is 9 m.

PRESTRESSED CONCRETE LNG TANK

SHEET NO. VIII.6.8



VIII.7. Concrete shells, chimneys and towers

VIII.7.1. General introduction

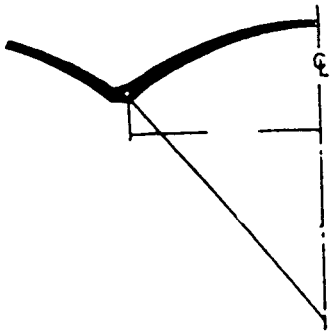
Various concrete shell roofs in reinforced concrete have been designed to provide for large uninterrupted roof spans for industrial and other buildings. Various attempts have been made to classify the types of shells. The most popular classification is based on Gaussian curvature and is given below.

- (a) *Shells of Positive Gaussian Curvature (Synclastic Shells)*. Here the surface curves are away from a tangent plane at any point on the surface. They lie completely on one side of the plane. Examples are: spherical dome and elliptic paraboloids.
- (b) *Shells of Negative Gaussian Curvature (Anticlastic Shells)*. They are formed by two families of curves which are opposite in direction. Examples are: hyperbolic, paraboloids, conoidal shells and hyperbolas of revolution.
- (c) *Shells of Zero Gaussian Curvature (Singly Curved Shells)*. They lie between positive and negative Gaussian curvature. Examples are: cylindrical shells, cylinders and cones.

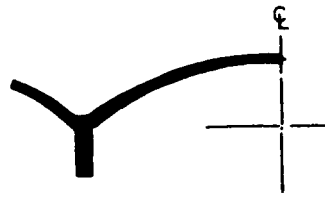
In addition to the above classification, shells are classified on the basis of shells of rotation and shells of translation. The shells of rotation are domes (spherical, elliptical, conoidal) and the shells of translation are hyperbolic paraboloids, elliptical paraboloids, etc. Sheet Nos VIII.7.1 and VIII.7.2 give some of these examples.

CYLINDRICAL SHELL

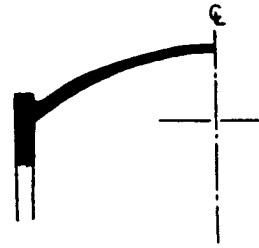
SHEET NO. VIII.7.1



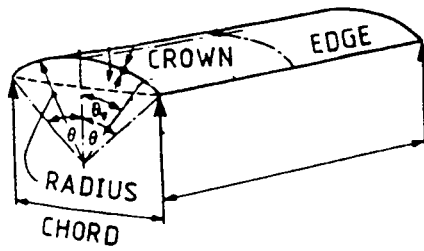
FEATHER EDGE SHELL



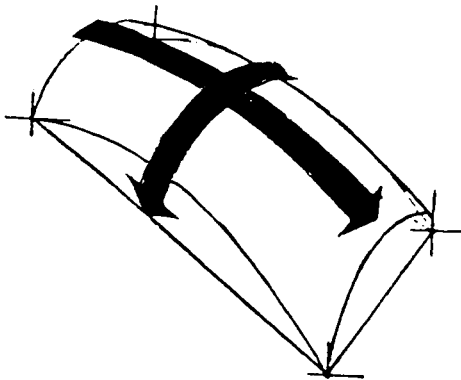
SHELL WITH VALLEY BEAM



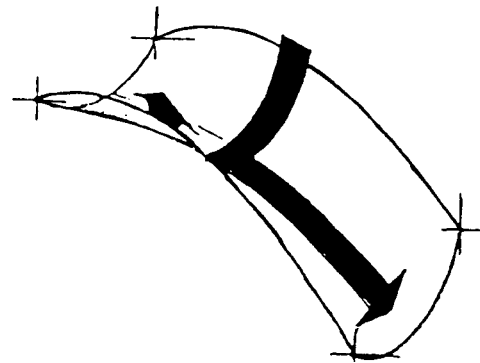
SHELL WITH
EDGE BEAM



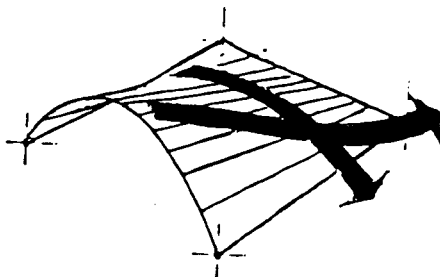
BARREL VAULT



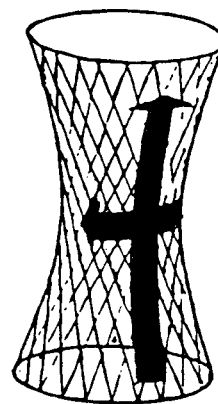
SYNCLASTIC OR ELLIPTICAL SURFACE



ANTICLASTIC OR HYPERBOLIC SURFACE



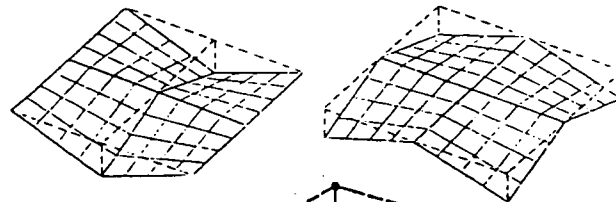
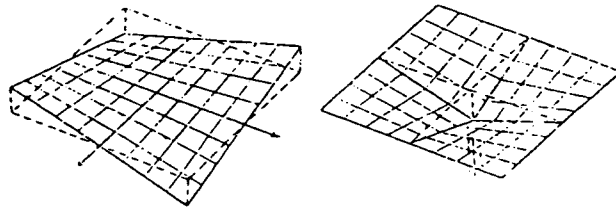
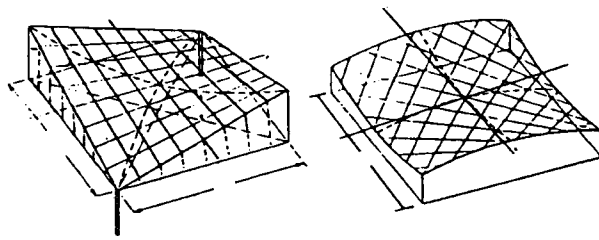
CONOID



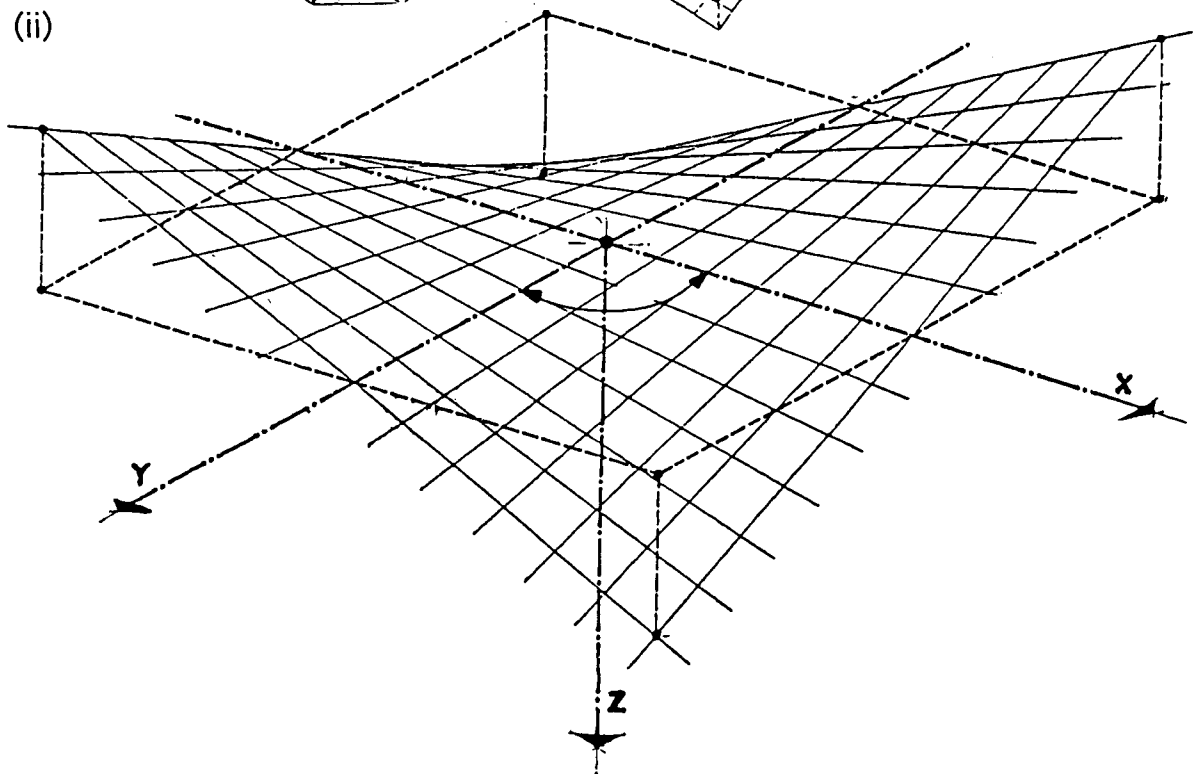
HYPERBOLOID

HYPERBOLIC PARABOLOIDS

(i)



(ii)



VIII.7.2. Shells

A cylindrical shell's surface is generated by moving a straight line parallel to itself along a cylindrical surface. The cylindrical surface can be a circular arc or any segment of a cylinder. At the ends, longitudinal edge beams simply supported or continuous are provided to stiffen the shell against the edge disturbance, bending and shear. These shells can be single ones spanning over many supports or can be multiple shells in transverse directions spanning over a single span. Sheet No. VIII.7.3 shows the reinforcement layout for a cylindrical shell with valley beams. The radius, thickness and span of the shell are 100 mm, 9.25 m and 22 m respectively. The edge beams are 0.25 m wide \times 1.75 m deep. A similar layout will be required for non-circular cylindrical shells. Sheet No. VIII.7.3 shows a photograph of the completed structure.

The shell shown on Sheet No. VIII.7.3 has been designed for a load of 360 kN/m² excluding the weight of the edge beams. Longitudinal steel has been provided because of the tensile force occurring in that direction. Transverse and diagonal steels are provided for transverse stresses and principal tensile stresses. Starter bars are provided in the end zone to connect shell and beam reinforcement and to offset stresses occurring due to edge disturbance.

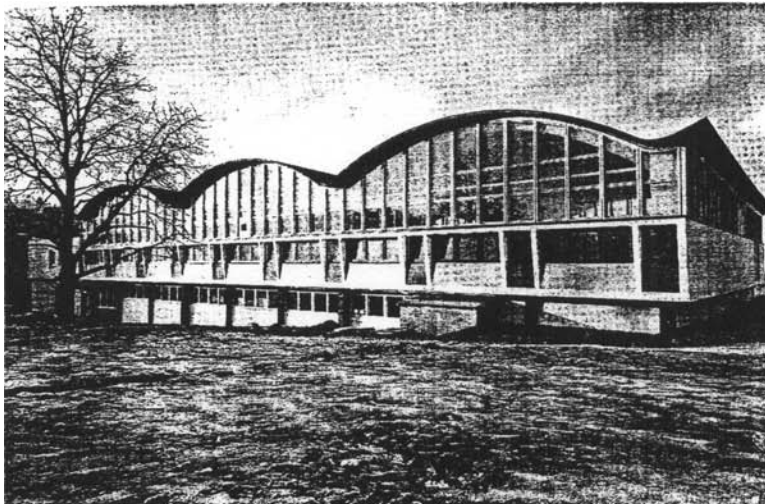
The hyperbolic paraboloid (hypar) shells can either be seen as a warped parallelogram or as a surface of translation. A number of these hypar shells are shown on Sheet No. VIII.7.2. The equation of the surface is first defined for the hypar types; forces and stresses are determined in the main shell and in the edge beams. Sheet No. VIII.7.4 shows an inverted umbrella-type hypar shell 24.38 m \times 24.38 m. The roof is a combination of four units and of thickness 75 mm and is designed for a uniformly distributed load of 345 kN/m². Sheet No. VIII.7.4 shows a detailed reinforcement plan and section of such a shell.

Sheet No. VIII.7.5 shows a sectional elevation and a plan indicating reinforcement details of a kite-shaped hypar shell designed for a conference hall. Sheet Nos VIII.7.6 to VIII.7.8 show complete details of the hypar shell layouts for Edens Theatre at Northbrook, designed and detailed by Perkins and Will of Chicago. The reader is left with original drawings in empirical units. Bar sizes and dimensions, etc., are converted into metric units using a standard conversion given in the text. Both kite-shaped hypar shells adopted are identical. The one shown in Sheet No. VIII.7.5 is half the size of that one given for Edens Theatre. These shells are also known as saddle shaped hypar shells. Generally their thicknesses are no more than 75 to 100 mm. The hypar shell types given on Sheet No. VIII.7.2 can similarly be designed and detailed once the geometry of the type given on Sheet No. VIII.7.6 has been decided. A typical drawing of the saddle shaped hypar between the dimensions (heights and plane projection, i.e. shell projected plan) is shown in Sheet No. VIII.7.2(ii).

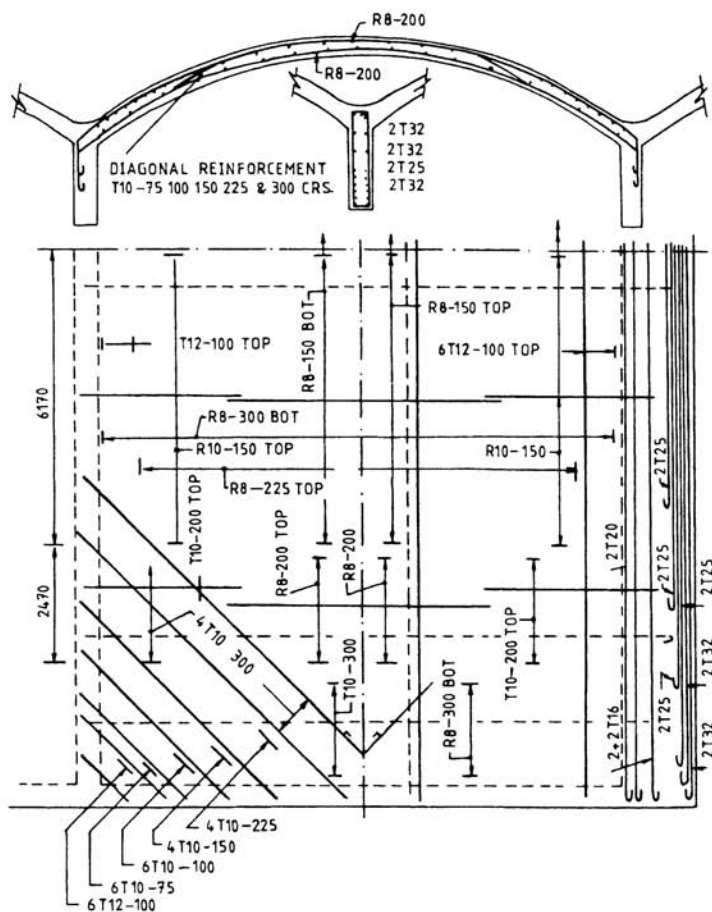
Hyperbolic shells and the hyperboloid of revolution of one sheet which have a graceful appearance have been exploited. Cooling towers and water tanks are just two examples. A great advantage is that their surface is generated by two families of intersecting straight lines. A typical view of earthenware pots and the curvatures shaped by potters are the basis of such shells given on Sheet Nos VIII.7.9 to VIII.7.12. Sheet Nos VIII.7.9 and VIII.7.10 show general dimensions of a water tank of a hyperbolic shell and its reinforcement details with cut-off bars at specific levels. The bars are placed along the longitude and latitude of the shell. The cover is of domical type integrated with the hyperbolic shell part.

THIN CYLINDRICAL SHELL AT HELSINKI, FINLAND

SHEET NO. VIII.7.3

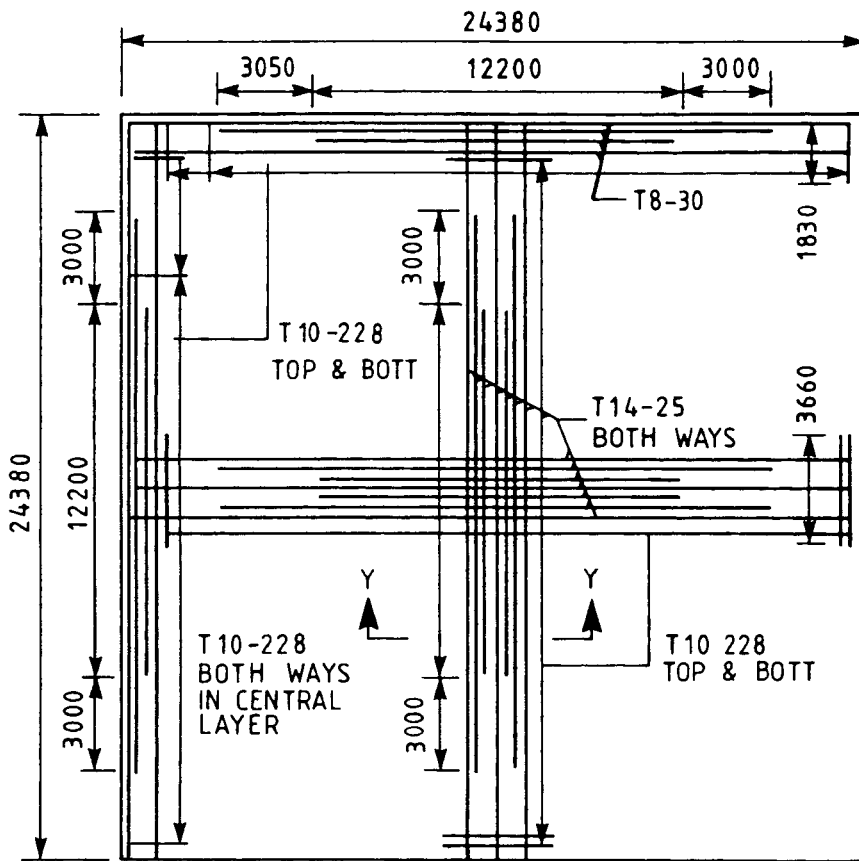


(Courtesy of S. Eggwertz, Consulting Engineer)

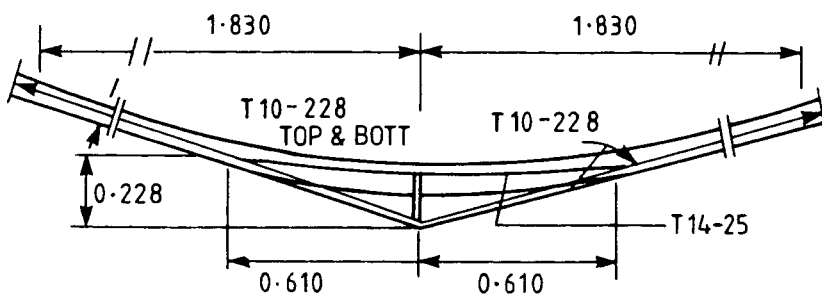


INVERTED UMBRELLA-TYPE HYPAR SHELL

SHEET NO. VIII.7.4



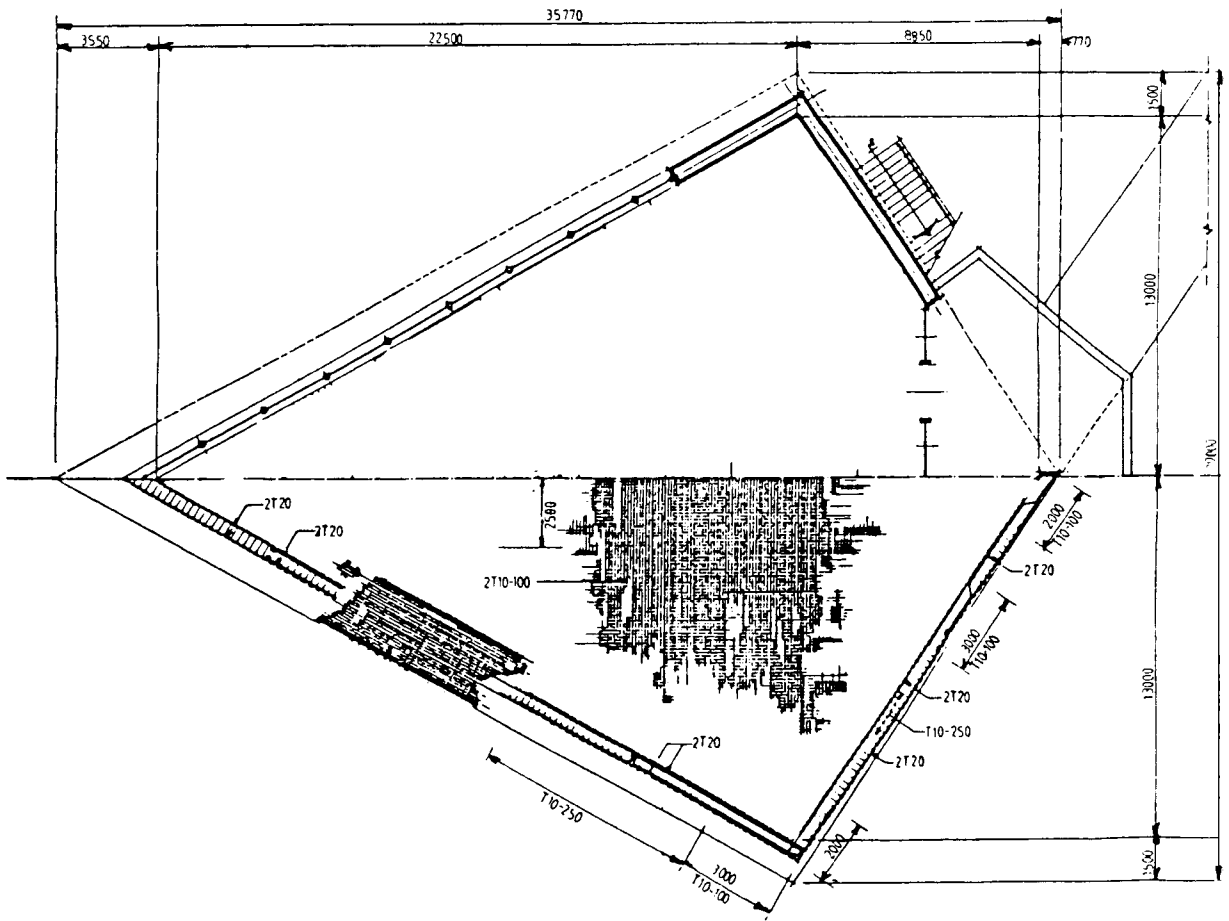
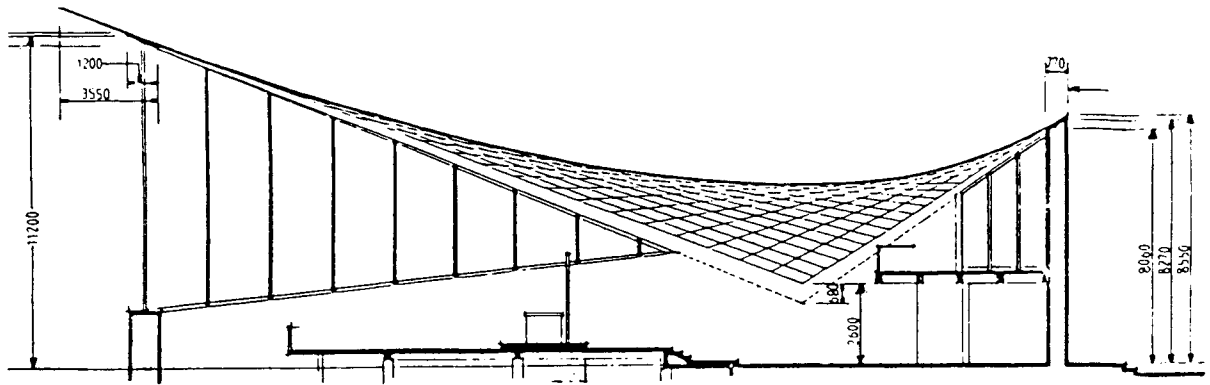
PLAN



SECTION Y-Y

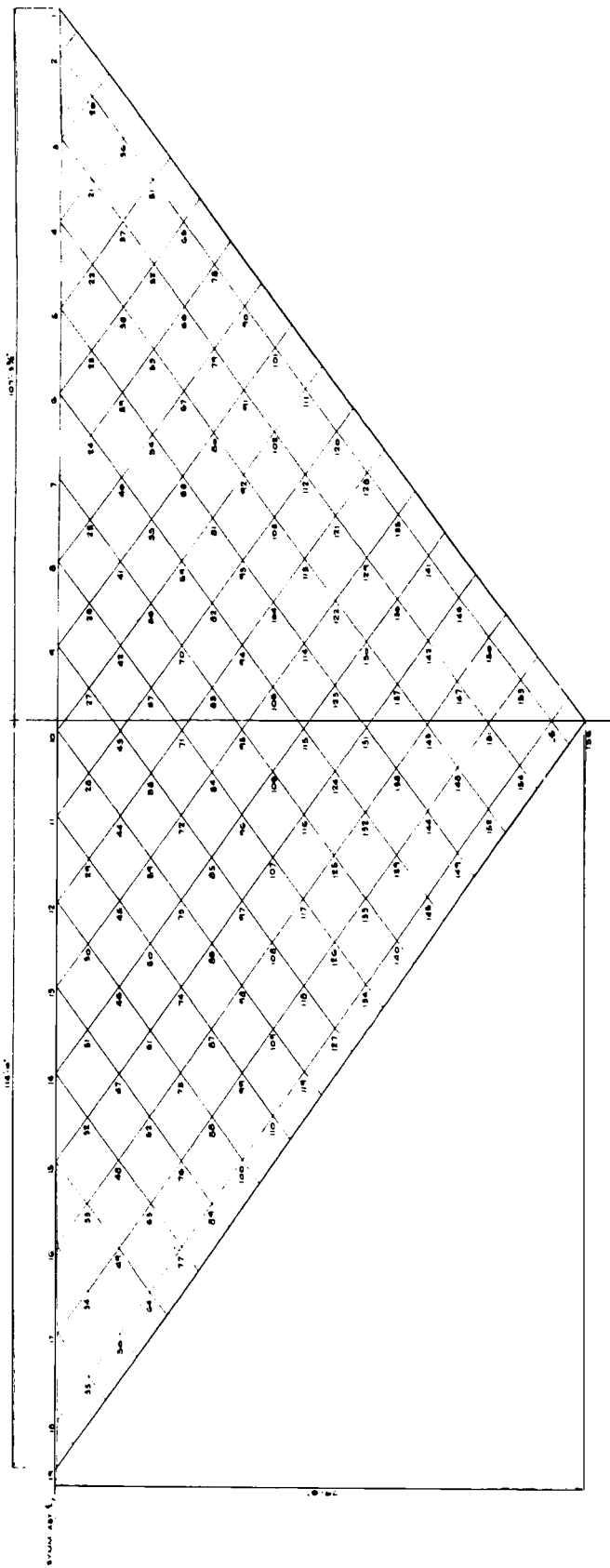
HYPAR SHELL IN BRAZIL

SHEET NO. VIII.7.5



HYPAR SHELL (EDENS THEATRE) GEOMETRY

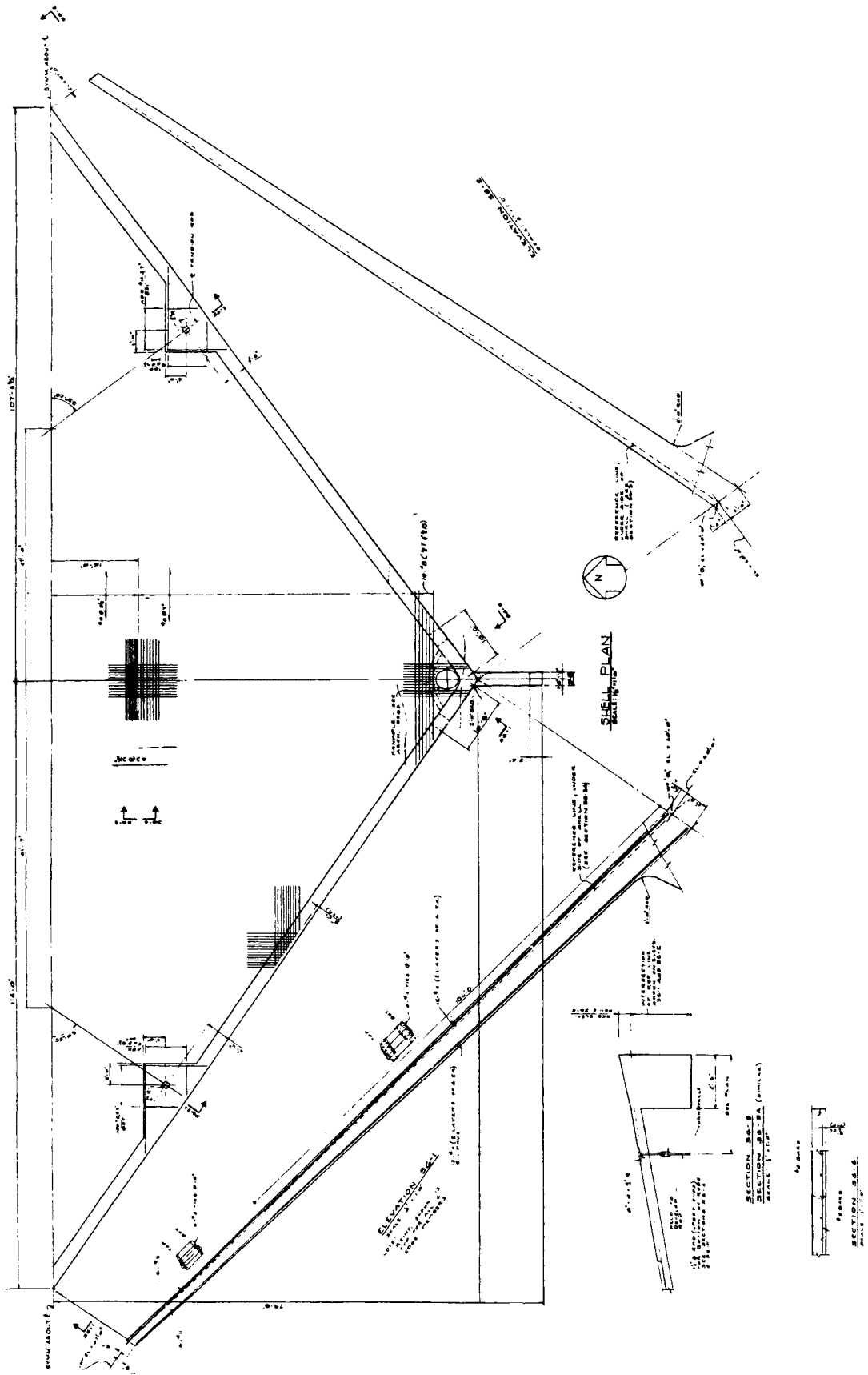
SHEET NO. VIII.7.6



HORIZONTAL PLAN OF ROOF IN TILTED POSITION
SCALE: 1/100

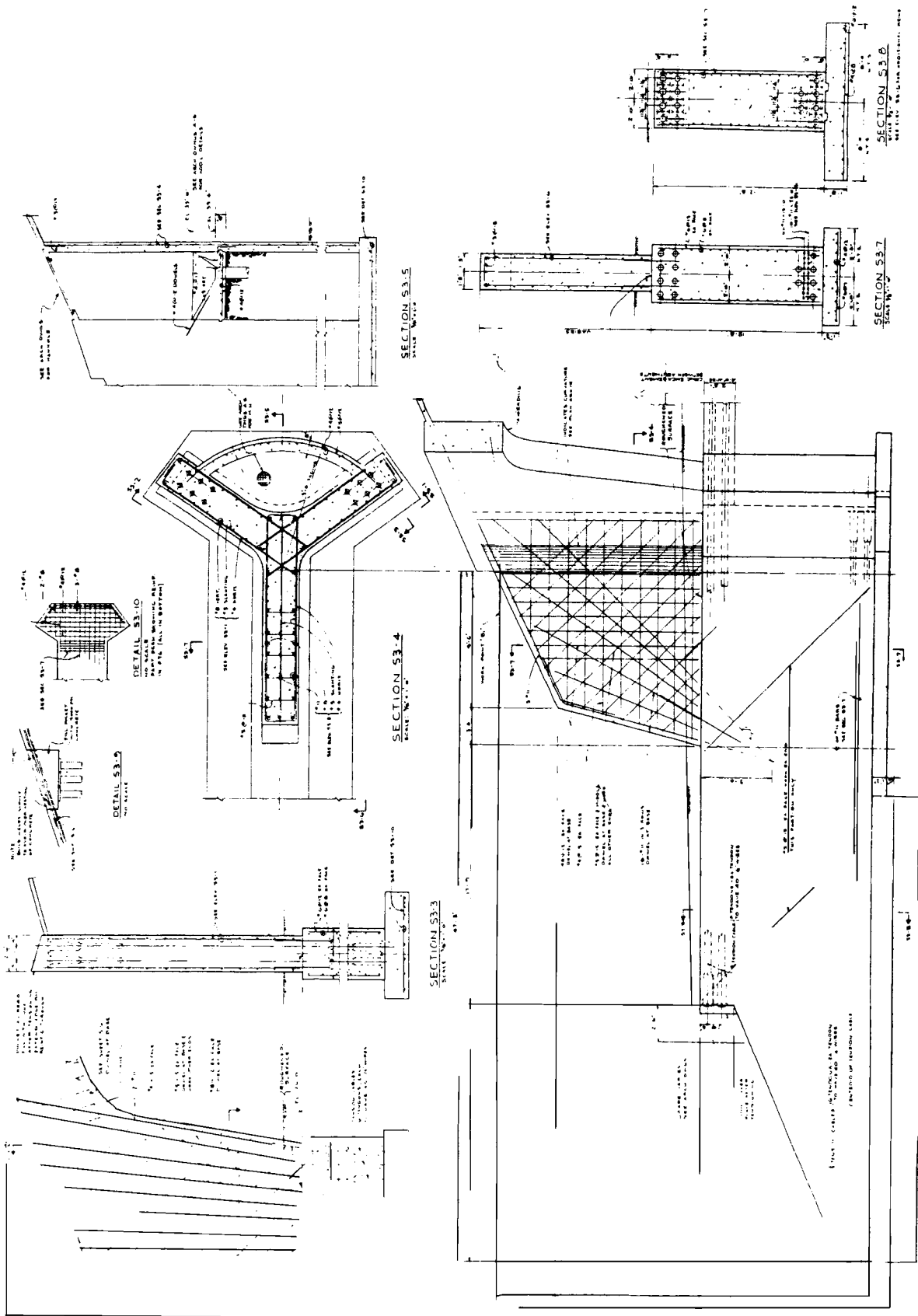
HYPAR SHELL (EDENS THEATRE) REINFORCEMENT DETAILS

SHEET NO. VIII.7.7



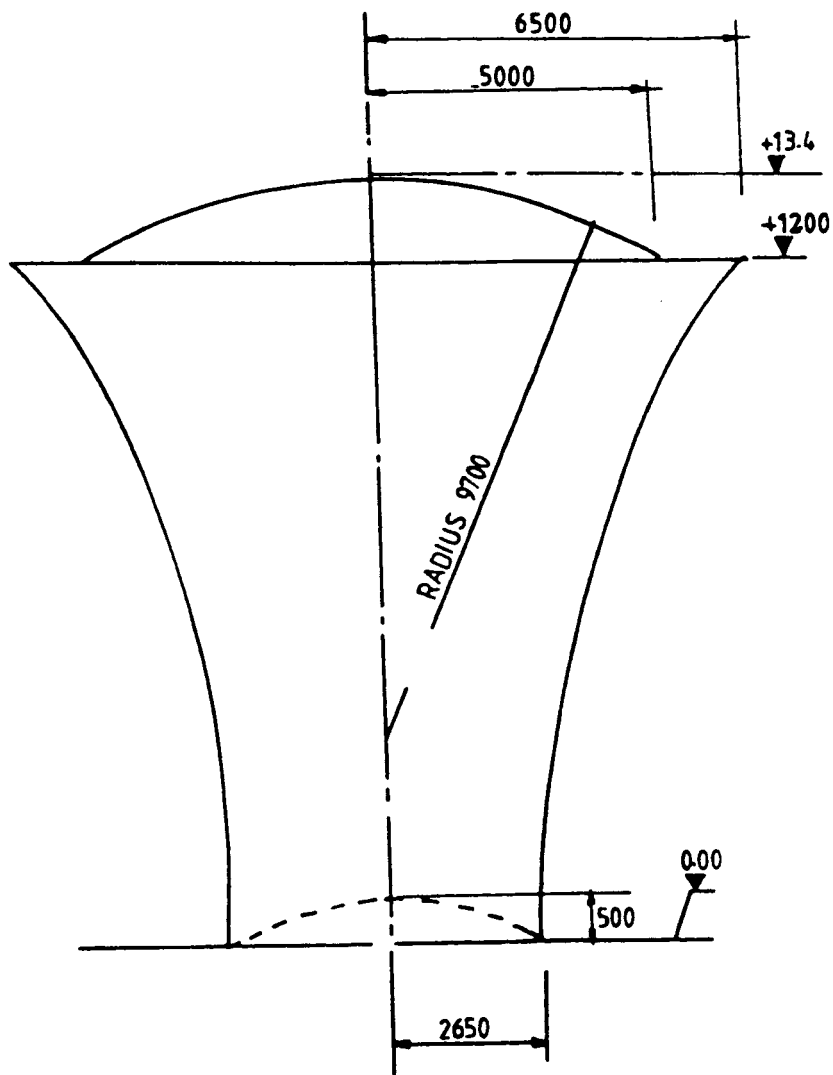
HYPAR SHELL (EDENS THEATRE) ADDITIONAL STRUCTURAL DETAILS

SHEET NO. VIII.7.8



GENERAL DIMENSION OF WATER TANK SHELL TOWER

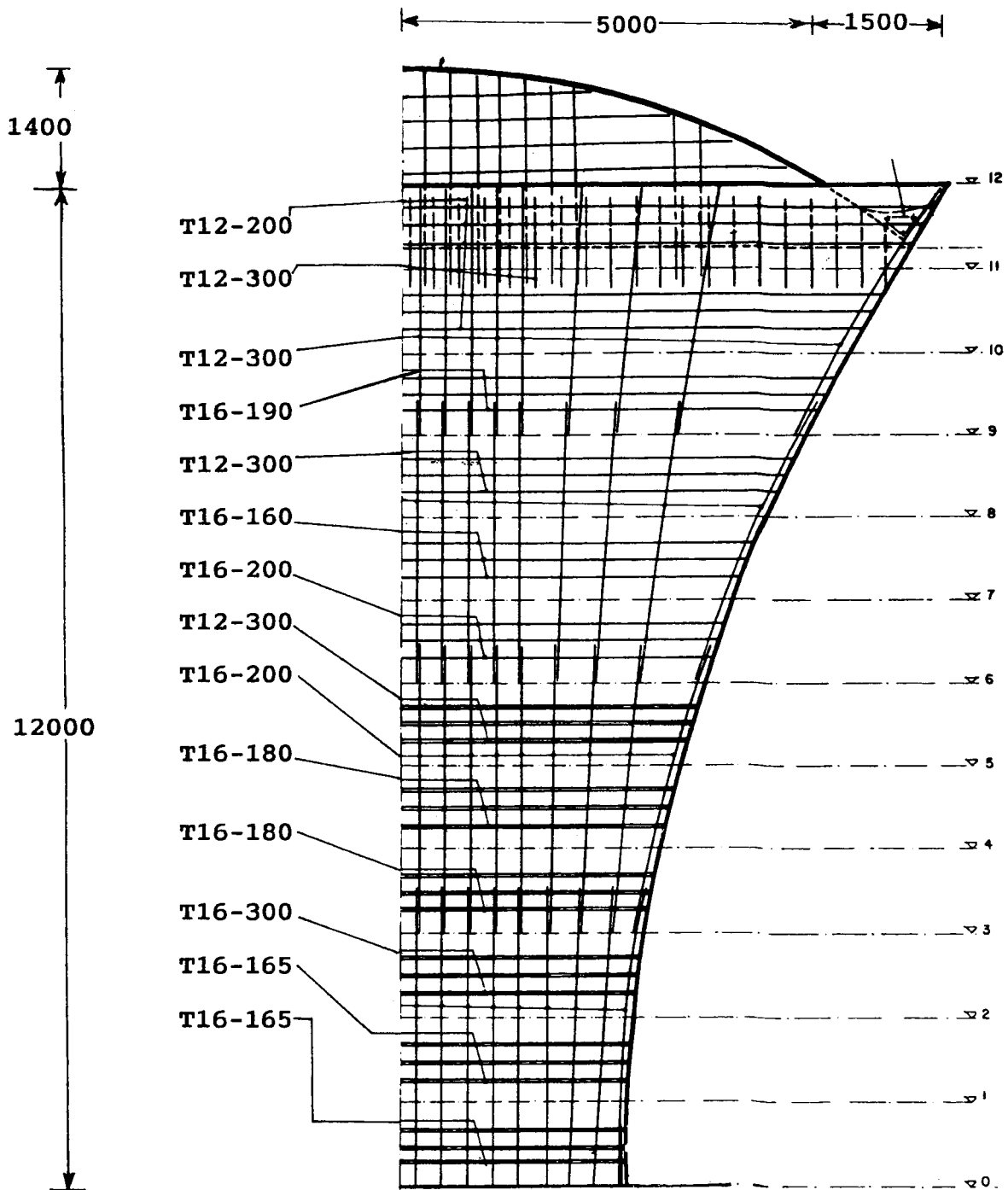
SHEET NO. VIII.7.9



HYPERBOLIC SHELL

SHEET NO. VIII.7.10

REINFORCEMENT DETAILS OF
WATER TANK SHELL

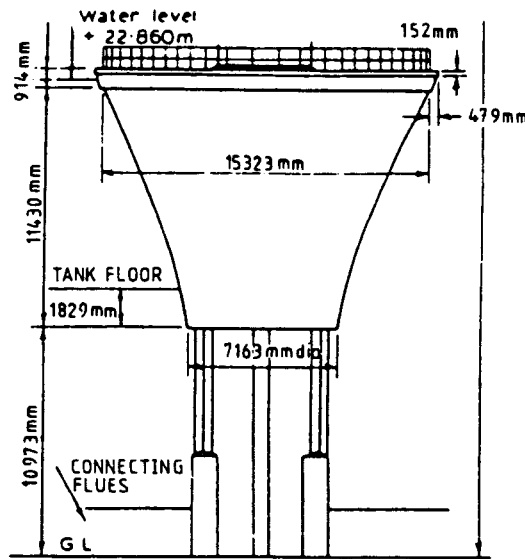


(Courtesy of S.D. Castillo)

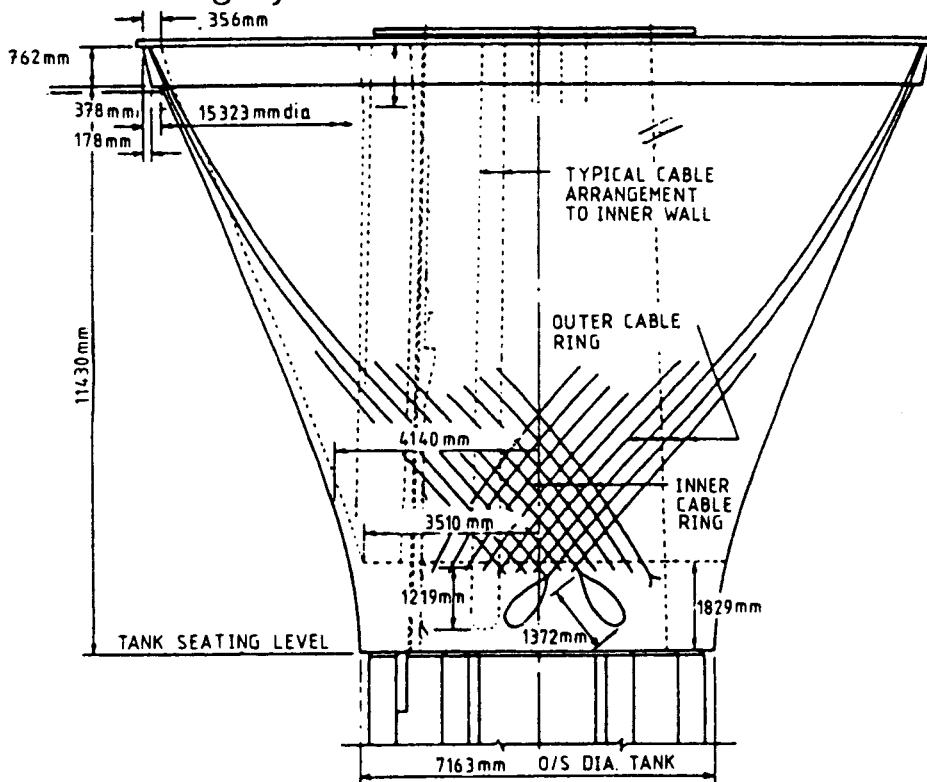
PRESTRESSED HYPERBOLIC SHELL OF WATER TANK

SHEET NO. VIII.7.11

(a) General layout

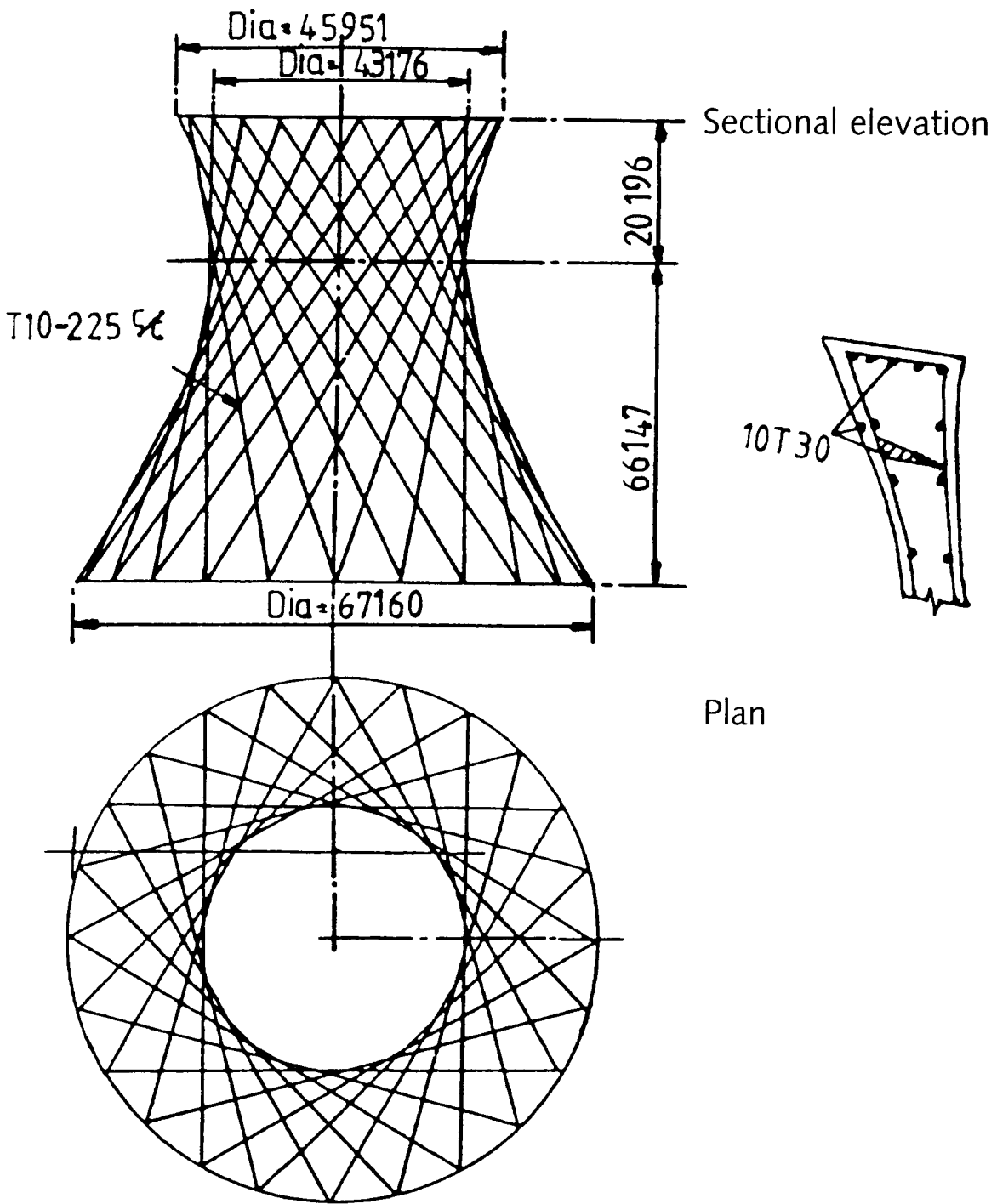


(b) Prestressing layout



HYPERBOLIC SHELL (COOLING TOWER REINFORCEMENT)

SHEET NO. VIII.7.12



Sheet No. VIII.7.11 shows a hyperbolic shell surface of a water tank. Owing to its much larger size it became necessary to prestress the shell. The entire tank is supported on columns. Prestressing is done by continuously winding the prestressing strands around the tank. The strands are of low relaxation type given in Section V. (Some other types of shell structures adopted for water tanks or water-retaining structures are discussed in Section VIII.3.)

The design of the cooling towers must take into consideration buffeting wind loads and sometimes earthquake effects. They are needed as exhausts for power stations. In plan they are placed in straight or zigzag rows. They stand in the higher region of the layer of air flow, which is usually unsteady and turbulent. A special dynamic analysis has been carried out to check the reinforcement layout of the cooling tower given on Sheet No. VIII.7.12.

P. C. Varghese and A. C. Mathai designed Christ Chapel in Irinjalakuda, Kerala, India, which was consecrated in May 1971. This wonderful shell design is unique, consisting of 12 identical shell units of groin type, each shell placed with a column at the end set at 30° intervals around the periphery of the hall. The shell roof has a varying diameter (24.38 m to 26.52 m) in plan. The columns are placed at 5.97 m centres. Sheet No. VIII.7.13 gives the longitudinal section and the plan, which gives the variable shell thickness and the reinforcement details. The reinforcement layout and designation are based on current practices and are thus modified.

Folded plates, known as hipped plates, are developed by joining a series of rectangular slabs at suitable angles of inclination. They are monolithic along their common edges and they span between diaphragms. The various types formed are V-type, through type, cylindrical type, Z-shaped type, north-light roofs, bunker shaped and troughs with lights at the top. Sheet Nos VIII.7.14 and VIII.7.15 give the reinforcement details for a cylindrical hipped plate and the north-light shell.

Spherical domes are common in structural engineering. Reinforced concrete domes are comparatively popular. A constant thickness is considered. The reinforcement details are shown on Sheet No. VIII.7.16 for a segment of a spherical dome and its ring beam.

When the following modifications are carried out a similar layout can be prepared for either conoidal or elliptical domes in reinforced concrete:

Conoidal dome: The central line of revolution is moved outward from the centre line of revolution of the spherical dome to a distance $(r \cos \varphi - r')$, r is the radius of the spherical dome and r' is the distance moved beyond this. The angle φ is the latitude of the dome.

Elliptical dome: The dome surface is defined as:

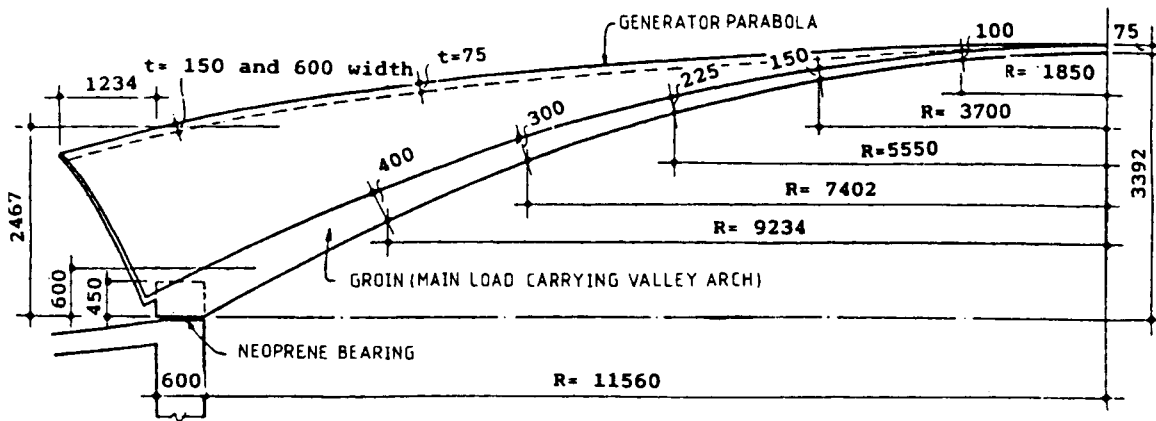
$$x^2/a^2 + y^2/b^2 = 1$$

where a and b are major and minor axes, x and y are coordinates.

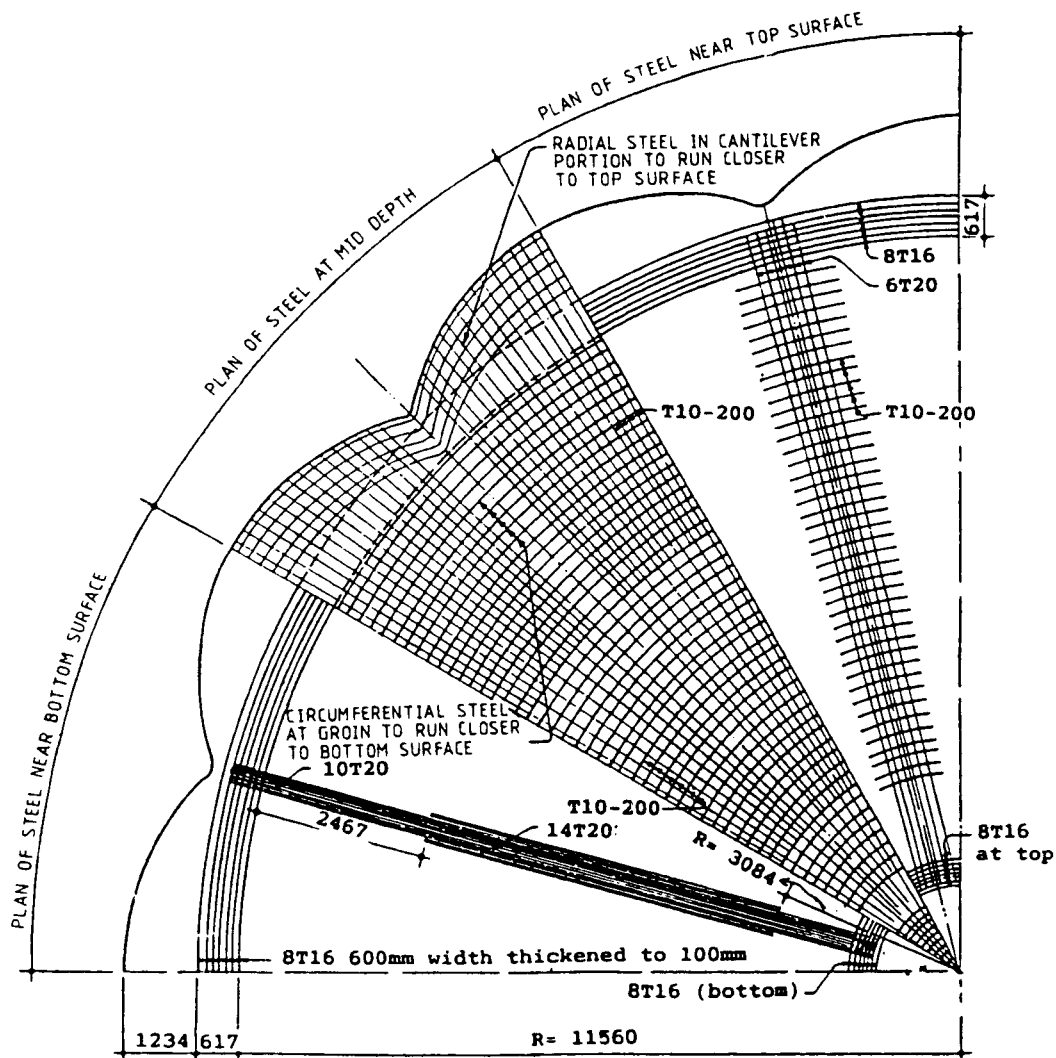
Reinforced concrete chimney shells are generally lined with steel liners so that the flue gases which are too hot and corrosive are prevented from having direct contact with the reinforced concrete. The chimneys are about 180 to 215 m tall and their minimum diameter is about 4 m. They are subjected to wind and earthquake loads and thermal gradients. Sheet No. VIII.7.17 gives a basic reinforcement layout for a chimney. The size or reinforcement depends on the height and the wind/seismic loads.

SHELL OF CHRIST CHAPEL

SHEET NO. VIII.7.13



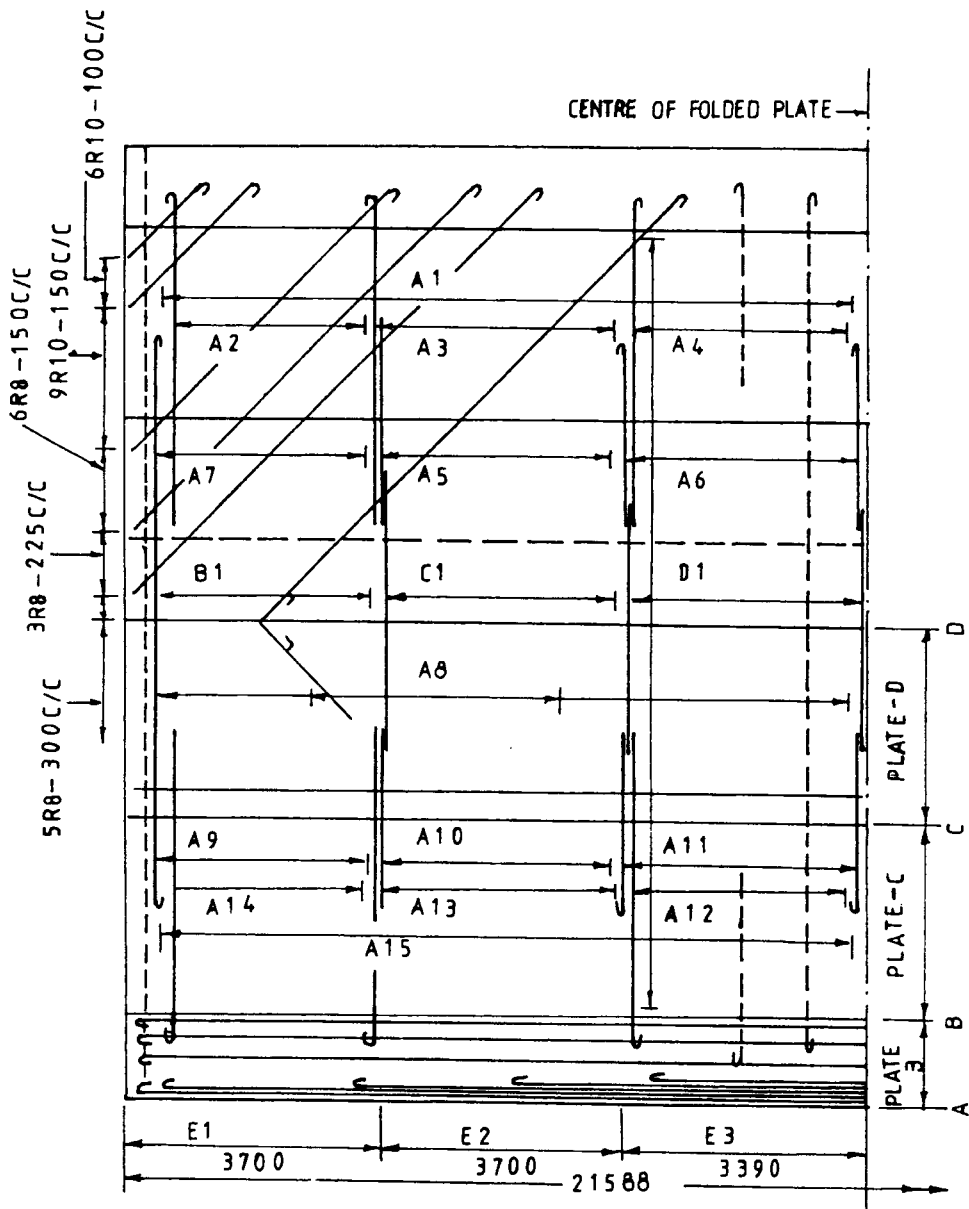
(a) Longitudinal section of groin showing the variation of thickness



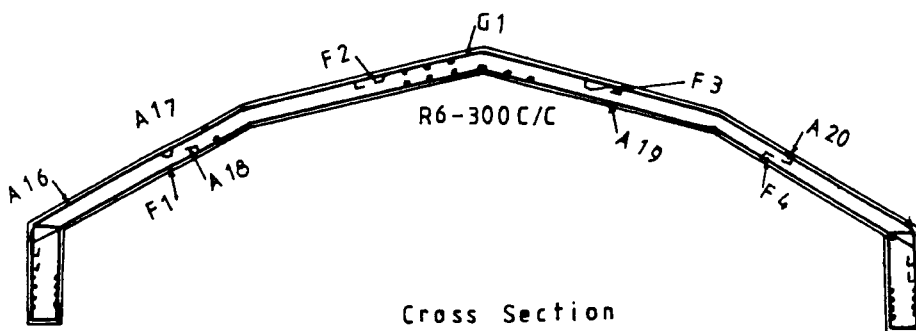
(b) Plan showing layout of reinforcement in the shell

A FOLDED PLATE ROOF—REINFORCEMENT LAYOUT

SHEET NO. VIII.7.14



(a) Developed plan

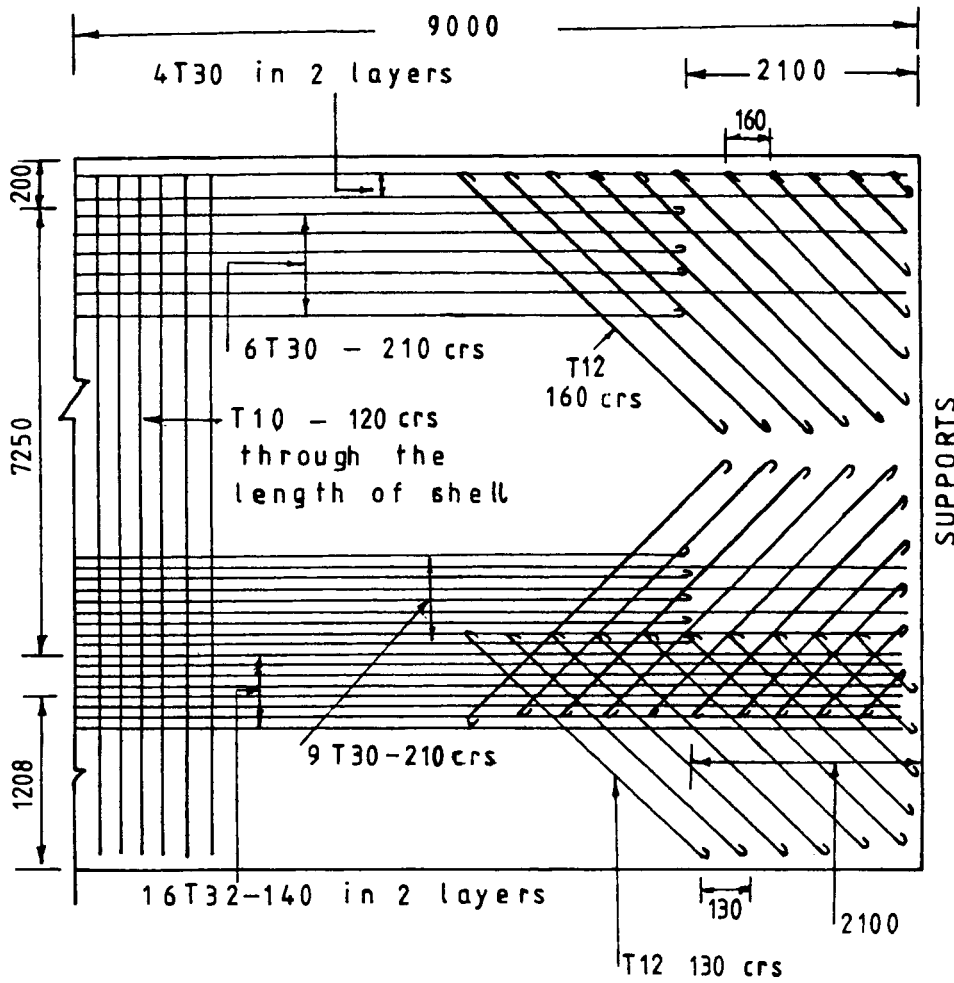


Cross Section

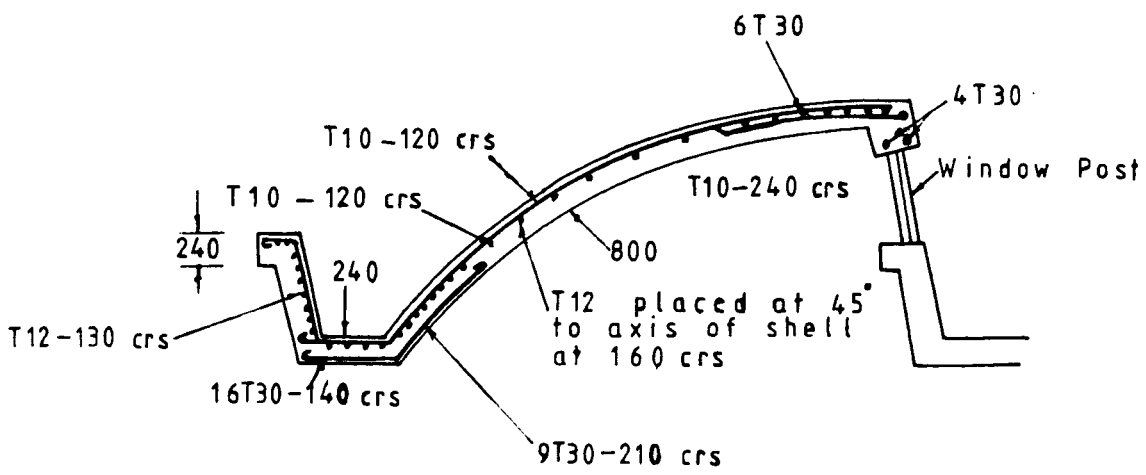
(b) Reinforcement details of folded plate 450,620,700 c/c

NORTH-LIGHT SHELL — REINFORCEMENT LAYOUT

SHEET NO. VIII.7.15



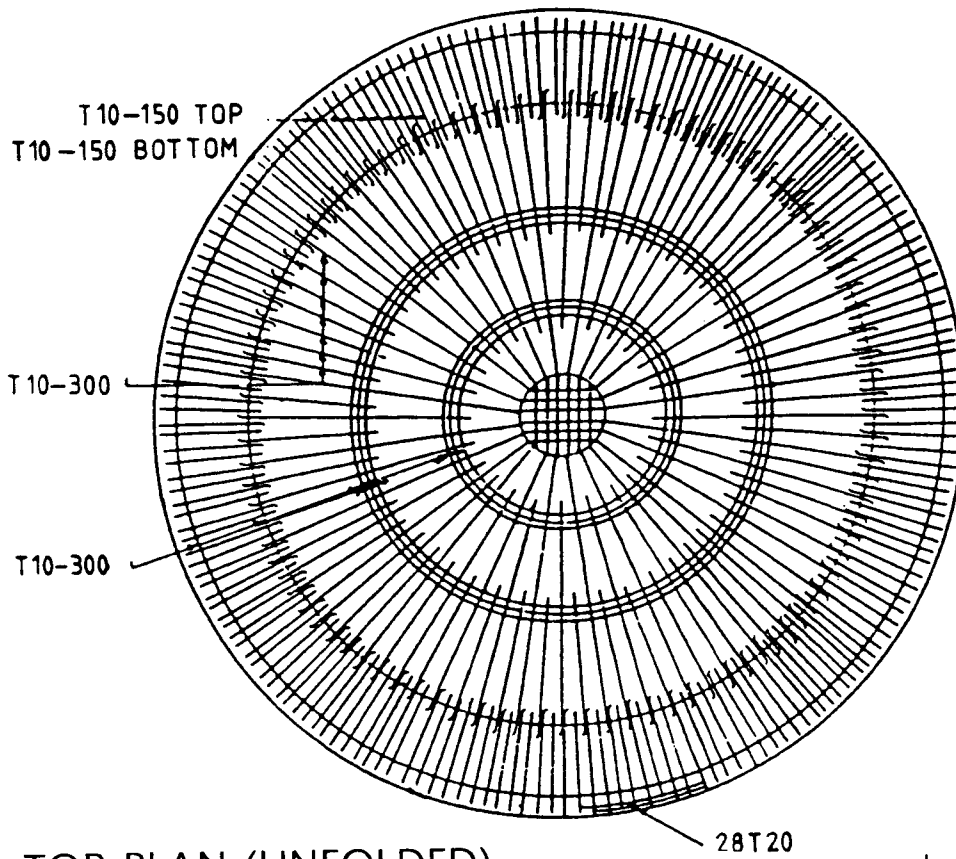
PLAN



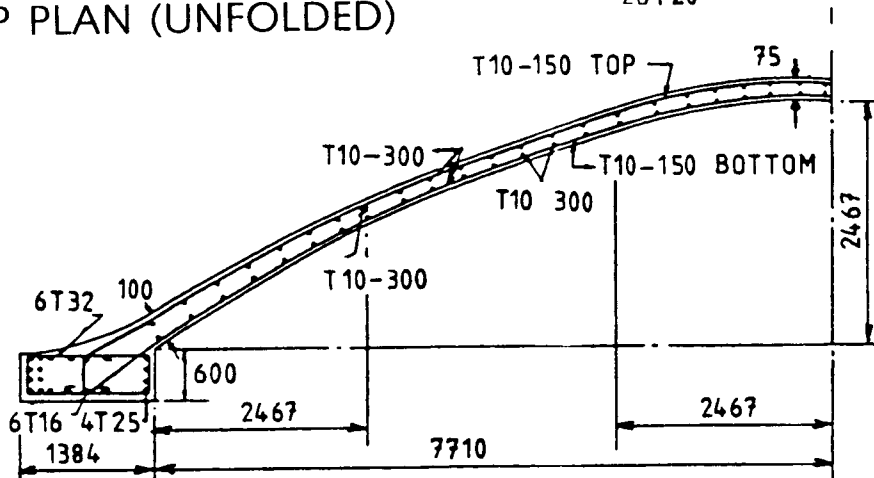
SECTION

SHALLOW DOME

SHEET NO. VIII.7.16



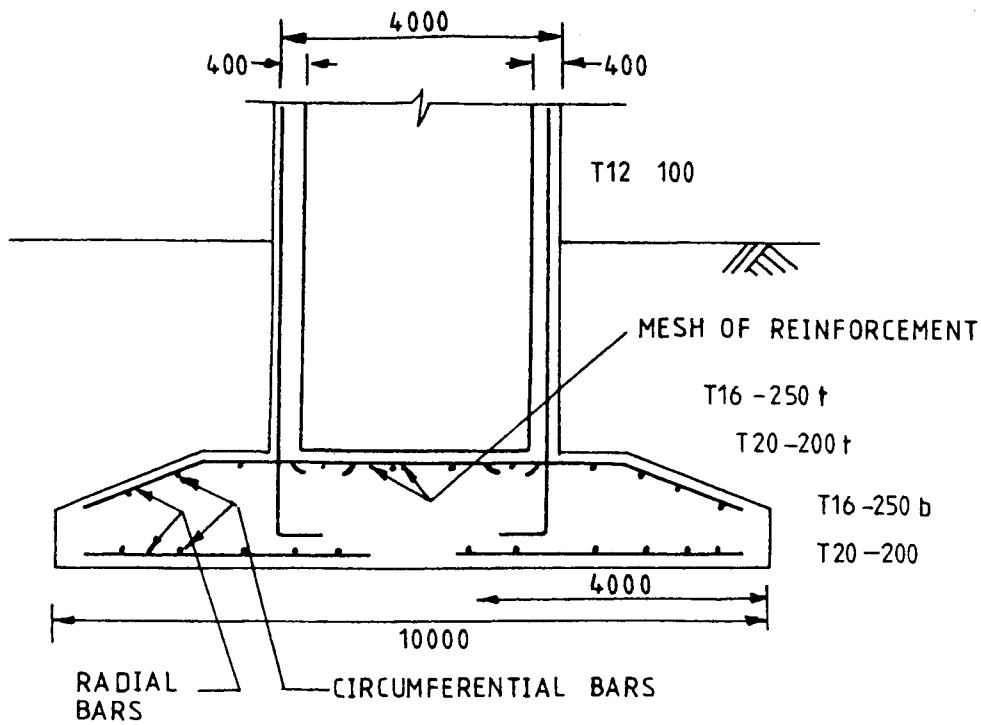
TOP PLAN (UNFOLDED)



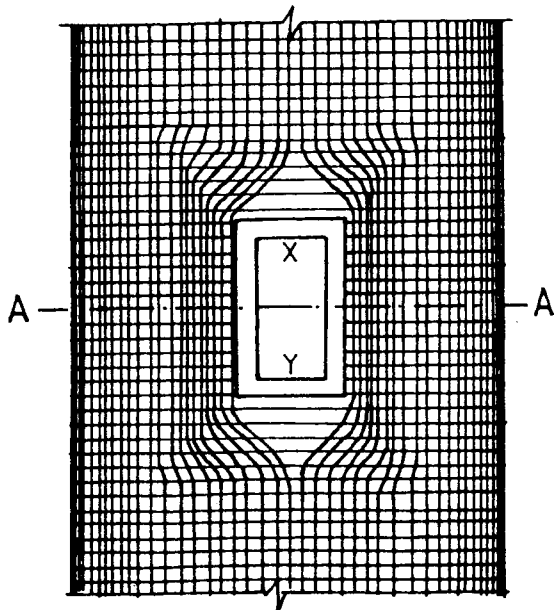
SECTION

CHIMNEY SHELL

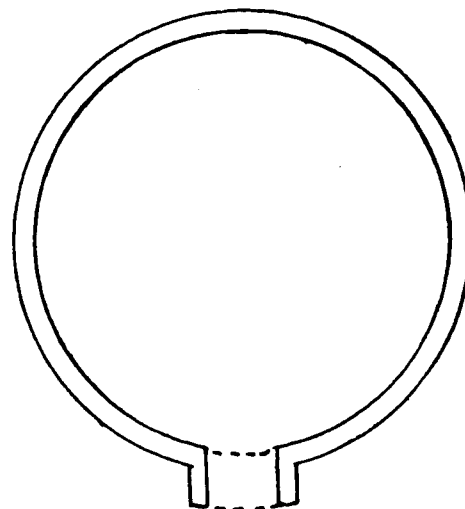
SHEET NO. VIII.7.17



(a) Section through foundation of chimney



(b) Front view of reinforcement round an opening in a chimney shell



(c) Section A-A through opening

VIII.7.3. Case study: Santa Famiglia Parish Complex Grosseto, Italy

All those curved section members of the segments that, starting out spaced wide from the edge of the circular plan, come to join together at the apex of the dome, all those sickle-shaped windows that create alternating zones of light and shadow, they too starting from the base and coming together at the apex. The dome is created with truncated conical elements.

The complex comprises four nuclei:

- (a) the truncated-conical campanile or Belfry partly compenetrating in the portico, or circular-plan two-columned portico
- (b) the circular-plan worship area, with dome roof and, below H, a room for meetings and entertainment, lying between elevation -1.70 m and $+3.00$ m
- (c) the sacristy and parish offices, on one floor only, of rectangular plan, above the plate
- (d) catechism classrooms and, above them, the parish priest's living quarters.

VIII.7.3.1. Structural characteristics

The whole complex has a reinforced concrete structure. The bodies holding the offices, the classrooms and priest's quarters and the plate are the usual frame structures with their outside white-cement concrete walls left fair face.

The body comprising the circular room with the overlying worship area comprises 16 radial frames jointed at their feet and connected each to each at the elbow and centrally by ring beams and by an in-situ-cast hollow clay brick and concrete floor structure, whose upper slab forms the floor for the worship area.

Sixteen radial trusswork arches that join together in the lantern ring beam at the crown are set down on the outer ring beam. These arches are connected two by two at extrados and soffit by a spherical slab 20 cm thick that forms the inner and outer groovings in the dome.

VIII.7.3.2. Foundations

The foundations comprise a 28.70 m outside-diameter ring beam of cross-section 2.10 m \times 1.00 m, calculated as an elastic beam on elastic soil.

The hinges are of concrete; the pier base is of cross-section 600 mm \times 400 mm; the bottom has a section of 200 mm \times 360 mm, and falls within the geometric design rules according to Monnig-Hetzel.

The pressure on the concrete was checked out to be 165 kg/cm². The reinforcing comprised three layers of 6 dia. 10 bars.

VIII.7.3.3. Materials: the study of the concrete mix

The white-cement concrete was mixed with white aggregates coming from the Zandobbio quarry in Bergamo and with 575 white cement.

The composition of the aggregates mix was worked out to achieve a perfectly impermeable concrete for the dome mantle, which had to be of moderate thickness (200 mm) and had also to do without waterproofing linings so as to preserve the aspect of the fair-face white-cement concrete.

Preliminary impermeability tests made on a small tank of 200 mm wall thickness made of the design concrete were successful.

In a first approximation calculation, with a class 350 aggregate and 575 white cement being projected, a mix having a cement content of 370 kg/m³ and a water/cement ratio of 0.65 provided a cone slump of 11 cm, owing to the maximum aggregate diameter used.

Preliminary tests displayed a compression strength of:

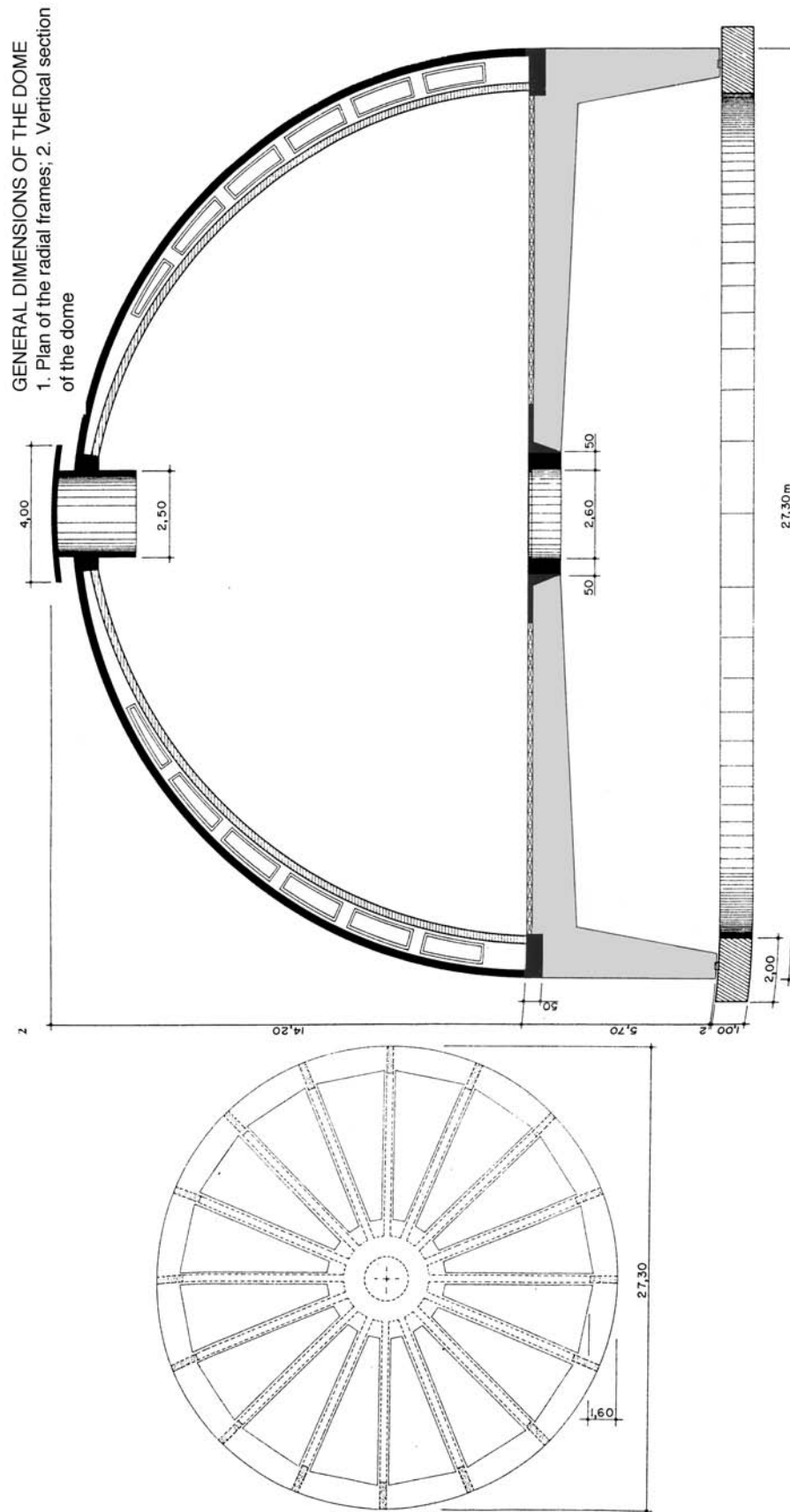
- (a) 1 day: 15.3 N/mm²
- (b) 2 days: 25.1 N/mm²
- (c) 7 days: 38.2 N/mm²

The general dimensions of the dome indicating the plan of the radial frames and vertical section of the dome are shown on Sheet No. VIII.7.18. General details of the tower of Belfry are given on Sheet No. VIII.7.19. The tower has a base 6.5 m diameter and reduces to conical shape in a truncated manner as shown on various cross-sections. This is known as a truncated-conical campanile with narrow deep grooves. They have wider spacings at the base and slowly rise over the full height of the tower 49.10 m until trays converge at the apex. Sheet No. VIII.7.20 gives the foundation ring and the reinforcement details of the radial frames. The ring girder forming the foundation has an external diameter of 28.7 m and its section is 2.10 m × 1.0 m. The upright of the radial frames are provided with the starter bars. A total number of 16 radial frames is provided, freejointed at the foot levels but interconnected at the elbow central areas.

Sheet No. VIII.7.21 shows reinforcement details of the dome and curved members. The detailing of the bars in the vertical and section of the upper ring are clearly shown. In order to clearly visualise the reinforcement detailing, the section is also given together with the curved member development with bars in different directions.

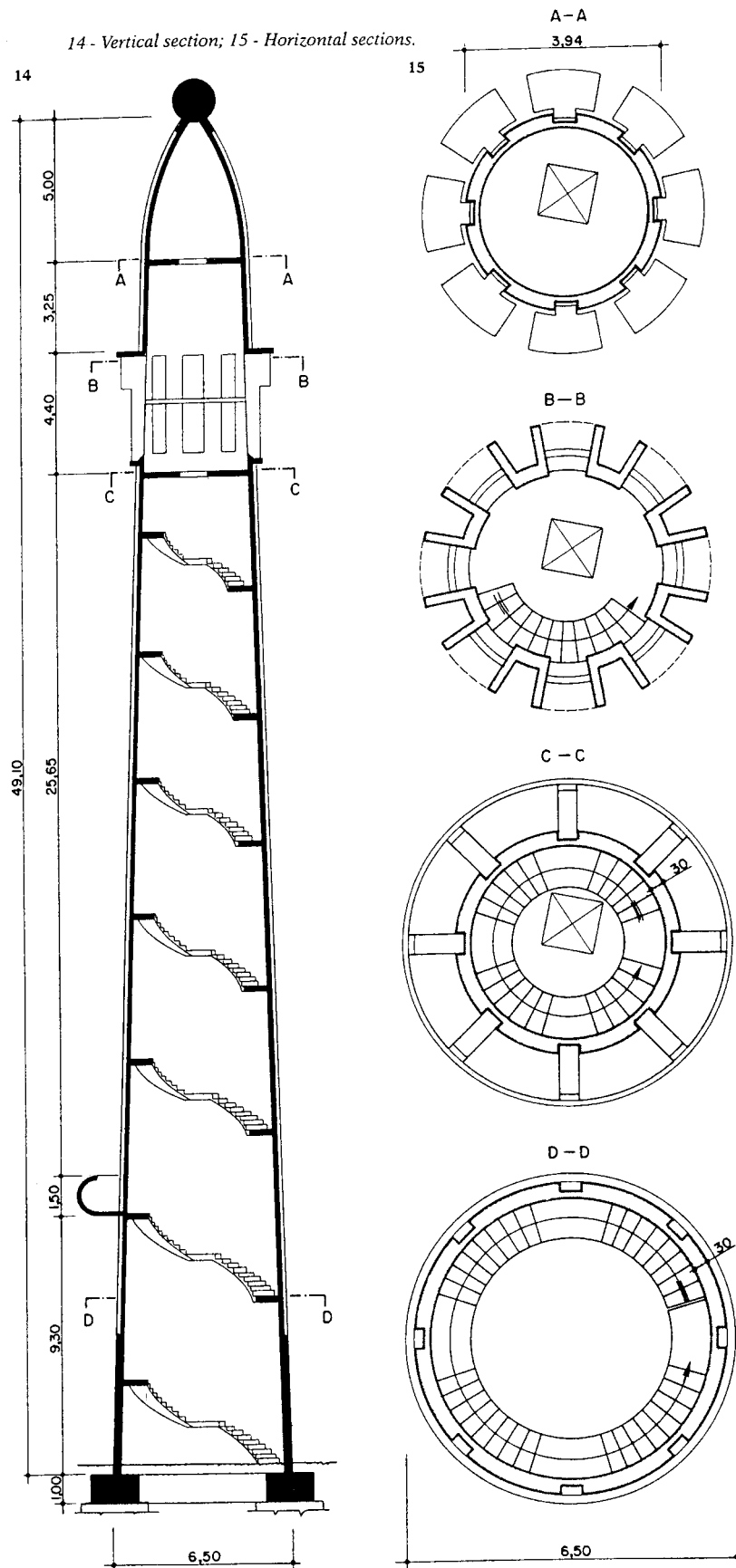
RADIAL FRAMES AND DOME

SHEET NO. VIII.7.18



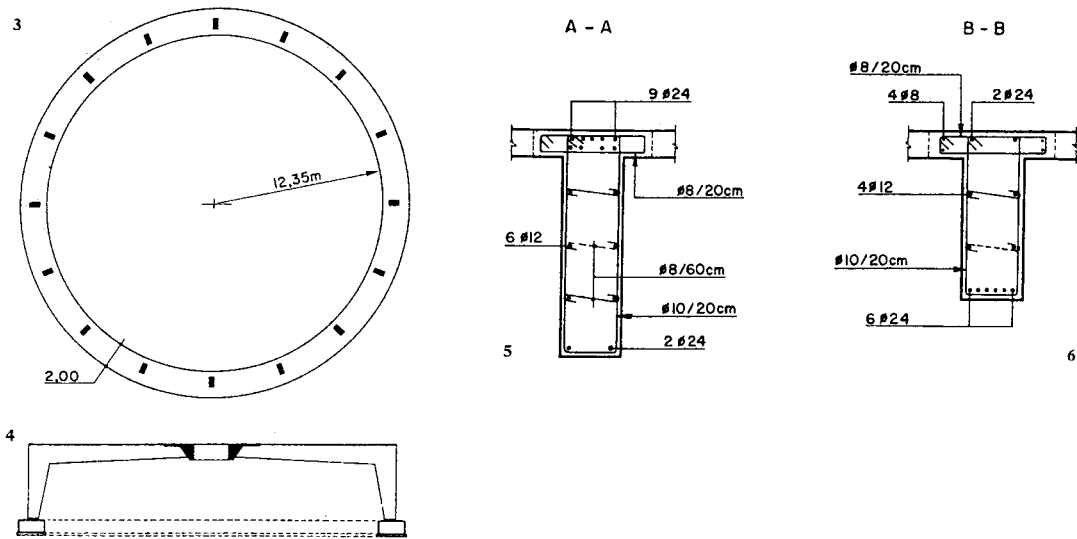
**GENERAL DETAILS OF THE TOWER OR BELFRY
(EUROPEAN PRACTICE)**

SHEET NO. VIII.7.19

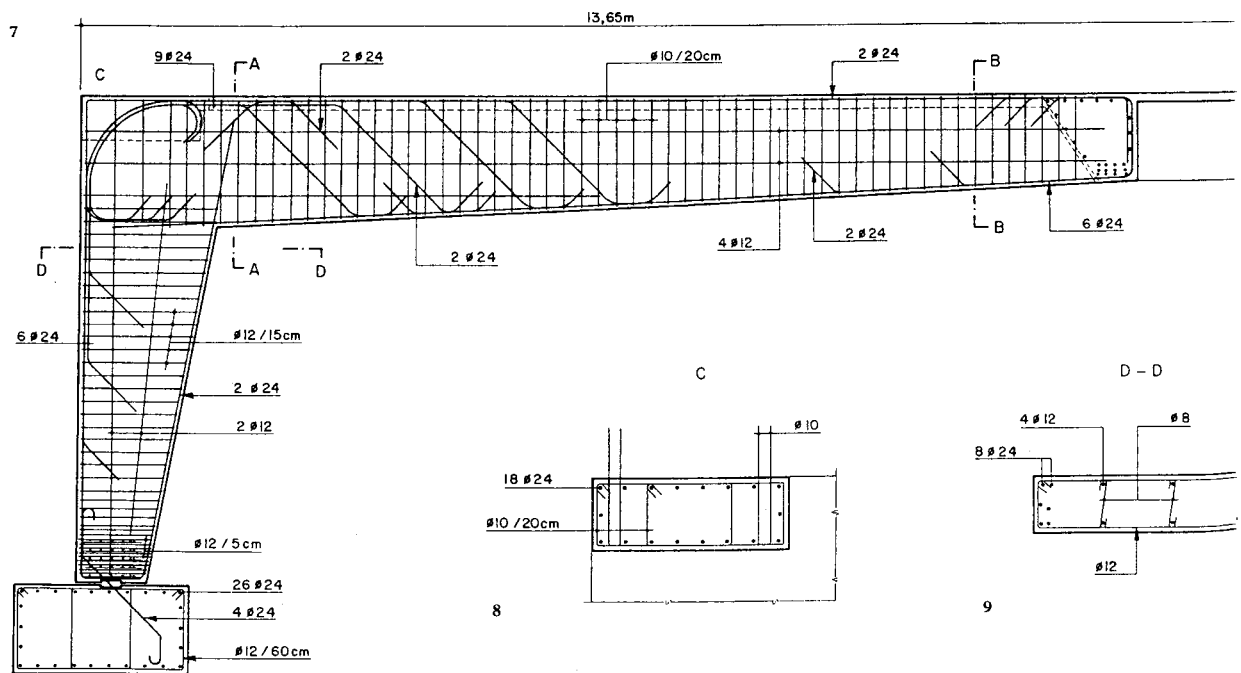


**FOUNDATION RING AND RADIAL FRAMES—
REINFORCEMENT DETAILS (BASED ON EUROPEAN PRACTICE)**

SHEET NO. VIII.7.20



3 –Foundation ring; 4-5-6-9 –Longitudinal and cross sections of frames;
7 –Half longitudinal section of a frame; 8 –Cross section of the edge beam.



**DOMES AND CURVED MEMBERS—
REINFORCEMENT DETAILS (BASED ON EUROPEAN PRACTICE)**

SHEET NO. VIII.7.21

10 –Vertical section; 11 –Section of the upper ring of the dome; 12 –Cross section; 13 –Development of a curved member.

